

DRAFT 2026 UPDATE

Technical Guide – River and Stream Systems: Flooding Hazard Limit

Ministry of Natural Resources

Cette publication hautement spécialisée, Technical Guide – River & Stream Systems: Flooding Hazard Limit n'est disponible qu'en anglais conformément au Règlement 671/92, selon lequel il n'est pas obligatoire de la traduire en vertu de la *Loi sur les services en français*. Pour obtenir des renseignements en français, veuillez communiquer avec le ministère des Richesses naturelles au hazard.policy@ontario.ca.

Disclaimer

This technical documentation has been prepared by His Majesty the King in right of Ontario as represented by the Ministry of Natural Resources (the “Ministry”). No warranties or representations, express or implied, statutory or otherwise shall apply or are being made by the Ministry with respect to the documentation, its accuracy or its completeness. In no event will the Ministry be liable or responsible for any lost profits, loss of revenue or earnings, claims by third parties or for any economic, indirect, special, incidental, consequential or exemplary damage resulting from any errors, inaccuracies or omissions in this documentation; and in no event will the Ministry’s liability for any such errors, inaccuracies or omissions on any particular claim, proceeding or action, exceed the actual consideration paid by the claimant involved to the Ministry for the materials to which this instructional documentation relates. Save and except for the liability expressly provided for above, the Ministry shall have no obligation, duty or liability whatsoever in contract, tort or otherwise, including any liability or negligence. The limitations, exclusions and disclaimers expressed above shall apply irrespective of the nature of any cause of action, demand or action, including but not limited to breach of contract, negligence, strict liability, tort or any other legal theory, and shall survive any fundamental breach or breaches.

Cette publication spécialisée n’est disponible qu’en anglais.

Additional Information

This document does not comply with all the applicable guidelines for accessible digital documents. For an alternative format please email the Ministry of Natural Resources at hazard.policy@ontario.ca.

For more information about this document, please contact the Ministry of Natural Resources at hazard.policy@ontario.ca.

Contents

Disclaimer	i
Additional Information.....	ii
Contents.....	iii
List of Figures.....	x
List of Tables.....	xiv
Explanatory Note.....	xviii
1 Introduction	1
1.1 Background.....	1
1.2 Document Purpose and Scope	1
1.3 Flooding Hazard Standards to support flood hazard identification in River Systems.....	3
1.3.1 Flood Resulting from Rainfall Actually Experienced During a Major Storm Event	4
1.3.2 Flood Based on Exceedance Probability of Occurrence.....	4
1.3.3 An Observed Flood of Greater Magnitude than the Storm Criteria Outlined in Section 1.3.1 and 1.3.2 and Actually Experienced in a Particular Watershed	7
1.4 Flooding Hazard Limits for Lakes Less than One-Hundred Square Kilometres .	7
1.5 Municipal Official Plan Flooding Hazard Limit Policies.....	8
1.6 Flooding Hazard Limit – Zonation Concepts	8
1.7 Special Policy Area Concept.....	11
1.8 Access and Floodproofing.....	12
1.9 Public Safety	13
1.10 Impacts of a Changing Climate	13

1.11	Overview of flood magnitude and water level computation methods	14
1.11.1	Methods to Estimate and Check Return Period Flood Flows where no streamflow gauging station data are available	19
1.11.2	Combined Individual and Regional Analyses.....	21
1.12	Summary	22
1.13	Overview of Technical Modelling Protocols and Policies Applied in Ontario.	26
1.13.1	Hydrologic and Hydraulic Modelling -Technical Policies.....	26
1.14	Document Structure – How to Read this Document.....	31
2	Data and Approaches Supporting Hydrologic and Hydraulic Analyses: General Procedures.....	34
2.1	Hydrometric and Spatial Data Required	34
2.1.1	Streamflow Records	34
2.1.2	Historical Floods	35
2.1.3	Rainfall Actually Experienced from a Specified Meteorological Event	35
2.1.4	Stage-Discharge Relationships	36
2.2	Land Use.....	37
2.3	Land Cover Data	38
2.4	Soils Data.....	39
2.5	Addressing Regulated Flows.....	41
2.6	Meteorological and Physiographic Data.....	41
2.6.1	Regional Flood Frequency Analysis	41
2.6.2	Hydrologic Modelling	41
3	Hydrologic Analysis.....	43
3.1	Flood Frequency Analysis versus Rainfall Simulation Approach.....	43

3.2	Flood Frequency Analysis	44
3.2.1	Data Requirements and Assumptions to Support Flood Frequency Analyses	44
3.2.2	Single Station Flood Frequency Analysis	49
3.2.3	Regional Flood Frequency Analysis	53
3.2.4	Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins	55
3.3	Runoff Simulation from a Specified Meteorological Event.....	58
3.3.1	Calibration and Validation.....	59
3.4	Rainfall Analysis.....	61
3.4.1	Introduction.....	61
3.4.2	Historical Storms	66
3.4.3	Use of Return Period Storms.....	75
3.5	Snowmelt and Winter Precipitation	88
3.5.1	Snowmelt Flood Generation and Flood Hazard Mapping	89
3.5.2	Snowmelt Computations.....	90
3.5.3	Snow Monitoring Data	92
3.6	Hydrologic Modelling of Flood Flows to Define the Flooding Hazard	94
3.6.1	Types of Hydrologic Models	94
3.6.2	Computational Procedures Used in Hydrologic Models.....	100
3.6.3	Recommended Model Selection.....	106
3.6.4	Hydrologic Parameter Selection	108
3.6.5	Model Calibration and Validation.....	108
4	Hydraulic Modelling.....	115

4.1	Planning for Hydraulic Modelling.....	115
4.2	Hydraulic Model Selection and Parameterization Considerations.....	118
4.2.1	Level of Analysis Required	119
4.2.2	Solution Methods – Steady vs. Unsteady Flow.....	121
4.2.3	One-Dimensional, Two-Dimensional and Quasi- and Coupled 1D-2D- Approaches.....	123
4.2.4	Grid and/or Mesh Resolution	127
4.2.5	Cross-Sections	128
4.2.6	Critical Depth	128
4.2.7	Velocity Distribution	128
4.2.8	Roughness	128
4.2.9	Use of High Water Marks.....	129
4.2.10	Plotting Routines	129
4.3	Criteria and Associated Data to Support Hydraulic Analyses in defining the Flooding Hazard	129
4.3.1	Hydraulic Coefficients.....	129
4.3.2	Elevations of High-Water Marks	130
4.3.3	Aerial Photographs and Remotely Sensed Data	132
4.3.4	Cross-Sections	132
4.3.5	Lake Levels	134
4.3.6	Boundary Conditions	134
5	Water Level Computations – Open Water Hydraulic Modelling	138
5.1	General	138
5.2	Backwater Profiles	138

5.3	Flood Routing	139
5.3.1	Hydrologic Routing	139
i.	Muskingum method	140
5.3.2	Hydraulic Routing	141
i.	Kinematic Wave.....	141
ii.	Dynamic Routing	141
5.4	Special Considerations in Hydraulic Modelling.....	142
5.4.1	Effects of Lakes and Reservoirs	142
5.4.2	Confluence with Rivers	145
5.4.3	Waterway Crossings and Encroachments	145
5.4.4	Bridges and Culverts	148
5.4.5	Split Channel Flow.....	149
5.4.6	Dykes.....	149
5.4.7	Flood Spills from the Channel.....	150
5.4.8	Other Specific Considerations	151
5.5	Hydraulic Model Calibration	153
5.6	Verification	155
5.7	Testing and Sensitivity	156
6	Ice-Affected Flooding - Water Level Derivations and Computations	158
6.1	Evaluating Ice Induced Flooding as a Flooding Hazard	158
6.2	River Ice Processes and Influence on River Stage	160
6.2.1	Classification of Thickened Ice Jam Type and Elaboration on Equilibrium Ice Jam Conditions:.....	165

6.2.2	Ice Affected Flooding in Large Rivers Relative to Smaller Rivers and Streams.....	167
6.3	High Level Assessment of Ice Jam Levels and Ice Jams as a flooding Hazard	177
6.4	Methods to Estimate Ice Influenced Flood Stage at Specific Areas of Interest on Rivers and Streams	178
6.4.1	Biophysical Approach	179
6.4.2	Historical Flood Approach.....	181
6.4.3	Flood Envelope Approach	182
6.4.4	Hydrotechnical Approach.....	182
6.4.5	Summary	183
6.4.6	Empirical Ice Jam Models for Determining Potential Ice Jam Stage.....	184
6.4.7	Direct Methods Including Data Sources.....	194
6.4.8	Indirect Methods and Approaches to Ice Affected Stage Derivation and Computation.....	204
6.5	Procedure for Estimating the Ice-Affected Effect Stage and Comparison to Open-Water Flooding Hazard Standards.....	223
6.6	Historical Documentation and Collection of Ice-Effect Level Data.....	226
6.6.1	Historical Flood Level Data.....	229
6.6.2	River Geometry	230
6.6.3	Ice Monitoring	231
6.6.4	During an Ice Jam	232
6.6.5	Post Ice Jam Observations.....	232
6.6.6	Report and Documentation.....	233
6.6.7	Ice-Affected Flooding: Influence of a Changing Climate.....	233

6.7	Conclusions.....	235
7	Technical Report and Documentation	237
7.1	Technical Report.....	237
7.2	Third party technical reviews.....	242
8	References.....	244
9	Glossary.....	269

List of Figures

Figure 1-1 Three Flooding Hazard Criteria Zones of Ontario	6
Figure 1-2 One-Zone Flooding Hazard Limit Concept (not to scale)	9
Figure 1-3 Two Zone Floodway-Flood Fringe Concept	10
Figure 1-4 Summary of 100-year Primary and Secondary Flood Peak Computation....	24
Figure 1-5 Computation of the Flooding Hazard and Flooding Hazard Limit (Flood Levels).....	25
Figure 2-1 Spatial extent of the Southern Ontario Land Resources Information System (SOLRIS).....	39
Figure 3-1 Process flow chart providing an overview of steps followed during a flooding hazard estimation study where non-stationarity is a potential issue (adapted from, Faulkner & Warren, 2020).	48
Figure 3-2 Flooding Hazard Standard Selection (segmented lines and boxes highlight where another return period or other storm has been approved for use by the Minister)	62
Figure 3-3 Hurricane Hazel Isohyets (Source: adapted from Mason et al., 1955).....	67
Figure 3-4 Hurricane Hazel 12 Hour Hyetograph (a) Hourly % Rainfall (of Total) and (b) Accumulated Rainfall (%) of Total	69
Figure 3-5 Hurricane Hazel Area Reduction.....	71
Figure 3-6 Timmins Storm 12-Hour Hyetograph (a) Hourly (% of Total Rainfall)) and (b) Accumulated Rainfall Total (%)	73
Figure 3-7 Provincial distribution of mean annual precipitation (mm) for 1981-2010 (Source: Watershed Hydrology Lab, MNR)	78
Figure 3-8 Areal Reduction Curves according to a) World Meteorological Organization (1973) and b) U.S. National Weather Service (Myers & Zehr, 1980), Curves	82
Figure 3-9 MSC Twelve-Hour Rainstorm Distributions for (a) Southern and (b) Northern Ontario	84

Figure 4-1 Example 1D and 2D model geometry and outputs (Source: Adapted from - U.S. Department of Transportation, 2019)	125
Figure 4-2 Comparison of Steady and Unsteady State Model Inflow Requirements...	136
Figure 4-3 Example Hydrograph Summation	137
Figure 5-1 Selection of flood standards at the confluence of lake, rivers and streams	147
Figure 6-1 Relationship between discharge and stage along a river reach, for open-water conditions (green line – “Site-specific rating curve”) and for ice conditions (coloured polygon areas). The segmented horizontal line defines the stage sufficiently high to cause flooding (Adapted from Barrette and Gaiqui (2023) after Turcotte et al. (2019)).....	160
Figure 6-2. Typical processes contributing to ice jams (source: NRCan, 2023)	162
Figure 6-3. Peak Water Level (W.L) caused by and ice jam as dependent on the location of interest and relative to the location and extent of the ice jam. The dashed line represents the open-water surface profile under the same discharge (Source: Beltaos, 2012).	164
Figure 6-4 Ice jam release with associated wave (or ‘jave’) and ice run. (Source: adapted from Barrette et al, 2025).	165
Figure 6-5. Longitudinal profile of hanging dam in the LaGrand River (Source: Michel and Drouin, 1981, adapted by Beltaos, 2019). Blue line depicts solid ice cover in flat portion of reach, downstream of rapids section.....	166
Figure 6-6 Schematic Illustration of a river ice jam and associated backwater level (Source: Rokaya et al., 2022).....	171
Figure 6-7 Example peak ice-influenced water levels plotted versus associated discharge, as recorded at the Water Survey of Canada hydrometric station on the Restigouche River near Rafting Ground Brook, Canada, 1972-1988. (Source: Beltaos, 2021).....	173
Figure 6-8 Schematic Illustration of an Equilibrium Ice Jam (Source: USACE, 2011).	174
Figure 6-9 Flood map showing open water flood zones and the extent of a past ice-jam flood event (highlighted in green) along the Des Hurons River, a small stream draining a catchment of approximately 75 km ² in Stoneham-Tewksbury, north of Quebec City,	

Canada. Streamflow direction is toward the bottom of the image (adapted from Turcotte et al., 2017).	176
Figure 6-10 Water elevation affected by ice jam on April 27-29, 1979 and elevations of ice scars caused by this event (Source: Gerard, 1981).....	180
Figure 6-11 Ice scar record with assigned ‘perception levels’ for three separate time periods, Peace River near Fort Vermillion, AB. (Source: Modified from Gerard, 1989).	181
Figure 6-12. Summary of non-dimensional discharge (ξ) versus non-dimensional ice-jam stage (water depth) (η) established from field data from breakup case study relationships for wide-channel equilibrium jams, including confidence bands. Source (Beltaos, 2013; after Beltaos, 1995b with edits)	189
Figure 6-13 Dimensionless discharge (ξ) and dimensionless ice jam stage (η) (water depth) relationship for breakup ice jams (Source: Burrell et al., 2015, after Beltaos (1983) with changes).....	190
Figure 6-14 Equilibrium ice jam of thickness t along a river reach of slope S and width W for a discharge Q with an overall backwater staging depth due to an ice jam H and depth of flow (h) under the ice jam (Source Lindenschmidt et al. (2025))......	192
Figure 6-15 Example cumulative probability distribution for ice-influence (open square) and open-water (open circle) annual peak stages, where stages above the intersection of the two lines are due to ice jams (Source: Beltaos, 2021, adapted from Gerard and Karpuk, 1979).....	197
Figure 6-16 Illustration of various perception stages and their utility in assigning suitable frequencies to stages reported by different data sources (adapted from Gerard and Karpuk, after White and Beltaos, 2008).....	200
Figure 6-17 Chronology of historical ice related flood levels summarised on a chronological chart producing a flood summary for Attawapiskat, Ontario (Source: Kriwoken and Brown, 1988)	201
Figure 6-18 Stage-frequency curves for open-water and ice-covered peaks showing range of sources of data for various data points (Source: Gerard and Karpuk, 1979). A cleaner, revised version of Figure 6-18 without multiple lines or the depiction of various separate data sources as points, is shown in Figure 6-16.....	202

Figure 6-19 Schematic illustration of discharge-dependent upper and lower envelopes of peak ice jam breakup stages, depicting conceptual rating curves for open water, sheet ice cover and an equilibrium ice jam scenarios; where QJ and QNJ, relate to 'jam' and 'no-jam'.	208
Figure 6-20 Comparison of synthesised and measured ice-related and open water stage probability distributions (Source: after Gerard and Calkins (1984).	211
Figure 6-21 Ice affect on a portion of the stage discharge curve taking into account non-jam conditions at low and high flows (Source: Tuthill et al., 1996; with changes, from Beltaos, 2010)	213
Figure 6-22 Plot of peak breakup stage (H_m) versus breakup flow (Q_m), along with upper and lower ice related envelopes, for Peace River at Peace Point, Partially based on Water Survey of Canada archived data (Beltaos, 2011).	215
Figure 6-23 Probability distribution of breakup stage, for WSC gauge 01BJ007, Restigouche River above Grounding Brook, NB, Canada (Source: Beltaos, 2021, adapted from Beltaos, 2012).	219
Figure 6-24 1D model of a hanging dam configuration (shown in burgundy) in the lower section of the Kaministiquia River in northwestern Ontario (KGS, 2007).	222
Figure 6-25 Annotated Illustration of instantaneous stage with time during ice breakup, where H_m is equal to maximum breakup stage and ΔH_m is the difference between the actual stage and the effective stage (i.e., ice effect on stage) (After Beltaos, 1990).	228
Figure 6-26 Decision tree for probability of seasonal ice affected flooding occurrence given future changes in air temperature and streamflow. For information purposes only (Source: Rokaya et al., 2022).	234

List of Tables

Table 1-1 Design Return Period in Years (T) For Different Design Life (L) and Risk Values	5
Table 1-2 Alternative Methods of Flood and Water Level Calculations	16
Table 1-3 Computation of Different Criteria Floods	17
Table 3-1 Comparison of Historical Storms used for Flood Hazard Computation	64
Table 3-2 Comparison of Return Period Storms used for Flood Hazard Computation ..	65
Table 3-3 Hurricane Hazel, 36th through to 48th Hour Rainfall Depths	68
<i>Table 3-4 Hurricane Hazel - Areal Reduction</i>	70
Table 3-5 Timmins Storm - Rainfall Depths.....	72
Table 3-6 Timmins Storm – Areal Reduction.....	74
Table 3-7 Rainfall Distributions – Percent (%) distributions per hour	86
Table 3-8 Alternative Rainfall Distributions.....	88
Table 3-9 Meteorological Service of Canada - Snowmelt Equations.....	91
Table 3-10 Calibration performance metrics	113
Table 6-1 Lower and upper limits of parameters that discriminate streams and smaller rivers from larger rivers (after Turcotte et al., 2017a)	168
Table 6-2 Typical ranges of f_o , f_i/f_o and μ for different riverbed slopes, compiled by Beltaos (1993a) and Lindenschmidt (2024) as reported in Lindenschmidt et al. (2024; 2025).	192

List of Acronyms

A-D	Anderson-Darling test
ADCP	Acoustic Doppler Current Profilers
AEP	Annual Exceedance Probability
AES	Atmospheric Environmental Services
AIC	Akaike Information Criterion
AMC	Antecedent Moisture Condition
BIC	Bayesian Information Criterion
CA	Conservation Authority
CN	Curve Number
DEM	Digital Elevation Model
DFM	Distributed-function Method
DTM	Digital Terrain Model
ECCC	Environment and Climate Change Canada
EV1	Extreme Value Type 1
FDC	Flow Duration Curve
FDRP	(Canada-Ontario) Flood Damage Reduction Program
FFA	Flood Frequency Analysis
GEV	Generalized Extreme Value
GIS	Geographic Information System
GNSS	Global Navigation Satellite System
HEC	Hydrologic Engineering Centre

HEC-HMS	Hydrologic Engineering Centre – Hydrologic Modeling System
IA	Initial Abstraction
IDF	Intensity Duration Frequency
K-S	Kolmogorov-Smirnov test
LiDAR	Light Detection and Ranging
LMOM	L-moments
LN	Lognorma
LN3	Three Parameter Lognormal
LP3	Log Pearson Type 3
MDRA	Multi gauging station Drainage Area Ratio
MECP	Ministry of the Environment, Conservation and Parks
MIFM	Modified Index Flood Method
MNR	Ministry of Natural Resources
MSC	Meteorological Services Canada
MTO	Ministry of Transportation Ontario
NOAA	National Oceanic and Atmospheric Administration (U.S.)
NRCS	Natural Resources Conservation Service (U.S.)
NWS	National Weather Service (U.S.)
OFCF	Offline Flood Control Facility
PPS	Provincial Planning Statement
PUB	Prediction in Ungauged Basins
SCS	Soil Conservation Service (U.S.)

SOLRIS	Southern Ontario Land Resources Information System
SWM	Stormwater Management
SWMF	Stormwater Management Facility
UOFM	Unified Ontario Flood Method
USACE	U.S. Army Corps of Engineers
USACE-HEC	U.S. Army Corps of Engineers - Hydrologic Engineering Centre
USGS	U.S. Geological Survey
WISKI	Water Information System Klsters
WMO	World Meteorological Organization
WSC	Water Survey of Canada
WWTP	Wastewater Treatment Plant

Explanatory Note

This document supersedes and replaces the primary chapters of the *Technical Guide – River & Stream Systems: Flooding Hazard Limit* (2002).

Chapter B, Section 4. Special Flood Hazard Conditions, of the 2002 Technical Guide is replaced by the *Technical Bulletin – Special Flooding Conditions in River and Stream Systems* [MNR, 2026(Draft)].

Appendix 3, 4, 5 (Part A) and 6, from the 2002 *Technical Guide* remain the current guidance for these topics until at which time they are replaced.

Information herein is organized and updated to ensure the currency, representativeness applicability and suitability of the methods presented.

1 Introduction

This Technical Guide is part of a guidance series which supports flood hazard identification and management activities in Ontario.

Guidance provided in this document is specific to the flood related inundation of areas adjacent to river and stream and small inland lake systems. Flood-related guidance specific to Large Inland Lakes, or the Great Lakes St. Lawrence River System is provided in other MNR technical guidance.

This document is reflective of the recommended best practices in the field of hydrology and hydraulics at the time of its release. This guidance, developed by the Ministry of Natural Resources (MNR) will be amended from time to time to ensure it remains current with best practices and procedures in the field of hydrology and hydraulics and to ensure alignment with provincial policies concerning flood hazard identification and management.

In the process of identifying and managing flooding hazards, there may be a need to review additional Technical Guides and supporting Technical Bulletins created by MNR.

1.1 Background

Ontario's approach to managing risks associated with flooding, erosion and other natural hazards is based on the five pillars of emergency management: prevention, mitigation, preparedness, response, and recovery. The objectives of this approach are to reduce risks to public health and safety, reduce public costs, build community resilience, maintain long-term prosperity, reduce the social disruption associated with emergencies, and help assure the continuance of critical infrastructure.

The province uses a combination of legislation, regulations, policies, and guidance to achieve these objectives. Two main tools used to manage risks posed by natural hazards and achieve the stated objectives above are the Provincial Planning Statement (PPS) issued under the *Planning Act (1990)*, and natural hazard-related regulations issued under the *Conservation Authorities Act (1990)*.

1.2 Document Purpose and Scope

The primary purpose of this Technical Guide is to assist in the implementation of the natural hazard policies outlined in the PPS. To promote consistency across the province, this Technical Guide should also be used by conservation authorities (CAs) to support identifying and managing development within areas over which they have jurisdiction under the *Conservation Authorities Act (1990)*.

Policy 5.2.1 of the PPS states that planning authorities shall, in collaboration with conservation authorities where they exist, identify hazardous lands and hazardous sites and manage development in these areas, in accordance with provincial guidance.

As stated within the PPS, provincial guidance, including guidance material, guidelines and technical criteria may be issued from time to time to assist planning authorities and decision-makers with implementing the policies of the PPS. The information, technical criteria and approaches outlined in provincial guidance are meant to support implementation but do not add to or detract from the policies within the PPS. If there is a conflict between the guidance material and the PPS, the PPS would prevail. While not specifically referenced within the *Conservation Authorities Act* or associated regulations, provincial guidance can also support conservation authorities in implementing regulatory requirements and mandatory programs and services prescribed by the *Conservation Authorities Act (1990)*. If there is a conflict between provincial guidance and the *Conservation Authorities Act (1990)* (including associated regulations), the *Conservation Authorities Act (1990)* would prevail.

This Technical Guide presents provincial guidance on the hydrologic and hydraulic studies needed to conduct riverine flood hazard analyses, in support of identification of the flooding hazard, through such tools as flood hazard modelling and mapping. This Technical Guide should be read in conjunction with the PPS as well as other MNR flood related technical bulletins, as applicable.

Flood hazard studies should be undertaken by a certified Professional Engineer. The resulting flooding hazard limit (i.e., flood lines) and floodproofing standards should be reviewed and approved by a Professional Engineer using accepted engineering principles and, in a manner, consistent with methods, procedures and technical policies outlined within this Technical Guide.

This Technical Guide is not intended to be a list of mandatory instructions on technical methodologies to be rigidly applied in all circumstances or to replace best professional

engineering judgement when undertaking site-specific flood hazard analyses. Rather it serves to assist technical staff experienced in water resources in the selection of the most appropriate computational methods and flexible implementation measures, provided the decisions made are consistent with the latest PPS.

The need to apply professional engineering judgement when undertaking site-specific flood hazard analysis should not be misconstrued as authority to disregard the methods, procedures and technical policies outlined within this Technical Guide. Planning authorities and decision-makers are expected to demonstrate that this Technical Guide was reviewed, considered, and used to inform their decisions. Any departures from these guidelines should be noted within the resulting technical report and supported by clear documentation demonstrating that reasonable grounds for the departure exist. The PPS natural hazard policies represent minimum standards. Within the framework of the provincial policy-led planning system, planning authorities and decision-makers may go beyond these minimum standards to address matters of importance to a specific community, unless doing so would conflict with the PPS. Similarly, there may be circumstances where local conditions dictate that the methodologies outlined within this Technical Guide may not be sufficient, and adopting a higher standard of care may be more appropriate.

This guidance, developed by the MNR, will be amended from time to time to ensure it remains current with practices and procedures in the field of hydrology and hydraulics as well as to ensure alignment with PPS policies. Unless explicitly stated otherwise, terms in this document have the same meaning as in the PPS.

This document consists of six parts.

[Section 1](#) introduces the general approach to delineating the flooding hazard in Ontario, and provides a brief description on data requirements, flood flow calculations and water level analysis, including a description of the general hydrologic and hydraulic procedures recommended for flood hazard delineation in Ontario.

[Section 2](#) details the data, approaches and procedures that support hydrologic and hydraulic analyses for modelling flood hazard mapping.

[Section 3](#) discusses procedures for hydrologic analysis needed to quantifying flood magnitudes, including flood frequency analysis and runoff simulation.

[Section 4](#) covers procedures for hydraulic analysis including model selection, considerations, and data requirements to support water level computation.

[Section 5](#) presents water level computations. It outlines and compares hydraulic analysis and modeling techniques to estimate water surface elevations under the flooding hazard, in support of flood hazard delineation.

[Section 6](#) focuses on ice-induced flooding. It outlines ice jam processes and procedures as well as their effects in hydraulic modelling; and discusses monitoring and data sources for ice jam flood hazard assessment and delineation.

1.3 Flooding Hazard Standards to support flood hazard identification in River Systems

In Ontario, a flooding hazard is defined in the PPS as the inundation, under the conditions specified below, of areas adjacent to a shoreline or a river or stream system and not ordinarily covered by water, which along a river, stream and small inland lake system, is the greater of:

- 1) the flood resulting from the rainfall actually experienced during a major storm such as the Hurricane Hazel storm (1954) or the Timmins storm (1961), transposed over a specific watershed and combined with the local conditions, where evidence suggests that the storm event could have potentially occurred over watersheds in the general area;
- 2) the one hundred year flood, or;
- 3) a flood which is greater than 1) or 2) which was actually experienced in a particular watershed or portion thereof, which has been approved for that specific area by the Minister of Natural Resources.

These flooding hazard standards are respectively outlined further in [Section 1.3.1](#), [1.3.2](#) and [1.3.3](#), and depicted in Figure 1-1 Three Flooding Hazard Criteria Zones of Ontario.. The criteria used to define limits of flooding hazards across Ontario are described in the PPS. Similar criteria may also be outlined in CA regulations issued under Section 28 of the *Conservation Authorities Act (1990)*.

A brief description is provided on data requirements, flood flow calculations and water level analysis. At a fundamental level, the criteria refer to floods based on probability (i.e., 100 year or 1% Annual Exceedance Probability (AEP)) or in relation to a specified meteorological event, whichever produces the greatest flood magnitude.

Further details regarding common data and methods to support hydrological and hydraulic methods are presented in [Section 2](#). Further detailed descriptions on specific

methods for computing flood flows and water levels in Ontario are given in [Sections 3 and 4](#), respectively.

1.3.1 Flood Resulting from Rainfall Actually Experienced During a Major Storm Event

These storms are used where evidence suggests that the storm event has or could have potentially occurred over watersheds in the general area of interest.

Examples of a specified meteorological event include the Hurricane Hazel storm (1954), and the Timmins storm (1961), transposed over a specific watershed and combined with local conditions, where evidence suggests that the storm event could have potentially occurred over watersheds in the general area

1.3.2 Flood Based on Exceedance Probability of Occurrence

A flood based on probability must be determined by conducting a frequency analysis of either recorded flood peaks, simulated peak flows generated by continuous models, or, where such information is inadequate, through a regional flood frequency analysis. The analysis should produce the best estimate for the required exceedance probability of occurrence. At a minimum, this includes the 100-year flood, or other larger return period flow as approved by the Minister of Natural Resources.

Although the return period has historically been used as a risk criterion, it can have disadvantages unless it is translated in terms of other informing criteria. Given a particular return period or AEP, it is not particularly clear what level of risk is being accepted in a specified engineering approach or operation. To provide a more complete picture, three components are required to describe the design criterion, specifically in this case, design return periods:

- 1) Design return period for flood (T)
- 2) Design life (L)
- 3) Risk of being equalled during design life (r)

These three are related by the equation:

$$r = 1 - \left(1 + \frac{1}{T}\right)^L \quad (1.1)$$

[Table 1-1](#) outlines the magnitude of risk for which the 100-year flood could be exceeded within a 50 or 100-year design life. A 100-year return period would include a 39% risk of being equalled or exceeded over a 50-year period, and a 64% risk over a 100-year design life/duration.

Table 1-1 Design Return Period in Years (T) For Different Design Life (L) and Risk Values

Risk (r)	Design Return Period for 50-year Design Life	Design Return Period for 100-year Design Life
1%	4977	9953
10%	475	950
22%	200	400
25%	174	348
39%	100	200
50%	73	145
64%	50	100

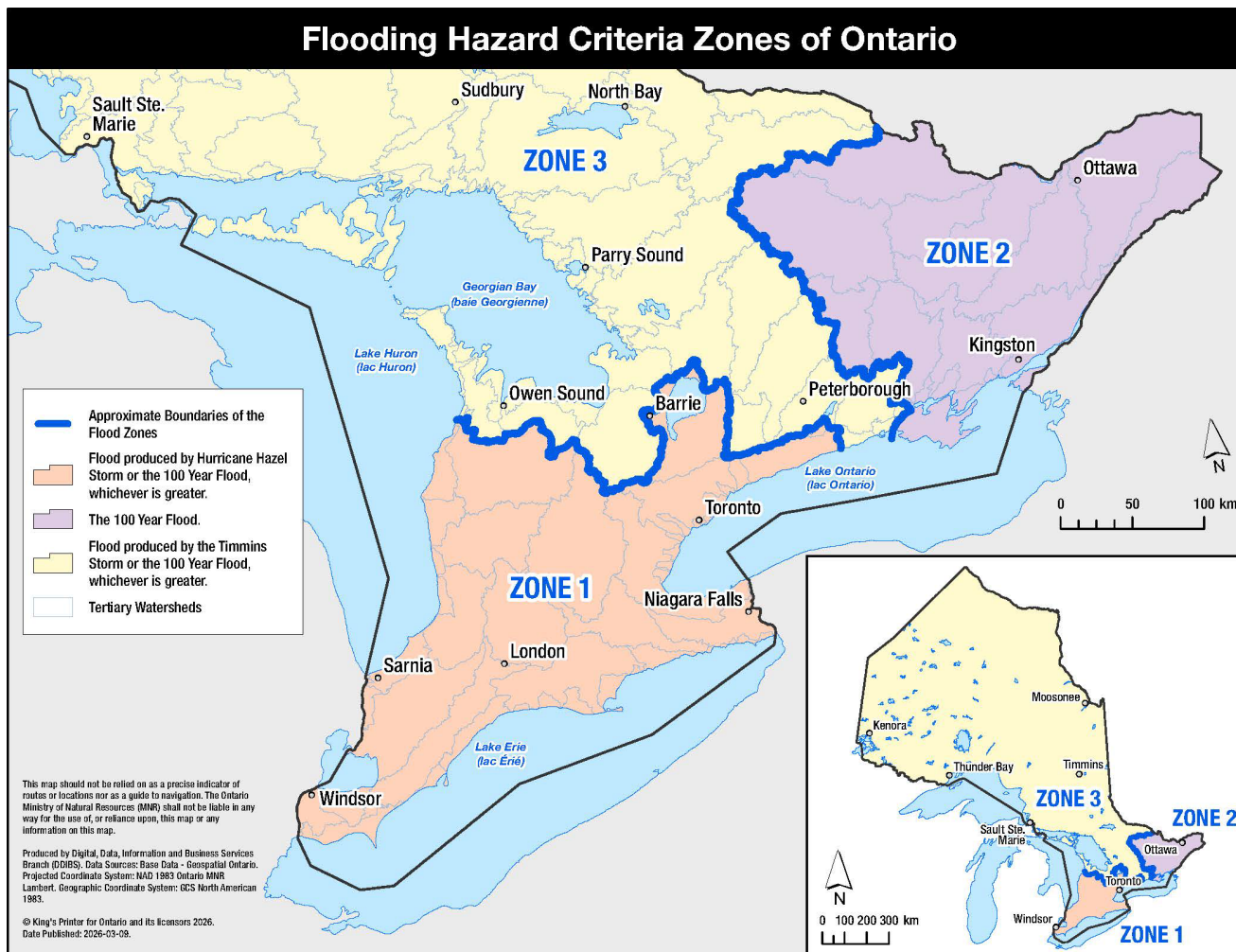


Figure 1-1 Three Flooding Hazard Criteria Zones of Ontario

1.3.3 An Observed Flood of Greater Magnitude than the Storm Criteria Outlined in Section 1.3.1 and 1.3.2 and Actually Experienced in a Particular Watershed

Observed floods greater than the 100-year flood level may be used to define the flooding hazard limit. Observed floods may be the result of precipitation, snowmelt, ice jams, other such causes, or any combination thereof. Observed floods must meet acceptable documentation requirements and be approved by the Minister of Natural Resources.

Independent of the flooding hazard standard, previously observed floods (e.g., magnitude of discharge, water levels and extent of inundation) are useful in illustrating to the public, the severity and extent of the defined flooding hazard. The limits of an observed flood may be chosen and included in public mapping products for comparison purposes, to highlight the relative difference to the extent of flooding under the flooding hazard limit.

1.4 Flooding Hazard Limits for Lakes Less than One-Hundred Square Kilometres

The basic principles of defining flooding hazards along lakes are similar to the principles adopted for rivers. There are however additional factors that need to be considered, such as the effect of wind, when selecting flooding hazard standards for lake levels.

Smaller lakes, with a surface area less than 100 km² may or may not respond to a single runoff event. Similarly, riverine lakes and large rivers (e.g., Ottawa River) which due to their size and fetch (>3 km), may be subject to wind set up, generating water levels higher than the flooding hazard standard selected for the river.

The standard for defining the flooding hazard limit along small lakes and large rivers is the same as that used for rivers (i.e., Hazel centred, Timmins centred, maximum observed or 100-year flood, whichever is applicable), plus an appropriate allowance for wind setup. The wind set up is to be calculated based on the mean wind speed and applicable maximum effective fetch. The computed wind set up will then apply along the entire lakeshore. Where the maximum fetch is less than 3 km, the lake can be treated as an integral component of the river system, therefore the flooding hazard standard will be the same as that applied to the river system.

1.5 Municipal Official Plan Flooding Hazard Limit Policies

Consistent with policies in the PPS, municipalities, in collaboration with CAs where they have been established, shall identify hazardous lands that could be unsafe for development due to naturally occurring processes. With regards to flooding along river, stream and small inland lake systems, this means the land, including that covered by water, to the furthest landward limit of the flooding hazard. These lands shall be identified in their Official Plans and policies be incorporated to address new development consistent with the PPS. Municipalities and planning boards, in consultation with the local CA or the Ministry of Natural Resources, as required, where no CA has been established, should include in their official plans:

- a) Policies whereby uses permitted in the flooding hazard are cognizant of flood susceptibility and flood risk;
- b) Policies whereby development will be directed away from areas where there is an unacceptable risk to public health or safety or of property damage and will not create new or aggravate existing hazards;
- c) Policies addressing additions or alterations to existing buildings or structures and replacement of buildings or structures located in the flooding hazard; and
- d) Policies addressing such public and private works that must be located in flood plains by nature of their use.

Municipalities and planning boards should identify in their official plans, the planning controls required to give effect to the flooding hazard policies of the province. Zoning and development permit by-laws are also important for the implementation of the PPS. Where no official plan exists, the zoning document affecting the area should contain provisions to reflect the PPS.

1.6 Flooding Hazard Limit – Zonation Concepts

Generally, the flooding hazard limit will consist of one zone, defined by the selected flooding hazard standard (). New development within the flooding hazard limit is to be prohibited or restricted. Where the one-zone concept is applied, the following applies:

- Municipalities and planning boards should include policies in their official plans that explain the intent of the one-zone concept.

- The flooding hazard limit should be appropriately zoned in conformity with the official plan designation to reflect its prohibitive or restrictive use, and;
- The entire flooding hazard limit should be treated as the floodway.

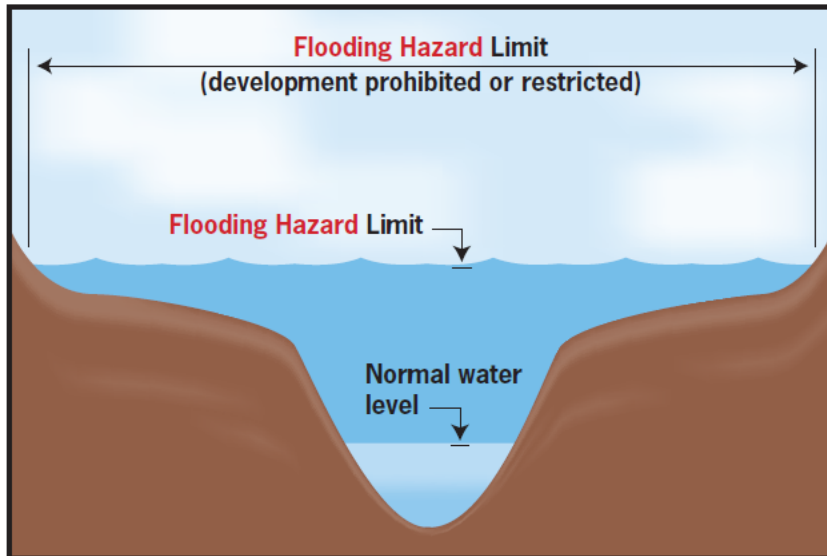


Figure 1-2 One-Zone Flooding Hazard Limit Concept (not to scale)

The two-zone concept recognizes that the flooding hazard limit can conceptually and safely be divided into two zones. This includes the floodway, where most of the flow is conveyed and the flood fringe that exists on either side of the floodway.

Where the two-zone concept is applied, the floodway is the contiguous inner portion of the flood plain, representing the area required for safe passage of flood flow and/or that area where flood depth/velocities are considered to be such that they pose a potential threat to life and/or property damage.

The floodway for a river system is the portion of the flooding hazard limit where development and site alteration would cause a danger to public health and safety or property damage. Uses which by their nature must be located within the floodway, such as flood and/or erosion control works, or where appropriate, minor additions or passive, non-structural uses which do not affect flood conveyance and flows may be permitted. New development in the floodway is to be prohibited or restricted.

For outer portions of the flooding hazard limit that could potentially be safely developed with no adverse impacts, municipalities, in cooperation with conservation authorities where they have been established, have the option of selective application of the two-zone (floodway- flood fringe) concept. The extent of the floodway is to be

determined based on local watershed conditions, such as critical flood depth and velocity, existing and proposed development and the potential for upstream and/or downstream impacts. Generally, flow depths exceeding 1 metre and/or flow velocities above 1 m/s can create significant hazards for development.

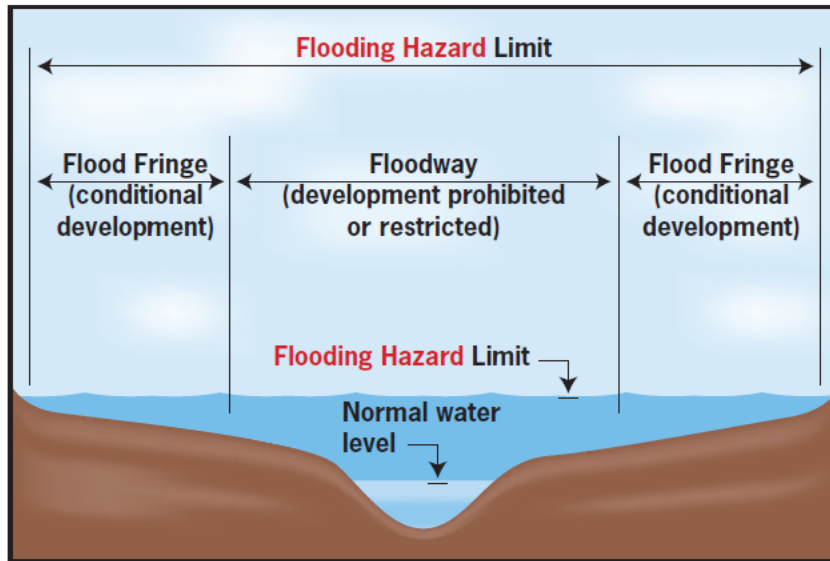


Figure 1-3 Two Zone Floodway-Flood Fringe Concept

The application of these critical flood depth and velocity values recognizes that the impact of encroachment caused by filling and/or development in the flood fringe can result in:

- Increases in upstream flood elevations;
- Increases in downstream flows;
- Increases in downstream velocities; and/or
- Changes to the timing of flows

The use of the hydraulic criterion, with consideration of upstream and downstream impacts on the existing and proposed developments, allows hydraulic modelling, discussed further in [Section 4 Hydraulic Modelling](#) and [Section 5 Water Level Computations – Open Water Hydraulic Modelling](#), to be used to better quantify and understand changes in flood levels, flows and velocities caused by various degrees of flood fringe encroachment. This includes levels of encroachment under the flooding hazard standard, as well as lesser floods. The final selection is based on the proposed development scenario with the least upstream and downstream impacts.

Pending the outcome of the above analyses, development in flood fringe areas may be permitted subject to certain established standards and procedures, including but not limited to conditions that exist under the flooding hazard standard, such as the depth of water, velocity of flow, a combined depth and velocity metric as well as the provision of vehicle access and structural integrity. Factors to be considered and the application of procedures for a two-zone designation are presented in Appendix 4 “Application of the Two-Zone Concept – Factors to be Considered” of MNR (2002), with hazard to life and access and egress criteria presented in Appendix 6 “Floodproofing” of MNR (2002). Any new development that may be permitted in the flood fringe should be protected to the level of the flooding hazard standard.

Where the two-zone concept is proposed to be applied or is considered to be a plausible option, municipalities should include policies in their official plans that explain the intent of the two-zone concept and the potential developability of the flood fringe versus the flood way. Where the two-zone concept is applied, the flood fringe should be zoned in conformity with the municipal official plan designation, and the flood hazards and requirements for floodproofing be recognized in the zoning document. The floodway should be appropriately zoned to reflect its prohibitive or restrictive use.

1.7 Special Policy Area Concept

Special Policy Area means an area within a community that has historically existed in the flood plain and where site-specific policies, approved by both the Ministers of Natural Resources and Municipal Affairs and Housing, are intended to provide for the continued viability of existing uses (which are generally on a small scale) and address the significant social and economic hardships to the community that would result from strict adherence to provincial policies concerning development. A Special Policy Area is not intended to allow for new or intensified development and site alteration, if a community has feasible opportunities for development outside the flood plain. The criteria for designation and procedures for approval are established by the Province.

Where strict adherence to one and two zone policies is not feasible, the concept of Special Policy Area status has been recognized as a possible option for flood prone communities or portions thereof. Municipalities have historically been able to apply for Special Policy Area status, in accordance with established procedures, and controlled development may be permitted once such status is obtained.

If approved, municipalities should delineate Special Policy Areas in their official plans and include policies indicating the circumstances under which new development may be

permitted and identifying the minimum acceptable level of protection required for new development.

Factors to be considered for a Special Policy Area designation and the procedures for approval are addressed in Part A of Appendix 5, in MNR (2002) and MNR (2009), respectively.

1.8 Access and Floodproofing

Any new development permitted in the flood plain, in accordance with the PPS, should be protected by acceptable floodproofing actions or measures. However, the new development must not include institutional uses or essential emergency services (e.g., police, fire) or disposal, manufacture, treatment or storage of hazardous substances. Where such uses are pre-existing, a higher floodproofing standard should be considered. Ingress and egress for new buildings should be such that vehicular and pedestrian movement is not prevented during times of flooding. The potential for hazard to life within the flooding hazard limit is the highest during storm events. Access involving both ingress and egress should be addressed prior to approval of developments near flooding hazard limits. Ingress and egress to and from the development should be such that movement of people and vehicles is not prevented during conditions that exist under the flooding hazard (i.e., depth, velocity and combined depth velocity factors). The highest priorities for access to emergency vehicles should be given to police, ambulance and fire services, especially where evacuation is a distinct possibility in areas surrounded by flooding. All local agencies involved in local emergencies should be consulted regarding the adequacy of access.

The Ontario Building Code applies to all new buildings in the Province. It is administered by the Ministry of Municipal Affairs and Housing and is implemented by the Chief Building Officials appointed by local municipalities.

Any floodproofing measure must conform with Part 4, Structural Design, which describes types and designs of construction materials and the design requirements to minimize the hazards caused by a potential structural failure. The Code indicates where dynamic loading may apply, what the allowable loads or bearing pressures are and where hydrostatic uplift applies. Also, the floodproofing must conform with Part 9, Housing and Small Buildings, which describes the detailed requirements for the construction of houses, including details on waterproofing, as well as surface and subsurface drainage. Part 9

also includes a specific section addressing buildings in flood plains, including reference to hydrostatic pressures and preserving the integrity of exits during floods.

More detailed descriptions of floodproofing and access metrics are presented in Appendix 6 of MNR (2002).

1.9 Public Safety

Notwithstanding the above policies, the PPS prohibits development and site alteration in the flooding hazard where the use is:

- Associated with institutional services, such as hospitals, long-term care homes, retirement homes and schools.
- Associated with essential emergency services such as those provided by fire, police and ambulance stations and electrical sub-stations.
- Associated with the disposal, manufacture, treatment or storage of hazardous substances.

1.10 Impacts of a Changing Climate

The PPS requires that planning authorities prepare for the impacts of a changing climate that may increase the risks associated with natural hazards. While there is uncertainty associated with what may be observed with the impacts of climate change on flooding hazards, it is important that planning authorities have awareness of areas identified to be potentially at greater risk under future climate scenarios.

This Technical Guide does not provide guidance on how planning authorities should conduct climate change analysis. It is however recommended that in addition to conducting analyses to support identifying flooding hazards, additional analyses be undertaken to estimate the flooding hazard under a future climate scenario. A future climate scenario would typically involve consideration of a rainfall amount greater than that of the designated storm event for the watersheds being modelled in order to estimate the extent of the climate informed flooding hazard.

The results of this analysis can be used to inform the potential for increased risk throughout the jurisdiction under a changing climate. Planning authorities can use this information to determine if and how to mitigate the increased risk, as appropriate, when conducting land use planning activities or making permitting decisions.

1.11 Overview of flood magnitude and water level computation methods

Hurricane Hazel, the Timmins storm, the 100-year flood criteria or other approved flooding hazard standard require multi-stage computations to determine the water levels and the extent of the flooding hazard, as shown below. Where the criteria are based on an observed flood, these computations are not required.

The first stage involves the computation of the magnitude of flood standard to be used for delineating the flooding hazard. The second stage involves the computation of the corresponding flood level, and the final stage in the process of defining the flooding hazard in a watershed involves the delineation of flood lines on maps. [Table 1-2](#) summarizes the alternative methods available to compute floods and water levels.

Hurricane Hazel, Timmins or 100-year floods can be calculated from flow data or from meteorological input, by using hydrological models or statistical calculations. The corresponding water levels can be computed from the floods by hydraulic models. The only exception is the rare occasion when the 100-year water level to be determined coincides with a location where long-term flood level data are available. In such a case, if the future watershed conditions will be similar to the conditions reflected in the historical data, a frequency analysis of water levels could establish the 100-year water level, at the location where the data were acquired. A hydraulic model is required to determine water surface elevations under the flooding hazard, for all other situations ([Table 1-2](#)).

The seven different alternative methods listed under the '100-year flood' in [Table 1-2](#) can also be used for computing various return period floods (in addition to the 100-year flood), that may be required where MNR Minister Approval has been granted to apply a flooding hazard greater than the 100-year flood and less than Hurricane Hazel or the Timmins storm.

Numerous flow computation procedures were reviewed to identify their appropriateness for Ontario conditions. Three criteria were used for the selection of flow estimating procedures:

- i) Accuracy was based on the comparison of flood frequency estimates obtained from the selected procedures, with flood frequency estimates determined from suitable streamflow records at selected test sites.
- ii) Consistency identified the ease of reproducing similar results by different users at the site based on the same procedures.

- iii) Cost-effectiveness was an indication of effort required to compute flows compared to the study requirements.

Generally, floods and corresponding flood lines can be calculated from 1) peak flows or from 2) flood hydrographs. Each of these two approaches has several alternative methods available to compute the flood. [Table 1-3](#) lists the methods described in this document.

As outlined in [Table 1-2](#), while a range of approaches exist for flood calculations (e.g., flow simulation, modelling), fewer endorsed approaches are available for water level computations.

Table 1-2 Alternative Methods of Flood and Water Level Calculations

Criteria	Data Used for Flow Calculation	Flow Simulation Model	Computed Flood	Water Level Computation	Flood Level
Hurricane Hazel	Hurricane Hazel Storm	Rainfall-runoff	Hurricane Hazel flood	Hydraulic model	Hurricane Hazel flood level
Timmins storm	Timmins Storm	Rainfall-runoff	Timmins storm flood	Hydraulic model	Timmins storm flood level
Observed Flood	Recorded storm flow or level	Rainfall/snowmelt-runoff or frequency analysis	Observed flood	Hydraulic model	Observed flood level
100 Year Flood	Single site flow records	Single station frequency analysis	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	Regional flow records	Regional frequency analysis	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	Combined single site and regional flow records	Frequency analysis	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	Series of historical storms	Rainfall/snowmelt-runoff or frequency analysis	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	Continuous meteorological (precipitation, temperature) records	Rainfall/snowmelt-runoff or frequency analysis	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	100-year design storm	Rainfall/snowmelt - runoff	100-year flood	Hydraulic model	100-year flood level
100 Year Flood	Water level records	N/A	N/A	Frequency analysis of levels	100-year flood level

Table 1-3 Computation of Different Criteria Floods

Peak Flood Flows: Method for Defining Flood Peak	Peak Flood Flows: Alternative Methods for Data Checking	Hydrograph: Single Event Storm	Hydrograph: Series of Storms
- Single station frequency analysis	- Index Flood method	- Storm simulation, frequency of flow assumed to be same as frequency of storm	- Series of large storms or continuous precipitation simulation, followed by frequency analysis of annual maximum flows
- Regional flood frequency analysis	1) New index flood study or method		
1) New regional frequency study	2) Modified Index Flood Method (MTO, 1996)		
2) Index Flood (Moin & Shaw, 1985)			
3) Regression Equation (Moin & Shaw, 1986)	- Regional regression equations		
4) Dimensionless Flood Frequency (Cumming Cockburn Ltd., 2000)	1) Northern Ontario Method (MTO, 1986)		
5) UOFM (MTO, 2016b)	2) Multiple Regression Method (Cumming Cockburn Ltd., 2000)		
- Combined Individual and Regional Analysis	3) Regression Equation Method (Cumming Cockburn Ltd., 1986)		
1) New Combined individual and Regional Analysis			
2) FDC Spatial Interpolation Approach			

Note: Hydrograph methods are recommended if:

- *Watershed is outside the range of regional regression applicability;*
- *Watershed has different land uses than the regression equations which were based on rural land use conditions;*
- *Streamflow in watershed is regulated (i.e., flow regime altered by dams and other water management structures);*
- *Hurricane Hazel Storm or Timmins Storm flood computation is required; and/or*
- *Remedial measures or watershed requirements warrant it.*

Before the appropriate method of computation is selected, the engineer should establish the need to compute a peak flow or a complete hydrograph. The following list of criteria as well as the information presented in [Table 1-3](#) will assist in the selection process.

Hydrograph simulation is required normally where:

- i) Watersheds for which the required parameters of the regional frequency relationships fall outside the range of applicability (i.e., for small or urban watersheds; e.g., $\leq 50 \text{ km}^2$);
- ii) Watersheds which have undergone or are expected to undergo land use changes which make the regional frequency relationships developed from a database of rural watersheds inapplicable. This will generally be as a result of urbanization but may also be related to major changes in agricultural practices;
- iii) Watersheds which are subject to significant human-influenced, artificial regulation effects due to manipulation of storage or flow within the drainage area;
- iv) Watersheds for which the criteria remain the Hurricane Hazel or Timmins storm as currently defined; and
- v) Where different watershed development/management options are to be tested.

Peak flow calculations are relatively easy to perform where long-term reliable streamflow records are available which reflect the present and future land use conditions. Unfortunately, such individual station frequency analysis results would only be applicable at the gauging station locations. Otherwise, where no flow data is available, regional data can be used to compute peak flows. A combination of the single station and regional flow analysis is recommended where only short-term data is available. Where the single station data include less than a 10-year period of record, the records should not be used in the frequency analysis. A summary of suggested single station data and regional frequency analysis for (primary) instantaneous and (secondary) mean daily peak flow computation methods, categorized by period of record streamflow, is shown in [Figure 1-4](#). The use of flood frequency analysis methods is further detailed in [Section 3.1](#).

Where the practitioner has the choice of computing the flows by hydrograph and the peak flows methods, the computer model results should be confirmed/checked by comparing the results with flows obtained from other endorsed methods, including those highlighted in [Table 1-3](#) and detailed in [Section 3.1](#). Normally the results should be within one standard error (i.e., standard deviation). When computing the peak flow using these methods for checking results, the outcome may identify a wide range of flows. Particular attention should be given to those situations where parameters of regional frequency analysis/regression equations fall near the limits of range and applicability. The accuracy and utility of these methods can suffer considerably in these situations, and their use should be avoided in such circumstances.

Hydrograph simulation through modelling should be carried out to compare the results obtained using regional frequency analysis/regression equations. The practitioner should use the standard error computation and a considerable amount of professional judgement and consideration prior to selecting the return period peak design flow. Results from a single station flood frequency analysis may be used to calibrate the model.

1.11.1 Methods to Estimate and Check Return Period Flood Flows where no streamflow gauging station data are available

Where no gauging station data are available for analysis to support a single site flood frequency analysis, a regional frequency analysis should be undertaken to determine the peak flow. The default approach should always be a new analysis, applying the most current hydrometric data and frequency analysis methods to estimate the flood peak. Where it is deemed unfeasible to undertake an updated analysis to define these regional analyses using up-to-date data, specific regional frequency analyses may be used to derive an estimated peak flow.

A variety of suitable options exist for estimating peak flows. These methods are listed in order of priority in [Table 1-3](#). These can be differentiated to include:

- (1) Primary methods for the purpose of defining the magnitude of the flooding hazard.
- (2) Secondary methods that are suitable for the purposes of checking recently or previously computed peak flow estimates for a particular site or catchment including to compare and contrast the outputs with the primary methods.

It must be emphasized that all methods have inherent limitations that must be accounted for in their application. Furthermore, these methods should not be used in or use data from urban watersheds, nor should they be used for Hurricane Hazel or Timmins storm and Historical flood computations.

It is essential that the practitioner strictly consider the various watershed parameters relative to the parameter boundaries of the individual methods and associated conditions when applying each method. Where considerable dispersion is noted amongst remaining results, one possible approach to address this variability is to average the flood peaks returned from the suitable methods employed, to derive a mean value. Caution should be applied when using the older regional analyses (i.e., Moin & Shaw, 1986) as 'primary methods' as the data used to derive these relationships may not represent current conditions. Furthermore, the statistical veracity of the relationships amongst physiographic and related parameters and peak flows can be affected by a range of factors. These include, but are not limited to, how physiographic parameters are

estimated including using new or different technologies to derive these variables compared to relationships in the original study and alterations in a variety streamflow response factors (Stainton, 2010).

The general methods are highlighted below and are further detailed in [Section 3 Hydrologic Analysis](#), where applicable. A succinct summary of the other methods listed in [Table 1-3](#) can be found in MNR (2020). Additional details are provided in [Section 3.2.3 Regional Flood Frequency Analysis](#) and [3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins](#)

1.11.1.1 Index Method

The index method (Dalrymple, 1960), (also known as the index-flood method) can be used to determine the relationship between catchment and hydrologic characteristics at one location, which can then be applied to estimate peak flow in an ungauged location. The process involves initially undertaking a single hydrometric station analysis and the corresponding frequency is developed. The variable used for single station analysis is the maximum instantaneous peak flow of the annual series. Then, single station frequency curves are combined to give the regional frequency curves. This is completed in two steps. The first step is the development of the dimensionless frequency curve which represents the ratio of the flood of any frequency to an index flood known as the mean annual flood (i.e., the peak flow corresponding to the 2.33-year return period). The second step is the development of the relationship between mean annual floods to the drainage area of the catchment.

The index method assumes that flow is natural, or with very minimal regulation, and the region is homogeneous. There is no limit to the drainage area of the homogeneous region. The recurrence intervals are computed with a minimum of 10 years of record. The index method is popular, as it is computationally simple involving the calibration of one index equation, the index ratios assume consistency across return periods and most importantly, it requires only the drainage area of the watershed, making it suitable for watersheds where physio-meteorological data are sparse. This method is suitable for data checking and corroboration purposes.

1.11.1.2 Modified Index Flood Method

A variant of the index method is the the Modified Index Flood Method (MIFM) (MTO, 1986; 1996; 1997). The MIFM is based on the basic form of the index method used for the estimation of the mean annual flood, however the procedure is modified for the prediction of the Q_{25} quantile as outlined in (MTO, 1996; 1997), with all other peak flows calculated by applying a frequency factor to the 25-year value. This method has been used in the

past to prorate flows from a larger catchment to a smaller catchment or sub-catchment that share similar land cover, topography, watershed morphometrics and other physiographic characteristics. MIFM is its applicability to drainage areas in both Canadian Shield type and Southern drainage basins. The MIFM cannot be applied to drainage basins less than 25 km² and is limited to estimating design floods in the 2.33-year to 100-year range in rural watersheds. The MIFM was last updated in 1996 (MTO, 1997) and has been superseded by the Unified Ontario Flood Method (UOFM) (MTO, 2016b), profiled in [Section 1.11.1.3 Unified Ontario Flood Method \(UOFM\)](#). Regardless, MIFM can serve as a useful data checking and corroboration tool.

1.11.1.3 Unified Ontario Flood Method (UOFM)

The Ministry of Transportation's UOFM (Sehgal & Drake, 2016; MTO, 2016b) is another technique that can be used to estimate peak flow of an ungauged location, using three stages of non-parametric testing, station frequency analysis, and multiple regression of flood quantiles. To ensure confidence in the results, it is recommended to compare the results with alternative regional flood frequency analysis methods, such as those outlined by Moin and Shaw (1985), the Regional Frequency analysis undertaken by Cumming Cockburn Ltd (1986) or other robust comparator. Caution should be used to ensure the watershed matches size limitations of the UOFM, as well as other noted limitations, including that data used must represent natural flood conditions and unaltered natural flow. A useful aspect of the implementation of this method is that it produces upper and lower quartile results giving you a range of values, to account for errors. This can be helpful in comparing to results from other methods, which don't often factor in error. As previously outlined, the UOFM supersedes previous MTO methods.

1.11.2 Combined Individual and Regional Analyses

1.11.2.1 FDC based Spatial Interpolation of Flow Time Series

An additional suitable method is the spatial interpolation Flow Duration Curve (FDC) technique of Hughes and Smakhtin (1996) and as applied in Ontario (Metcalf et al., 2003; 2005; 2013). Findings suggest that the spatial interpolation and regionalization methods using FDCs offer an initial, parsimonious approach for simulating natural flow regimes at ungauged sites in Ontario. This is corroborated by results from other areas of Canada suggesting that this method (Gupta et al, 2014), and adaptations (Shu & Ouarda, 2012) can produce robust results. Flood frequency analyses on the annual peak flows derived from this method may be employed to estimate return period flows. This approach is highlighted in further detail in [Section 3.2.4.2 Spatial Interpolation of Flow Time Series using Flow Duration Curves and Flood Frequency Analysis](#).

1.11.2.2 Use of a Hydrologic Model

Hydrological modelling is another approach that may be used to estimate peak flow and includes the calibration of a hydrological model in a gauged catchment, regionalization of the calibrated model parameters to estimate runoff time series for the ungauged catchment, and finally using the predicted runoff time series to generate hydrologic signatures of the ungauged location that can be used to estimate peak flow. Further information on using hydrologic models is provided in [Section 3.6 Hydrologic Modelling of Flood Flows to Define the Flooding Hazard](#). Hydrologic modelling is an important tool for estimating flow hydrographs, including peak discharges, for flooding hazard identification where statistical techniques based on regional analysis of flow records are inappropriate.

1.12 Summary

Other design hydrograph estimation techniques may also be used to estimate peak flow, such as those supported by the Prediction in Ungauged Basins (PUB) initiative (IAHS, 2023), where suitable, appropriate and justified. With all flood peak estimates, it is valuable to check and compare calculated values to better understand their relative magnitudes, between and amongst approaches. For checking new or previously computed flows, two peak flow computation methods developed for Ontario conditions are presented in [Table 1-3](#) specifically, (i) Regional Frequency analysis, and (ii) Index Flood method. It must be emphasized that the Regional Frequency Analysis for Ontario was undertaken in 1986 (Cumming and Cockburn Ltd, 1986) based on earlier period of record streamflow data, with the Index Flood Method study for Ontario released in 1985 (Moin & Shaw, 1985). These methods require updating and analyses to confirm that they continue to reflect streamflow and associated landscape variable relationships

While these data and analyses are not current, they can provide some utility for defining relative magnitude of flood peaks. Furthermore, this does not preclude practitioners from undertaking similar regional frequency analyses and using results from these studies for planning and checking purposes. It should also be acknowledged that data from previous flood frequency studies (i.e., single site frequency analyses) conducted by Moin and Shaw (1985) for the Great Lakes basin were updated to the 2012 record year, by MNR (MNR, 2014). Similarly, single-station annual flood frequency analysis for the gauges of the Nelson and Hudson Bay watershed systems were completed in 2013 (MNR, 2013). It is important to highlight that the two latter studies by MNR did not update the regional analyses.

Other work (e.g., Tetford & Desloges, 2023) has been completed for specific localised regions of the province with the aim of updating previous studies and associated relationships and may be of value for review; other relevant work in Ontario has focussed more on the mechanics of flow regionalisation (e.g., Razavi and Coulibaly, 2013). As other rigorous methods are developed for use in Ontario, including by the MNR, they may serve to replace those highlighted above as new authoritative methods.

[Figure 1-4](#) summarizes the recommendations on the use of single station data and regional frequency analyses for (primary) instantaneous and (secondary) mean daily peak flows, according to period of record for the source hydrometric data.

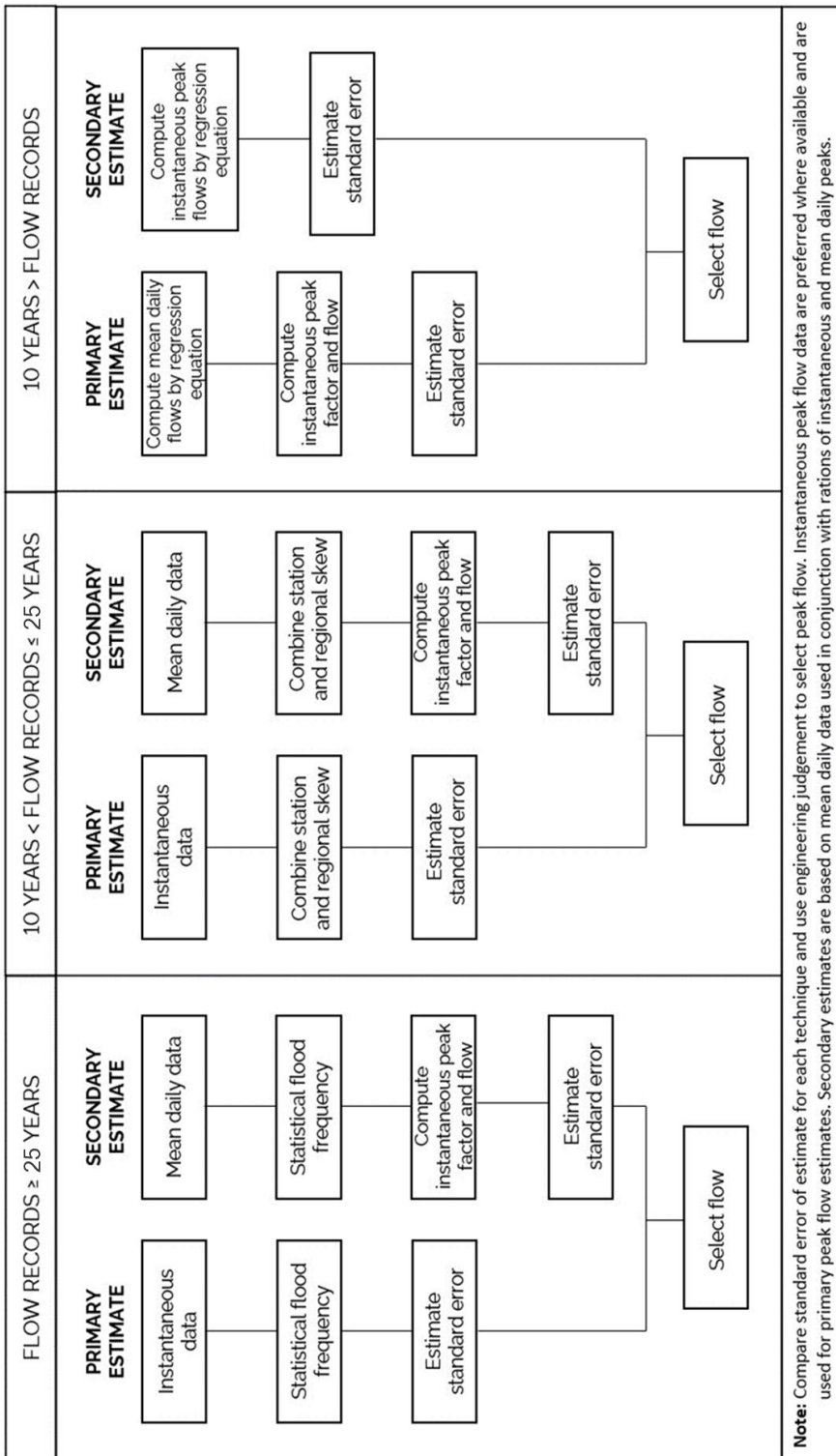


Figure 1-4 Summary of 100-year Primary and Secondary Flood Peak Computation

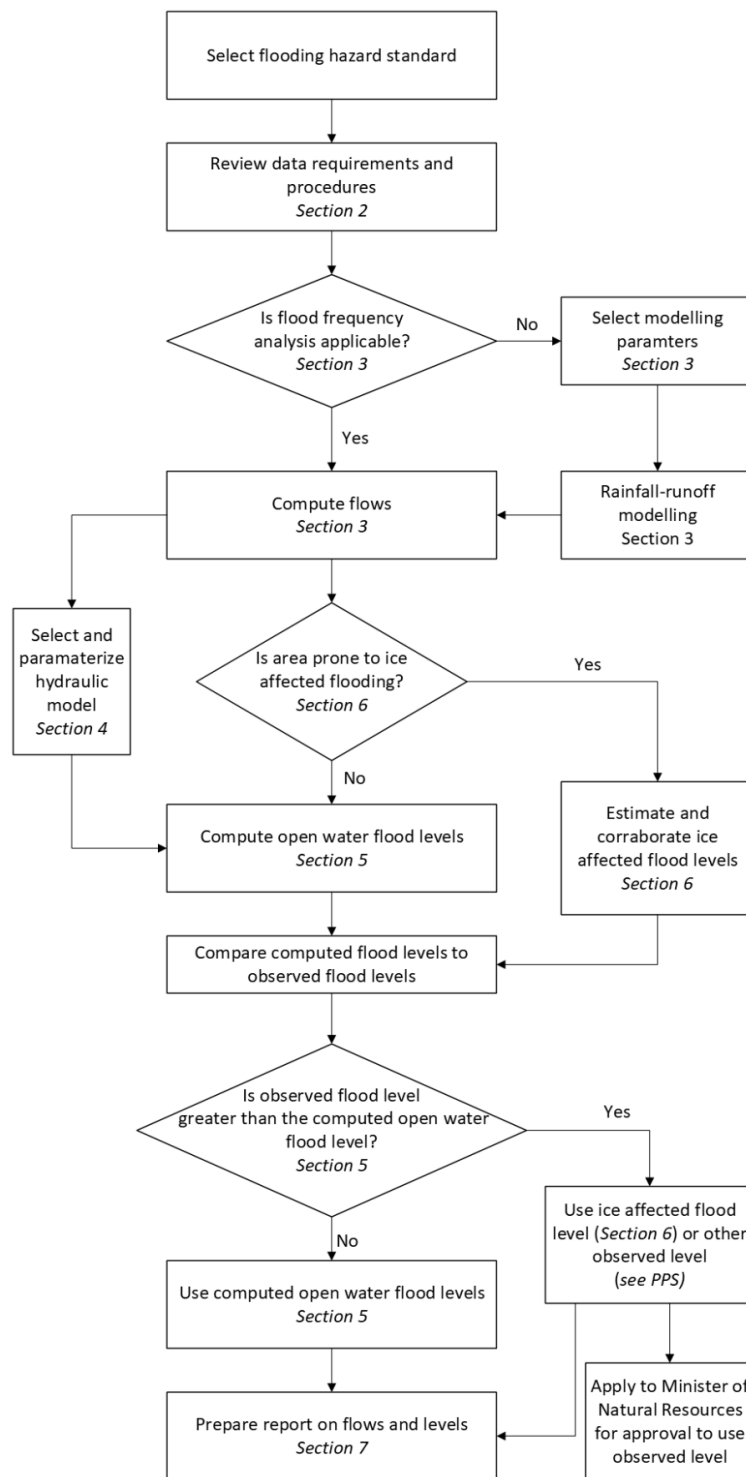


Figure 1-5 Computation of the Flooding Hazard and Flooding Hazard Limit (Flood Levels)

1.13 Overview of Technical Modelling Protocols and Policies Applied in Ontario

The province's prosperity, environmental health and social well-being depend on reducing the potential for public cost or risk to Ontario's residents from natural hazards. This section highlights the range of prudent, conventional approaches to hydrologic and hydraulic modelling integral to Ontario's guidance for defining flooding hazards, that are aligned with the goal of protecting public health and safety and include an appropriate margin of safety. This section serves to outline and highlight these protocols for the purpose of clarity. References are provided to relevant sections in the document where the content is addressed, as appropriate. Application of these recommended protocols ensures that analyses and resulting flooding hazard limits are applied in a consistent manner throughout Ontario and to provide assurances that methods and procedures are appropriately protective of people and property.

Technical policies included herein should be read in conjunction with policies outlined in the *Technical Bulletin: Special Flooding Conditions in River and Stream Systems* [MNR, 2026(Draft)] to ensure consistency in application of approaches.

1.13.1 Hydrologic and Hydraulic Modelling -Technical Policies

The overarching conventional protocols to be used when undertaking hydrologic and hydraulic modeling for the purposes of flood hazard identification and mapping are outlined below.

1.13.1.1 Hydrologic Modelling Technical Policies

a) Use of unregulated, or unmitigated flows are the recommended flow management approach to apply downstream of flow attenuating structures:

- Where regulated flow conditions exist (i.e., downstream of a dam or other flow attenuating structure) or where the flow regime is otherwise affected by artificial storage, the unregulated/natural flow condition is used to define the flooding hazard (See MNR, 2026 Draft)
- The recommended approach involves using unregulated, unmitigated or naturalized flow conditions to define flooding hazards, this includes downstream of dams and flood control structures, as well as stormwater management ponds or any other feature that includes artificial storage (See MNR, 2026 Draft)

- Naturalized flows are river flows that could be expected if human-made artificial surface storage structures (dams, reservoirs, stormwater management (SWM) facilities, offline flood control facilities (OFCFs), flood diversions) did not alter the timing and magnitude of flood flows.
- Naturalized flows are determined by removing the effects of artificial regulation from the flood peak estimates under the flooding hazard and calculating what flood peaks would be if dams, reservoirs, SWM facilities, OFCFs and/or similar were not able to effectively store water during a flood.
- These flows reflect a situation where all of the water flowing into a reservoir, or SWM facility, is simply passed through the structure, as if the structure was not there, other than by accounting for its backwater effect, as applicable.

SWM facilities can play an important role in reducing the impact of development on streams and rivers and their use is well established under existing guidance in Ontario (MECP, 2003).

For the purposes of delineating the flooding hazard, however:

- Stormwater management facilities should not be used to provide any reduction in flood flows and hence should not be relied upon in the establishment of flood hazard limits (i.e., flood lines).

Acknowledging the policies and protocols above, artificial storage (dams, stormwater management facilities, flood control structures), via hydrological modelling should be addressed using the following approaches to ensure the veracity of the hydrologic model for estimating unmitigated flows under the flooding hazard:

- 1) Flood magnitudes should be modelled and assessed, accounting for all artificial and natural storage features. This includes stormwater ponds, hummocky terrain, reservoirs, dams, among other existing attributes. This “existing conditions” model is then calibrated and validated with the local stream gauge network, to ensure the model closely matches observations. This is a critical step as it provides a level of confidence in the model and its calibration with actual conditions.
- 2) Once the model has been successfully calibrated to the local stream gauge network (or other supporting empirical methods/evidence, if no gauge network exists) all artificial storage features are then removed from the model, to reflect the “unmitigated flow” condition/system. This includes removing all dams, stormwater ponds, culverts, and similar. This model, without the benefits of any artificial

storage, will then be used to assess the flow conditions under the flooding hazard by revising the land use parameters (i.e., infiltration, runoff coefficients) reflective of the various development build-out conditions planned for the watershed. (see [Section 2.2 Land Use](#))

According to this policy, the recommended approach that no credit or accounting is afforded to SWM facilities, dams or other artificial storage in the watershed when modelling of peak flood flows under the flooding hazard, nor to subsequent hydraulic modelling and mapping of the flooding hazard limit.

For more information regarding special flood hazard conditions and criteria related to these storage structures, please consult the *Technical Bulletin – Special Flooding Conditions in River and Stream Systems* [MNR, 2026(Draft)].

b) Conversion of Regulated flow to Natural Conditions

Where frequency analysis is to be used to define the flooding hazard standard, if a stream or river is regulated or where streamflow includes the effect of artificial regulation which have a significant effect on peak flows, flows need to be converted to natural conditions (i.e., effect of regulation removed) prior to undertaking the frequency analysis.

c) Structures Used to Divert Excess Water from Rivers to Reduce the Magnitude of the Flooding Hazard

Other than endorsed methods that improve flood flow conveyance of rivers and streams, flood mitigation measures that are constructed to reduce or divert excess water or otherwise reduce flood peaks with the intent of alleviate flooding, shall not be accounted for to reduce the extent of the flooding hazard limit, as they cannot be consistently relied upon to work under flood conditions. These measures may also not consistently function as intended, and therefore cannot consistently be relied upon. This includes but is not limited the following:

- Flood diversion through sewers, tunnels, or culverts of any type, regardless of whether or not they include openings or orifices that can be blocked or plugged by debris; and with or without trash racks.
- Structures designed to store and release storm flows more gradually to rivers and streams, which may include underground storage of flood flows in tanks or other similar containment and release structures.

Where this type of infrastructure has been used, any impact on reducing flood peaks, other forms of flood attenuation, should not be accounted for in flood modelling or subsequent mapping.

1.13.1.2 Hydraulic Modelling - Technical Policies

a) Model Dimensionality

- One-dimensional (1D) modelling is the standard approach for flood hazard modelling.
- Two-dimensional (2D) modelling may be used to characterising flows, levels and flow direction in complex floodplains, outside of the river channel.
 - When 2D modelling is used for out of bank modelling including for spill hazard modelling and mapping, “quasi-steady” flows shall also be applied in which the flow input is gradually increased and sustained at the peak flow for an extended period with constant boundary conditions to achieve steady state conditions throughout the system/study area.
 - This “quasi-steady” approach enables the user to follow standard practices of not accounting for or not preserving storage components within the flood plain during 2D modelling.
- Use of coupled 1D-2D models is recommended, whereby a standard 1D model is used on the main river channel, and in the flood plain where flow paths are less defined, a 2D mesh connected to the 1D modeled river channel can be used.

b) Steady-State Hydraulic Analyses

- Hydraulic modelling for the purpose of flood hazard mapping assumes is to assume steady state flow as a standard practice.
- Where there is a desire to use 2D modelling (inherently unsteady state flow) for characterising flows, levels and flow direction in complex floodplains, outside of the river channel, it is recommended to conduct “quasi-steady” flow simulations in which the flow input is gradually increased and maintained at the peak flow for an extended period to achieve steady state throughout the system.
 - The use of a quasi-steady flow regime provides the combined benefits of a conservative approach (storage areas will become filled and therefore not

ultimately affect flood hazard mapping) and 2D modelling advantages (defining complex overland flow interactions including flood spill areas).

- This requires sufficient simulation time to allow for flow to enter, pass through and exit the domain as well as filling all storage areas. Determining this type of convergence necessitates that, modellers compare flows entering and leaving the domain to ensure that they match.

c) Linear Embankment Structural Measures - Berms, Dykes and Floodwalls

- Dykes, flood walls and berms are not regarded as permanent flood control structures and are not recommended to be considered to reduce the extent of flooding
- The effect of the linear embankment on increasing water surface elevations in the stream or river in the vicinity of the structure should be modelled, and these increased elevations used to extrapolate behind the structure to define the extent of the flooding hazard limit

d) Temporary ponding and conveyance around bridges and culverts

Roadway and other transportation related crossings, such as railways, are not intended to act as dams although the design often has to accommodate temporary ponding behind the structure. This can increase the flooding hazard limits upstream and reduce downstream flood peaks; hence the downstream hazard may be reduced. When the culvert in the structure is enlarged or the structure is removed the temporary backwater ponding is reduced or eliminated, thereby potentially changing both the upstream and downstream flood lines (See [Section 5.4.4 Bridges and Culverts](#)).

- Flooding hazard limits (i.e., flood lines) upstream of bridges and culverts shall make allowance for backwater effects and associated ponding caused by these structures.
- Downstream of a culvert or bridge the natural, unmitigated flood line shall be used for delineating the flooding hazard limit, making no allowance for the upstream temporary ponding.

e) Waterway Crossings

Further to item d), flood profile computations, for flooding hazard mapping purposes, should be based upon existing structural and hydraulic conditions along the river ([See Section 5.4.3 Waterway Crossings and Encroachments](#)).

- Where a structure may be replaced in the future, or upon inspection of preliminary hydraulic results, a structure may prove to be unstable or breach

on overtopping in such cases, additional hydraulic analyses of the resulting downstream flood wave should be carried out.

f) Stream Bed Scour

- No allowance should be made for potential scour of the stream bed during a flood in the assessment of hydraulic losses. (See [Section 5.4.4 Bridges and Culverts](#))

g) Spills of flood waters from the valley and unconfined systems:

- Under circumstances where flood waters exceed the capacity of the valley system and flow into adjacent areas away from the river corridor, no reduction should be made for the spill in the downstream flooding hazard computations; and the extent of the spill should be characterised hydraulically and mapped (See [Section 5.4.7 Flood Spills from the Channel](#)).

1.14 Document Structure – How to Read this Document

The following seven steps outline the hydrologic and hydraulic related flood hazard delineation tasks to be followed, identifying the relevant chapters/sections within which detailed descriptions on the recommended procedures can be found.

Step 1 – Select flooding hazard standard ([Section 1.3 Flooding Hazard Standards to support flood hazard identification in River Systems](#))

- Review policies and select flooding hazard standard(s) relevant for the area of Ontario being investigated

Step 2 - Review data requirements, methods of hydrologic and hydraulic calculations ([Section 2 Data and Approaches Supporting Hydrologic and Hydraulic Analyses: General Procedures](#))

- Data requirement: streamflow, water levels, meteorological and physiographic data
- Flood magnitudes: flood frequency analysis for 100-year floods, or hydrologic modelling of flood from a specified meteorological event,

- Hydraulic modelling, type of flow, cross-section data, roughness, bridge and culvert losses, plotting
- Review data requirements for mapping (for detailed specification see MNR, 2023).

Step 3 - Select hydrologic modelling parameters and model(s) for computing flood flows ([Section 3 Hydrologic Analysis](#))

a) Model Parameters/Model Parameterization

- Select precipitation input to modelling of flood standard: Hazel, Timmins, 100-year storm depth, duration, distribution, snowmelt
- Select soil data (Refer to Section 2.4 and MNR, 2023)
- Select land use data (Refer to Section 2.2 and MNR, 2023)
- Antecedent conditions

b) Model/Computational Procedure

- Select hydrologic model: single and continuous models
- Select computational procedures: snowmelts, infiltration, soil moisture account, base flow, watershed routing
- Incorporate reservoir routing, as appropriate
- Model calibration, validation and sensitivity analysis

Step 4 - Compute water levels for open water conditions, ([Section 4 Hydraulic Modelling](#) & [Section 5 Water Level Computations – Open Water Hydraulic Modelling](#))

- Determine method of calculation: backwater computations vs. flood routing
- Select model software
- Develop cross-sections, incorporating effect of lakes and reservoirs, waterway crossings and encroachments,
- Model calibration, validation and sensitivity analysis

Step 5 - Compute ice affected water levels, where appropriate, ([Section 6 Ice-Affected Flooding - Water Level Derivations and Computations](#))

- Determine the need to compute ice jam levels for the site
- Select ice jam computational method
- Estimate frequency of ice jams

Step 6 – Map Flooding Hazard Limit/Flood line

- Refer to MNR (2023).

Step 7 - Prepare Technical Report and Documentation ([Section 7 Technical Report and Documentation](#))

2 Data and Approaches Supporting Hydrologic and Hydraulic Analyses: General Procedures

The purpose of this section is to outline the data, approaches and procedures that are common to both hydrologic and hydraulic modelling for the purposes of flood hazard mapping in Ontario. This includes highlighting the different flood standards to be applied and the associated data required to support quantification of both flows (i.e., hydrology) and levels (i.e., hydraulics) under the flooding hazard.

2.1 Hydrometric and Spatial Data Required

To determine the peak flows for floods to be mapped and their corresponding water surface elevations, it is necessary to obtain a considerable amount of data from numerous sources. It will not be necessary to collect all of the following data for each specific case; the requirements will vary depending upon the types of floods to be mapped, the methodology to be used, the location, and the availability of requisite data. The general types of information required will be common to all studies or analyses.

2.1.1 Streamflow Records

Streamflow records should be collected for the watercourse in question and any tributaries involved. Where streamflow records are not available and cannot be reasonably obtained, the availability of streamflow data for similar nearby watercourses should be investigated. Methods to support transposing data from nearby and outlining factors that influence a catchments flow regime are outlined in [Section 3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins](#). If a regional flood frequency analysis is to be carried out, records must be collected for other streams in the same meteorological region. The main source of such data is the Water Survey of Canada, but records may also be available from other organizations such as provincial agencies (e.g., MNR, MECP), conservation authorities, municipalities, and waterpower companies. Data should be obtained from every available source and thoroughly reviewed for accuracy. To ensure that the most up-to-date records are obtained for published hydrometric stations, streamflow data should be retrieved directly from the data available on the [Water Survey of Canada's website](#).

For a flood frequency analysis, an annual maxima series will generally be required. Although a record of instantaneous peaks is desirable, in many cases, mean daily flows will be the only available data. Should either partial duration series or joint probability analysis be required, it will then be necessary to obtain records for more than just the highest peak flow in each year (i.e., several high peaks throughout the year). Where instantaneous data is not available, daily data must be adjusted to reflect instantaneous values. A variety of methods exist to support this adjustment (Sangal, 1981; Chen et al., 2017a), however the user must be cognizant of the performance of the methods (i.e., under prediction/over prediction), particularly in relation to catchment size (Chen et al., 2017a). Additional details are provided in [Section 3.2](#).

To enable routing of a flood from a gauging station to another location downstream, sufficient data must be obtained to define the complete flood hydrograph.

2.1.2 Historical Floods

Historical flood refers to an event that occurred either prior to records being kept on a river or stream, or floods that are greater than the minimum 100-year flood but have occurred since records have been kept. There are many cases where reasonable estimates can be made of the magnitude of a historical flood from information on high water marks. Such information may be found in newspaper files, public archives, or online resources.

If the historical flood was unusually large and a good estimate of its magnitude can be made, the information can be used in a flood frequency analysis leading to increased confidence in the estimate of the flood used for flooding hazard limit definition.

When documenting historical floods, two or more sources of information should be cross-referenced to augment the reliability of the flood estimate.

2.1.3 Rainfall Actually Experienced from a Specified Meteorological Event

In Zones 1 and 3, ([Figure 1-1](#)) the flooding hazard standards are based on specified meteorological events outlined in the Provincial Planning Statement. The rainfall volumes and their time distributions for the Hurricane Hazel and the Timmins Storm are defined in [Section 3.4.2 Historical Storms](#).

If a flood is to be based on another specified input, relevant data on snowmelt and precipitation rates must also be collected.

2.1.4 Stage-Discharge Relationships

Once the peak flow discharge of the flood event is determined, the next step is to develop a stage-discharge relationship within a reach and to convert the discharge to a water surface elevation. If the water surface elevation is to be determined at a different location upstream where no stage-discharge curve is available, or at a series of points along a stream, then the discharge must be converted to a water surface profile. This procedure normally entails backwater computations based on a known or downstream water surface elevation. In this regard, a stage-discharge relationship can be used to determine the starting water surface elevation for the calculation of the water surface profile, and to calibrate the backwater calculation model at the same intermediate or upstream location. If a stream gauge is available at the location, this is achieved by using the stage-discharge or rating curve for the gauging station.

When converting a discharge to water surface elevations, it is important to note that smaller floods usually fall within the range of existing discharge measurements and can be converted to stages with a fair amount of confidence. This is only true if measurements are taken at a stable section where the curve remains constant with respect to time. At unstable sections, the curve changes with physical changes in the stream channel and considerable judgement and experience regarding the site are necessary to use the curve. Judgement is also required in the extrapolation of stage-discharge curves that will be necessary for floods of large magnitude. A small error in extending a curve can lead to sizeable errors in estimating the stage corresponding to a large discharge. Therefore, such estimates are best made by personnel who are both experienced in this type of work and have a good knowledge of the location. This may be done for gauging stations maintained by the Water Survey of Canada, conservation authorities, and/or municipalities.

For control points where no gauging station exists, it will be necessary to develop a stage-discharge relationship. This entails a considerable amount of field work to survey the cross-section and take discharge measurements over a range of stream flows. It is not adequate to measure discharge only at times of low flow, which is the optimum time for surveying cross-sections, but it is also essential to take measurements during medium and high flows.

In most parts of Ontario, where flood flows often occur in the spring, this would entail collecting measurements over a period of several months to establish a reasonable range of the rating curve, across a spectrum of flows and stages. Many discharge measurements are necessary at a new station to define the stage-discharge relationship throughout the entire range of the stage (WMO, 2020). Adequate measurement of

discharge during flood and under ice conditions is of prime importance (WMO, 2020). Streamflow measurements should be as accurate as possible; this can be achieved by following the procedures described in the hydrometric field manuals for the measurement of streamflow (Teriz, 1981) and of stage (Moore, 2019).

2.2 Land Use

The land use types within a watershed may be determined from maps and/or aerial photos, remotely sensed thematic map analysis. Where the runoff analysis requires the computation of agricultural farming practices, Census of Canada data may be used to estimate the extent of crop, pasture, or wooded area, as well as the classified remotely sensed imagery such as Southern Ontario Land Resources Information System (SOLRIS), available for southern portions of the province (as highlighted in [2.3 Land Cover Data](#))

Land uses may be grouped according to similar hydrologic characteristics:

Crops

- Areas under cultivation, including summer fallow; also improved land such as farm lanes and yards

Pasture

- Seeded and natural pasture and other unimproved farmland

Woodland

- Farm woodlots, bush, forested areas and cutover land reverting to forest. Variations in types of woodland should normally be ignored. Small clearings and cutover land can be included in the wooded areas.

Urbanization

- Large areas of urbanization should be measured from maps, including digitally classified remotely sensed data and/or air photos. Roads, scattered houses and other relatively small non-agricultural areas can be included as part of the agricultural area in all but the smallest drainage basins.

The future extent of urbanization should be extracted from a range of sources to evaluate the planned and potential maximum extent of land use change. This should include consideration of the following scenarios:

- a) Official Plans or other Municipal land use planning documents and the planning horizon that should preferably identify planned development that extends a minimum of 20 to 25 years into the future.
- b) Where Official Plan Amendments are proposed, for example as part of an urban boundary expansion, an additional land use scenario may be necessary based on the proposed Official Plan or other Municipal land use planning document (e.g., Secondary Plan).
- c) Full Build Out Scenarios -The maximum extent of urbanization feasibly possible within the watershed should be evaluated to estimate potential future development impacts on flooding hazards, modelling and mapping, under a full-build-out scenario, particularly where projections are in greenfield watersheds, sub-watersheds.
 - Evaluating flooding hazard limits under the full-build-out scenario can provide useful insight into where and how future development may need to be planned or modified to mitigate impacts of development on flooding hazards under such a scenario, and allow for foresight in planning for future development, particularly as it relates to potential impacts on flooding hazards.

2.3 Land Cover Data

Aerial photographs, maps and classified digital land cover layers are tools that may be useful to determine land use types at both local and watershed scales to support hydrologic modelling. There are many benefits to using aerial photography for land use characterization as aerial photographs are relatively easy to collect for any given location, generally have a long time series dating back to the 1930s and may be used to track changes in land use over time or to gain insights for areas that are data scarce.

Common methods for interpreting aerial photographs include the delineation of boundaries for each polygon, or areas with similar properties, and using various characteristics such as the size, shape, texture, or patterns, among others, to characterize land use within a specified location.

The [Southern Ontario Land Resource Information System \(SOLRIS\)](#) is a primary data layer that provides a comprehensive, standardized landscape level inventory of natural, rural and urban lands in Ecoregions 7E, 6E and 5E ([Figure 2-1](#)). SOLRIS is based on MNR's Ecological Land Classification for southern Ontario. Maintained on a consistent update cycle, SOLRIS provides a standardized geospatial information layer that allows for reporting on changes in natural and anthropogenic features such as wetland extent, forest afforestation/deforestation, and built-up areas (impervious/pervious). This layer

may provide useful information to support hydrologic model parameterization and in quantifying changes in relevant hydrologic landscape parameters over time.

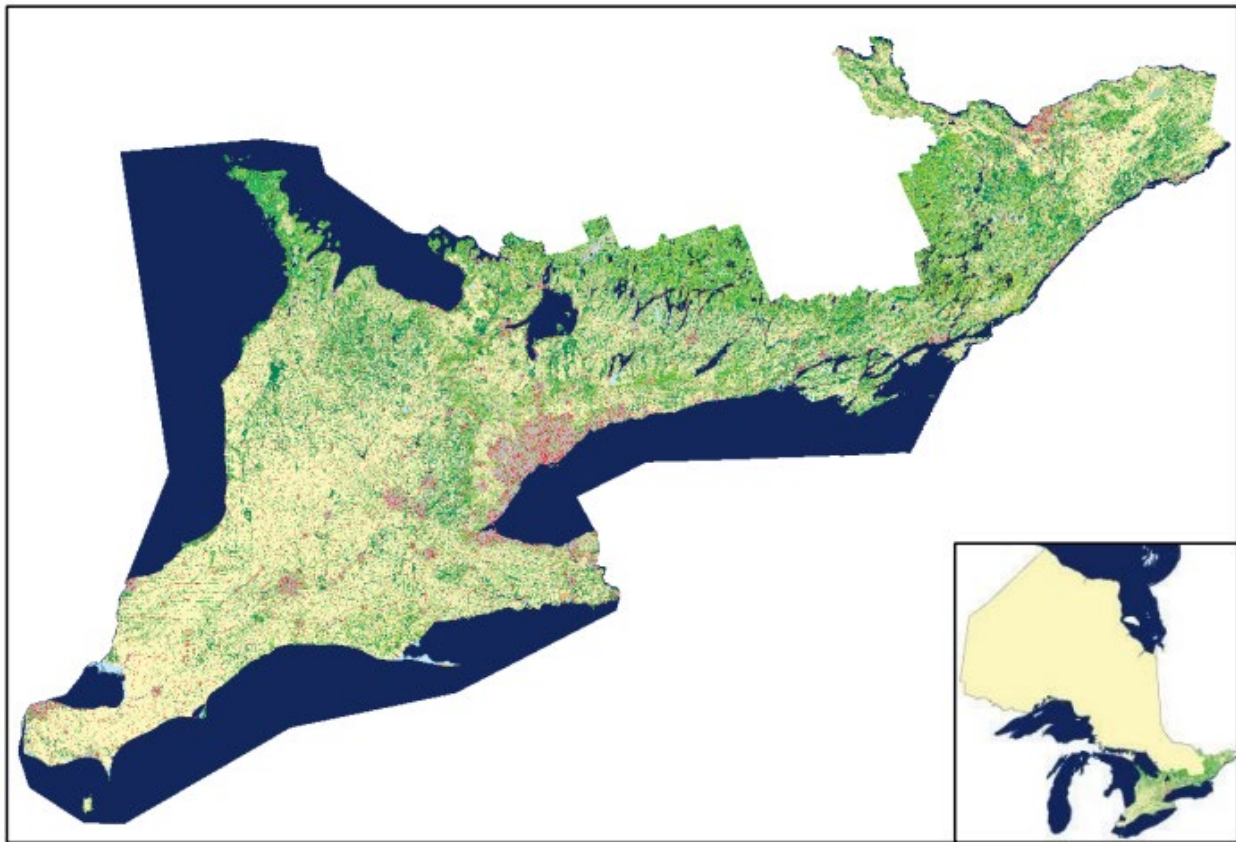


Figure 2-1 Spatial extent of the Southern Ontario Land Resources Information System (SOLRIS)

2.4 Soils Data

Soil type and texture can strongly influence the runoff characteristics of a basin. A considerable amount of published data is available on Ontario soils and their runoff characteristics. For instance, a multivariate regression of estimated flood quantiles in Ontario using principal component and discriminant analysis revealed that surficial geology provides the greatest predictive ability for high magnitude, less frequent flood events (Tetford & Desloges, 2023), emphasizing the importance of surficial geology in understanding and defining the flooding hazard.

The [OMAFRA Soil Survey Class](#) information illustrates soil survey coverage as well as soil type, texture, and slope, among others, and can be found on the Ontario GeoHub.

This open data resource was last updated in 2019 and may be subject to periodic updates.

In hydrologic calculations, according to the system developed by the U.S. Soil Conservation Service, soils may be classified into four hydrologic soil groups, A, B, C and D, on the basis of their texture and drainage condition. Descriptions of the four groups, modified slightly by the staff of the MTO to suit Ontario conditions (MTO, 1986) are as follows:

- A. High infiltration and transmission rates when thoroughly wet (i.e., deep, well-drained to excessively-drained sands and gravels). These soils have a very low runoff potential.
- B. Moderate infiltration and transmission rates when thoroughly wet (i.e., open textured loam, moderately deep to deep).
- C. Slow infiltration and transmission rates when thoroughly wet (i.e., clay loams with a high swelling potential).
- D. Very slow infiltration and transmission rates when thoroughly wet (i.e., clay loams with a high swelling potential). These soils have the highest runoff potential.

It should be noted that the classification depends not only on the texture of soils, but also on the presence or absence of impeding layers and/or high-water tables. According to the SCS criteria (U.S. Soils Conservation Service), soils with a layer that impedes downward movement of water are classified as Group C and soils with a permanent high-water table are classified as Group D.

In Ontario some important soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC or CD as appropriate.

The four main hydrologic soil groups (A, B, C and D) and the three interpolated ones (AB, BC and CD) were identified by the Ontario Ministry of Transportation (MTO, 1986) for various soils and are identified in various publications such as:

1. General Soil Types.
2. Surficial Geology Maps.
3. Land Classification Maps.
4. Soil Associations in Southern Ontario.

A detailed identification of hydrologic soil groups for principal soil types are shown in the MTO's Drainage Manual (MTO, 1986).

Field verification of soil types selected from maps and reports are an important part of the data preparation, it should be done before the runoff computation phase.

2.5 Addressing Regulated Flows

If the stream under consideration is subject to a significant artificial regulation by dams, diversions, and similar, that have significant effects on peak flows, it is necessary to obtain data on the effect of such regulation, or at a minimum understand the significance of regulation, to enable a conversion of streamflow to natural conditions prior to undertaking a flood frequency analysis. A variety of methods exist to support removing the influence of artificial regulation on streamflow time series, including peak flows. A summary of methods for establishing these hydrologic reference conditions is included in [Section 3.2.1.2](#) and presented in more detail in [Metcalf et al.\(2013\)](#).

2.6 Meteorological and Physiographic Data

Meteorological and physiographic data will be necessary for three primary areas of study, regional flood frequency analysis, hydrologic modelling and hydraulic modelling.

The actual variables required depend upon their relative significance or the information necessary to calibrate and validate a hydrologic model.

2.6.1 Regional Flood Frequency Analysis

For regional flood frequency analysis, the following factors are commonly considered, but may not all be significant: drainage area, area of lakes and swamps, basin slope, channel slope and channel length, soil type and soil permeability classification as well as mean annual runoff, precipitation, as well as snowfall. Other variables may also be of significance depending upon the region, its topography and climate.

2.6.2 Hydrologic Modelling

For hydrologic modelling purposes, many of the above variables are used as well as information on forest cover, groundwater, land use, infiltration rates and soil moisture conditions. To operate a model, a great deal of meteorologic data is required, and may include rainfall, temperature and snowfall records (snow water equivalent, snow cover area, snow depth), radiation data, snowmelt coefficients, lapse rates, evapotranspiration.

The data required vary widely depending on the model used. The best test of the model's validity can be measured by how well the model estimates compare to the recorded flows for flood events that are similar in magnitude. Otherwise, the calibrated model parameters may have to be adjusted to allow for possible non-linear effect of watershed response in a manner similar to the adjustment of Manning's "n" value described in [Section 4.2.8 Roughness](#).

Other methods rely on meteorological and topographic data to estimate flood hydrographs. Radar rainfall data can be used with appropriate hydrological models for estimating flood hydrographs. These models can integrate real time radar and rain gauge data, streamflow/reservoir release data where applicable, and watershed characteristics to estimate flows.

The use of remote sensing to obtain hydrologic or water level information also presents potential opportunities. Satellites monitoring the earth surface can determine the extent of water bodies or flooding, type of land cover, snow depth, soil moisture and river ice will soon be able to estimate flood conditions for every satellite pass. As satellite information will only be available at daily intervals at best, this method would be more applicable for large and remote watersheds, or to inform for post-hoc analyses, including information such as water elevation at peak river stage under extreme flood events.

3 Hydrologic Analysis

This section serves to outline suggested procedures and methods to be used to quantify flood magnitudes under the flooding hazard event standards outlined in [Section 1.3 Flooding Hazard Standards to support flood hazard identification in River Systems](#) and as depicted spatially by flood hazard criteria zones in [Figure 1-1](#). This includes details regarding the suitable data sources used to support the computations as well as characteristics of those data.

Whether the flooding hazard standard is to be assessed based on a flood frequency analysis or the resulting runoff and flood peak produced by a specified meteorologic event, there is considerable investigation necessary to develop a reasonable estimate. This is the main part of the hydrologic investigation required and should be carried out using the best techniques available. A high standard of analysis along with good engineering judgement will be required to obtain realistic results, which can be defended if legally challenged.

3.1 Flood Frequency Analysis versus Rainfall Simulation Approach

The use of flood frequency or meteorologic input approaches will be determined by the desired flood event to be modelled and available data. Flood frequency methods will determine floods of a particular recurrence interval and require a sufficient period of streamflow record for analysis. Meteorologic input approaches will determine resultant flooding of a specified storm or rainfall event and require the use of a hydrologic model to convert basin rainfall into runoff and discharge.

Much of Canada uses Flood Frequency Analysis (FFA) processes to define riverine flood hazards; this includes some of the largest watersheds in Canada (e.g., Saskatchewan River). Conversely, in small, flashy catchments effective flow monitoring to capture quickly passing peak flows can be difficult. In these instances, and particularly if they are developed catchments, the use of design storms and runoff simulations is recommended. In urbanized catchments, infrastructure has the potential to greatly impact flow measurements and, therefore, the results of an FFA may not be accurate or appropriate based on other provincially recognized criteria. Examples of such impacts are outlined further in the proceeding paragraphs.

Infrastructure such as stormwater management facilities or road/rail embankments with undersized culverts create backwater effects and reduce downstream peak flows. Large

embankments such as rail and roadway crossings, with small openings (e.g., culverts), create storage effects. If the embankment is high, with a small opening and the upstream storage is significant, there is potential for significant effects on hydrologic flows downstream. Current policies do not allow for the inclusion of the storage effects of these restrictions on the downstream flows. However, if using an FFA approach, the effects of these restrictions will be inherently included in the data set and, therefore, flow naturalization may be required (refer to [Section 3.2.1.2](#)).

Urban areas with storm sewer infrastructure and, even more so, those with combined sewers, can have different contributing areas during low flow events up to the capacity of the sewer system vs during large flood flow events (e.g., an area with a combined sewer may convey all flows up to a 2-year event to the wastewater treatment plant (WWTP)) but flows above that are conveyed overland to a storm sewer or creek system. Though programs for sewer separation and downspout disconnection continue to advance, it cannot be assumed that these smaller storm flows will always be conveyed away from the creeks. A FFA based on infrastructure-affected flow monitoring could provide a poor estimate of potential flood flows in a creek.

Another reason to use design storms in urban areas is that the parameterization is relatively easy as detailed topography and land use is typically well known. However, in rural, hummocky, wetland, and/or forested areas, depression storage and initial abstraction parameters can be harder to estimate. In these areas rural areas FFA may be more appropriate than design storms.

3.2 Flood Frequency Analysis

In some cases, the floods to be mapped will be determined by frequency analysis. Flood frequency analysis involves using observed annual peak flow discharge data to calculate statistical information and that are used to construct frequency distributions with the goal of determining the magnitude of a flood event with a specific return period (i.e., annual exceedance probability - AEP).

3.2.1 Data Requirements and Assumptions to Support Flood Frequency Analyses

Peak flow data is generally subject to considerable uncertainty. As flood frequency analysis is sensitive to a small number of high and low streamflow observations, care is needed to understand the uncertainties that are implicit in the measured data. Furthermore, missing data from truncated and incomplete records should be filled in wherever possible. Once data are screened and reviewed for quality, practitioners should

plot and review the record to determine its likelihood to produce a satisfactory flood frequency analysis. Outliers, step-changes and slope breaks, when floods occurred could indicate that different flood-generating processes may be present and can indicate non-stationarity of the data. Low outliers that could influence the flood frequency analysis (e.g., zero-flow years, potentially influential low flood observations) should be screened and removed using an appropriate statistical test. Procedures outlined in Bulletin 17C (England et al., 2019) can be followed to address such situations. Once removed, their occurrence must still be reflected in the overall analysis.

Extreme flood peaks (high outliers) may be identified using a similar process but are usually retained due to the importance of high flows for design flood purposes. There is the potential that high outliers may be the result of a different flood-generating process, requiring a more complex analysis (England et al., 2019). As well, observed historical information, when known, should be incorporated into the analysis and the inclusion of such information would also be reflected in the plotting position. Many of these components of the analysis are further elaborated upon in the proceeding sections on flood frequency analysis.

While it may be difficult to specify some of the aspects of flood frequency analysis, there are some basic rules that must be followed and assumptions made of the data being analysed, including that:

- a) Flood peaks are derived from instantaneous data;
- b) Flood peaks are representative of natural flow conditions (i.e., unregulated);
- c) There is stationarity in the flow record; and/or
- d) Streamflow has a satisfactory or requisite period of record.

Under circumstances where these assumptions are not met, methods exist to address these data issues. These are outlined in the following sections.

3.2.1.1 Use of Instantaneous Flood Peak Data

For a flood frequency analysis, an annual maxima series will generally be required. Although a record of instantaneous peaks is desirable, in many cases, mean daily flows will be the only available data. Where instantaneous data are not available, daily data must be adjusted to reflect instantaneous values. A variety of methods exist to support this adjustment (e.g., Sangal, 1981; Fill & Steiner, 2003; Chen et al., 2017a; Khaliq, 2024), however the user must be cognizant of the performance of the methods (i.e., under prediction/over prediction), particularly in relation to catchment size (Chen et al., 2017a).

3.2.1.2 Conversion of Regulated Flows to Natural Flow Conditions

In flood frequency analysis of peak flows, the initial assumption is made that floods are random and independent events that can be described by a particular probability distribution. If a stream is regulated sufficiently to affect the resulting peak flows, then they are no longer random and independent events; a probability distribution which assumes randomness and independence is not applicable. The first step in undertaking a frequency analysis is to evaluate the streamflow time series to determine the influence of regulation on streamflow. If necessary, the conversion of regulated streamflow to natural conditions is achieved by removing the effect of dams, diversions and other regulating factors that impact the flow regime, including the magnitude and timing of peak flows.

Given adequate data such as diversion flows, reservoir stages, outflows, and stage-storage curves, it is possible to convert flows to natural conditions by reverse reservoir routing. If such data are not available, the problem becomes more difficult. Records are generally available for major installations, which may be most likely to affect peak flows. Where there may not be data available for smaller projects, it may be necessary to estimate their effect by various empirical formulas depending on the type of installation. Evidence from urban and urbanizing catchments suggests that roadway and railway crossings can have a significant effect on streamflow, particularly where many crossings exist. The impacts of these transportation networks on the flow regime/flood peaks will also need to be addressed. However, this can be more difficult for major installations, as similar supporting data are not available for these circumstances. In any situation where the effects of regulation cannot be removed, these flow data should not be included in flood frequency analyses.

As mentioned in [Section 2.5 Addressing Regulated Flows](#), streamflow that includes the effect of artificial regulation needs to be converted to natural conditions prior to undertaking a frequency analysis. Under scenarios where the use of regulated flows is supported by adherence to criteria outlined by MNR (2026 - Draft) in relation to consideration of artificial storage for defining the flooding hazard it must also consider these in relation to structural design elements for downstream bridges, floodwalls and flood channels, in a decision to use regulated flood flows. Mitigated flows may be used in the case of credited constructed storage including Offline Flood Control Facilities (OFCFs), where specific purpose-built criteria are met. In these cases, converting to natural conditions may not be required. However, in most cases naturalization of flows is expected.

Many of the methods highlighted to predict flows and flood peaks in an ungauged basin may also be suitable to use to produce representative natural flow regimes, in regulated

basins. Once an estimate has been made of the natural flood magnitude, it is necessary to reconvert the flow back to regulated conditions, to support calibration and validation with the existing (regulated) streamflow gauge network/conditions. Also, the data used for the original conversion will be necessary to determine and document probable operating procedures of the installations under conditions of the flood selected for defining the flooding hazard, where such installations exist.

Common methods used to support naturalizing flow time-series are outlined in Metcalfe et al. (2013). The spatial interpolation method highlighted in [Section 1.10.1](#) and detailed in [Section 3.2.4.2](#) has proven to be a useful tool to support the generation of natural streamflow time-series data, upon which FFA can subsequently be performed.

3.2.1.3 Streamflow Record Stationarity

When records of historical peak flows are used for a frequency analysis, it is assumed that all the data are samples from a single population. This implies that conditions in the watershed have remained unchanged during the period of record. In some cases, considerable change in a basin over the years will affect streamflow and the flood regime. Anthropogenic changes at a local scale, forestry operations, urbanization, agricultural drainage, and irrigation can have a considerable effect on streamflow. Similarly, anthropogenic alterations at a more global level, including climate change, can have a significant regional and local impact on streamflow, including flood magnitudes. If this effect is significant, it is necessary to assess the changes that have occurred with time to support the development of a stationary record for analysis.

Approaches for non-stationary frequency analyses exist to assist in addressing these situations, generally involving more complex statistical techniques. These include concepts and techniques that are widespread in academia but until recently have been applied very little in flood estimation/flood management and include tools and techniques to assist in detecting and accounting for non-stationarity in flood frequency analysis. Generally, these approaches include trends tests, consideration of temporal and physical covariates, understanding probabilities and return periods in a non-stationary context and way to address uncertainty (Faulkner & Warrant, 2020, Faulkner et al., 2020; Maria et al., 2024). The UK Environment Agency have developed tools to allow hydrologists and water resource engineers to carry out trend and change point detection and non-stationary flood frequency analysis using annual maximum flow data (Faulkner & Warren, 2020). An overview of the steps in this process are provided in [Figure 3-1](#).

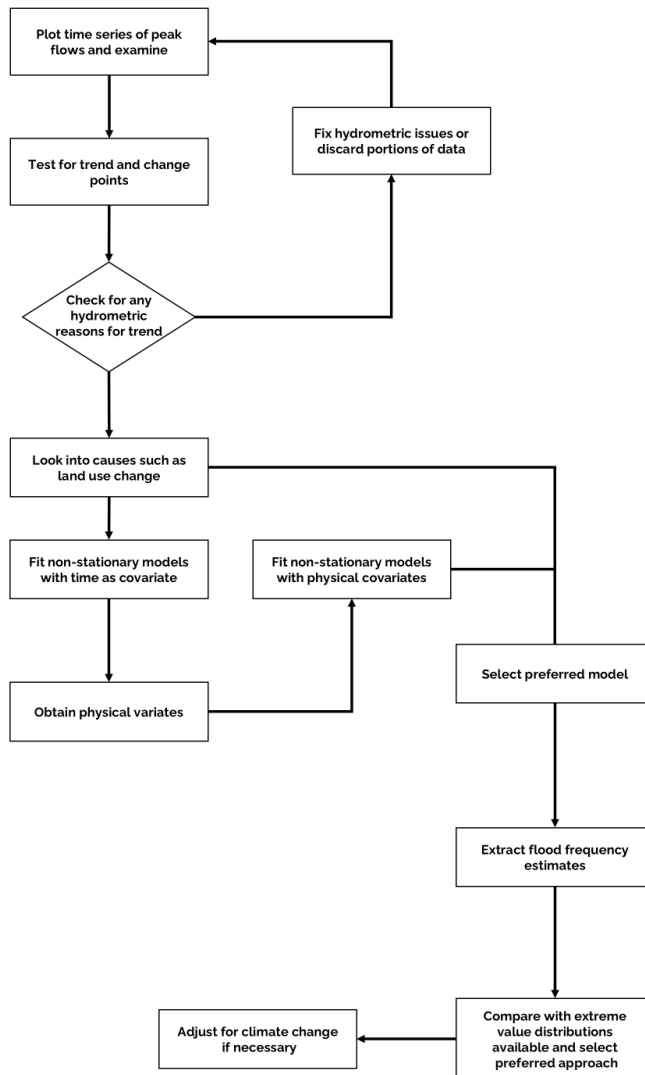


Figure 3-1 Process flow chart providing an overview of steps followed during a flooding hazard estimation study where non-stationarity is a potential issue (adapted from, Faulkner & Warren, 2020).

3.2.1.4 Extension of Streamflow Records

It has been past practice, particularly when undertaking a regional flood frequency analysis, to extend the records of flood flows at short-term stations by correlation with adjacent streams. Generally, this has been used to make a more accurate assessment of the plotting positions of the recorded floods when graphical techniques are used to define a frequency curve. Such a procedure is not used with statistical techniques and should normally be avoided.

The only case where extending a streamflow record can be justified is where the stream has a short period of record and there is not enough data available to carry out a regional analysis. In such a case, the record can be extended based on a larger sample at an adjacent stream or on meteorological data in order to estimate the desired flood event. The estimated flows are then analyzed along with the observed data in a frequency analysis. One recommended approach for record extension and included in the USGS Bulletin 17 C (England et al., 2019) is the maintenance of variance extension technique. This involves using concurrent data at a nearby long-term gauging station having similar catchment characteristics to the site of interest. This approach requires that there be at least 10 years of overlapping data for the two stations and that the correlation coefficients for these data exceed a critical value defined in England and associates (2019) and be considered under circumstances when the site with the short record is less than 20 years and has a minimum 10-year length.

3.2.2 Single Station Flood Frequency Analysis

A single station flood frequency analysis (FFA) for the stream in question will only be adequate if the record is sufficiently long and reliable. If the record is not of sufficient length to calculate an extreme event, (i.e., 30 to 40 years) or there is some doubt about its reliability, a regional flood frequency analysis should be carried out, in which case several single station analyses are combined. Single station FFAs should not be conducted in cases where the period of record is less than 10 years, and as a rule extrapolating to return periods more than double the length of the available record should be avoided.

The first step in a frequency analysis is to obtain the available data and assess its reliability. Whenever possible, instantaneous peak flows should be analyzed rather than mean daily values. Where the data are not available, mean daily values should be converted to instantaneous. The record should be checked for the possibility of ice or log jams which may cause an increase in stage which could lead to erroneous discharge values.

Once the record has been assessed and is judged to be reliable, a frequency analysis of annual peak flows must be carried out. There are various theoretical probability distributions that can be used for this purpose. Those commonly used in Canada include: (1) Extreme Value Type 1 (Gumbel) distribution (EV1); (2) Lognormal distribution (LN); (3) Three-Parameter Lognormal distribution (LN3); (4) Log Pearson Type 3 distribution (LP3), and (5) Generalized Extreme Value (GEV) distribution. Various flood frequency analysis software packages (e.g., HEC-SSP, HYFRAN-PLUS, CFA) exist that can analyze streamflow data using some, or all the probability distributions mentioned above.

The notable topographic and climatic variations across the Canadian landscape, including within Ontario, can make identifying a single distribution for flood flow frequency analysis a challenging task (Zaghloul et al., 2020). However, recent Canadian investigations have shown that the GEV distribution can produce better results than Generalized Logistic, Pearson Type III, and log Pearson Type III distribution for at-site frequency analysis, fitting of annual maximum data and instantaneous peak flow data (Zhang et al., 2020). Practitioners should be cognizant that the performance of flood frequency distributions in Ontario can differ between hydro-climatic regions (Yue & Wang, 2004), and land use types and/or surficial geology (Tetford & Desloges, 2023), hence one or more of the distributions may produce suitable results in some of these regions. Results from other areas of North America have suggested that GEV distribution may outperform the LP3 distribution in estimating floods with return periods of 50 years or more (Berton and Rahmani, 2024).

Two distributions are advocated for use in Ontario, based on the results of studies considering the relative performance of a range of probability distributions, these include the LN3 and the GEV. The LN3 has been, and continues to be a preferred distribution in Ontario, as previous studies have generally found it to produce the best fit (Sangal & Biswas, 1970; Sangal & Kallio, 1977; Moin & Shaw, 1985; Cumming Cockburn Limited, 2000; MNR, 2014). The principal advantage of the LN3 distribution is that it provides a simple method for preserving the first three moments of the observed data (Burgess et al., 1975). Findings from other northern temperate climates, including Canada, have supported the use of the LN3, particularly for less frequent, high magnitude (i.e., extreme right tail event) flood peaks as it is less likely to underestimate peak flows than some other distributions (Kousar et al., 2020; Faghfoury et al., 2023). Secondly, the GEV distribution is also recommended for use in Ontario, based on results of recent Canadian studies (Burn & Whitfield, 2016; Faulkner et al., 2016; Gaur et al., 2018; Zhang et al., 2020; Zhang & Stradnyk, 2020), as well as from other local (Millington et al., 2011) and provincial-scale studies in Ontario (Glaves & Waylen, 1997; Nguyen et al., 2017).

It is also important to highlight that there may be some spatial specificity in the best-fit performance of the LN3 versus the GEV distribution, based on relative location in the province. For instance, the GEV distribution may best describe flood peak distributions for areas in northern portions of Ontario, with the LN3 best characterizing flood peaks in central and southern areas of the province (Yue and Wang, 2004). The performance of three-parameter distributions, including LN3 and GEV, can differ based on the hydroclimatic properties of the basin in question, affecting flood frequency properties such as the mean variance, and tails or respectively the location, scale, and shape parameters of the probabilistic distribution (Reinders & Munoz, 2024). Practitioners should select the

most appropriate distribution based on careful consideration of the results from flood frequency analysis plots and goodness-of-fit testing.

Another factor which should be considered is the possibility of including a historical flood in the analysis. If a good estimate can be obtained of a large flood which occurred prior to records being maintained on a stream, it should be considered in the flood frequency analysis. For this purpose, the value of the historical flood and its year of occurrence must be known, as well as the fact that it was the largest flood between that year and the start of the recorded series. In such a case, all available information on a stream can be used in the analysis to give the best estimate of the frequency curve. This type of analysis can be carried out using Extreme Value Distribution with parameters estimated by maximum likelihood. Although the inclusion of a historical flood may not have a significant effect on the frequency curve, it does reduce the variance of the estimate, thus increasing the reliability of the curve.

Several methods have been developed to estimate the parameters in hydrologic frequency analysis models. The Method of Maximum Likelihood, Method of Moments, and the method of L-Moments (LMOM) are the most used methods in hydrology for parameter estimation, owing to the lower sensitivity to outliers and to the better estimation of the tail heaviness in presence of short data records (Hosking, 1990). While the maximum likelihood method is widely regarded as one of the most powerful methods for estimation of parameters, as well as an efficient method as it produces parameter estimates with the smallest sampling variance (Hamed & Rao, 1999). The method of L-Moments (Hosking, 1990; Hosking & Wallis, 1997) has received considerable endorsement (e.g., Hu et al., 2020) and use (e.g., Zaghloul et al., 2020); as the method is almost unbiased and less sensitive to outliers in comparison to the conventional product moments (e.g., mean, variance, skewness and kurtosis). L-Moments are widely used and less computationally involved and are frequently associated with the application of the GEV distribution (Zhang & Stradnyk, 2020).

The preferred methods of estimating distribution parameters are that of maximum likelihood, and L-moments. If no solution can be found using these methods, the method of moments should be used. Computed or graphical estimates based on empirical plotting of positions should be avoided.

It is often difficult to determine which distribution best describes a given data sample. There are various hypotheses tests based on goodness-of-fit, such as Chi-square, Kolmogorov-Smirnov (K-S), and the Anderson-Darling (A-D) tests, that can be applied for checking the adequacy of fitting of probability to the recorded data. Specifically, the A-D test, is considered a more sensitive alternative that gives more weight to the tails than the

K-S test, with the latter being more distribution free, in that its critical values are more independent of the specific distribution being tested (Chen et al., 2017b). Information-based criteria tests such as the Bayesian Information Criterion (BIC) and Akaike Information Criterion (AIC) assist in selecting the best fitting probability distribution for flood frequency analysis (Di Baldassarre et al., 2009; Rahman et al., 2013). It is noted that the AIC and BIC are information criteria that help choose the best distribution by balancing fit and complexity, with the K-S and A-D tests being more typical goodness of fit tests that evaluate how well the data distribution matches a specified distribution.

Although graphical estimates should not be used alone, it is always worthwhile plotting the data and the computed frequency curve to give a visual indication to confirm it is the best fit. The use of L-moment ratio diagrams is an effective tool to provide a visual goodness-of-fit measure to effectively evaluate various distributions and assist in identifying the best-fit distribution amongst all distribution candidates (Robson & Reed, 2008; Rahman et al., 2013). Various R programming language software packages exist and can easily implement the L-Moments ratios diagrams for practitioners (e.g., Hosking, 2024). This can be used in conjunction with other factors such as a comparison of sample and distribution statistics, the variances of the estimates and, where applicable, confidence bands using a preferred confidence limit of $\pm 95\%$. Various statistical flood frequency analysis software packages can be used to compute the parameters, statistics, frequency curves and standard error of estimate for the four distributions mentioned above. The inclusion of a plot routine and a guide to the interpretation of output provides the user with all the pertinent information necessary for choosing the most suitable distribution.

In the absence of a well-defined standard of practice, approaches endorsed for selecting the “final” flood frequency analysis results include:

- Using results from a single “best-fit” distribution, where “best fit” is apparent.
- Using an average or weighted average of all reasonable distributions.
- Using the most conservative result from a set of “acceptable” distributions.
- Providing confidence bands for the estimates for the return period storm(s) of interest.

It is always good practice to plot the observed peaks at empirical plotting positions and the computed frequency curve on the relevant probability plot to gain a visual impression of the frequency curve. In some cases, a broken line effect is indicated which is not described by any theoretical distribution. If this occurs, the data must be thoroughly

examined to establish the cause of the non-homogeneity of the sample. There are several possible reasons for this such as moving of a gauging station, change in regulation patterns, urbanization, climate change, or other changes in watershed characteristics which should be considered prior to analysis. It is also possible that the sample may be biased by the inclusion of several rare events within a short time-period. It is most likely, however, that the explanation for the broken line effect will be that peak flows were caused by two or more flood-generating mechanisms. In this case, the flood series contains samples from more than one population and should be treated as such.

Regardless of the final choice for the analysis, practitioners should document and justify the choice of analysis methods, the data used to derive the best-fit distribution, and the method used to develop the model parameters.

3.2.3 Regional Flood Frequency Analysis

A flood frequency regime based on the analysis of data from a single station may not accurately represent the regional characteristics. A regional flood frequency analysis tends to overcome this problem by including data samples from several stations within ideally a hydrologically homogeneous region. A number of tools exist that may aid in performing a regional flood frequency analysis, including best available information on [GeoHub](#) or programs such as [FloodNet RFA](#). Caution should be used to ensure programs or tools used to aid in the regional flood frequency analysis are kept up-to-date and maintained.

This type of analysis should be used when the record for the stream in question is short or unreliable and in cases where there is some doubt of the validity of the single station frequency curve. In cases where there is little or no data for the stream, a regional frequency is a possible solution; it may also be used when a stream is gauged at some distance from the reach to be mapped.

The first task in carrying out a regional frequency analysis is to determine the extent of the homogeneous region, as the homogeneity of a group of stations in a regional analysis is an essential assumption. This must be based initially on knowledge of meteorological and physiographic conditions so that all streams used would appear to have similar runoff characteristics, with confirmation through tests of statistical homogeneity. For instance, the Hosking-Wallis procedure based on the variability of L-moment ratios (Hosking and Wallis, 1993; Hosking, 1990). For each stream in the region with sufficient length of record (at least 10 years), a single station frequency analysis is carried out, accounting for all the factors described earlier. The same probability distribution must be used in all cases, so one must be chosen, that is applicable to the region. It is common to use a two-parameter

distribution for regional analysis as the necessity for estimating a regional coefficient of skew is avoided; however, a three-parameter distribution should be used if indicated by the single station analyses.

At this point, a technique must be chosen to develop regional characteristics from the single station frequency curves. There are three methods that are commonly used, to develop these regional characteristics including: (i) index flood method, (ii) estimating return period floods directly by multiple regression analysis, and (iii) estimating distribution parameters. Each of these methods is outlined further in the proceeding sections.

3.2.3.1 Index Flood Method

For each of the stations used in the analysis, a dimensionless frequency curve is developed by plotting floods of various return periods as ratios of the mean annual flood. The mean used should be that of the distribution rather than the sample mean. Confirmation of the homogeneity of the region should be made prior to conducting the analysis. Applicable confidence limits must be computed.

Once it is established that all streams are within a homogeneous region, a dimensionless regional flood frequency curve can be developed. For each of several return periods, the median of the individual ratios to the mean annual flood is determined. These median values, when plotted on the appropriate probability paper, define the dimensionless regional flood frequency curve.

The final step entails developing and verifying an equation to estimate the mean annual flood for any stream within the region. Multiple regression analysis or an envelope curve is used to develop a relation between the mean annual flood and various physiographic and climatic parameters. In the original Index Flood Method, the mean annual flood was related to the drainage area only. When additional variables such as mean annual runoff, channel slope, basin slope, areas of lakes and swamps and soil characteristics are considered, it becomes a modified index flood method. While the use of the modified index flood method has been advocated for use in the past by MTO and others, this method has been superseded through the release of the Unified Ontario Flood Method by MTO (2016). Use of the modified index flood method is advocated for data checking and corroboration purposes only.

3.2.3.2 Estimating Floods of Various Return Periods by Multiple Regression Analysis

A second technique is to estimate floods of specified return periods directly through multiple regression analysis. The independent variables chosen are similar to those used

to estimate the mean annual flood, but a series of relationships is developed; one for each of selected return periods. The dependent variables used in the analysis are floods of the selected return periods.

Since the same meteorological and physiographical data sets are used by both the modified index flood and multiple regression analysis methods, the flood estimates for various return periods are generally expected to be similar, but this depends on the primary variables defining respective regression methods. The regression parameters and equations developed by Moin and Shaw (1986) have not recently been validated, and flood values calculated using those methods should only be used for basic data checking purposes, rather than primary data sources. This does not preclude practitioners from undertaking studies to verify the veracity of these equations or undertaking rigorous studies to derive new regression methods using current streamflow, meteorological and physiographic variables.

3.2.3.3 Estimating Distribution Parameters using Multiple Regression Techniques

A third technique involves using multiple regression techniques to estimate regional values of the parameters of a probability distribution. The relevant parameters are first derived for each of the individual stations in the region, then regression equations are developed to estimate them for ungauged streams. These are generally done for the purposes of selecting the best distribution or delineating a homogeneous region.

3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins

In many cases there will be no gauge on a stream at the location where mapping of the flooding hazard is required. If there is no gauge on the stream at all, or the gauge is far from the desired location, regional techniques must be used to estimate flood magnitudes. If a stream is gauged sufficiently close to the required point, a transfer of flows by streamflow routing or simpler techniques can be achieved.

Streamflow routing should be used if there is significant storage between the gauging station and the required control point. Routing can be carried out either prior to or after the frequency analysis. In the first case, recorded flood hydrographs are routed to the desired location, then a frequency analysis is carried out on the routed peak discharges. In the latter case first a frequency analysis is carried out, then the estimated flood is routed through the system. It becomes necessary to develop a complete flood hydrograph for this purpose rather than only estimating a peak discharge.

3.2.4.1 Simple Flow Transposing/Flow Proration

The drainage-area ratio method is predicated on the assumption that the streamflow for a site of interest can be estimated by multiplying the ratio of the drainage area for the site of interest and the drainage area for a nearby streamflow gauge by the streamflow for the nearby streamflow gauge. Depending on the temporal characteristics of the streamflow data used (e.g., hourly, daily, monthly, seasonal) these data can serve various purposes including predicting flow hydrographs in ungauged basins as well as predicting flood peaks in ungauged basins using flood frequency analysis. The method estimates peak flow at an ungauged site by multiplying the measured peak flow at the nearby reference gauging station by the area ratio of the ungauged to gauged watersheds:

$$Q_d = Q_s \left(\frac{A_d}{A_s} \right) \quad (3.1)$$

where:

Q_d = peak flow at the destination, ungauged location.

Q_s = peak flow at the source, gauged location.

A_d = catchment area at the destination, ungauged location.

A_s = catchment area at the source, gauged location.

When used to estimate peak flows at an ungauged location annual maxima data can be used and subsequently be analysed using FFA to derive return period flows at the ungauged site.

A variant of the drainage area-ratio method is the Multi gauging station Drainage Area Ratio (MDAR). In the MDAR method, the weighted sum of more than one streamflow gauging station is used to estimate the flow at the site of interest.

There are a series of key requirements for implementing the above approaches. Specifically, the source and destination catchments must be hydrologically similar, and include similar surficial geology, soil type, landcover, annual precipitation as well as watershed slope and the percentage of the basin comprised of lakes and wetlands, among other attributes, to employ this method. Drainage area is a boundary criterion when applying this method, whereby the difference between the source (gauged) and destination (ungauged) catchment size should not exceed approximately 25% (Moin & Shaw, 1985; McCuen et al., 2002; CDA, 2007).

The major assumption of the area ratio method is that flow scales directly with watershed area, such that as watershed area increases, flow rate increases at some fixed rate per unit area. This assumes that the flow per unit area is the same at both the ungauged location and gauged reference location. This simple flow transposing method should only be used as a checking or data corroboration method, rather than a primary data derivation method.

3.2.4.2 Spatial Interpolation of Flow Time Series using Flow Duration Curves and Flood Frequency Analysis

A more robust spatial interpolation can be used to transfer a known streamflow time series from one or more locations to estimate flood magnitudes in ungauged locations or those with limited observed flow data. This method as applied in Ontario, uses flow duration curves (FDCs) from observed flow data, which may be supplemented by observed rainfall records, to illustrate the proportion of time that specified discharges or flow is equaled or exceeded within a given period (observed or simulated) (Metcalf et al., 2003; 2005). This includes three steps: 1) estimate a regional FDC; 2) calculate an FDC for the specified site; and 3) generate a continuous streamflow hydrograph for the specified site using one or more representative streamflow gauges. A fourth step, 4) undertake a flood frequency analysis to estimate flood flows under the desired return period, would be added for the purposes herein. This method is subject to many of the requirements highlighted for flow transposing (e.g., similar hydrology, location, physiographic setting) (Metcalf et al., 2005). This, among other methods are profiled in Metcalf et al. (2013). This method is analogous to the Pragmatic Method used by Gupta and colleagues (2014), in Saskatchewan and as adapted by Shu and Ouarda (2012) for use in Quebec. Mohamoud (2008) highlighted the improved predictive performance of these FDC-based approaches relative to the area-ratio method profiled in [Section 3.2.4.1 Simple Flow Transposing/Flow Proration](#), also finding that the FDC based method showed less predictive uncertainty compared to the area-ratio method.

The method is derived from Hughes and Smakhtin (1996) who developed a spatial interpolation approach based on a FDC for patching or extending existing daily flow records. Over a range of river catchments with different physiography the method was found to perform at least as well as a more complex semi-distributed daily rainfall-runoff model in terms of several goodness-of-fit criteria applied. Smakhtin and associates (1997) and Smakhtin (1999) extended this technique for daily flow time-series generation at ungauged sites using regional FDCs constructed from limited observed flow data, which exist in a specified region. Smakhtin and Masse (2000) further extended the technique by including duration curves of observed rainfall records where flow records were not available.

Findings from other areas of Canada confirm that the FDC spatial interpolation approach can produce robust flow simulations. One study identified that the FDC approach compared favourably to an implementation of a comprehensive spatially distributed, physically based hydrological model (Gupta et al., 2014). In another study, analyses between the drainage area ratio method (i.e., traditional flow transposing) and the spatial interpolation FDC method, highlighted that the FDC-based method produces much more robust flow estimates (i.e., improved statistical performance) when summary statistics were compared (Shu & Ouarda, 2012). The FDC spatial interpolation approach has recently been implemented for flow prediction at ungauged basins in Alberta (Alberta Environment & Parks, 2022). Undertaking a flood frequency analyses on simulated data generated by these methods (using the approaches outlined in [Section 3.2.2](#)) can assist in acquiring the necessary flood peak information, for return period-based flooding hazard standards.

3.3 Runoff Simulation from a Specified Meteorological Event

When the flooding hazard is based on a specified input rather than on frequency analysis, it becomes necessary to convert the input data into discharge values. The first part of the process is to determine the likely meteorological event. Among the two likely inputs are an historical regional storm (the Hurricane Hazel or the Timmins storm) and a combination of snowmelt and precipitation.

If the specified input is to be a combination of snowmelt and precipitation, it is necessary to document the rates and areal extent of each, and to justify the specified values. It should be shown that such a combination is realistic by comparison with recorded meteorological data in the region.

The second major part of the process is the conversion of the precipitation/snowmelt input to basin runoff, and runoff to river discharge at the location required. This involves the use of a hydrologic model of the watershed of which there are many types currently available. Most modelling software are adequate for the region in which they were developed and for the size of the watershed they were designed to handle. Applying them to different regions and larger or smaller watersheds than recommended is not always successful, therefore, great care should be taken when choosing software. Watershed models can vary from those based on a triangular unit hydrograph to those that attempt to describe every aspect of the hydrologic cycle. In general, it is preferable to use the simplest model (i.e., having a minimum number of parameters) that is adequate to simulate observed discharges over a range of storm events. In many of the more complex

systems, enormous amounts of data are required apart from the basic physiographic and meteorological characteristics. Data on evaporation, soil moisture, infiltration rates, groundwater storage, and alike, are not available in many cases and so the model parameters must be estimated. Thus, it is common that the majority of the parameters are estimated rather than measured, which leads to a low level of confidence in the results.

Any model applied to a particular watershed should consider factors which have a major influence on the runoff characteristics. It should have the capacity to adequately describe the main physiographic aspects of a watershed as well as the effects of channel and lake storage and potentially groundwater influence, where groundwater is expected to be or has been shown to have a significant contribution to flood flows. Furthermore, the model should have the ability to incorporate those types of artificial regulation that may exist in the basin under study. This supports calibration of the model to observed conditions including streamflow gauges, prior to removing sources of artificial storage and regulation in the model when deriving the flooding hazard magnitudes.

As the major meteorological input to a model in this case may be a particular documented storm, it should be able to operate on a time scale that will both analyze the precipitation data and synthesize discharges at such intervals as are relevant for the watershed. For large basins (e.g., $>2000 \text{ km}^2$), daily computations will generally suffice; for small basins, however, the time interval may be very small. Also of concern is the type of input to be analyzed. If it is a storm that can only occur in summer or fall, no provision for snowmelt is required. If, however, the input is a specified combination of snowmelt and precipitation, a model must be chosen that can take both into account.

3.3.1 Calibration and Validation

The best test of a watershed model lies in its ability to adequately reproduce recorded flows from storms similar in magnitude to that of the selected flood. For the purposes of a flood hazard mapping program, ideally a hydrologic model is put through adequate testing for both calibration and validation of the model parameters. Generally, hydrologic models are calibrated for a basin by successive attempts at reproducing recorded data while varying those parameters that are not fixed, within reason, until an adequate reconstitution is developed. This procedure is followed for several historical events of varied magnitude so that model calibration is not based on a single sample. If sufficient records exist on the stream, several recorded discharges should be run, independent of the calibration runs, to verify the model. If there are no records on the stream in question, the model should be calibrated and tested on a similar nearby watershed where the variable parameters can be assumed to match those of the basin under study.

Where reliable long term monitoring data are not available, or where historical flow data do not capture flows comparable in magnitude to those derived/calculated for the flooding hazard, calibration and validation should be accomplished using a range of supported methods. These methods are elaborated upon further, below (also see [Section 3.6.5 Model Calibration and Validation](#)).

It is known that the runoff characteristics of a watershed can vary widely depending upon the quantity and intensity of precipitation, and the antecedent conditions, (i.e., the non-linear watershed response). If the synthesis of stream flows is to be based on the largest storm of record in a particular region, the calibration of model parameters should be attempted using other large storms rather than lesser events. Also, the antecedent conditions assumed should match those in effect prior to the occurrence of the historical storm.

One appropriate method can include constructing a rating curve using a hydraulic model (e.g. HEC-RAS) employing water surface slope, channel cross section geometry, calibrated to a reasonable assumed Manning's 'n' value. This can prove to be a useful and reliable tool at high water levels, that may not have been observed or gauged.

If a combination of snowmelt and precipitation is specified, model parameters relevant to snowmelt should initially be estimated from rainfall-free events. Similarly, estimates of rainfall-runoff parameters should be based on events free from snowmelt. Calibration and validation of the model should then be finalized using recorded discharges caused by both elements.

Stream flow simulation consists of the derivation of a discharge by transforming a meteorological input into a hydrological output. The simulation methods generally characterize the drainage basin of three separate types of storage elements: watershed, lake/reservoir and channel considered either in series or in parallel. The watershed storage input is represented by rain or snowmelt. Abstraction from gross water input will produce the runoff portion of the channel inflow. The total channel inflow includes a baseflow component. The channel storage is computed from channel routing representing a reach of the river, with input at the upstream end and the local inflows along the reach. The outputs are the downstream discharge. Reservoir/lake storage is computed using river inflow as input; the output represents the storage and outflow. The routing involves computation of the hydrograph modification as the water flows through the storage elements, decreasing and delaying peak flows, and therefore, extending the flow duration of the hydrograph.

Reservoir/lake routing transforms stream flow (peak or hydrograph) by its passage through the reservoir/lake.

Compared to the statistical estimates of peak flows, and corresponding standard error of estimates, errors in estimates obtained from streamflow simulation are not calculated. Streamflow simulation to establish flood hazard limits depends on the use of computer models, that mathematically represent the complex physical process inherent in flood generation. Thorough understanding of the flood characteristics of the river and the watershed and the calibration/validation of the model is essential to derive credible results. The user must also understand the calculations, processes and limitations of the software used in the streamflow simulation.

When flood flows and levels are computed for different land use scenarios (such as existing and future development conditions, including full-build-out scenarios) (See Section 2.2), abstractions should be computed separately for lands likely to be affected by different land use scenarios, to capture the range in runoff for the various combinations of scenarios that may occur. Infiltration in frozen grounds is generally assumed to be zero in modelling. This assumption is only valid for conditions when the ground is frozen solid.

A more detailed discussion of flood computation using hydrologic models is presented in [Section 3.6 Hydrologic Modelling of Flood Flows to Define the Flooding Hazard](#).

3.4 Rainfall Analysis

3.4.1 Introduction

The following discussion is a collection and presentation of design storms and distributions applicable to hydrologic studies within Canada, and particularly suitable for use in Ontario; and is intended to assist the user in simulating runoff from precipitation. It does not attempt to select a universal or standard distribution of storms. Rather, it attempts to present and compare storms and distributions so that the user may select the one which is the most suitable for the study.

Apart from the storms used for computing floods (generally the Hurricane Hazel, Timmins and 100-year storms), information is also provided on frequently occurring storms and distributions.

The storms and distributions have been grouped into two categories. The first, **Historical**, includes recorded events, while the second, **Return Period**, includes storm distributions and rainfall depth of a specific frequency or return period.

Environment and Climate Change Canada (ECCC), including Meteorological Services Canada (MSC) and the former Atmospheric Environment Service (AES), provide atmospheric data and analyses across the country.

The following characteristics will be used to describe each of the above storms:

- Total precipitation.
- Duration.
- Temporal distribution.
- Time step.
- Areal reduction.

The purpose of using a storm in the computation of floods is to generate simulated runoff for a specific event. A number of storms and rainfall distributions have been developed and are used to generate simulated runoff.

To compute the extent of flooding hazard limit, one may use either of the two Historical storms (i.e., Hurricane Hazel and Timmins storm) described under the flooding hazard standards or a return period storm depending on the zone the watershed is located in ([Figure 1-1](#)). [Figure 3-2](#) presents a decision diagram to assist the user in the selection process.

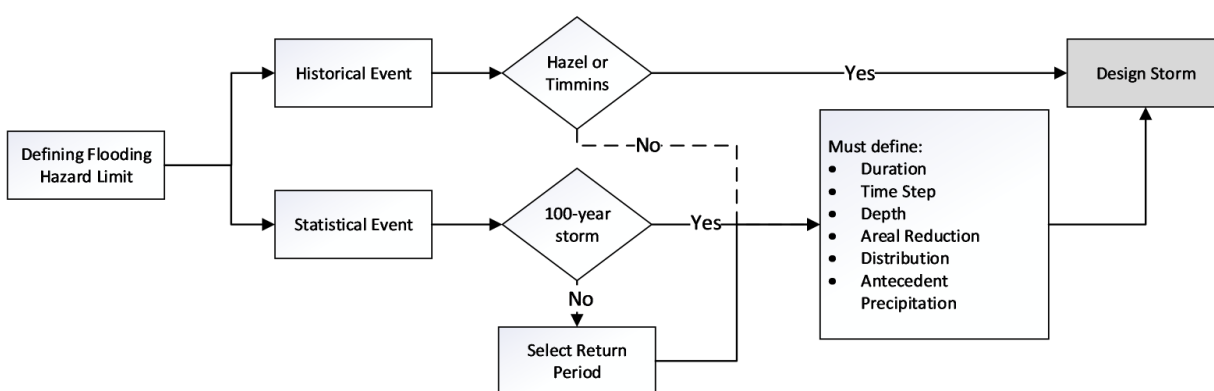


Figure 3-2 Flooding Hazard Standard Selection (segmented lines and boxes highlight where another return period or other storm has been approved for use by the Minister)

A summary of storms used in the past in Ontario are presented in [Table 3-1](#) and [Table 3-2](#). The variation in the duration of the storms is from 1 hour to 30 days. Many of the computed intensity-duration-frequency values do not compare very well with the published MSC data, as indicated in the table. In such cases, except for Hurricane Hazel or Timmins, storms of different durations may have to be applied, if applicable, in order to obtain the maximum peak flow rate for the selected frequency event.

Table 3-1 Comparison of Historical Storms used for Flood Hazard Computation

Historical Storms	Duration	Time Interval	Comparison with IDF Curves	Difficulty to Determine	Difficulty To Apply	Antecedent Precipitation*	Areal Reduction
Hurricane Hazel	48 hour	1 hour	Poor	Low	Low	Recorded	Recorded
Hurricane Hazel	36 hour	1 hour	Poor	Low	Low	Recorded	Recorded
Hurricane Hazel	12 hour	1 hour	Poor	Low	Low	Recorded	Recorded
Timmins storm	12 hour	1 hour	Poor	Low	Low	Recorded	Recorded
Severe Single Event	User Defined	User Defined	Poor	High	Low	Recorded	Recorded
Series of Single Events	User Defined	User Defined	Poor	High	High	Recorded	Recorded

Notes:

**The amount of run-off generated by a storm depends on the soils and land use in the watershed. The amount of rainfall lost to infiltration also depends on the moisture condition of the soil. If the soil is already saturated, the run-off will be much greater. The soil moisture condition is known as the Antecedent Moisture Condition (AMC). For determination of the following events defining the flooding hazard, the AMC's on a watershed must be as identified:*

Hurricane Hazel AMC III: wet condition; watershed nearly saturated

Timmins Storm AMC II: average condition

The selection of the appropriate antecedent condition for other design storms depends on the condition being simulated and requires the judgment of the engineer.

Table 3-2 Comparison of Return Period Storms used for Flood Hazard Computation

Return Period Storms	Duration	Time Interval	Comparison with IDF Curves	Difficulty to Determine	Difficulty To Apply	Antecedent Precipitation*	Areal Reduction
Huff	Variable	Variable	Poor	Low	High	User Assigned	User Assigned
Keifer & Chu (Chicago)	Variable	Variable	Good	Medium	Low	User Assigned	User Assigned
SCS Type II	24 hour	15 min	Fair	Low	Low	User Assigned	User Assigned
	12 hour	15 min	Fair	Low	Low	User Assigned	User Assigned
	6 hour	15 min	Poor	Low	Low	User Assigned	User Assigned
MSC - 1 Hour	1 hour	5 min	Poor	Low	Low	User Assigned	User Assigned
MSC - 12 Hour	12 hour	1 hour	Poor	Low	Low	User Assigned	User Assigned
Snowmelt	1-30 days	1 day	NA	Medium	Low	User Assigned	User Assigned

Notes:

The amount of run-off generated by a storm depends on the soils and land use in the watershed. The amount of rainfall lost to infiltration also depends on the moisture condition of the soil. If the soil is already saturated, the run-off will be much greater. The soil moisture condition is known as the Antecedent Moisture Condition (AMC). For determination of the following events defining the flooding hazard, the AMC's on a watershed must be as identified:

100 year AMC II: average condition

The selection of the appropriate antecedent condition for other design storms depends on the condition being simulated and requires the judgment of the engineer.

Acronyms: SCS = Soil Conservation Services ; MSC = Meteorological Services Canada

3.4.2 Historical Storms

Two of the floods suitable for flooding hazard delineation are based on historical storms called “Hurricane Hazel” and “Timmins storm”, both of which are described below. For the meteorologically-similar region in which each applies, the Hazel and Timmins storm events are to be transposable from the location in which they occurred to any watershed within that region as shown on [Figure 1-1](#). The storms are to be centered over the watershed for which the flooding hazard is to be calculated so as to result in maximum run-off.

3.4.2.1 Hurricane Hazel

Hurricane Hazel was a tropical storm of 48 hours’ duration; the first 36 hours of which were of low intensity (i.e., $\approx 2\text{mm/hr}$) and created near saturated conditions on the watershed but did not produce high stream flow. The last 12 hours of the storm were of high intensity and produced the maximum run-off and peak stream flow. Hurricane Hazel was adopted by the MNR as the standard to define the flooding hazard for watersheds located within Zone 1 in the area delineated in [Figure 1-1](#) (MNR, 2002). Although the path of Hurricane Hazel was located to the west of Toronto, studies have indicated that it could have occurred anywhere within the delineated area.

The 12-hour design storm ([Table 3-3](#)) was developed from rainfall gauge data located at Snelgrove, just north of Brampton. The storm is to be applied to watersheds with areas less than 25 km^2 as indicated in the [Table 3-3](#). For larger basins, the rainfall amounts listed in [Table 3-3](#) are to be modified by the areal reduction factor (ARF) percentages shown for different drainage areas in [Table 3-4](#). ARFs are used to reduce the point rainfall amounts to reflect the way rainfall amounts tend to fan out and diminish from the eye of a storm.

The Equivalent Circular Area (ECA) method accounts for typical basin elongation by incorporating the longest length of the watershed from the point of interest to the further upstream portion of the catchment (i.e., as the diameter). This empirical fixed-area ARF method, was first implemented in the areal reduction factor approach of Myers & Zehr (1980). The diameter of the area-equivalent circle and the centroid location of the footprint of rainfall data/storms are commonly used in meteorology, for simplicity, as a substitute for the actual elliptical footprint of gridded rainfall (e.g., Mesnard & Sauvageot, 2003; Hirose & Okdada, 2018). Similarly, The ECA approach is used in hydrological modelling to avoid overestimating runoff for large watersheds when using point rainfall or hyetograph data, by accounting for the fact that high intensity rainfall does not cover large areas uniformly.

In the case of extremely elongated watersheds, the isohyetal technique can also be applied to determine the rainfall input for the runoff simulation. This method provides the rainfall depth which falls on each portion of the drainage area. This is computed by: 1) measuring the areas between adjacent isohyets, 2) calculating the mean rainfall for each area, 3) multiplying each calculated mean rainfall by its respective area, 4) summing all the products, and 5) dividing by the total catchment area. [Figure 3-3](#) shows the isohyets developed following Hurricane Hazel based on rainfall gauge data. Isohyets can be transposed and centered over the drainage area being studied.

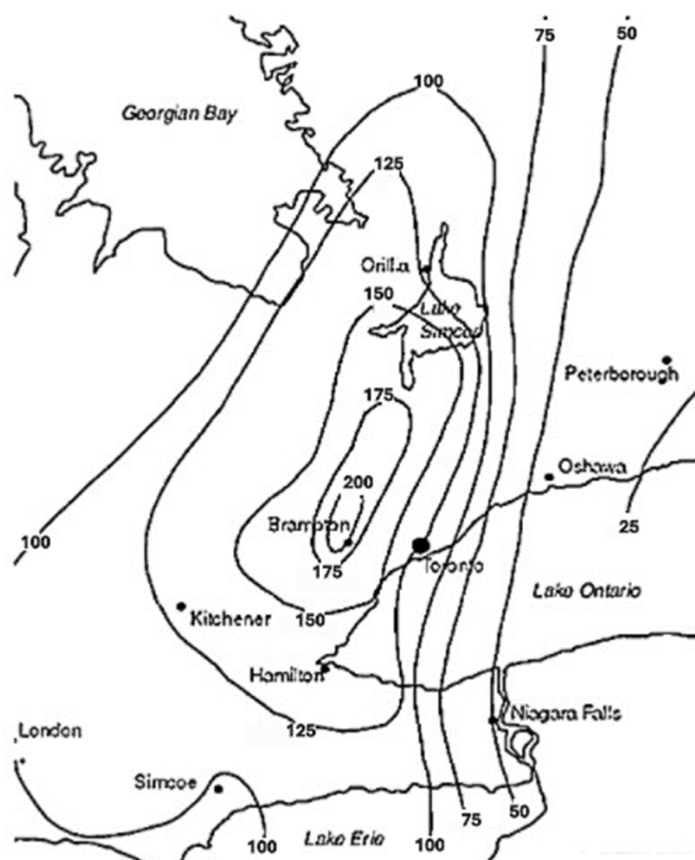


Figure 3-3 Hurricane Hazel Isohyets (Source: adapted from Mason et al., 1955)

A hyetograph and a dimensionless storm distribution curve of the 12-hour design storm (36th and 48th hours) are shown in [Figure 3-4](#).

For illustrative purposes, rainfall records were used to produce areal reduction curves for the 6, 12, 24, 36, and 48-hour portion of Hurricane Hazel for a range of drainage areas, as illustrated in [Figure 3-5](#).

Table 3-3 Hurricane Hazel, 36th through to 48th Hour Rainfall Depths

Timing	Depth (mm)	Percent of Last 12 hours
First 36 hours	73	-
37th hour	6	3
38th hour	4	2
39th hour	6	3
40th hour	13	6
41st hour	17	8
42nd hour	13	6
43rd hour	23	11
44th hour	13	6
45th hour	13	6
46th hour	53	25
47th hour	38	18
48th hour	13	6
TOTAL	285	100

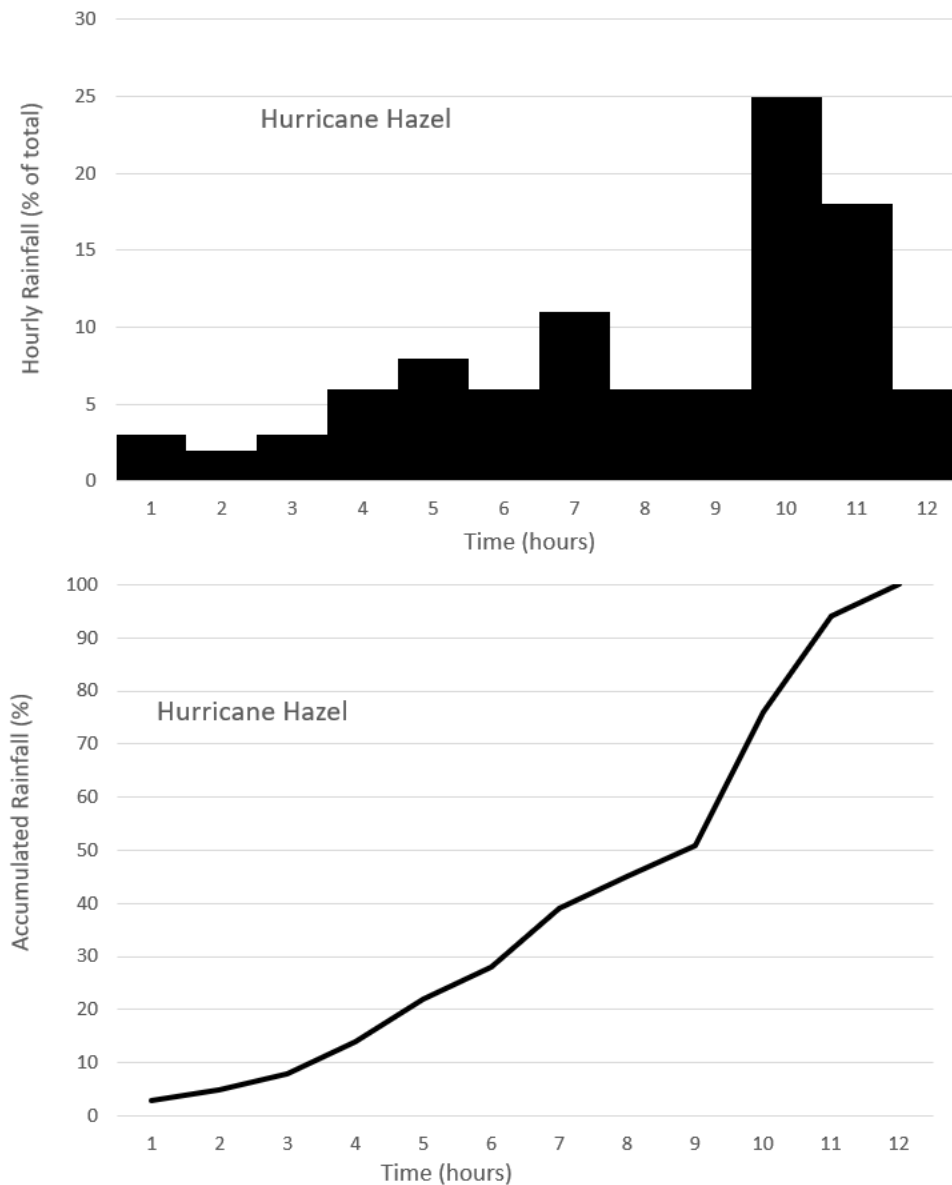


Figure 3-4 Hurricane Hazel 12 Hour Hyetograph (a) Hourly % Rainfall (of Total) and (b) Accumulated Rainfall (%) of Total

Table 3-4 Hurricane Hazel - Areal Reduction

Area (km²)	Reduction Factor Percentage (%)
0-25	100.0 (no reduction)
26-45	99.2
46-65	98.2
66-90	97.1
91-115	96.3
116-140	95.4
141-165	94.8
166-195	94.2
196-220	93.5
221-245	92.7
246-270	92.0
271-450	89.4
451-575	86.7
576-700	84.0
701-850	82.4
851-1000	80.8
1001-1200	79.3
1201-1500	76.6
1501-1700	74.4

1701-2000	73.3
2001-2200	71.7
2201-2500	70.2
2501-2700	69.0
2701-4500	64.4
4501-6000	61.4
6001-7000	58.9
7001-8000	57.4

Note: Reduction factor to be multiplied by the rainfall depth for watersheds larger than 25km².

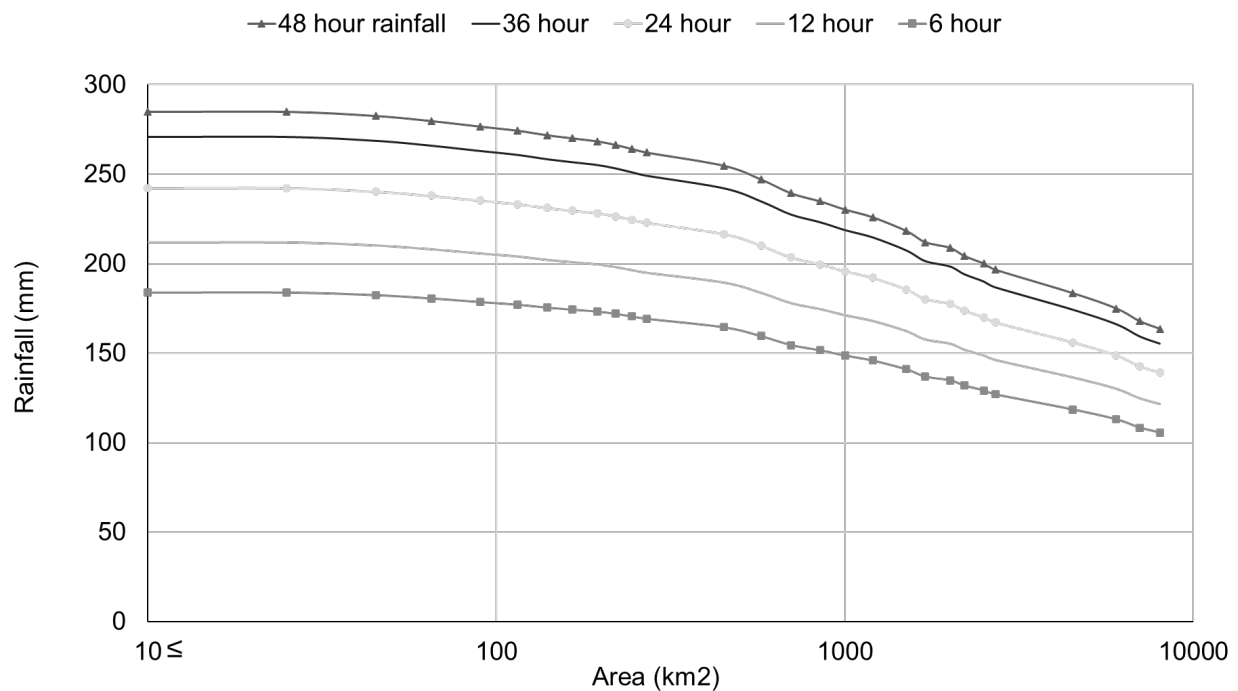


Figure 3-5 Hurricane Hazel Area Reduction

3.4.2.2 Timmins Storm

“Timmins storm” is the name applied to the summer storm which occurred over Timmins, Ontario on September 1, 1961. The event is described in Circular 3746 of the Meteorological Branch of the Department of Transport (McMullen, 1962). The storm created severe property damage and resulted in loss of life on the banks of Town Creek.

The 12-hour design storm ([Table 3-5](#), [Figure 3-6](#)) was developed from information assembled from several gauges located in Timmins. It was adopted by the MNR, as the storm to produce the flood standard for watersheds less than 25 km² in size within Zone 3 delineated in Figure 1-1. Similar to the Hazel storm, an areal reduction is applied to the Timmins point rainfall, for drainage areas larger than 25 km² as shown in [Table 3-6](#), based on the 24 hour isohyets.

Table 3-5 Timmins Storm - Rainfall Depths

Hour	Rainfall Depth	Percent of last 12 hours (%)
1st	15	8
2nd	20	10
3rd	10	6
4th	3	1
5th	5	3
6th	20	10
7th	43	23
8th	20	10
9th	23	12
10th	13	6
11th	13	7
12th	8	4
TOTAL	193	100

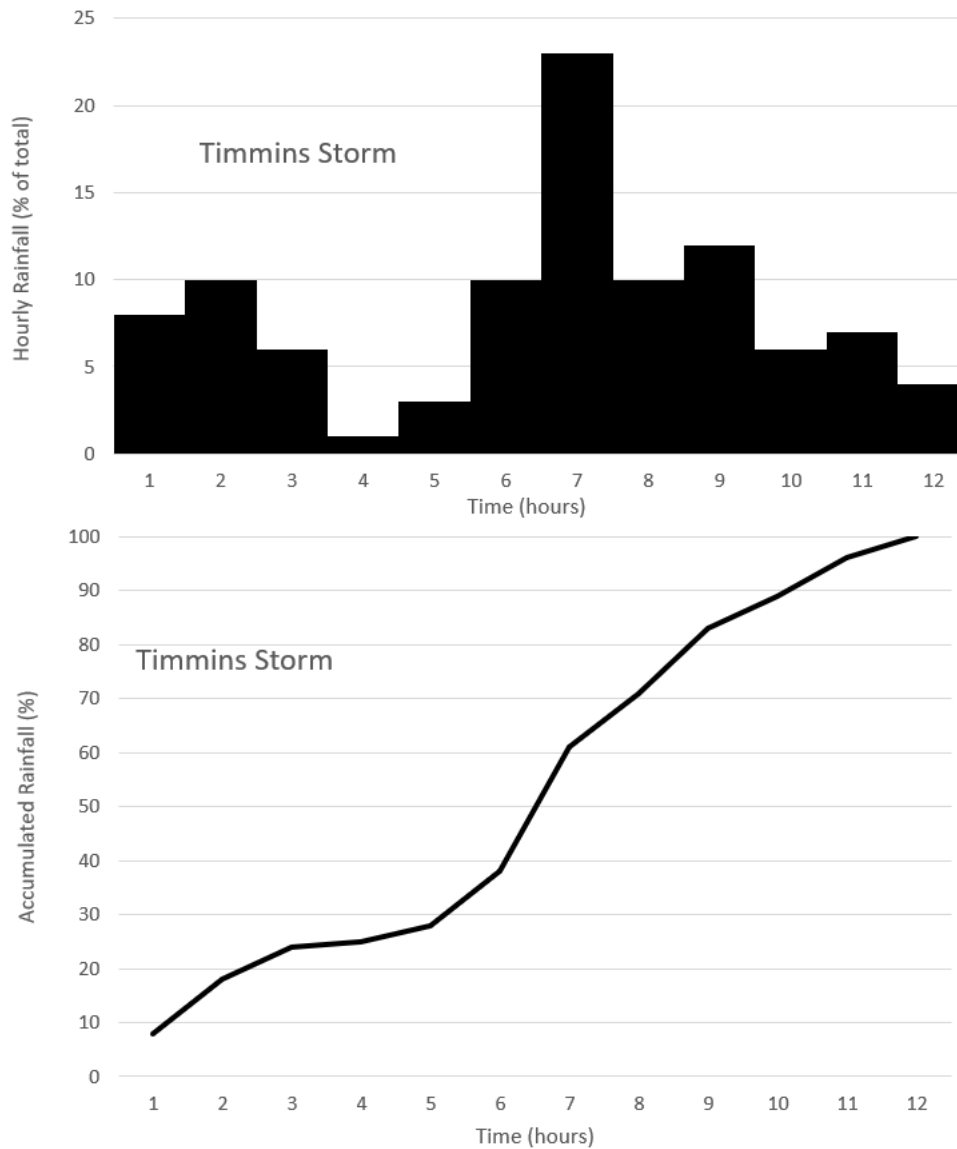


Figure 3-6 Timmins Storm 12-Hour Hyetograph (a) Hourly (% of Total Rainfall)) and (b) Accumulated Rainfall Total (%)

Table 3-6 Timmins Storm – Areal Reduction

Area (km²)	Reduction Factor Percentage (%)
0-25	100
26-50	97
51-75	94
76-100	90
101-150	87
151-200	84
201-250	82
251-375	79
376-500	76
501-750	74
751-1000	70
1001-1250	68
1251-1500	66
1501-1800	65
1801-2100	64
2101-2300	63
2301-2600	62
2601-3900	58
3901-5200	56

5201-6500	53
6501-8000	50

Note: Reduction factor to be multiplied by the rainfall depth for watersheds larger than 25km².

3.4.3 Use of Return Period Storms

The 100-year flood is determined statistically by a frequency analysis of maximum stream flows on record for the river(s) in the watershed under consideration where a sufficiently long period of record is available, and rather than from rainfall records. Particular attention should be given to the Harrow Storm (1990) in Southwestern Ontario, the Peterborough Storm (2004) in East-Central Ontario and the 49th Parallel Storm (2002) in Northwestern Ontario in the determination of the 100-year return flood event.

Where very short records or no stream flow records are available, a synthetic hydrograph derived from the 100-year rainfall may be used for small watersheds. Methods for deriving synthetic flood hydrographs for the 100-year return period snowmelt or snowmelt plus rainfall for larger watersheds where few or no stream flow records are available.

However, as for all statistically determined floods where synthetic hydrograph methods are used to derive them, values to be used in and results obtained from these methods must be compared and correlated with values obtained from analyses of stream flow records in the same or adjoining watersheds, adjusted for differing watershed size and variation in watershed characteristics.

The second group of storms used for flooding hazard definition are the return period storms. In absence of adequate streamflow records, rainfall data is used to synthesize stream flows. Rainfall records are more frequently available and have longer duration than stream flow records. Although a precipitation gauge can only provide information for a small area, the general characteristics of precipitation usually vary in a regular manner. Once the rainfall information is determined, there are several techniques available to convert it into an estimate of flow.

Intensity-duration-frequency (IDF) curves provide the relationship between rainfall events and return period storms using historical rainfall data. The Ministry of Transportation Ontario has created a tool to search for IDF curves by location, using rainfall data from MSC and National Oceanic and Atmospheric Administration (NOAA) stations located within 150km of Ontario. This tool is available online through the MTO [IDF Curve Lookup](#)

(MTO, 2016a) and may be subject to periodic updates. Gauge data from municipalities and conservation authorities should also be included.

When computing flows from a given return period rainfall, the commonly made assumption is that rainfall of a given frequency produces streamflow of the same frequency. The two most important parameters to define a return period storm are rainfall duration and depth. However, a rainfall event is made up of many components, each with a unique return period.

Similarly, catchment conditions can vary with time, including seasonal variation in land cover (i.e., crops, bare fields, snow cover) and antecedent moisture conditions. These factors can result in rainfall of a given frequency producing streamflow of a different frequency. However, the common assumption that these frequencies are the same is often required. Practitioners should take care to clarify between references to rainfall frequency and references to streamflow frequency.

Methods can be divided into two categories: single event and continuous simulation. The single event can be based on a historical storm, or a synthetic storm based on a statistical analysis of recorded data.

The continuous simulation approach is based on the input of long-term precipitation data, rather than the assumption of a single return period event. The simulation of numerous events will allow a statistical analysis of the simulated flows and thus derive an estimate of the return period for the runoff event.

3.4.3.1 Rainfall Duration

The duration of the required storm varies with the type of analysis. Generally, storm durations should reflect watershed characteristics and should be equal to the time of concentration of the watershed. Large watersheds or small watersheds with storage facilities (dams, reservoirs, significant stormwater storage) may require the use of long duration storms such as 12 or 24 hours.

Smaller watersheds without storage facilities, such as older urban watersheds, may require a 1 to 3-hour duration storm for rainfall-runoff simulation. If in doubt, several storm durations should be tested.

Similarly, the time step should be carefully selected in the computation. Small urban area studies may require 5 to 10-minute time steps. Any further reduction such as a 1 or 2-minute time step may result in unrealistic high flows when used with some of the design

storm distribution, such as the Chicago method. Large watersheds and long duration storms such as a 24-hour storm, may be analyzed with 15-minute, to hourly time steps.

3.4.3.2 Selection of Rainfall Depth

A summary graphic highlighting the spatial distribution of mean rainfall depths throughout Ontario is shown in [Figure 3-7](#). This information can be useful for highlighting areas of the province with a propensity for higher rainfall amounts (on an annual basis).

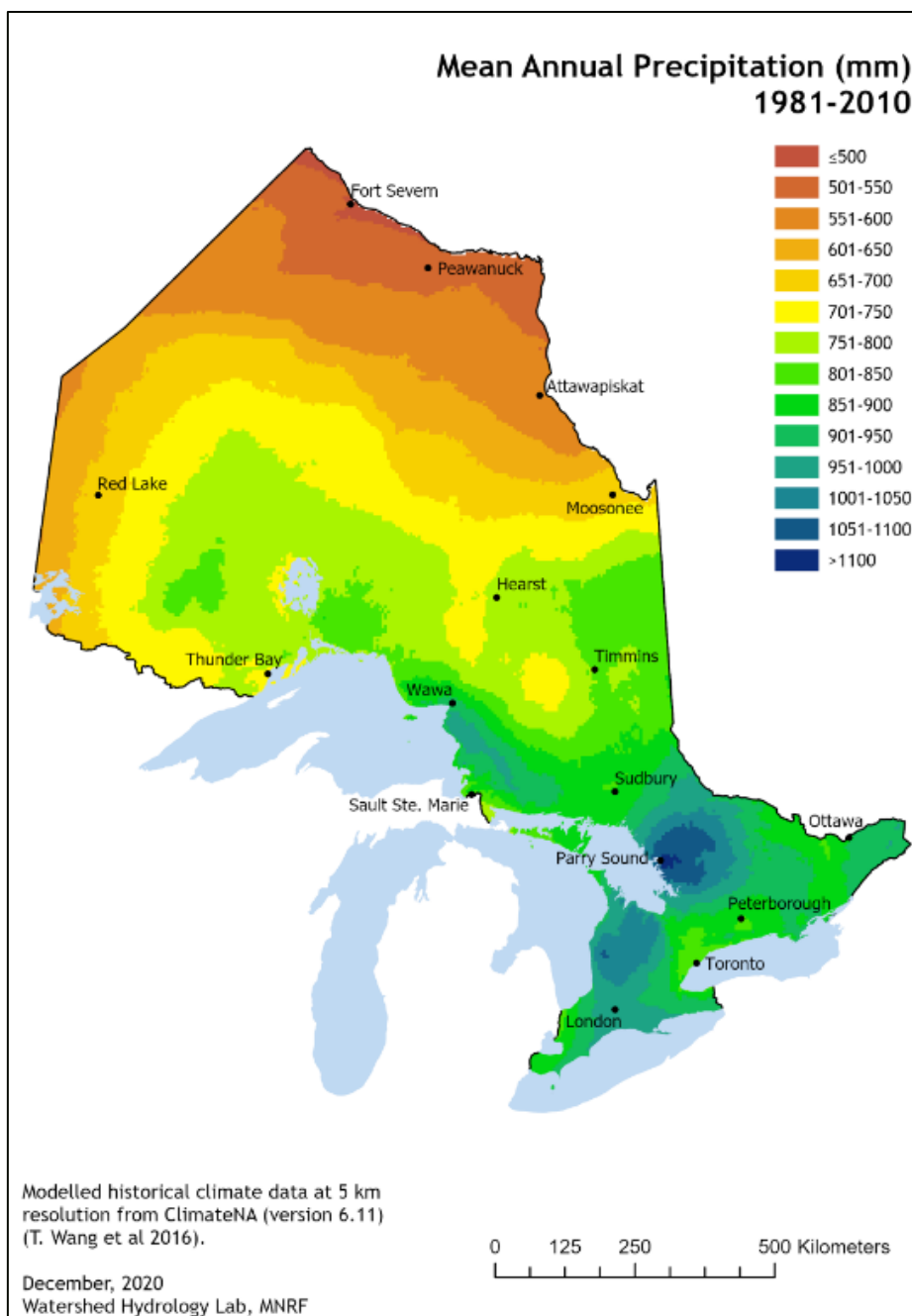


*Figure 3-7 Provincial distribution of mean annual precipitation (mm) for 1981-2010
(Source: Watershed Hydrology Lab, MNR)*

i. Locations Near a Rainfall Gauge

In Ontario, sources of precipitation data may include MSC, conservation authorities, municipalities, and other appropriate sources which provide continuous data on duration and intensity of events. In most cases, there will be at least one such station data available for rainfall-runoff simulation at or near the project to estimate flows. The user should obtain the most recent intensity-duration-frequency curves and tables for the relevant station(s) from the local municipality or CA.

Intensity-duration-frequency curves were traditionally the most widely used form of meteorological data, especially in urban drainage design, because of the dominant role of the Rational Method.

Most designs are based on maximum values drawn from records of complete annual rainfall. There are situations, however, where runoff from winter rainfall on frozen ground, coupled with snowmelt, may exceed runoff from annual maximum rainfall.

Intensity-duration-frequency curves are calculated on data recorded during several independent storms. As an example, the highest depth of rainfall recorded for a 5 and a 10 min. period may have occurred during different storms. The largest rainfall intensity value for each duration for each year of record is used to prepare intensity-duration-frequency curves.

It should be noted that extrapolation of frequency curves for return periods with a insufficient period of record will result in increased inaccuracies. The best practice is to proceed with caution for return periods that are between one half and two times the period of record, and to avoid extrapolating for return periods that are more than two times the period of record. Expressed another way, where N is the period of record in years, caution be applied in extrapolation between $N/2$ (Robson & Reed, 2008) to $2xN$ (Swain et al., 2004) with no extrapolation beyond $2xN$ (Swain et al, 2004).

ii. Determining Rainfall Depth without a Rainfall Gauge

The [MTO IDF Lookup](#) (MTO, 2016a) can be a useful source of rainfall data for areas between existing rainfall gauging stations. In the past, information on rainfall for selected duration and for different return period events between existing stations were presented in Rainfall Frequency Atlas for Canada (Hogg & Carr, 1985). This publication contained maps describing extreme rainfall events, including regional maps of both the mean and standard deviation of annual extremes of rainfall, provided for the following durations: 5, 10, 15, 30, 60 minutes and 2, 6, 12 and 24 hours. The 24-hour duration mean annual

extremes and the corresponding standard deviation for Ontario included in Hogg and Carr (1985) may prove useful to some practitioners where rainfall data are sparse and a comparison to the MTO IDF Lookup Tool (MTO, 2016a) is desired for checking purposes. However, it must be emphasized that these data are older, and the analysis may not be representative of current extreme events. Other modern, up-to-date tools may also be available and appropriate for use in Ontario.

Although limited to southern Ontario, Paixao and associates (2011) regionalized heavy rainfall into nine homogeneous regions based on 24-hour rainfall extremes using data from daily and sub-daily stations. The spatial extent of these homogenous regions may assist practitioners with decisions regarding which rainfall gauge data could be employed, relative to the location of the flood hazard mapping project.

iii. Rainfall Depth – Areal Reduction

In contrast to the Hurricane Hazel and Timmins storms, which have known areal distributions, return period storms require the computation of areal reduction factors.

Intensity-duration-frequency curves describe the variation of point rainfall with time for a given frequency. The curves do not include an adjustment for the variation with area. Examples of two common methods of adjustment are shown in [Figure 3-8](#). The first method is used by the World Meteorological Organization (WMO) (1979), and the second by the U.S. National Weather Service (NWS) (Myers & Zehr, 1980) for non-historical storms. The areal reduction factors that apply in a region should be determined through the analysis of area-specific data.

Rainfall characterization with the watershed and neighbouring watersheds is recommended for insight into the storm patterns that are creating the design storm events. For example, if 1:2-1:10 year return-period events are resulting from springtime rainfalls that tend to be widespread events, less areal reduction would be required for these design storms. Similarly, if 100-year events are a result of highly localized summer convective events, more areal reduction would be appropriate for this design storm.

Areal reduction factor approaches for return period storms follow the same approach as historical events. These include use of the upstream drainage area or with the assumption of a circular area, if required. Isohyets may not be available for return period storms. Refer to [section 3.4.2.1](#) for details on calculating drainage area.

Either method is acceptable to reduce point rainfall, especially for a large area; however, the WMO curves give a ratio of 1.0 for areas of less than 25 km², which is more realistic

than the NWS curves which will converge at 0 area for a ratio of 1.0. Therefore, caution must be taken when using these curves for smaller watersheds. Historical events documented herein demonstrate that recorded peak rainfall remain widespread from the storm centre, prior to any areal reduction.

Generally, for studies undertaken in Ontario, no reduction to point rainfall is used for areas under 25 km².

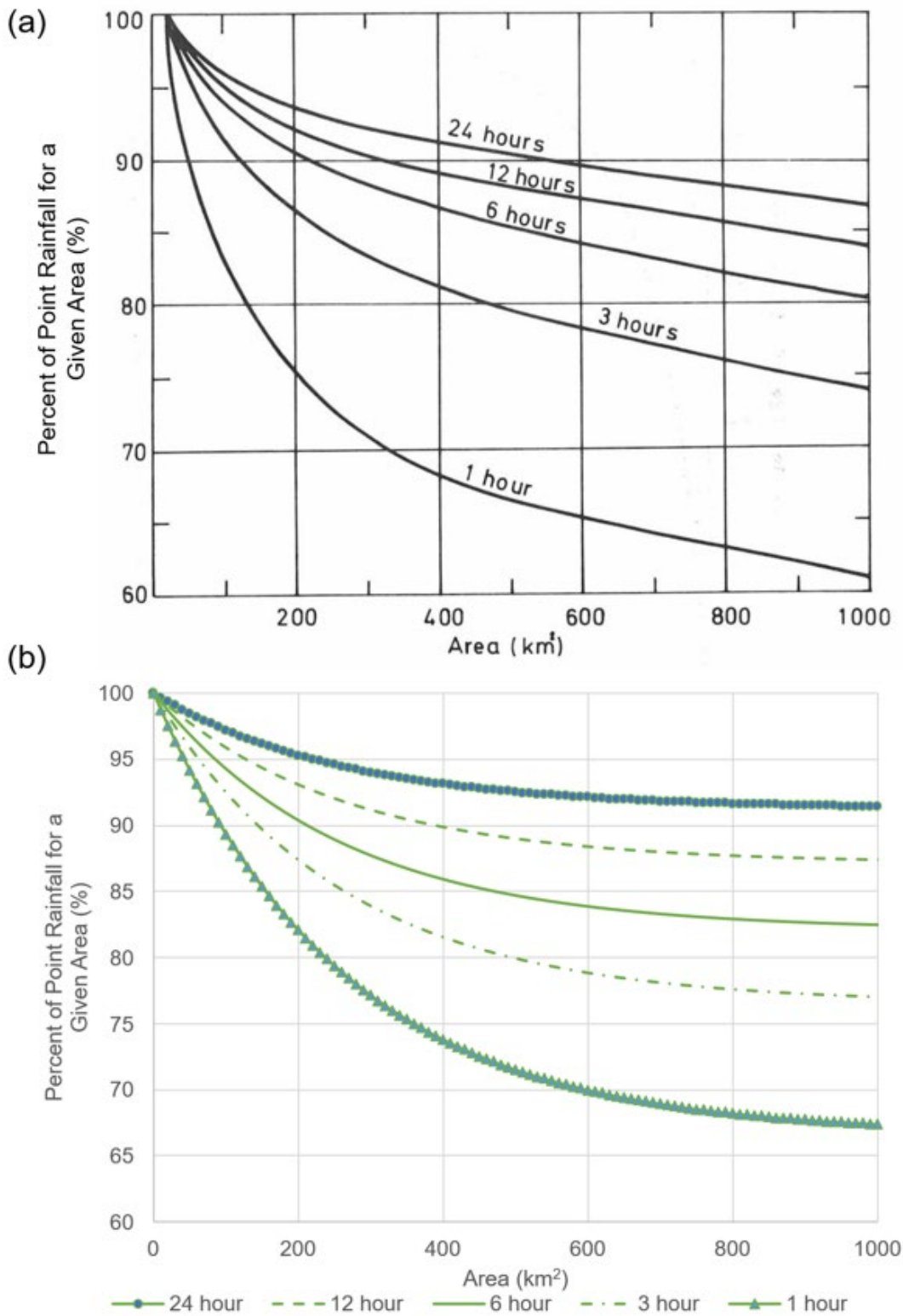


Figure 3-8 Areal Reduction Curves according to a) World Meteorological Organization (1973) and b) U.S. National Weather Service (Myers & Zehr, 1980), Curves

3.4.3.3 Synthetic Design Storm Distributions

Numerous historical and return period storm distributions are available for the practitioner. [Table 3-6](#) summarizes the distributions recommended for the various storms by different agencies.

For the Hurricane Hazel and Timmins storms, the distribution is provided by the MNR. The use of the final 12 hours of the Hazel hyetograph and adjusted hydrologic parameters to represent Antecedent Moisture Conditions (AMCIII) may still be used as appropriate. However, the infiltration and abstraction methods in existing hydrologic models are sufficiently sophisticated to track the moisture conditions. Hydrologic model development and maintenance can be simplified by using the 48-hour hyetograph and avoiding the need for AMCIII parameter calculation.

For Return Period storms, the temporal distribution of the rainfall will determine the peak intensity and time to peak. The most used temporal distributions are (a) those based on Ontario and Canadian data, such as the MSC distribution, or (b) those developed in the U.S., such as the Chicago or the SCS distribution.

The results of a Flood Frequency Analysis (FFA) can be used to assist in the selection of an appropriate design storm distribution. The design storm event volumes are compared against the FFA results in context with the snowmelt plus rainfall volume. The objective is to have a storm distribution that best replicates the volume, peak flow, and timing of the FFA to be used for discrete event modelling. If necessary, a sensitivity analysis can be performed to assess various design storm distributions.

With the appropriate storm distribution selected, the hydrology model can be run with selected events to determine the return period/AEP flood flows at ungauged locations.

i. Canadian Storm Distributions

Meteorological Services Canada staff undertook an investigation on the variability of the time distribution of rainfall in storms across Canada. Data from almost 2,000 extreme events were used to develop regional time-probability distributions for the 1 and 12-hour storms. While these analyses were completed in the 1980s, they remain widely in use today, in part because no substantive updates have been released that update these analyses.

Examples of the 12-hour rain distribution curves produced by MSC for southern and northern Ontario are shown on [Figure 3-10](#). These durations were selected to provide samples of both convective shower and synoptic scale events, with the distributions plotted in cumulative form for various probability percentiles (i.e., 30%, 70%, 90%).

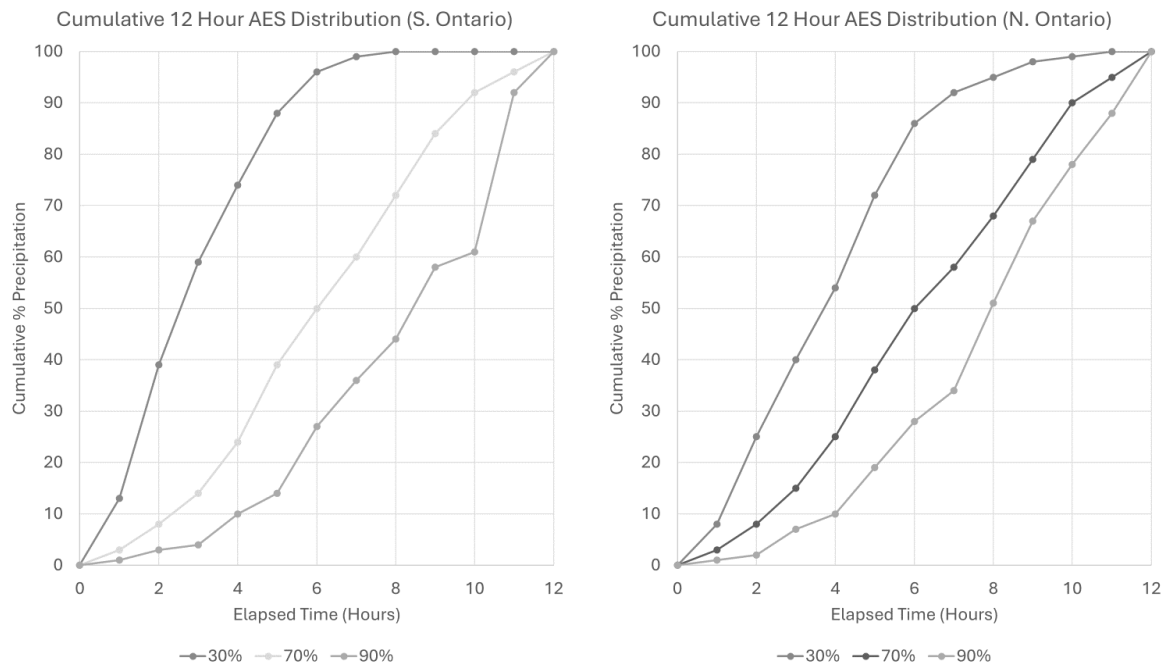


Figure 3-9 MSC Twelve-Hour Rainstorm Distributions for (a) Southern and (b) Northern Ontario

The engineer must exercise caution when selecting rainfall distribution from the enclosed graphs ([Figure 3-10](#)). For example, the selection of a 10% curve (not shown) would result in an advanced pattern of excess rainfall, as the early portion of the rain contains a large percentage of the total precipitation. This distribution will have a significant effect in the runoff computation, as the high losses during the initial part of the high intensity storm will result in relatively low runoff. At the other extreme, the selection of the 90% curve will provide a delayed pattern resulting in high rainfall values at the end of the storm; and may be a pragmatic distribution to apply. Such a distribution will produce higher runoff as the losses are usually lower at the tail end of the storm.

The use of the 50% curve can give misleading flow results, as the intensity during the storm would be fairly uniform, which contradicts observed distribution data. It is suggested that the user should analyze local storms to establish appropriate distributions. Where no such data are available, the 30% curve is suggested for use.

A more detailed analysis of 1-hour storms presented below suggests that the peak occurs within the first half of the storm event ([Table 3-7](#)).

Table 3-7 Rainfall Distributions – Percent (%) distributions per hour

Type	Reference	Storm Distribution	1st hour	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th hour
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	Soil Conservation Service Type II	24 Hour 2 Hour Increment	2	3	3	4	6	48	16	6	4	3	3	2
	Meteorological Services Canada (MSC), 30% Southern Ontario	12 Hour	13	26	20	15	14	8	3	1	0	0	0	0
	MSC, 30% Northern Ontario	12 Hour	8	17	15	14	18	14	6	3	3	1	1	0
	MSC, 70% Southern Ontario	12 Hour	3	5	6	10	15	11	10	12	12	8	4	4
	MSC, 70% Northern Ontario	12 Hour	3	5	7	10	13	12	8	10	11	11	5	5

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

ii. Keifer and Chu (Chicago) Distribution

The main advantage of this method, published first in 1957 (Keifer & Chu, 1957), is that the design storm is created from the intensity-duration-frequency curve. Therefore, results will fit the published IDF curve.

The Chicago distribution is best suited for use in urban or urbanizing basins where there is a high percentage of impervious area. This method requires the computation of the time to peak and the ratio of the time before or after the peak, divided by the storm duration. This ratio can only be derived from existing rainfall records. The method involves calculating the mean values of mass antecedent rainfall and the mean location of the peaks for various rainfall durations for a series of excessive rainfall events.

When analyzing thunderstorm type rainfall in urban areas, storm distribution becomes an important factor. Variation in rainfall intensity over small areas, such as 5 km², can be significant. Unfortunately, due to lack of rainfall data, no guidelines are available on storm distribution within small urban areas.

It is suggested that when the Chicago distribution is applied for flood hazard assessments, that its duration be limited to less than 12 hours (i.e., 12-hour maximum duration).

iii. Soil Conservation Service (SCS) Type II Distribution

The original distribution derived by its author was based on a 24-hour storm. Subsequently, the 6-hour and 12-hour distributions were obtained by selecting the increments within the central 6-hour and 12-hour periods of the original 24-hour storm. This distribution is typically applied to rural or underdeveloped basins, where there is a low percentage of impervious areas. SCS is now known as the Natural Resources Conservation Service (NRCS).

Prior to the development of distributions for Canada and Ontario by MSC, the Chicago and SCS distributions were widely used in Ontario. Historically, comparisons of the Chicago and SCS distributions with Ontario data found that for Ontario applications, the MSC distributions may give more realistic results; however, this is highly catchment specific. Additional analyses are required to substantiate the currency of these results.

iv. Region Specific Rainfall Distributions

The development of a regional storm distribution can be achieved through a variety of other methods. These use existing IDF information for a region or rely on the statistical analysis of a set of rainfall profiles. Some examples of a few of these alternative rainfall distribution methods are outlined in [Table 3-8](#). A review of these and other methods (Alfieri et al., 2008) to evaluate the peak discharges of design hyetographs during flood events noted a significant underestimation of the design flood results from the adoption of any rectangular hyetograph in the context of the rational formula. [Figure 3-8](#) highlights the regional differences in average mean annual rainfall in Ontario.

Table 3-8 Alternative Rainfall Distributions

Distribution Name	Type	Citation
Normalized Rainfall Mass Curves	Rainfall profile analysis	Huff, F.A. (1967)
Triangular approximation	Geometric shape tied to single point IDF information	Yen, B.C., and Chow, V.T. (1980)
Composite Method	Use of multi-point IDF information	Wenzel Jr., H.G. (1982)
Sifalda	Geometric shape tied to single point IDF information	Sifalda, V. (1973)
Pilgrim and Corderoy	Rainfall profile analysis	Pilgrim, D.H., & Corderoy, I. (1975)

3.5 Snowmelt and Winter Precipitation

The previous chapters have discussed total rainfall and rainfall distributions. A review of Canadian flow records will reveal that many basins generate annual peak flow rates and volumes during the spring period, usually resulting from a combination of snowmelt and rainfall. Experience has indicated difficulties in selecting design parameters to calculate accurate estimates of snowmelt. This section discusses simplified snowmelt modelling procedures for determining annual maximum snowmelt plus rainfall values. It is important that the chosen model or equation best mimic the extreme snowmelt conditions

experienced in the area of interest as snowmelt processes and their contribution to flooding will differ throughout the province.

Extreme precipitation data is derived from storms that occur mainly in the summer or fall, which represents rainfall that falls on pervious surfaces that permit infiltration and thereby reduce runoff. In the winter months, rainfall is less intense, but may fall on frozen ground that prevents infiltration, and may be supplemented by snowmelt runoff.

Characterization of floods at stream flow gauges in the watershed or near the watershed is an important step in any flood hazard assessment. Characterizing the annual maximum flows (floods) provides insight into the seasonality and drivers of floods and to what extent snowmelt contributes to flooding in each watershed. Professional judgement of the approving agency and individual professional or professionals will be required to determine if snowmelt calculations are required or if annual maximum flows (floods) are predominantly within the open water seasons.

3.5.1 Snowmelt Flood Generation and Flood Hazard Mapping

The use of snowmelt and rain-on-snow derived floods needs to be appropriately conservative, but also pragmatic. If using flow frequency methods to determine return-period events, snowmelt and rain-on-snow events will naturally be accounted for. In this case the 100-year event, or other, as appropriate, should be included in the flood event considerations.

A prudent exercise for evaluating whether snowmelt deserves more consideration in relation to defining the flooding hazard, includes reviewing hydrographs for the basin of interest or adjacent gauged basins, if the basin is not gauged, including comparing the time of year and making inferences about flood generating mechanisms (e.g., snowmelt, rainfall, rain on snow). Where snowmelt is suggested to be the dominant flood producing mechanism and available data are sparse, consideration of the use of a snowmelt model may be most warranted. Modelling results should be compared and corroborated with data from gauged basins sharing similar physiographic and landscape parameters.

The method for including both winter and open-water events when modelling is used to determine flows, is more uncertain. In these cases, open water return frequency flows are typically based on modelling from return frequency rainfall data, or IDF data. The methods for including flows from snowmelt and rain-on-snow conditions are undefined. Some combination of snowmelt and rainfall to create an additional flow condition for comparison to the rainfall-derived 1:100-year flood and/or the other flooding hazard standard events should be considered in watersheds where snow is shown to be a significant factor in flood generation.

3.5.2 Snowmelt Computations

Snowmelt computations are typically handled through continuous hydrologic modelling to determine the amount of runoff from a snowmelt event within a watershed over time. Models used to calculate snowmelt runoff range in complexity and may require various input data to complete the analysis. The chosen model should provide a good representation of the watershed and may depend on available data for snow course and meteorological conditions.

3.5.2.1 Meteorological Service of Canada (MSC) – Snowmelt Equations

A common practice in snowmelt computations is the use of daily MSC snowmelt equations which output daily snowmelt amounts. This approach is relatively simplistic and can be used where the data for more detailed methods is not available. To function in floodplain modeling, daily snowmelt amounts would have to be further distributed into hourly melt amounts, however this is not a simple calculation.

Historically, hourly air temperature monitoring data was not available and, therefore, some procedures like the MSC equations were based on daily information since daily air temperature data was the only readily available data. Daily synoptic maximum and minimum air temperature data can do a poor job of reflecting frontal air temperature events where temperatures do not necessarily track standard diurnal patterns and the maximum temperature can even occur overnight. Indeed, frontal types of events are often the most important from a flooding perspective as the melt's "thermal momentum" is not dampened by an overnight cooling. Hourly air temperature better describes the energy inputs for the snowmelt and refreeze calculations.

Five sets of snowmelt plus rainfall frequency values were previously developed by MSC for Canada using the degree day method. Two of these prepared for Ontario are presented in [Table 3-9](#). A more detailed description can be found in *Extreme Value Estimates of Snowmelt* (Louise & Hogg, 1980) and in the *Hydrology of Floods in Canada* (Watt, 1989).

In the past, Environment and Climate Change Canada provided snowmelt plus rainfall frequency values for MSC stations for durations of 1 to 30 days and return periods up to 100-years. These daily values can be converted to shorter durations, such as hourly contribution, and used as input in the precipitation/runoff simulation process, especially in cases where it exceeds the summer rainfall values.

Table 3-9 Meteorological Service of Canada - Snowmelt Equations

Model	Region	Snowmelt Equation	Units (SM)	Temperature Variable	Variable Definition
Model 4	Southern Ontario	$SM_4 = 0.91 \times T_{max}$	mm/day	Tmax	Maximum daily air temperature (°C)
Model 5	Southern Ontario	$SM_5 = 3.66 \times T_m$	mm/day	Tm	Mean daily air temperature (°C)

The input data used in the MSC snowmelt analysis and in the calculation of the snowmelt estimates are daily maximum and minimum temperatures, daily rainfall total and daily depth of fresh snow measurements by ruler. A snow density of 0.1 is assumed to convert snow depth into its water equivalent.

Snowmelt models use an algorithm that is based upon synthetic snowpacks which are accumulated according to the daily snowfall measurements and depleted according to the snowmelt as determined by the individual model, during days with the average daily temperature above the base temperature. The algorithm ceases to operate when the synthetic snowpack is reduced to zero. Evaporation (sublimation) from snow is accounted for using this method. Daily rainfall is added to the daily snowmelt as calculated by each model and the maxima of maximum series for the different durations.

Calibration of the equation selected from using single station or regional frequency estimates should be carried out wherever feasible.

3.5.2.2 Hydrologic Modelling Methods

One example of open-source software that can be used for modelling snowmelt routines is the Hydrologic Engineering Centre – Hydrologic Modeling System (HEC-HMS) model, supported by the U.S. Army Corps of Engineers (USACE). This model includes a temperature index snowmelt method and allows the user to input both point and gridded data.

Computer models continue to be refined and developed. Ultimately, selection of the most appropriate model must be left to the professional judgement of the approving agency and individual professionals undertaking the modelling. Any model based upon the

principles discussed in this chapter, and which can be calibrated and verified by the user, is widely used and applied by practitioners, is an acceptable choice.

3.5.2.3 Flood Frequency Analysis

When FFA is used as the method for determining flood flows, instead of modelling design storms, the contributing snowmelt flows are automatically naturally included. In this case it is not likely that snowmelt calculations are required.

3.5.3 Snow Monitoring Data

Appropriate procedures in Ontario for snow monitoring and snow hydrology in support of calculating snowmelt runoff, including for flood hazard identification can be found in a range of publications (MNR, 1985, 1989b, 2019). As part of a series of snow hydrology studies undertaken by the MNR, Dillon (1994) prepared a comprehensive study of 195 Ontario snow gauges. The study provided estimates of maximum snow water equivalent for a range of 2 to 100- year return periods. Work by MNR (Metcalf et al., 2018) has suggested a shift in snow cover and associated snow water equivalents for the period 1980-2010, showing a significant decreasing trend in the water year maximum, snow water equivalent for Ontario. While these values were representative of averages over significant geographic areas, it was noted that greater decreases may be occurring locally. These findings suggest that historical data analysed for snow water equivalents (e.g., Dillon, 1994) may not be fully representative of current conditions. Further analyses and additional studies are anticipated.

Differences in rainfall and rainfall/snowmelt events affect the amount of runoff generated, due to the seasonal differences in rainfall losses. The excess rain for undeveloped land can be significantly higher during the winter and early spring than during the summer. Runoff coefficients for undeveloped land could be 3 to 4 times higher during the winter compared to the summer conditions.

In urban areas, the differences in runoff between the two seasons are less significant. However, the long duration events, required for the design of storage facilities, may show that rain plus snowmelt in urban areas could generate runoff in excess of the runoff caused by summer events.

Currently, the provincial snow course monitoring network plays a particularly important role regarding observed snowpack information. Snow course information is used to assess current snow cover, frozen ground conditions, snowpack, potential snowmelt and contributions to streamflow. The Dillon (1994) study used MNR snow course monitoring data to prepare frequency analysis information and snowpack water equivalent mapping,

which updated previous studies undertaken by MSC. Snow surveys are typically completed on a semi-monthly basis and may not capture the peak snowpack water equivalent depending on the timing of significant snows, melts, and snow sampling. The limitation of periodic snow sampling needs to be considered when completing frequency analysis of snow survey monitoring data. Furthermore, limitations of point scale mean values have been identified in Ontario, however it has also been highlighted that these biases can be consistent within homogeneous landscapes and that this bias can be corrected by scaling factors (e.g., depth, SWE) derived from a limited number of transects (Beaton et al., 2019)

At the present time, manual surveys of snow depth and snow water equivalent are conducted bi-weekly in winter at 270 locations throughout the province to support flood forecasting. Surveys are conducted by MNR districts, conservation authorities, Ontario Power Generation, and Parks Canada (Metcalf et al., 2018). Snow depth has been collected at approximately 90 Environment and Climate Change Canada climate stations in Ontario using automated acoustic sensors, with approximately 75 stations currently active (Metcalf et al., 2018). Ontario Snow Survey location and data can be found on Ontario GeoHub. Additional studies related to snow, including the use of high-resolution reanalysis gridded snow data as a source of snow water equivalent to support streamflow modelling applications, including flood modelling, continue in Ontario (Beaton et al., 2024) with the involvement of MNR.

Climate change can have significant impacts on the seasonality of flooding. Climate change is expected to change snowpack and mid-winter thaw patterns that will vary by location throughout the province. Urban development has also been shown to alter the seasonality of watershed flooding. The impacts of higher ground temperatures in urban areas, road salts, and plowing result in increased winter flows, decreased spring flows, and increased volume in summer flows (Dickinson et al., 2018). The data analysis for flood characterization should consider changes in the watershed. Traditionally much of Ontario, has been considered a nival regime, however flow patterns in some of these systems particularly in southern Ontario may be transitioning from a nival to a pluvial system in a warmer and wetter climate (Tiwari et al., 2025). A decrease in annual flow variability has been observed, likely the result of earlier and lower spring freshet flows due to warmer winter conditions, causing more winter thaw events and winter rainfall; both of which resulted in less snow to contribute to the snow melt peak (Tiwari et al., 2025).

3.6 Hydrologic Modelling of Flood Flows to Define the Flooding Hazard

Hydrologic models incorporate calculations to simulate hydrologic processes of transport, storage, and water balance. Hydrologic modelling is an important tool for estimating flow hydrographs, including peak discharges, for flooding hazard identification where statistical techniques based on regional analysis of flow records are inappropriate.

A hydrologic model can be used to estimate the:

- response of the physical system to rainfall and/or snowmelt;
- frequency distribution of high flows;
- attenuating effects of storage elements such as channel routing and reservoirs; and
- effect of changes to the watershed.

3.6.1 Types of Hydrologic Models

The following summary provides guidance on how to select a hydrologic model for a flood hazard mapping application.

There are two main approaches to mathematical modelling:

1. **Deterministic** (or parameter-based) modelling: this whose response is equivalent to the physical system. This may be either through a model that uses semi-empirical equations that describe the rainfall/snowmelt-runoff process (converts precipitation to runoff); or one that uses a series of processes of the system (e.g., evapotranspiration, rainfall, snowmelt, runoff).
2. **Stochastic** modelling: This approach seeks to describe the hydrological time series of several measured variables involving distributions in probability. This approach determines statistical distributions of statistical parameters describing the response of the system (e.g., evapotranspiration, rainfall, streamflow) to generate a record which is statistically indistinguishable from the measured record.

For the most part, the models used in relation to flood hazard mapping are of the first type, namely deterministic models. Hence, the remainder of this section will be restricted to the discussion and documentation of models of this type.

Within the class of deterministic models, there are a number of model types which can be identified. The differences between these types are important since they affect the selection of a particular model for a particular situation. The most significant distinctions which can be made are:

- continuous event *versus* single event;
- lumped *versus* distributed;
- semi distributed; and,
- rural *versus* urban.

3.6.1.1 Continuous Event Models

Continuous simulation models allow the synthesis of long sequences of streamflow data based upon long-term records of meteorological conditions in the modelled watershed.

In general, they operate by maintaining a continuous account of various important hydrologic conditions in the watershed. These conditions include interception, depression, and soil moisture storage, ground water storage, channel storage, and snowpack conditions. The components of the hydrologic cycle are represented within such models to varying degrees of detail. These components react to a given watershed input, (e.g., precipitation, depending upon the current status of the system), to produce an estimate of the resulting streamflow. If the model is well calibrated and verified, the resulting time series of streamflow can be used as a substitute for observed flow records. Using these results, a flood frequency analysis could then be performed using the annual maxima peak extracted for each year.

There are several advantages to this technique. The continuous model needs only one startup calculation. It is easier to run it continuously through the next event than to attempt to ascertain the correct startup parameters and then adjust them for the next event. It is easier to calibrate these models when accurate continuous data is available because it is easier to run them for a continuous period concentrating on calibration of the parameter rather than continue trying to get the startup correct. If the model drifts, one can change any of the temporary files and restart the model from that file concentrating on the drift and not the startup parameters. This technique was a natural development from attempts to convey startup conditions from one event to the next. The advantage of continuous simulation approaches to event-based methods is that the catchment moisture state prior to the flood-producing rainfall event (i.e., antecedent conditions) is implicitly incorporated

into the modelling framework, provided the model has been calibrated and validated to produce reasonable simulations (e.g., Pathiraja and associates [2012]).

3.6.1.2 Single Event Models

Single event models share many features with continuous models in that the hydrologic cycle is represented by various model components. However, no attempt is made to track watershed conditions continuously in time. For such a model, the initial moisture storage, snowpack conditions, among other indicators, must be obtained by some external means prior to each runoff event of interest. In some situations, this can be carried out effectively based upon observed data; in other cases, an arbitrary selection of initial conditions may be necessary.

Many single event models use startup conditions and then run for a period limited by their application or the number of times steps it takes to fill their internal arrays. Some single event models have been made continuous by the use of 'carry-over' files. The technique used is to retain the conditions for the end of the one run of the model in a temporary file, then use this file as the startup conditions for the next run of the model. However, some single event models are more complex and require seasonalization of the parameters in order to run in continuous mode. Therefore, caution should be exercised when taking this approach.

Single event models can be used to generate a hydrograph associated with an observed storm event, or from a synthetic design storm. In the latter case, rainfall statistics recorded at a meteorologic station are used to construct a rainfall event which is then input to the model. Generally, this type of single event modelling assumes that there is no alternative to the rainfall statistics during the transformation of rainfall to runoff (i.e., 100-year rainfall yields a 100-year peak flow).

Multi-event simulation can be carried out with single event models. It consists of the simulation of a series of storms, each with its own antecedent moisture conditions followed by a frequency analysis of resulting flows. This method avoids the use of a design storm.

3.6.1.3 Lumped and Distributed Models

The terms lumped versus distributed describe the degree to which the application of a particular model requires the averaging of watershed characteristics over its area. A number of models are purely lumped models, using average values of parameters such as maximum infiltration rate across the entire watershed. This is why lumped models are also often referred to as spatially-averaged models. In these models, the watershed is

modelled as a single unit and does not account for spatial variability. Other models are distributed in the sense that they permit the discretization of a watershed into a number of subareas. Using this semi-distributed model, flows from individual subareas are combined and routed through the stream channel network to obtain flows at downstream points. However, within an individual subarea, the parameters are lumped representations of the area.

Distributed models require small scale input data, such as radar, distributed weather information, or satellite information and combining this with small scale watershed parameters and antecedent conditions would result in a highly distributed model. These models often use a gridded approach, in which the watershed is represented by grid cells and runoff is calculated for each area to reflect spatial variability within the system. In this way, even distributed models perform spatial averaging, but generally do so over a much smaller scale than typical spatially averaged models, for instance at the scale of a grid cell. Streamflow calculations from a very large number of segments of a watershed are routed overland and through the ground into the streams. Using smaller and smaller pieces of the watershed to perform the calculations produces a highly distributed model and allows for increased accuracy in both the timing and quantity of water moving through the watershed.

3.6.1.4 Semi-Distributed Models

Semi-distributed models are variations of lumped models with features of distributed models and are often considered to be optimized to benefit from the strengths of both models, making this type of model very common. This approach divides watersheds into sub-areas often referred to as hydrologic response units, analogous to small, lumped units. In this way these models can have data that are separated within a catchment, but homogeneous or lumped within a sub-area (Beven, 2012). Spatially, this classification of model is considered to represent variability better than lumped models but at less detail than distributed models. The benefits of semi-distributed models are their fast computational time and ability to use fewer parameters and less data than a distributed model (Sitterson et al., 2017). Manipulation of input data is considered a drawback of these models, for example spatially distributed rainfall data must be averaged within sub areas or point rainfall gauge data must be disturbed to these areas using Thiessen polygons to define the area of influence around the sample point (Sitterson et al., 2017).

3.6.1.5 Urban and Rural Models

The earliest (historically) computerized hydrologic models were developed for use in rural watersheds in various watershed planning and design studies. In urban areas the

Rational Method was generally applied for estimating design flows for drainage projects. However, with the heightened awareness of the environmental impact of storm and combined sewer discharges, a new set of models specifically designed for use in urban watersheds have been developed. These models provide detailed information on urban runoff for use in studies relating to the control of storm runoff. This has, however, led to the creation of two distinct sets of models: one for rural watersheds and another for urban areas. The use of a “rural” model in an urban area and vice versa has sometimes led to erroneous results. In flood hazard mapping projects, there is frequently an urban component associated with the hydrology of the areas of interest. This often means that it is necessary to utilize an urban hydrologic model to analyze part or all of the subject watershed.

Urban models, such as PCSWMM, should not be used without testing in a rural area since default values and runoff response simulations are often not representative of rural conditions. Generally, urban models should simulate separately runoff from pervious and impervious areas, averaging can lead to significant errors in runoff volume.

The precipitation/runoff models can simulate the flows associated with existing urban development as a prerequisite to developing remedial strategies (particularly related to sewer overflows). With an increasing emphasis on controlling flows from new developments, these models were adopted to estimate post-development flows. However, a number of these models were generally unsuitable for estimating flows from rural areas and hence, the existing rural models were used for “pre-development” cases. Unfortunately, many of these models are based upon different computational procedures. It is, therefore, very difficult to determine to what extent changes predicted by this approach are the result of different modelling techniques rather than real differences caused by land use changes. This problem of incompatibility of different models has been recognized, and such an approach is not recommended if at all avoidable.

The intermediate approach of utilizing a “single” model which contains separate components to simulate the urban and rural areas appear to reflect the current practice. As an example, the Visual OTTHYMO model contains commands to include urban and rural areas for single event hydrologic modelling. This allows the user to tailor model parameters depending on the characteristics of the area being modelled to create distinct unit hydrographs for urban and rural areas. The urban component uses parallel instantaneous unit hydrographs for both impervious and pervious regions within the specified watershed. The rural component uses an n-linear reservoirs conceptualization first suggested by Nash (1957) focused on instantaneous unit hydrograph. Many models exist to model flow in these different landscapes.

While impacts of urbanization are clearly visible and relatively well understood, the effects of rural land use changes are generally more subtle and the subject of diverse opinion. Although there are many ways in which changes may occur, the following have been identified in past studies to be of some importance:

- installation of agricultural drainage to improve productivity of existing agricultural lands and to bring new “drained” areas into production;
- modifications of crop types and cropping practices; and
- deforestation or reforestation

Modelling of agricultural drainage such as municipal or tile drains, can be carried out in one of several ways, including:

- empirical adjustments or watershed model parameters based upon literature and experience;
- field scale modelling of tile-drainage using a model that can then be projected to a watershed scale by modifying the parameters of a watershed model;
- modelling the modifications to municipal drains using a channel routing component of a watershed model; and
- using a watershed model that has a specific component designed to simulate the effect of agricultural drainage.

Modelling potential impacts of cropping practices on watershed response would ideally be carried out using a physically based model in which specific areas would be modelled with the appropriate characteristics. However, given the empirical nature of models and computation techniques, the most that can be achieved is a judicious modification of model parameters based on experience and literature sources. In the case of a relatively complex model, parameters related to overland flow length, slope roughness, infiltration and soil moisture storage may be modified to reflect altered practices. In the case of a simpler model based on the SCS runoff method, the average Curve Number (CN) can be recomputed to reflect a different make-up of land use.

3.6.1.6 Forested Watershed Modelling

Forest cover reduces the yield from a watershed, thus resulting in lower annual streamflow and decreased ground water recharge.

Although evapotranspiration rates and soil water moisture storage are reportedly diminished during the growing season by the removal of wooded areas, summer streamflow rates are usually only marginally greater. The most significant increase in watershed yield following forest clear cutting is experienced during the spring snowmelt season with incremental runoff volumes reportedly approaching 30 percent of annual yield.

It appears that flow is higher from cutover forest land during the growing season, with an average response difference of between 10 and 200% being possible. The effect of the removal of cover on peak flows will vary depending on the soil disturbance, intensity and duration of the storm event and the treatment of the land after forest removal. Larger infiltration and soil storage capacities commonly found in forests are known to eliminate overland flow. These forest processes are particularly important during small storm events. During severe rainfall, forest discharges to stream channels via subsurface flow can be significant.

Forest cover may either increase or decrease individual flood peaks caused by snowmelt or rainfall plus snowmelt events. Although wooded areas will prolong snow cover during the spring period, the most rapid snowmelt of the year may be late in the season when high temperatures and radiation occur. If combined with rainfall, forest cover can be a liability rather than a flood control asset. Maintaining dissimilar land uses over a watershed appears to be the accepted practice of desynchronizing snowmelt related peak flows and thereby reducing flood potential throughout the snowmelt season.

The most suitable models for examining impacts of forest cover on flood peaks are those that contain specific parameters related to forest cover. These parameters act as controls on the snowmelt and evaporation processes and can be varied directly in accordance with anticipated changes. Other factors such as changes in wind speed from open to forested areas, changes in surface storage and infiltration rates between forested and open lands must also be correctly modelled.

3.6.2 Computational Procedures Used in Hydrologic Models

A description of the techniques in general use is given below. It is intended that this will assist the user in better understanding the types of models available.

Most methods of computing runoff from rainfall and/or snowmelt using a deterministic model involve two processes: the computation of the volume of rainfall/snowmelt contributing to river flow from each storm, and the distribution of this volume with respect to time.

In the determination of runoff volume, most models currently used utilize the “classic” concept of “Hortonian” overland flow. In general, this consists of the separation of input rainfall or snowmelt into a surface or direct runoff component and a sub-surface component which may or may not contribute to streamflow within the period of interest. The separation is usually controlled by determining the infiltration capacity of the surface of the watershed at the time of the input. If the rate of input exceeds the infiltration capacity, surface runoff occurs: if not, then all input is absorbed into the ground. The infiltration capacity is generally dependent upon the level of soil moisture prior to the event of interest. Many models maintain an account of the soil moisture status by budgeting inputs from infiltration versus losses from evapotranspiration and deep percolation. In continuous simulation models, a ground water component is frequently linked to the deep percolation loss, which, in turn, controls baseflow between events.

The estimated runoff volume is generally subject to some form of watershed routing to represent the lag time and attenuation found in most watersheds. In more complex models, channel routing and reservoir routing may also be included.

It should be noted that this “classic” concept of the genesis of streamflow has been widely questioned in the past. An alternative is the so-called variable source area concept, which explains that surface runoff is generated by the “non-Hortonian” or saturated overland flow, due to the rising of water table in the (wet) areas immediately adjacent to the stream channels. However, most hydrologic models employ this “classic” concept in one form or another.

3.6.2.1 Snowmelt

There are three generalized snowmelt models that are commonly used in current hydrologic practice:

- a) **Energy balance models** which characterize some or all aspects of the heat balance of a snowpack but utilize approximate formulae to model the snowmelt process.
 - i) Additional empirical equations may be used to define the extent of areal snow cover, the storage and delay of water in the snowpack, and any heat deficit which may arise.
- b) **Index models** which identify snowmelt processes explicitly, but without the use of extensive meteorological input data.

- c) **Temperature index models** in which air temperature is used as an indicator of snowmelt

Other models (e.g., GAWSER) contain a snowmelt routine that considers what happens to the snow as it melts, and it reflects the observations that as the snow leaves the centre of a field, significant amounts remain at the edges.

3.6.2.2 Infiltration

The following infiltration methods are considered acceptable in hydrologic modelling:

i. **Horton's equation/method**

Horton's method (Horton, 1941) is an empirical formula to estimate infiltration. It describes how infiltration capacity declines as the precipitation event advances and is based on generating runoff when rainfall intensity exceeds the infiltration capacity. The Horton equation can be derived from the Richards equation by using a proper set of assumptions that simplifies the physics of the Richards equation to derive an equation with an exact analytical solution (Chow et al., 1988).

ii. **Holtan Infiltration equation/method**

The Holtan method (Holtan, 1961) is based on the desire to estimate infiltration based on the physical properties characterizing soil water storage in the Horton equation (Holtan and Lopez, 1971). This method links the infiltration rate with the soil properties and fits the equation based on field measurements. By making this physical connection, the Holtan method further made the recovery a function of the evapotranspiration, coupled with available water in the soil through introducing field capacity (when no more water can be stored in the soil and runoff occurs), and wilting point, which is the point at which plants can no longer draw water out from the soil.

iii. **S.C.S. Curve Number Procedure**

The Soil Conservation Service (SCS) (now the Natural Resources Conservation Service) method predicts runoff from an expression for a rainfall-runoff curve that varies according to a single parameter called the curve number (CN). The dimensionless CN parameter describes the antecedent potential water retention of a watershed. Tables are used to provide a CN value according to soil type, hydrologic condition, antecedent moisture condition, and land use (USDA-NRCS, 2021). The CN tables make it easy to transfer GIS data into a rainfall-runoff model based on the SCS-CN method. Conceptually, the application of the SCS-CN method generally assumes a homogeneous watershed with a spatially uniform runoff.

iv. Modified S.C.S. Curve Number

The modified SCS-CN was developed to address some of the original method's shortcomings regarding practical applications in different geographical conditions, than for where the method was originally developed (i.e., regional data from agricultural landscapes in the U.S. Mid-West). Among the adjustments, include refinements to the initial abstraction (IA) coefficient/methodology to improve its representativeness. The initial abstraction represents the amount of rainfall that is retained in the watershed storage as interception, infiltration, and surface storage before runoff begins, which by convention is a fraction of the antecedent potential retention. Some evidence suggests that in Ontario the prescribed IA may overestimate the initial abstraction, especially in areas with low curve numbers. Research and monitoring from rural and urban catchments in Canada have been used to improve the performance of the SCS method. This involves using a fixed initial abstraction (IA) parameter (rather than a varying IA parameter as in the SCS-CN Method) and the CN is rather varied or altered to define the Modified CN as a function of the IA and total rainfall. This modified SCS-CN approach is integrated into some hydrological models, including Visual OTTHYMO. Others have worked to extend the traditional SCS-CN method with a quantitative spatial characterization of runoff variability and a new representation of the runoff response, to further address the method's shortcomings (e.g., Bartlett et al., 2016).

v. Green-Ampt Infiltration equation/method

The Green-Ampt infiltration method is based on a saturated upper layer, called the wetted zone, where the water is percolated to an un-wetted zone with an initial soil moisture content. This makes the model only valid for saturated conditions. Darcy's law provides the infiltration velocity through the saturated wetted zone, including hydraulic conductivity and wetting front suction as parameters. As this model is only valid for saturated conditions, prior to saturation a common procedure is to assume that the infiltration rate is equal to the rainfall intensity (Rossman & Huber, 2015). Recovery in the Green-Ampt model is calculated based on three recovery parameters, (a) thickness of the upper crust, (b) a recovery constant, and (c) the time needed for a complete recovery.

3.6.2.3 Soil Moisture Accounting

Soil moisture is generally modelled using moisture storage reservoirs. A variety of formulations have been utilized ranging from a simple one reservoir system of total subsurface moisture to a complex four reservoir system. Several models utilize the SCS Curve Number procedure (USDA-SCS, 1972) for the calculation of runoff volumes which

is based upon external information such as antecedent soil moisture indices. In single event models, it is necessary to initialize whatever index of soil moisture is used based upon external information. In the case of continuous simulation models, an account of moisture conditions is maintained. This is achieved by balancing inputs from infiltration against losses due to direct outflow to the channel system, evaporation to the atmosphere and percolation to deeper soil moisture zones.

3.6.2.4 Groundwater Storage/Baseflow

Baseflow or groundwater response should be considered in determining peak flows where there is considerable evidence that a rapid subsurface response can generate a large proportion of flows for the watershed. Models such as HydroGeoSphere, MIKE SHE and other current and maintained integrated or coupled groundwater-surface water models may be used to reproduce such responses. In general, groundwater response can be modelled as a linear storage reservoir.

3.6.2.5 Reservoir Routing

Reservoirs, lakes and onstream storage facilities affect both the flows and water levels along a river system. Hydrologic models may be employed to estimate the effect of reservoirs on streamflow, and subsequently the hydraulics can be computed with the revised flood flows. Alternatively, the physical properties of the reservoir can be incorporated directly into a hydraulic flood routing model of the river. More frequently, it is necessary to establish a starting water level at a lake for a backwater computation, or a local flood line around a lake. In such cases, a process of reservoir routing may be applied, in the absence of direct water level recordings.

The most used method of routing through a reservoir is the Storage-Indication method, also known as the modified Puls method or level-pool routing. This is a hydrologic routing method by which a flood hydrograph routed through a reservoir is both delayed and attenuated as it enters and spreads over the reservoir surface. Water stored in the reservoir is then discharged gradually through flow control devices such as gates, orifices and overflow spillways.

Storage-Indication method/modified Puls method is also used by some hydrologic models, such as GAWSER and HEC-HMS in their respective reservoir routing routines.

In flood hazard mapping studies, the analysis of flow-frequency relationships requires the conversion of regulated flows to de-regulated or natural flows. If the streamflow under consideration are subject to artificial regulation by dams or diversions, which have significant impacts on peak flows, it is necessary to estimate the effect of such regulation

to enable a conversion of streamflow to natural conditions prior to undertaking a flood frequency analysis (see [Section 3.2.1.2 Conversion of Regulated Flows to Natural Flow Conditions](#)). When the degree of flow regulation is not negligible, results from single-station flood frequency analyses cannot be used in a regional analysis unless the observed flood flows have first been converted to their natural state. Otherwise, the regional regression equations would underestimate the flood magnitude at ungauged watersheds.

For reservoirs, records of outflows, stage and stage-storage curves are required, and for diversions, the quantity diverted into or out of the system. Rule curves or operating procedures must also be obtained to enable a reversion from regulated conditions to a natural flood estimate. It may be necessary, if an installation was made within a period of record, to collect data on the times of installation, cut-off, and reservoir-filling, among others.

Given adequate data, such as diversion flows, reservoir stages, outflows and stage-storage curves, it is a simple task to convert flows to natural conditions. The regulated flows may then be converted to natural flows by a simple ratio equal to the degree of regulation determined. Once the flows are unregulated/unattenuated, a normal flood frequency analysis can be carried out. Comparison of streamflow records before and after reservoir construction can also be used to estimate the overall effect of regulation.

3.6.2.6 Watershed Routing

A great deal of research effort has been expended in refining this aspect of hydrologic modelling. Hence many different methodologies have been proposed. The most frequently used techniques are:

- the Unit Hydrograph method (both linear and non-linear);
- the Kinematic Wave method; and
- the Time-Area Curve method.

3.6.3 Recommended Model Selection

At the present time, there is no universal model that can be generally recommended for all applications. Therefore, each user must consider several alternatives in deciding what model is best for a given application.

A variety of procedures have been presented in the literature for use in model selection. Important considerations when selecting hydrologic modelling software include the following:

- 1) **Development** – Is the model actively being updated with new features to add functionality to the model or keep pace with the state of hydrologic science?
- 2) **Maintenance** – Is the model actively supported by a developer or user community who can address issues such as errors in the model code or compatibility with new operating systems? Is the software supported by an institution or other long-term agreement?
- 3) **Documentation and training** – What degree of software documentation exists (e.g., user manuals, example models)? Are training courses available for the software?
- 4) **Cost** – What is the capital cost of investing in the software as well as ongoing costs? Some models require payment of annual maintenance fees to access software patches and new versions of the model. This consideration is key not only for model development, but also for each future user of the developed model.
- 5) **User base** – Does the software have a broad user base both in academic and applied (consulting) settings? Are peer-reviewed applications described in academic journals and conferences? A large user base may provide valuable resources through the collective experiences of applying the software and confidence in the abilities of the model.
- 6) **Representation of Hydrologic processes** – Models vary in their representation of hydrologic processes and depending on the context of the applications, some features may not be relevant. Some common processes related to flood flow generation are described below:
 - i) **Precipitation** - Uniform versus spatially distributed precipitation inputs. Spatially distributed inputs may be relevant for large catchments where significant variation in rainfall intensity may be present.

- ii) **Snowmelt processes** – Snowpack accumulation and depletion is a complex process. Snow processes may include the following:
 - a. Solid and liquid snowpack – Release of liquid water from snowpack only after a threshold fraction of snowpack has been converted to water.
 - b. Snow redistribution, snow sublimation, snowmelt from solar radiation, snowmelt from rain on snowpack (thermal melting), and dynamic snow albedo.
- iii) **Depression storage** – Representation of the storage of water that accumulates in depressions in the ground surface during a rainfall event.
- iv) **Infiltration** – Infiltration process represents water entering the subsurface and is a very common feature of hydrology models. Most pertinent in continuous modelling.
- v) **Channel flow** – Channel routing processes are used to simulate flow times in channels.
- vi) **Urban hydrology** – Representation of sewers, stormwater management-type features (e.g., retention ponds), while respecting Ontario’s technical protocols related to not accounting for or ‘crediting’ artificial storage in flood hazard mapping.

Elaborate model evaluation is often not practical or possible due to a lack of published information on the accuracy of computational procedures. However, many peer-reviewed articles and related guidance documents exist to guide model selection and evaluation. While a preliminary screening of available hydrologic models was carried out in the preparation of this Technical Bulletin, a list of hydrologic models suitable for use in Ontario has not been included herein at this time.

3.6.3.1 Use of New Modeling Software

It is expected that new modelling software or existing recoded software will continue to appear on a regular basis. When choosing software to be used, modellers should keep several factors in mind to ensure the relevancy and appropriateness of the models for use in Ontario. These are:

- The software is well documented and known to other practitioners.

- The software has proven results in Ontario or a similar jurisdiction with respect to climate and hydrology.
- The software is backed by peer reviewed scientific literature.
- Past iterations of the software have been widely used and accepted by the flood hazard estimation and mapping community.

In addition, the models should first be tested on specific demonstration watersheds to validate the models' applicability. Consultation with hydrology and hydraulic specialists is encouraged during the review procedures. This should include documentation of the advantages and disadvantages of the candidate models including evaluation from both theoretical and practical points of view. Generally, only those models which have been demonstrated to be superior to some of the currently accepted models used by the agency should be advanced.

3.6.4 Hydrologic Parameter Selection

Selection of values for hydrologic parameters requires a high degree of engineering judgement. Physically derived parameters (e.g., area, length, slope, % imperviousness) should be determined from available GIS data. Initial selection of other hydrologic parameters should be based on the best available data. There are many sources for other parameters that can be referenced, and the best data sources may vary by project location. For example, the Federal soils layer includes Green & Ampt parameters estimated in the tabular data associated with the soil layer, and the MTO Drainage Management Manual (MTO, 1997) includes many tables of standard parameters and appropriate ranges. Some municipalities and conservation authorities have developed standard hydrologic parameters for use within their jurisdictions, for example, Technical Guidelines for Flood Hazard Mapping (Doherty et al., 2017) was developed in coordination by a group of conservation authorities. Other sources of standard and appropriate ranges of parameter values include academic textbooks and modelling software manuals.

Models should initially be run with standardized parameters based on the best available base data and agreed upon sources of parameter ranges. Good initial parameter selection can reduce the efforts required to complete the calibration process.

3.6.5 Model Calibration and Validation

Model calibration is an important part of the flooding hazard investigation. The accuracy of hydrologic predictions from rainfall runoff estimation and models is strongly dependent

on the input parameters used to derive these models, making model calibration and validation an important requirement of the flood peak estimation process. Best practices for hydrologic modeling include calibration whenever possible.

3.6.5.1 Calibration, Verification and Validation

Calibration of hydrological models is the process in which model parameters are adjusted to best represent the basin of interest, by matching observed hydrologic conditions pertaining to specific storm events observed within the watershed. While some parameters may be able to be determined through measurement, many parameters represent less measurable characteristics of the basin. These parameters can be determined through manual calibration, where the modeler adjusts the parameters to best match observed data, and automatic calibration where algorithms are used to find parameters that provide optimized results.

When streamflow gauge data are not available, model verification is considered a minimum best practice. Model verification uses high water marks, photographs, and other evidence of water levels during historic events to check that the model results are reasonable. An appropriate hydraulic model could be required to estimate flows associated with the high-water levels. Another option when there is no gauged data available within the study area, is to use gauge data from adjacent hydraulically similar watersheds for model verification. This level of scrutiny does not constitute a calibrated model.

Following calibration, the model is then tested to confirm that the parameter adjustments are representative of major hydrologic processes given different climatic inputs; this modelling procedure is called validation. Validation is the testing of the calibrated model performance to independent data that were not used in calibration. Validation provides insight to ensure that the model can perform adequately for its intended purpose (Klemes, 1986). The validation of a model is equally important, as this procedure will assess how the calibrated model predicts a set of events not used for calibration.

3.6.5.2 Calibrating for Large Events and Flood Hazard Mapping

The selection of data for calibration should reflect the intended use of the model. If the model is to be used exclusively to estimate flood flows of high return periods, then the calibration data set should be biased towards such events. If on the other hand the entire annual flow cycle must be simulated, then a broad range of flows must be included (i.e., wet, average, and dry years). As a rough guideline, for continuous simulation models a minimum of five years of calibration data and five years of independent validation data would be appropriate. For a single event model, five events of significant and varying

magnitude are recommended, this will prove invaluable for testing the accuracy of the calibration and should not be overlooked.

The developed model will almost certainly be used for events larger than the calibration data when the primary goal of the model development is flood hazard mapping. For flood hazard mapping the primary focus is on accurately modelling water surface elevations and associated inundation under peak, regulatory flood magnitudes. As such, bias should be towards larger calibration events to reduce the errors of extrapolation.

Care should be taken to not over-calibrate. Parameters, and in particular those that are physically derived, need to be varied only within reasonable limits. Parameters should not be varied outside of reasonable limits, or anomalies will result from model simulations. For example, during a calibration process, if parameters are adjusted outside of the reasonable limits to match flows for smaller recorded events, the effects of these adjustments will likely cause unrealistic results for large events. It may be better to have a less than perfect fit for the monitored events and confirm the large event results with judgement checks.

The accuracy of flow records is largely dependent on rating curves at gauge sites; every effort should, therefore, be made to investigate and carry forward the degree of error which may be inherent in rating curves due to the type of control section or the flow range. When calibrating for high water elevations during spring breakup, it is important to consider ice conditions that can cause false high flow readings in the rating tables. For gauged watersheds, the rating curve at the gauge should be reviewed and deemed appropriate for the range of measured levels being used in the calibration, or validation. Refer to the rating curve discussion in [Section 2.1.4 Stage-Discharge Relationships](#) for more information. For ungauged watersheds, gauged data from similar, reference watersheds can be used as a surrogate. Methods such as the Regional Flood Frequency Analysis (Watt et al., 1989) can be used.

3.6.5.3 Parameter Calibration

Calibration of models involves the adjustment of model parameters to reduce the differences between observed and simulated events. If there are no observed events available for calibration the uncertainty of parameter values can be reduced if the model is verified on a hydrologically similar watershed. Generally, the more complex a simulation model, the more data and expertise that is required for calibration. This will invariably result in a more expensive study, with results that are not necessarily more accurate.

Model parameters can be obtained by either measurement from physical data, computation from available records of flow or selection based upon previous experience.

Certain parameters such as drainage area, channel slopes and impervious fraction of the watershed can be measured directly from topographic mapping or other sources. Other types of parameters which must be initialized are those which can be estimated from available records. This would include baseflow recession constants, direct runoff unit hydrograph parameters, areal reduction factors applicable to rainfall and degree-day coefficients for snowmelt calculations. This group of parameters would be subject to optimization in the calibration process but would be expected to vary within a relatively narrow range in a suitable model of the given watershed.

The final group of parameters is that for which it is difficult to find a direct physical interpretation and that must be estimated based upon general experience, or experience from using the specific model on another watershed. This would include parameters such as maximum soil moisture deficit, factors to modify potential evaporation to actual evaporation, factors to adjust convection melt estimates to field conditions, interflow recession constant, average infiltration rates, among others. A fairly wide range of fluctuation of these values during calibration would be anticipated. Final values which are consistent with watershed characteristics would increase the user's confidence in the suitability of the model for the intended application.

In general, it is not feasible to directly estimate all model parameters from the physically measurable characteristics of the watershed. It is, therefore, desirable to calibrate and validate a model for each particular application. From a calibration viewpoint it may save time to select a model with the fewest parameters. However, the selection of a model that has been calibrated on a similar watershed could assist in the calibration.

The reasonable limits vary for each parameter. Generally, the hydrologic modeller should have an understanding of the acceptable range and sensitivity of each parameter based on the watershed conditions. As the parameters and coefficients applicable to each hydrologic modelling method vary, acceptable parameter limits are not provided herein. Rather, the reader should refer to documentation for the modelling method at hand, or academic textbooks for guidance on parameters that cannot be measured. Refer to [Section 3.6.4 Hydrologic Parameter Selection](#) for a discussion of available and local standards and guidelines regarding model parameter values. Many of these documents provide ranges of appropriate values. Summarizing parameter adjustment and reasoning is considered a required part of study documentation.

There will be situations where the "standard" parameters are not appropriate. However, these situations are not the norm and should be considered on rare occasion only. These situations will require appropriate documentation, review and approval from the appropriate regulating authority (e.g., the local municipality or CA).

3.6.5.4 Manual vs Automated Calibration Processes

Calibration can be completed either manually, by the hydrologic modeller, or using automated processes through various software. The pros and cons of each method are briefly summarized below. For example, Artificial Intelligence (AI), is a rapidly advancing and changing field. The considerations in this section may need to be verified and updated by the reader for their project.

Manual calibration utilises the modellers judgement on best selection of parameter values in combination with the overall fit of the volume, peak and timing of the hydrograph. Automated calibration utilizes modern computing power to run numerous model simulations to test numerous combinations of parameter values. The results of each simulation are tested for statistical best fit to provided observed data.

The availability of autocalibration software and AI does not relieve the hydrologic modeller of their responsibility to professional judgement and understanding of the meaning behind individual parameter values. Watershed and flood characteristics must remain top of mind throughout the process. While the statistical best fit generated by an autocalibration tool may include parameters related to snowmelt, in combination with fully grown crops, this is not likely an appropriate combination of parameter values. In many cases manual review and revision of the calibration is required. Sometimes the statistically 'good fit' model is better than the statistically 'best fit' model when professional judgement and characterization are included.

There are inherent limitations and uncertainty within each model and modelling software. These uncertainties should be understood and considered by the modeler before initiating an automated calibration process. Without this understanding, an over calibrated model could result in precision without accuracy.

3.6.5.5 Assessing Calibration Performance

Calibration processes should start with the most sensitive and uncertain parameters to obtain a reasonable match between simulated and observed hydrographs. Calibration efforts should focus on first matching total volume, then peak flow rates and finally other flood characteristics including time to peak. A number of metrics can be used to assess the suitability of the simulated model results including:

- Plots of simulated and observed streamflow with associated hyetograph;
- Time to peak;

- Nash Sutcliffe coefficient: measurement of best fit between simulated and observed data, where 1.0 is a perfect fit; and/or
- Percent bias: percent difference between simulated and observed volumes (positive percent difference means over estimation and vice versa).

General guidelines for evaluating calibration performance metrics are listed in [Table 3-10](#) Calibration performance metrics below.

Table 3-10 Calibration performance metrics

General Performance	Nash-Sutcliffe Coefficient	% Bias (Volume)
Very Good	0.65-1.00	$\pm < 15\%$
Good	0.55-0.65	$\pm 15\%$ to $\pm 20\%$
Satisfactory	0.40-0.55	$\pm 20\%$ to $\pm 30\%$
Unsatisfactory	< 0.40	$\pm > 30\%$

Source: Moriasi and colleagues (2007) as summarized in the HEC-HMS Application Guide.

A one-to-one line-of-best-fit chart of observed versus simulated peak flows and volumes for gauged locations is illustrative at showing whether the model is under- or over-predicting peak flows and volumes across the range of events being considered. Another option is hydrograph plots of observed and simulated for each event with a summary of fit using Nash–Sutcliffe efficiency or other statistical comparison.

Once initial runs have been completed and analyzed using the above process, calibration or adjustment of parameters should be considered/undertaken, along with inspection and scrutinizing of input data and observed flow data, if that has not already been done. Achieving better calibration fit alone should not be considered reason for adjusting parameters outside of the acceptable limits.

3.6.5.6 Uncertainty Analysis

An uncertainty analysis should include an assessment of potential sources of error including uncertainty associated with parameter values, hydrologic constants, input data, and methods. This analysis can be performed using numerical modelling or more

qualitatively by considering the results of sensitivity analysis, input data representativeness and goodness-of-fit for model calibration and validation.

4 Hydraulic Modelling

4.1 Planning for Hydraulic Modelling

When deciding on an approach to hydraulic modelling it is useful to consider the goal of the modelling and to identify the appropriate level of model detail and define the components that will be required to adequately model the river or stream in question. It is useful to develop a conceptual model prior to initiating a hydraulic modelling project, this can include at a fundamental level, representing the main flow routes and types of structures by highlighting them on a map. This can provide useful information on areas of interest and points of flow interaction, that potentially may require additional characterization. A pre-planning step such as this will also assist in deciding where model boundaries should be placed and the types and specifications of topographic information including survey information that may be required.

To support this, it can be useful to create hydraulic modelling method statement or plan, prior to initiating the modelling work. This provides an opportunity to get input from colleagues and other practitioners on the proposed approach. This hydraulic modelling plan or method statement can include information such as:

- Study objectives and a discussion of project scale and required outputs. This provides a venue to obtain additional flood related information (if deemed desirable) beyond the flooding hazard design storm and modelling flows and levels under said event;
- A review of previous and ongoing flood hazard delineation projects and data sets, including an opportunity to leverage those data or any lessons learned from those projects; and/or
- A discussion of potential assumptions and limitations of the modelling.

This type of plan or method statement should also include additional information such as:

- Develop/document model schematization;
- Document design event/flooding hazard standard to be used (magnitude determined from hydrologic modelling);
- Proposed topographic survey locations;

- Topographic data accuracy requirements and data sources (see MNR, 2023);
- Document regulated flow conditions that will need to be naturalized, including method to naturalize;
- Document/classify flow conditions of interest to mode including solution methods (i.e., steady, unsteady, and similar) and in what dimensions the modelling should be characterized in relative to the expected conditions experienced by flood flows and expected inundated areas (i.e., 1D, 2D, coupled 1D-2D) and how this will be undertaken to meet conventional protocols;
- Determine computational methods (i.e., 1D, 2D or coupled);
- Choose software, including naming conventions and file management;
- Boundary conditions and flow applications/types;
- Parameter definition (parameterisation);
- Calibration and validation;
- Proposed sensitivity testing; and/or
- Additional scenario development and data acquisition (e.g., coinciding high flows to be used for a receiving water body, tributary inputs).

Additional information and decisions at this stage will also be required based on the plan/method statement. For instance, for the purposes of 2D and coupled 1D-2D modelling, the following additional parameters should be considered, based on the needs of the study, specifically as it relates to grid cells or mesh elements.

Computational spacing for discretization of both 1D, 2D and coupled 1D-2D models is important to determine at the outset of any modelling exercise. Factors to consider include:

- i) **Resolution of available topographic data:** Certain types of data lend themselves better to different modelling approaches. High-resolution data sets such as LiDAR based DEMs are ideal for 2D modelling applications to generate a detailed terrain; however, contour information may be sufficient for 1D model preparation. 2D modelling requires high resolution terrain data to accurately define topography within the flood plain and therefore should only be used if such high-quality data is available.

ii) **Extent of topographic data:** A model, whether 1D or 2D, should extend far enough upstream and downstream to ensure that boundary conditions are not influencing model results. Models should also extend far enough into the floodplain or table lands to contain the inundated flow extents.

iii) **2D mesh type and/or size:** Depending on the modelling software, the user may have the option to use either a structured, rigid grid, in which all 2D cells are the same dimensions throughout the model domain, or a non-structured, flexible mesh, where fine resolution can be used in areas of interest and coarse resolution can be applied to flatter areas or those of lesser concern. Similarly, there may be options for mesh shape including rectangular, curvilinear, triangular or hexagonal. The mesh shape can influence resulting flow patterns.

Cell size is a sensitive parameter for 2D modelling applications (e.g., Musa et al., 2015); therefore, selecting the appropriate size should be done with care to ensure key topographic features are appropriately represented while managing computation time and data storage limitations.

There are a range of useful resources in relation to 2D and coupled 1D-2D modelling available, including some that reference the Ontario-specific modelling protocols (e.g., DHI, 2018).

A synthesis and assessment of the content of the model goals, the conceptual model and/or the model plan will assist in finding the most appropriate software platform to support the hydraulic modelling project.

The choice of software should be based upon the available data and information regarding the river or stream, and the desired amount and quality of outputs. Additional factors that might be considered include:

- a) **Development** – Is the software actively being updated with new features to add functionality to the model or keep pace with the state of hydraulic science?
- b) **Maintenance** – Is the software actively supported by a developer or user community that can address issues such as errors in the model code or compatibility with new operating systems?
- c) **Documentation and training** – What degree of software documentation exists (user manuals, example models)? Are training courses available for the software?
- d) **Cost** – What is the capital cost of investing in the software as well as ongoing costs? Some software requires payment of annual maintenance fees to access

software patches and new versions of the model. This consideration is key not only for model development, but also for each future user of the developed model.

- e) **User base** – Does the software have broad user base both in academic and applied (consulting) settings? Are peer-reviewed applications described in academic journals and conferences? A large user base may provide valuable resources through the collective experience of applying the software and confidence in the abilities of the model software.
- f) **Hydraulic calculation processes** – Hydraulic modelling software may support one or more of the various solution methods in relation to hydraulic processes (e.g., steady state vs. dynamic routing; 1D, 2D, or coupled 1D-2D). In addition to technical requirements, an important factor to consider when selecting the hydraulic approach is the needs of the study, available resources, and experience of the user.

Obviously, no model satisfies all requirements; a general rule should be to select the simplest model that provides the necessary information. In Ontario, 1D modelling continues to be the standard approach for flood hazard mapping for riverine valleys while 2D, or integrated 1D-2D modeling, is typically used for out of bank modeling including spill hazard mapping, employing a ‘quasi-steady’ state approach consistent with the conventional technical policy approach used in Ontario.

Computer models continue to be refined and developed. Ultimately, selection of the most appropriate model must be left to the professional judgement of the agency and individual professional or professionals undertaking the modelling. Any model that is based upon the methods discussed herein, including the principles discussed in this chapter, and which can be calibrated and verified by the user, is widely used and applied by practitioners and satisfies the factors outlined in the list (i.e., items a-f) above, can be considered an acceptable choice.

4.2 Hydraulic Model Selection and Parameterization Considerations

Once the magnitude of a flood to be mapped has been determined by one of the methods described in the hydrology sections of this bulletin, the next step involves converting the streamflow to a water surface elevation at a given location, generally downstream of the reach to be mapped, and computing the water surface profile for the reach.

To determine the stage corresponding to a large flood discharge, it is necessary to extend the stage-discharge curve. As mentioned earlier, for gauging stations maintained by the federal government, preferably, this should be carried out by staff of the appropriate District Office of the Water Survey of Canada. For control points where no gauging station exists, the stage-discharge curve developed for the site must be extended. There are several methods for this that can be found in various texts, but particular care must be taken when overbank flow is involved, which will nearly always be the case when mapping the flooding hazard limit.

Recognizing the different types of flood routing methods, the choice of a suitable modelling technique for a particular river can be very important. Generally, there are two major factors affecting the choice: namely the type of information required from the method and the data available about the geometry of the natural river and previous floods. For flood hazard mapping, the primary focus is on accurately modelling water surface elevations and associated inundation under peak, regulatory flood magnitudes.

4.2.1 Level of Analysis Required

When general flow and water level information is required at selected sections which are far apart (i.e., 20 times the width of the flood plain or more), then a routing method, utilizing simplified mathematical calculations combined with a coarse model of the river valley is usually sufficient, rather than a detailed model of the river channel, flood plain and valley wall. However, when more detailed information on flooding is required along the river, including water levels and mean velocities, then a more complicated numerical model is required.

Another important aspect of the information obtained from a flood routing method is the accuracy of the results. This accuracy will, of course, be a function of the accuracy of the data and the method itself. Apart from the accuracy of input data, the errors in the results will depend principally on the suitability of the basic equations to describe the phenomenon of flood propagation. In general, it is the errors arising from the unsuitability of terms in the basic equations which are the most difficult to eliminate. Numerical analyses are sufficiently well advanced that errors generated by the solution techniques, including finite difference schemes for the equations, boundary conditions and data handling techniques are not a significant problem.

The amount and quality of data from a natural river, both for the geometry of the channel and flood plain, and for previous flood discharges and levels, is another significant factor in the choice of a flood routing method. While established agencies are more likely to have access to robust in-house digital data sets, and geomatics support, others may not.

Fortunately, it is usually possible to obtain general information on a river from topographic maps, in-house GIS services, or from online tools such as the [Ontario Watershed Information Tool](https://www.ontario.ca/page/ontario-watershed-information-tool-owit) (<https://www.ontario.ca/page/ontario-watershed-information-tool-owit>). This information will, for instance, include the length of a reach, the slope of the channel, and the plan area of the flood plain. It is still necessary, however, to know something about the speed of flood peaks along the reach, and the corresponding peak discharges, particularly if water inundates the flood plain. If this information is of poor quality, then nothing is gained by using a flood routing method more complicated than a simple storage method.

Some rivers and streams may have at least one gauging station which can be used to produce hydrographs from previous floods for calibration. The usefulness of recorded water level information will increase if gauges are located within sites that have the potential to be flooded and possibly experience high damages, especially those that may require flood control works. However, it is common for the error in the calculated discharges to be more than 20% of the actual discharge, particularly for high flows. This large error is due to the difficulties in extrapolating the rating curve for flow over an adjacent flood plain. If there is no gauging station available, either a gauging station should be established or discharge hydrographs for previous floods can be derived at the upstream section of the reach using a hydrologic model. Otherwise, indirect determinations of flood discharge based on flood stage information involving hydraulic derivation of rating curves (e.g., Fenton and Keller, 2001; Reistad et al., 2007; Shao et al., 2018) or other indirect methods of discharge measurement (e.g., Brown and Metcalfe, 2014) may also be employed where applicable.

If a detailed numerical model is to be developed, the amount of calibration data increases with the amount of detail required. Such data can be expensive to obtain and depends, in the case of a flood hazard study, on the occurrence of appropriate floods during the course of the investigation. It should also be remembered that if the parameters for a given method have been determined for a particular range of floods, then the use of the method with the same parameters for larger floods can introduce errors (see [Section 5.5 Hydraulic Model Calibration](#)).

There are numerous computer models available for backwater and flood routing computations. These models have been developed over the years based upon established methods and have been tested and calibrated on real streams and rivers.

Computation of water surface profiles is often accomplished using hydraulic modelling software assuming either steady or unsteady-state flow conditions. There are several reliable backwater programs available, and the choice of a particular system can depend

on many factors. One should be used that has been well tested and applied successfully to many different conditions. It should have the capability to incorporate those conditions that will be met in the reach under study. Any proposed program should be deemed applicable and acceptable by the responsible agency.

Many modelling programs have more than one solver or computational process. Each solver has positives and negatives related to computational time, data resolution, precision and appropriateness for specific applications. The engineering practitioner must review the available solvers and choose an appropriate method for the task at hand.

Steady state flow conditions are ideally used in situations with gradually varied flow, minimal storage areas, and where the stage-discharge relationship can be represented by a single valued rating curve. However, there are still situations where unsteady flow calculations can be more appropriate than steady flow, particularly in instances with complex flow interactions and flow paths, tributary input, or reversed flow. Application of unsteady flow solution methods on a broad scale, is strongly discouraged. The following sections should be used as a guide and taken into consideration when selecting a program to compute water surface profiles.

The overall purpose of a hydraulic model is to provide understanding of the hydraulic conditions within a study area. Different types of models provide various levels of understanding of a given situation. As such, it is important to understand the strengths of each type of hydraulic model, including steady and unsteady-state solution types, 1D modelling, 2D modelling, and coupled 1D-2D modelling. Furthermore, when initiating a flood modelling and mapping project it is important to have a good understanding of the river system and where flooding is likely to occur. This allows informed decisions to be made regarding the suitability of a 1D, 2D or coupled 1D-2D flood modelling approach. A description of these options, the various solution methods available and some notable attributes of each is provided in the subsequent sections.

4.2.2 Solution Methods – Steady vs. Unsteady Flow

There are many programs available for steady, gradually varying flow only, using the standard step method of computation. Also, many of these programs were developed for subcritical flow although some can handle supercritical conditions. Steady state flow conditions are ideally used in situations with gradually varied flow, minimal storage areas, and where the stage-discharge relationship can be represented by a single valued rating curve. Additionally, some modelling packages are better suited to either steady or unsteady state analyses. The ultimate goal of the hydraulic analysis (i.e., flood hazard mapping) as determined during the planning stages of the work should be considered

when selecting the appropriate flow regime to be applied (refer to [Section 4.1 Planning for Hydraulic Modelling](#)).

At a fundamental level, unsteady flow describes changes in river discharge with time and downstream distance (i.e., the discharge (m^3/s) is not constant and changes with time). There are also several widely used programs available for unsteady state calculations using the St. Venant or diffusion wave equations. These include the effects of continuity and momentum and can be more physically correct than the steady flow energy equation in some cases.

The use of unsteady state modelling is less common than steady state modelling, and as previously stated, should not be the used as the default approach for modelling in Ontario as steady-state is the default. Unsteady state simulations should not be the default for spill analyses; however, they should be considered when steady state simulations can be shown to produce unrealistic results. Examples of situations where unsteady state analyses may be useful are provided below. The decision for use of unsteady analysis is up to the approval agencies / engineering practitioner based on the characteristics of the study area.

- Assessing flood levels in attenuation areas upstream of cross-channel embankments (i.e., rail and road crossings). Unsteady state modelling can provide realistic flood hazard limits related to actual runoff volumes in cases when steady state modelling provides excessively conservative attenuated volumes, or when quasi-steady state simulations cannot reach a steady state condition and water levels continue to rise to levels beyond the calculated runoff volumes. In such cases, unsteady state modelling can provide appropriately conservative representation for the area upstream of the cross-channel embankment. Steady state results should continue to be used for the area downstream of the embankment.
- When areas of significant natural storage occur within the floodplain. Unsteady state analysis may be applicable for determination of potential storage volumes in these areas relative to the available runoff from the upstream catchment. Steady state results should continue to be used for the in-channel flows.
- Tracking a spill which continues to travel overland for extensive distances without re-entering a channel. An unsteady state simulation can provide insight into the realistic extents of the spill hazard. This particular analysis may also require 2D modelling. Steady state results should continue to be used for the in-channel flows.

Some modelling platforms require unsteady flow or hydrographs as the flow input. This is particularly prevalent in 2D modelling. To be consistent with accepted technical protocols and policies applied in Ontario (refer to [Section 1.3 Flooding Hazard Standards to support flood hazard identification in River Systems](#)) for the purpose of flood hazard mapping, it is appropriate to conduct “quasi-steady” flow simulations in which the flow input is gradually increased and maintained at the peak flow for an extended period to achieve steady state throughout the system. In this manner the selection of 1D or 2D modelling methods is independent of the decision between steady state and unsteady state analyses. The use of a quasi-steady flow regime provides the combined benefits of a conservative approach (storage areas will become filled and therefore not ultimately considered in flood hazard mapping) and 2D modelling advantages (defining complex overland flow interactions and spill areas).

Due to the complexity of unsteady flow calculations, unsteady models tend to be more sensitive to variations in input parameters and therefore can be prone to instability. Input parameter sensitivity can also affect steady state models and model stability.

Sensitivity analysis is therefore a key aspect in both steady and unsteady hydraulic modelling to ensure model stability, as well as to demonstrate the validity of loss coefficients and assumed boundary conditions within the model. Sensitivity analysis is used to achieve an awareness of the 'models' uncertainties, optimize their functions, and identify the key parameters for which the model output is most sensitive (Song et al., 2015). Given the wide range of input parameter values outlined in the literature (e.g., Annis et al., 2020), it is important to know which parameters are the most sensitive (e.g., Pappenberger et al., 2008; Devak & Dhanya, 2017; Chen et al., 2018; Mohanty et al., 2020; Das & Lindenschmidt, 2021a; Mondal et al., 2023). Sensitivity analysis is further elaborated on in [Section 5.7 Testing and Sensitivity](#).

4.2.3 One-Dimensional, Two-Dimensional and Quasi- and Coupled 1D-2D- Approaches

4.2.3.1 1D Modelling

In Ontario, 1D modelling, based on steady state conditions continues to be the standard approach for flood modelling flood hazards. 1D modelling includes the basic assumption that water levels and flow in a river or stream channel occur and can be accurately represented as a 1D system. As such, the direction of flow is assumed to only occur in one direction (i.e., the downstream direction) and water levels across the channel remain constant. Any component of hydraulic forces that do not action in the streamwise direction are neglected in a 1D model. 1D modelling is ideal for systems in which flow in both the

channel and flood plain occurs in one dimension, usually along the channel or valley centreline. River systems suitable for 1D modelling typically include a well-defined channel and valley with gradually varied flows and where lateral spreading of flow is limited (flood plain width not more than three times the main channel width (Neelz & Pender, 2009)) or where storage areas and/or overland flow paths can easily be defined and incorporated (Goodell, 2016). The input data required for a 1D model is rather straightforward (typical surveyed cross-sections and structure measurements); therefore, 1D modelling is suitable in situations where topographic data is available in limited quantity and/or detail. Additionally, the assumptions of 1D flow may still be valid depending on the topographic complexity and size of the floodplain, particularly for steady state flow conditions. It is possible for 1D modelling to be undertaken using either steady or unsteady flow regimes, the availability of each depends on the selected software platform.

If flow in the floodplain is not well defined, such as within an unconfined valley or under a flood spill condition, and particularly in urban environments where surface drainage can be directed in various directions, and/or the extent of flooding and flow direction within the floodplain is not easily defined; then a typical 1D modelling approach may no longer be suitable on its own.

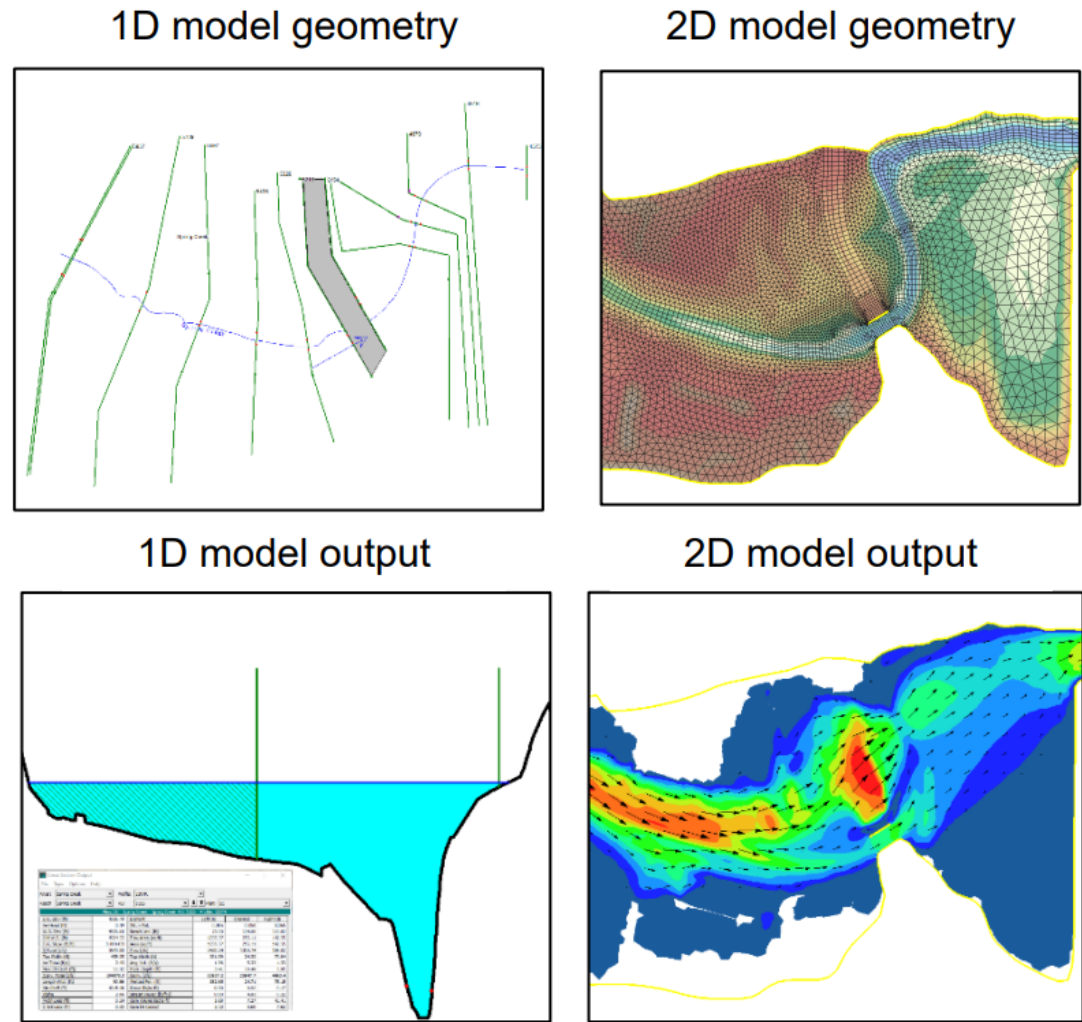


Figure 4-1 Example 1D and 2D model geometry and outputs (Source: Adapted from - U.S. Department of Transportation, 2019)

4.2.3.2D Modelling

2D modelling is appropriate for systems in which overland flow patterns are widespread and complex and cannot be easily defined (Goodell, 2016), or otherwise where flow in the channel and the floodplain is not constrained to a uniform 1D flow direction through these cross sections. It allows for accurate modelling of complex flow patterns around bridge crossings, roadway overtopping including circumstances in which flow does not return directly to the channel downstream, in addition to channel bifurcations, braided or highly sinuous channels, as well as wide, flat flood plains, particularly where flow is noted to flow in any horizontal direction. Systems suitable for 2D modelling typically include urban areas where flood flows are conveyed along roads and around buildings within the flood plain, often in directions other than the direction of main river flow, as well as in

varied terrain with depression storage and ponding areas. The direction of the overland flow in a 2D model is computed based on changes in topography using 2D shallow water equations. 2D modelling is also required in instances where knowledge of flood depths, velocities, direction, and duration are required for the purpose of risk assessment and flooding hazard characterization, as these results are not readily available using 1D models. For example, these results are required for identifying hazards to life safety in Special Policy Areas, or for better defining flood spill areas. 2D modelling requires high resolution terrain data (such as Light Detection and Ranging [lidar]) to accurately define topography within the flood plain and therefore should only be used if such high-quality data are available. Proposed specifications related to data, survey and mapping criteria are outlined in MNR (2023). In relation to these data, one advantage of using 2D modelling is that it can eliminate the need to select and manage river cross section locations and alignments (DHI, 2018).

2D modelling is typically completed under an unsteady flow regime (hydrograph-based input) and is able to account for flow routing and storage through the system. To comply with conventional practices and protocols in Ontario, “quasi-steady” flows shall also be applied in which the flow input is gradually increased and sustained at the peak flow for an extended period with constant boundary conditions to achieve steady state conditions throughout the system/study area. This “quasi-steady” approach enables the user to follow standard practices of not accounting for or not preserving storage components within the flood plain during 2D modelling.

At present, 2D riverine flood modelling is most suitable for modelling projects where there is a need for a detailed representation of overbank floodplain flows, including in unconfined river and stream systems. 2D modelling is not intended to be used across broad spatial scales and not to be used to model entire catchments.

4.2.3.3 Coupled 1D-2D Modelling

Coupled 1D-2D modelling provides the combined benefit of both 1D and 2D approaches. While it is possible to model river networks in either 1D or 2D in some cases it may be appropriate to use an integrated approach. Using the coupled 1D-2D approach, the main channel is modelled using 1D techniques, while the overbank and flood plain areas are modelled using 2D techniques. The two entities are coupled (typically through lateral weir connections) to allow for the dynamic exchange of flows from the channel to the overbanks and vice versa. This methodology provides increased understanding of complex overland flow while also providing improved representation of the channel as compared to a strictly 2D approach, in which very fine detail would be required to accurately represent the channel bathymetry. It also allows for improved accuracy as

compared to strictly 1D or 2D approaches as the channel can be modelled using traditional 1D calculations while the flow in the flood plain can be calculated in two dimensions to provide additional understanding of flow in multiple directions in the flood plain. To comply with conventional practices used in Ontario, the 2D modelling analysis shall be done in a manner to not account for storage, using the quasi-steady approach outlined in [Section 4.2.2](#).

The linkages or coupling between the 1D and 2D portions of a 1D-2D model requires engineering judgement and understanding of the modelling computational processes. The following considerations are required when developing the couplings in a 1D-2D model:

- spacing of cross sections relative to changes in elevation at the coupling;
- cell or mesh size along the coupling;
- flow characteristics at the coupling location;
- couplings are often calculated as a weir between the 1D and 2D components. A location that naturally functions as a weir is best;
- a weir coefficient used in the lateral link calculation; and
- review of mass balance and model outputs to confirm the coupling is functioning appropriately.

Advantages of the coupled 1D-2D modelling approach include that it combines the more classic approaches and formulations associated with 1D modelling, with an ability to more realistically and accurately, represent flooding in more complex areas outside of the main channel (via 2D model). Disadvantages include that it requires developing and maintaining two models as well as the model coupling elements/mechanisms and associated model file formats.

4.2.4 Grid and/or Mesh Resolution

Computational spacing for discretization of both 1D, 2D and coupled 1D-2D models is important to determine at the outset of any modelling exercise. For reference, see *Technical Bulletin – Delineating Flooding Hazards: Data Survey and Mapping Specifications* (MNR, 2023) for recommended raster cell sizes for hydraulic modelling, relative to hydraulic terrain and/or study area characteristics (see Table 3-2 in MNR, 2023).

4.2.5 Cross-Sections

Computer-based hydraulic modelling software should be able to incorporate cross-sections of any shape and should have the capability to subdivide the sections to enable separate analysis of the channel and overbank regions. The number of sub-divisions should be adequate to reflect the varying hydraulic characteristics of the entire cross-section up to the limits of the flooding hazard. It may also be advantageous to have the ability to interpolate between specified cross-sections where velocity changes are rapid.

The software should also be able to account for skewed cross-sections which is necessary where bridge crossings are not perpendicular to the channel.

4.2.6 Critical Depth

Critical depth should be computed at each cross-section to ensure that the water surface stays on the correct side of critical (sub or super critical) and that hydraulic jumps or drawdowns are accounted for as the flow state changes in a downstream direction from supercritical to subcritical or vice versa. This is necessary as some programs continue calculations assuming subcritical flow regardless of the critical depth. Minimum specific energy should be used to calculate critical depth, rather than a simplified approach.

If the flow depth crosses critical, some programs simply assume critical depth is reached at the next section whereas others interpolate between cross-sections to obtain a more accurate location.

4.2.7 Velocity Distribution

The velocity of flow is not uniform for a cross-section and necessitates the subdivision of the section into elements of approximately equal velocity. From these elements, a weighted velocity head can be computed for the section, the accuracy of which increases with the number of elements.

4.2.8 Roughness

It is very important that friction losses be computed as accurately as possible within an allowable range, either locally or regionally accepted or as defined in literature, to reduce model sensitivity to friction losses. The program should enable specification of several values of Manning's "n" for each cross-section as well as for different reach lengths between cross-sections if required. The ability to change roughness coefficients by a

given ratio is useful for testing the sensitivity of the water surface profile to the roughness values.

While modifying Manning's "n" values for sensitivity testing is a useful and beneficial practice, modellers should always ensure that Manning's "n" values remain realistic and within an anticipated range for the substrate and conditions that exist for the reach being model, as determined through field verification or other suitable manner. Modifying Manning's "n" values to the extent that they are no longer realistic or within a range expected for the substrate/conditions expected, particular to make the hydraulic model stable, is not a satisfactory technique or endorsed practice.

Manning's 'n' is a relative coefficient of roughness of the surface under which the water is flowing. This roughness changes based on land use but can also change temporally as vegetation grows throughout the year, snow covers the ground, or farmers rotate crops or let fields sit fallow. The ultimate goal of the hydraulic analysis (i.e., flood hazard mapping) should be considered when selecting roughness parameters. Care should be taken to not over-calibrate to a specific event as the resulting model needs to be representative of the overall conditions of the system.

4.2.9 Use of High Water Marks

It is an advantage for a program to have the ability to use known high water marks to calculate the roughness coefficients. In this case, only preliminary estimates are needed to initialize the program rather than specifying inflexible roughness coefficients which can lead to considerable errors in the water surface profile (Refer to [Section 4.3.2](#)).

4.2.10 Plotting Routines

The inclusion of routines for plotting profiles and cross-sections in the program has the advantage of simplifying the editing of input data and verifying assumptions, as well as providing an output that can be easily understood and directly included in a technical report.

4.3 Criteria and Associated Data to Support Hydraulic Analyses in defining the Flooding Hazard

4.3.1 Hydraulic Coefficients

To determine the water surface profile along the reach of a stream corresponding to a given discharge, it is usually necessary to undertake backwater computations. These

computations normally work in a stepwise manner, determining the water surface elevations at various points along the reach. The important variables to be considered are the hydraulic coefficients which affect the flow within each segment of the reach. Many of these coefficients are well established for all types of structures, such as bridge piers and weirs, and can be found in texts on hydraulics. The most important, however, is the roughness coefficient, Manning's "n", which has considerable influence on hydraulic computations. Although there are guidelines available for estimating this coefficient, considerable experience is necessary to determine realistic values. The coefficient must be estimated for the stream channel and the overbank area inundated by the various flood events. It should be noted that in many backwater programs the coefficient used also accounts for such things as bend and eddy losses. The combination of these is often referred to as Manning's "n" for the sake of simplicity.

As Manning's "n" is an important parameter in backwater calculations and is not a value that can be directly measured, it is worthwhile expending some effort in obtaining a good estimate. This can best be achieved using known water levels and the corresponding peak flow at various points along a reach for a past flood. As the effects of channel beds on the roughness coefficient are not the same for high and low flows, it is not sufficient just to obtain water levels when the flow is low; levels during higher flows are also necessary to determine how the roughness coefficient varies. In general, the value of the roughness coefficient decreases as the depth of flow increases. While Mannings 'n' values should be determined for each specific project, they can be obtained from relevant academic texts and reference documents as highlighted in [Section 4.2.8](#)

4.3.2 Elevations of High-Water Marks

Accurate readings of high-water marks of past flood events are very useful in backwater calculations. However, obtaining reliable readings is often difficult due to the passage of the time between the flood and the collecting of data.

While the methods outlined below may allow determination of the flow-stage of various flood events, it is acknowledged that the flood events of interest to define the flooding hazard correspond to the hundred-year, or greater (i.e., flood produced by Hurricane Hazel, or Timmins storm). These are large events that occur infrequently. However, the techniques outlined below highlight methods that can be employed when a flood meets or exceeds water surface elevations anticipated under the flooding hazard. This data can provide useful information to corroborate or inform existing flood hazard mapping and data. For instance, methods such as the slope-area method could be used to support the indirect determination of flood discharge after a flood has passed, in addition to other calculations of streamflow from measurements of stage (e.g., Brown & Metcalfe, 2014;

Fenton & Keller, 2001). While these techniques are of most value on ungauged catchments, they may also provide utility to gauged catchments, particularly for instances where data from the gauge may be in question.

4.3.2.1 Crest-Stage Gauges High-Water Mark Indicator

An excellent way to obtain these data is by the use of crest-stage gauges at various locations along a reach. They are inexpensive, simple to install and maintain, and provide an accurate elevation of high water. The installation of crest-stage gauges would be worthwhile even if the passage of only one flood peak was recorded, but obviously more data would be an improvement. It is recommended, therefore, that crest-stage gauges should be installed at various points along a reach to be mapped if it is likely that a flood peak will pass during the course of the investigation.

There may be cases where such gauges have been installed and maintained by various agencies, such as Water Survey of Canada, conservation authorities, and/or municipalities, and, if available, this information should be collected.

4.3.2.2 Manual Marking of High-Water Marks

High water marks can also be collected by direct survey at the time of passage of a flood peak. This is usually achieved by pegging or otherwise marking the water level at various locations at or near peak discharge. If the stage is fairly constant and the distances to be travelled are small, a good indication of the true high-water mark can be obtained. The elevations of the pegs or marks can be determined by field survey at a later date, although they should be revisited soon after the flood to find indications of the actual high-water mark. It is rarely known when the actual peak is occurring but, if pegs were placed close to the peak, they should serve as a good guide to locating marks of the maximum stage.

4.3.2.3 Use of Debris Lines and Other Remnant Evidence

Additional field techniques can be used following the passage of a flood peak to discern the maximum flood stage. Characteristics of the evidence used to identify evidence of high water, can vary based on whether the high-water marks were formed in slow moving water or fast-moving water. These include so-called 'trash lines' or debris lines or snags, wash lines, mud lines seed lines, cut lines and ice rings that can be used to define high water marks. These lines should be marked and measured within hours or days soon after a flood event as exposure to sun, wind and rain can quickly remove materials that serve to demarcate the maximum flood elevation. The U.S Geological Service, has produced some useful guidance on how to best utilize these methods to identify high water mark data in the field following floods (Koenig et al., 2016).

Other sources of information on past floods are newspaper files and public archives where documented examples of maximum water surface elevations can often be found.

4.3.3 Aerial Photographs and Remotely Sensed Data

In several cases where large floods have occurred, aerial photography programs have been undertaken to delineate the inundated area. This information can be very valuable for mapping purposes as it gives a true indication of the flood line for a given event.

If the stream is gauged and the discharge is known at the time of photography or the remotely sensed imagery acquisition, the information can serve as a check on water levels, roughness coefficients, and alike. Also, if a large, recorded flood that is to be mapped has been photographed, much of the work in estimating the extent of flooding is eliminated.

While surface roughness is often measured through site visits and field surveys, it can also be estimated using aerial photography based on features such as topography, texture, and vegetation cover. From aerial photographs, dominant landscapes can be compared with those of known Mannings 'n' values. Estimating surface roughness from aerial photography requires expertise and subjective judgement and should therefore be carefully applied.

4.3.4 Cross-Sections

To determine the water surface profile of a given flood discharge, it is usually necessary to perform a backwater analysis along the reach of the stream or streams considered. For this purpose, it is necessary to obtain information on the geometry of the channel and its flooding hazard which is accomplished by surveying cross-sections at various locations. For flood hazard mapping prepared at a watershed or subwatershed scale, a digital elevation model (DEM) is typically used. Cross section geometric data are often extracted from the DEM using RAS Mapper or other GIS enabled software. Since the LiDAR used to create the DEM does not return ground points below the water surface, it is necessary to supplement these areas with surveyed data to create accurate river geometry. Cross sections may be cut into a LiDAR derived DEM and surveyed cross section data fused or conflated into the cross sections generated, using tools such as RAS Mapper. Cross sections in the study reach should include reduced spacing at culvert crossings, bridges and other restricting structures to accurately represent channel flow. Local validation of these data sources is required to ensure currency and accuracy of data. Where bathymetric surveys are not possible due to access issues or health and safety concerns, channel dimensions may be estimated based on typical bankfull channel

dimensions within the reach. For detailed information on methods for acquiring cross-sectional information and bathymetry please refer to the Ministry's *Technical Bulletin – Flooding Hazards: Data Survey and Mapping Specifications* (MNR, 2023).

4.3.4.1 Location of Cross Sections

There must be enough representative cross sections throughout a stream reach to adequately describe the following:

- Contractions or expansions of the channel and/or floodplain;
- Changes in bed slope and roughness; and
- Significant changes in discharge.

Cross sections must also be added immediately upstream and downstream of the following features:

- Tributary inflow locations;
- Dams and other inline structures that act as internal boundaries (e.g., weirs, drop structures, natural drops in the bed profile);
- Bridge and culvert crossings; and
- Where any lateral hydraulic structures exist, including where they begin and end or change in elevation (e.g., levees, dykes).

Where abrupt changes occur in these parameters, several cross sections should be used to describe the change, regardless of the distance between cross sections. Additional essential locations for the positioning of cross sections to inform modelling, include at:

- The model limits;
- Sites of key interest to the modeller and project; and
- All flow and level gauging stations.

All of these features and attributes outlined above must be adequately captured to support robust hydraulic simulations and considering the intent of the modelling (i.e., flood hazard delineation), must sufficiently document the physical attributes that affect flood propagation as determined by storage, conveyance and controls. In some circumstances,

additional cross sections including other features and attributes than those listed may be required to adequately characterise water surface elevations under flood conditions.

For additional information, see the *Technical Bulletin: Data, Survey and Mapping Specifications* (MNR, 2023).

4.3.5 Lake Levels

A problem arises when a stream discharges into a large lake with a backwater effect. For a given flood flow in the stream, there is a wide range of possible lake levels that would be coincident. It is necessary to obtain lake level data in such cases to enable a reasonable judgement or assumption to be made. Decisions on the backwater effect must be based on the variability of water levels and the probable timing of the flood which will define the limits of the flooding hazard. For instance, for rivers and streams that outlet to the Great Lakes or their connecting channels, the 100-year flood level (i.e., peak instantaneous stillwater level) should be used to ensure that riverine modelling accounts for the backwater effect influenced by the receiving water body under defined flood water surface elevations for the specific waterbody. Refer to [Section 5.4.1 Effects of Lakes and Reservoirs](#) for additional details

4.3.6 Boundary Conditions

Boundary conditions are an important component in hydraulic modelling as they represent locations within the model where water either flows into or out of the modelled sequence and are required to determine the water surface elevation at the upstream and downstream sections of the river. It is also important that the model, whether 1D, 2D, or integrated 1D-2D, extends far enough upstream and downstream to ensure that the boundary conditions are not influencing the model results. Boundary conditions can include both external and internal boundaries at the end of each river reach. External boundary conditions typically include flow or stage hydrograph data upstream, and rating curve or normal depth downstream of the river, while internal boundary conditions may include flow data or gate operation. Hydraulic modelling software includes a wide range of boundary conditions that can be applied to the model, depending on the selected software platform and/or the location of the river reach.

A rating curve is a stage-discharge relationship that uses water height to convert into river discharge and will vary over time due to changes in the stream channel from man-made or natural changes. Rating curves are derived from multiple direct discharge measurements along the river or stream and can be constructed using a steady-state model or other online tools. Rating curves can be used from previously completed models

and observed data. For 2D or coupled 1D-2D models, rating curves or open boundaries can be applied to prevent excess water from accumulating at the edge of the domain and causing artificial backwater effects in the system.

4.3.6.1 Flow Inputs

In steady state hydraulic modelling, flow inputs are required at the upstream end of each branch and also at every confluence and flow node to represent the total flow at that location. Flow calculations including hydrograph addition is required outside of the hydraulic model.

In unsteady and quasi-steady hydraulic modelling the flow inputs are applied to the upstream end of each branch and the model calculates the addition of flow at confluences. Additional inflows can be added as flow nodes, but these are additional, not a revised total flow.

A generalized version of this concept is demonstrated in [Figure 4-2](#) below, using a schematic example of a tributary entering a main branch. In the steady state model example (left figure), flows are applied at the upstream ends and the flow downstream of the confluence is entered by the user as the summed flows from each branch as calculated by a hydrology model. Steady state hydraulic models do not take into account flow attenuation due to storage within the system and, therefore, the routing takes place in the hydrology model. In the unsteady (and quasi-steady) state example (right figure), the flows are applied to the upstream end of each branch. Flow at the confluence in an unsteady state model is calculated internally by the hydraulic model to be the sum of the flows in each branch (i.e., no flow change input is required at the confluence).

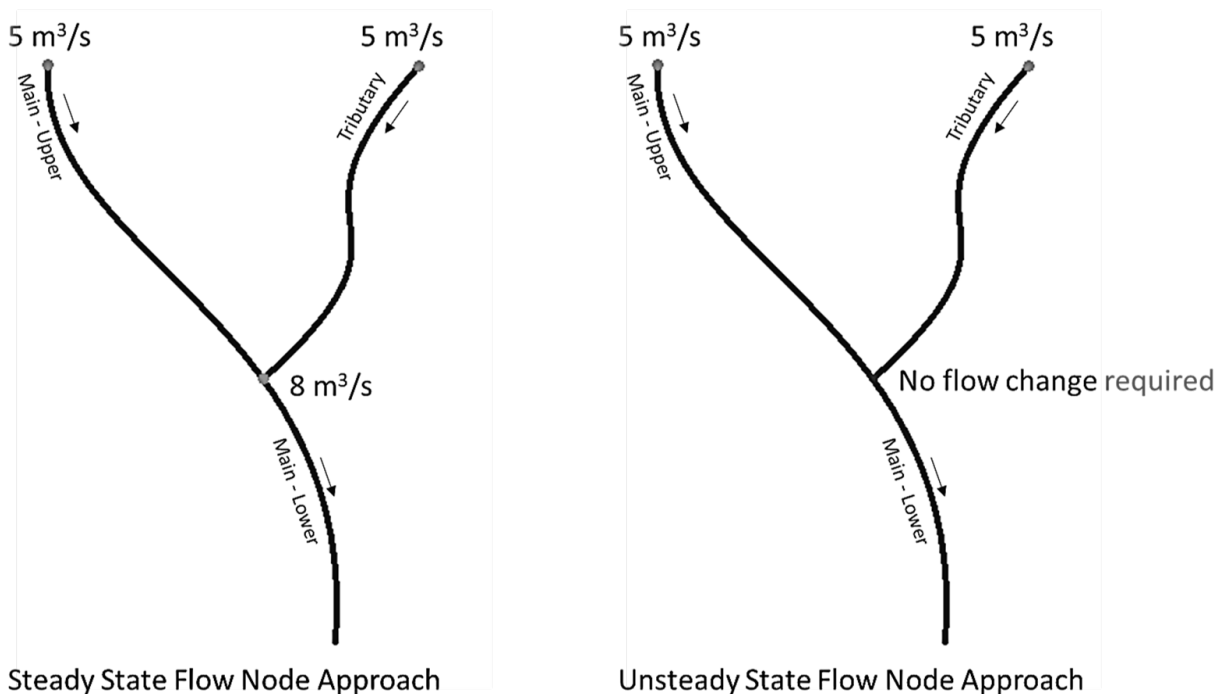


Figure 4-2 Comparison of Steady and Unsteady State Model Inflow Requirements

This example demonstrates the fact that the peak flows in each tributary to a channel typically do not sum directly. The summation of flows occurs through the superposition of hydrographs such that the total flow at the confluence accounts for differences in the time to peak of the two watercourses. This means that the peak flow downstream of the confluence is typically less than a direct sum of the peak flows from upstream. The summation can occur in the hydrology model, as is typically the case in steady and quasi-steady simulations, or in the hydraulic model, as is possible in unsteady models. A sample hydrograph summation following the previous example is provided in [Figure 4-3](#).

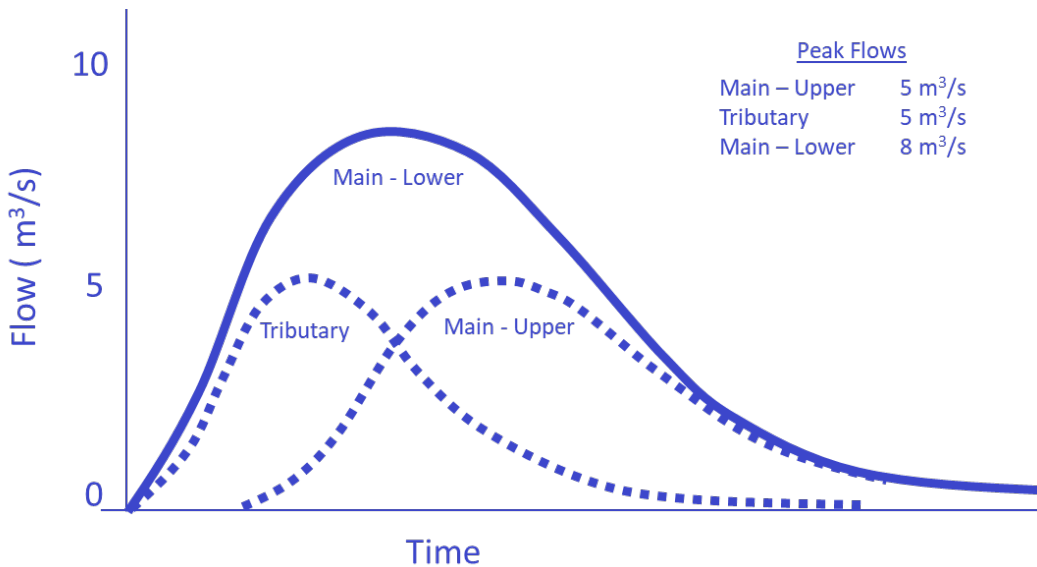


Figure 4-3 Example Hydrograph Summation

Quasi-steady state hydraulic models are intended to mimic the effects of steady state models in that they do not account for difference in timing of peak flows and, therefore, flow corrections may be required at confluences to ensure the summation of flows does not produce an unrealistic flow value. In locations where incoming tributary peak flows in the quasi-steady model sum to a higher value than the hydrologic peak flow downstream of the confluence (as exemplified above), a negative quasi-steady inflow can be applied at the confluence to reduce the peak flow and to ensure specified flows (e.g., from the hydrology model) are maintained downstream of the confluence. Continuing the same example, a negative quasi-steady hydrograph of 2 m³/s would be applied downstream of the confluence to bring the total flow at the confluence back down to 8 m³/s. Similar approaches can be used where natural floodplain storage or other routing reduces peak flows in a manner consistent with provincial direction.

5 Water Level Computations – Open Water Hydraulic Modelling

5.1 General

This section outlines and compares hydraulic analysis and modelling techniques that are suitable for application to a wide variety of Ontario rivers and streams to estimate water surface elevations under the flooding hazard, in support of flood hazard delineation. Where discharge or streamflow may be assumed to be constant over a period of interest, a set of steady, nonuniform flow computations, generally known as “backwater” computations, can be applied. On the other hand, where the discharge varies with time due to a rapidly changing runoff, or due to complex channel/storage interactions, unsteady non-uniform flow computations, known as “flood routing” computations, are necessary to depict the changes in discharge, depth and velocity.

5.2 Backwater Profiles

In most cases, steady flow conditions can be assumed along a particular length of watercourse or river. The water surface profile computations may be based on the solution of the one-dimensional energy equations for gradually varied flow (the Bernoulli equation).

The Standard Step Method is a commonly used method in computing water surface profiles under the above conditions. The calculations are made by successive approximation as follows:

1. For subcritical flow conditions, the calculations begin with a known water surface elevation at the start of a reach (or downstream boundary).
2. A water surface elevation is then estimated for the end of the reach, and energy losses are calculated.
3. The resulting water surface elevation is then computed by adding the calculated losses to the known water surface elevation at the start of the reach.
4. If there is a significant difference between the estimated and computed elevations, a new estimate is made and the computation repeated.

5. When the difference between these estimated and computed elevations is less than some allowable error (about 1 or 2 mm), the computed elevation is accepted, and computations are advanced to the next cross section.

In the case where the backwater computations are to begin at another river, the initial water level should be selected according to the respective travel times of both watercourses. For example, if the peak of the flood hydrograph on the study river occurs at an elapsed time of 16 hours, then the initial water level on the downstream river at the confluence would correspond to the same time on its own flood hydrograph, developed from the storm centred on its own watershed. Depending upon the relative times to peak of both rivers, the initial water level may occur before or after the peak elevation occurs on the downstream river.

Basic data required for calculating water surface profiles are discharge, starting water surface elevation, cross section geometry, roughness coefficients, and occasionally other types of energy loss coefficients. To determine the flow, the Manning formula for open channels is commonly used, and physical geometry is expressed in terms of a plan of channel alignment cross section coordinates, and distances between cross sections. Detailed surveys are needed to establish the boundary geometry at bridges, and photographs of the structure are desirable.

Field measurements of cross sections, discharges, and water surface profiles of actual floods should be used to verify the accuracy of calculated profiles. This procedure requires measurements of recorded flow and water surface elevations to calibrate the hydraulic roughness coefficient (Manning's "n") associated with the friction losses occurring along a study reach. When bridges and other constrictions are encountered, contraction and expansion losses are frequently more predominant than friction losses and should also be verified by field measurement. In this situation, expansion and concentration coefficients will be calibrated using the recorded water levels upstream and downstream of the bridge.

5.3 Flood Routing

In some cases, the problem cannot be reduced to a steady flow assumption because of rapidly varying inflows or tributary flows, the effects of significant channel storage, or complex interactions between channel flow and adjacent flood plain areas. Under these conditions, the flow is considered to be unsteady, and the process of computing the movement of a flood wave through a channel is called "flood routing".

5.3.1 Hydrologic Routing

Hydrologic routing is the term used to describe flood routing in which only the equation of continuity is employed. The form of the continuity equation used in hydrologic routing procedures states that for a given time interval (dt), the difference between the inflow to a reach (I) and the outflow from a reach (O) must equal the change in storage (dS), within the reach, expressed by:

$$I - O = \frac{dS}{dt} \quad (5.1)$$

When the continuity equation alone is used as the mathematical model of unsteady flow, the model becomes linear, in contrast to the fact that the flow is mathematically non-linear. Thus, the success of this model depends upon the extent to which the prototype flow is non-linear. The steadier the flood flow, the better the results of the hydrologic routing model. Linear mathematical models of river flow based on continuity alone do not take into account the upstream travel of waves, such as those produced at junctions, dams, or irregularities in channel cross section.

The simplest application of the continuity equation is obtained by assuming storage to be a function of outflow alone, (i.e., $S = KO$ or $O = (1/K)S$). While applicable for reservoir routing, these methods generally are among the least exact of those which utilize only the continuity equation.

i. Muskingum method

Probably the most widely used hydrologic routing method is the Muskingum method. This method assumes storage to be a linear function of both inflow and outflow, so that:

$$S = K[XI + (1 - X)O] \quad (5.2)$$

and/or

$$O = \frac{1}{k}S - x\left(\frac{dS}{dt}\right) \quad (5.3)$$

where X and K are empirical constants which are found by trial and error for a reach. The coefficient, x , is considered to be related to the conveyance of a reach, while K represents the time of propagation of a given discharge along a reach.

Although hydrologic routing is comparatively easier than other techniques, it has several disadvantages. A large amount of data is required for evaluation of coefficients, and the applicability of the coefficients beyond the range of calibration is doubtful. Also, backwater

is not included, and the solution is heavily dependent on the time and distance increments used in the computational procedures.

The hydrologic or storage routing methods are generally included in watershed simulation models to route or move an upstream hydrograph along a river to a downstream location, such as a reservoir, a confluence or a flood damage centre. These methods simulate stage and discharge in stream channels, when given an upstream hydrograph. When the “routed” hydrograph is combined with the “local” hydrograph representing the contribution of flows only between the upstream and downstream ends, a “total” hydrograph is obtained which represents the contribution of flows from all the areas upstream. Professional engineering judgement is required when deciding where in the sub-catchment to place the flow node within the hydraulic model. There is a need to balance the risks in placement of the flow node such that realistic results are derived (i.e., placement at the top of the sub-catchment risks too conservative results, at the bottom end risks flows that are not conservative enough).

5.3.2 Hydraulic Routing

Hydraulic routing describes those methods in which both the continuity and momentum equations are employed to route flood waves in a river.

i. Kinematic Wave

When lateral inflow is ignored in the momentum equation, it becomes equivalent to a rating curve for steady uniform flow, since the friction slope is equal to the bed slope. This approximation results in the kinematic wave method of routing. The kinematic wave method is considered superior to hydrologic routing methods since the degree of oversimplification is reduced and thus, the degree of uncertainty between model results and prototype behaviour is reduced.

Storage effects due to channel irregularities or off-channel storage may produce subsidence of the flood wave which is not predicted by the kinematic wave theory. Since kinematic waves are defined in terms of continuity considerations, they can therefore only travel downstream, thus backwater effects are ignored.

ii. Dynamic Routing

Dynamic routing models include some or all of the terms in the momentum equation which have been ignored in the kinematic wave models. These models may be 1-D or 2-D. Dynamic routing models possess the capacity of simulating upstream movement of a

flood wave (i.e., backwater effects are included). However, solution of the full equations, especially in two dimensions, is more difficult than the previously described simplified forms. Dynamic models may also be simplified. For example, the spatial derivatives of velocity in the momentum equation are often deleted, as the inclusion of these terms adds little to the computation. For a slowly rising flood the effect of these terms is minimal and, therefore, their removal may be justified.

It is important to recognize that unsteady or dynamic flow calculations have not typically been recognized accepted for flood hazard mapping in Ontario. While steady state analysis can be more conservative than flood routing techniques and flood plain storage is not typically considered in flood hazard modelling for the purposes of flood hazard mapping, it may be accepted for application in appropriate situations, but not on a broad scale (i.e., not catchment scale analyses).

5.4 Special Considerations in Hydraulic Modelling

The following sections describe unique situations and the modelling conventions used to address these situations. Situations requiring special consideration include:

- effects of lakes and reservoirs
- waterway crossings and encroachments
- bridge and culvert losses
- split channel flow
- dykes
- spills
- other factors (floodway, ice jams).

5.4.1 Effects of Lakes and Reservoirs

Lakes and reservoirs act to attenuate flood flows in a watershed; therefore, the storage and outflow characteristics of major lakes and reservoirs must be taken into account in the hydrologic analysis. The resulting design runoff should then be used for establishing required water surface elevations.

A lake also represents a discontinuity in the flood profile along a river that may sometimes be separately accounted for in the hydraulic computations. Therefore, the estimation of maximum lake or reservoir levels is a necessary component of many flood studies. These water levels may be used for:

- initial water levels for commencing backwater computations; direct plotting of flooding hazards (i.e., flood lines) around lake or reservoir perimeters
- estimating flood levels at watercourse inlets or outlets.

Where sufficient records exist, water levels corresponding to specified return intervals may be estimated by statistical means. Maximum instantaneous water levels should be fitted to extreme-value probability distributions similar to the method used in flood frequency studies.

Joint probabilities due to spring flood water elevations and wind set-up may also be estimated from available records.

If a significant amount of regulation exists in a lake or reservoir, it will be necessary to carry out detailed design flood routing computations through the lake or reservoir, according to appropriate seasonal gate settings and rule curves. In most cases, short interval time steps (approximately 1-hour) will be needed in order to define the maximum levels reached under various floods. Hydrologic models generally provide the means to perform these computations. Details on reservoir routing are outlined in [Section 3.6.2.5](#).

Water levels for the Great Lakes are presented in MNR's *Technical Guide for Great Lakes – St. Lawrence River System*, as amended from time to time. Flood levels are determined by calculating the probability of all possible combinations of monthly mean lake levels and wind setups which could result in a peak instantaneous water level having a total probability of being equalled or exceeded of 1% in any year. For rivers discharging to the Great Lakes – St. Lawrence system or its connecting channels, use of up-to-date water level data is important for characterizing backwater effects at river mouths and associated riverine flooding under these conditions.

Determining the relevant flooding hazard limit at the junction of a lake and river or stream is based on a comparison of which *flooding hazard* governs the site, namely the Great Lakes – St. Lawrence River System limit, the Large Inland Lake System limit, the flooding hazard limit for a specific lake that doesn't have a predictable response to a single runoff event (hereafter these three systems are collectively referred to as Large Lakes), or the river and streams hazard limit. In other words, the decision on which limit applies is based on which factors most influence the level of flood risk or hazard at a given location.

This determination may become particularly difficult at a wide river mouth due to the interaction of lake-generated waves with the channel currents and the complex bathymetry at the river mouth, as the waves propagate into the river or stream system flood plain. If the currents in the riverine flood plain are significantly high, the wave propagation pattern (i.e., refraction and diffraction) will be altered and the wave conditions along the connecting channel, which are essential for wave uprush calculation, may be affected. Please note, that where [Figure 5-1](#) refers to the term “Great-Lakes St. Lawrence River System”, this can be replaced with either Large Inland Lake or a lake that does not have a predictable response to a single runoff event, to meet a practitioners specific case.

Determining which flooding hazard limit applies should be based on the following principles:

- a) Rivers flowing into Large Lakes ([Figure 5-1](#)), where the high-water conditions at the confluence are generated by two independent flood events, the flood standard should be based on the higher of:
 - i. Mean annual lake level and the river and stream systems flooding hazard limit (Hurricane Hazel, Timmins Storms, observed or the 100- year event) as shown in [Figure 5-1](#) (Section A-A'), OR
 - ii. Large Lakes flooding hazard limit and the mean annual flood level in the river and stream system, as shown in [Figure 5-1](#) (Section B-B').
- b) Rivers flowing into Great Lakes-St. Lawrence connecting channels also require an analysis of the respective flood levels. Where the high-water conditions at the junction of the connecting channels and a riverine flood plain are generated by two independent flood events, the flooding hazard limit should be based on the higher of:
 - i. Mean annual flood level in the connecting channel (Great Lakes – St. Lawrence River System) and the river and stream systems flooding hazard limit, OR
 - ii. The flooding hazard limit in the connecting channel (Great Lakes – St. Lawrence River System) and the mean annual flood level in the river and stream system.

Where high water conditions at the junction of lake and river are caused by the same type of flood, such as in small lakes (less than 100 km²) or riverine lakes with an effective fetch less than 3 km, the river and the corresponding lake levels cannot be assumed to be caused by independent events. In such a case, the flooding hazard limit at the junction should be based on the flooding hazard limit of the lake and river.

In certain areas, ice jamming or the closure of barrier beach outlets, at the junction of rivers flowing into the Great Lakes, may result in flooding upriver which is greater than

either of the storm-centred events or the 100-year flood, for river and stream systems. In these situations, a detailed technical study using a suitable hydraulic model supplemented by local experience of actual floods should be used to establish the appropriate *flooding hazard* limit. For specific details regarding the Great Lakes – St. Lawrence River flood methods and levels, please refer to the Great Lakes – St. Lawrence Technical Guidance.

5.4.2 Confluence with Rivers

Rivers flowing into a large receiving watercourse, such as the connecting channels which convey flows between Lakes Superior, Huron, St. Clair, Erie and Ontario, also require an analysis of the respective flood levels. Where the high-water conditions at the junction of the two rivers are generated by two independent flood events, the flood hazard standard should be based on the higher of:

- i) mean annual flood level in the smaller river and the flood standard (Hazel, Timmins, observed or 100-year event) in the connecting channel; or
- ii) flood standard (Hazel, Timmins, observed or 100-year event) in the smaller river and mean annual flood levels in the connecting channels.

Where the high-water conditions at the junction of two rivers are caused by the same event, the flood standard is applied to both.

5.4.3 Waterway Crossings and Encroachments

Bridges, culverts, weirs, and embankments create local head losses and rapidly varied flow conditions; these also represent discontinuities in the flood profile and must be treated separately. Energy losses may be due to local contraction and expansion of the flow approaching and leaving the structure, as well as to friction loss through the structure itself.

The coefficient for expansion losses is a function of the change in velocity head between cross sections and is widely discussed in the literature. Bridge losses may be estimated from observations and measurements, or by use of an empirical equation, such as the Yarnell formula. These losses are then added to the total energy at the bridge section, and computations proceed to the next upstream cross section.

In most cases, however, computer models such as HEC-RAS will be applied for backwater analysis. Special routines are included in many of these models to directly handle the losses due to bridges and culverts, based upon the size, geometry, and configuration of these structures.

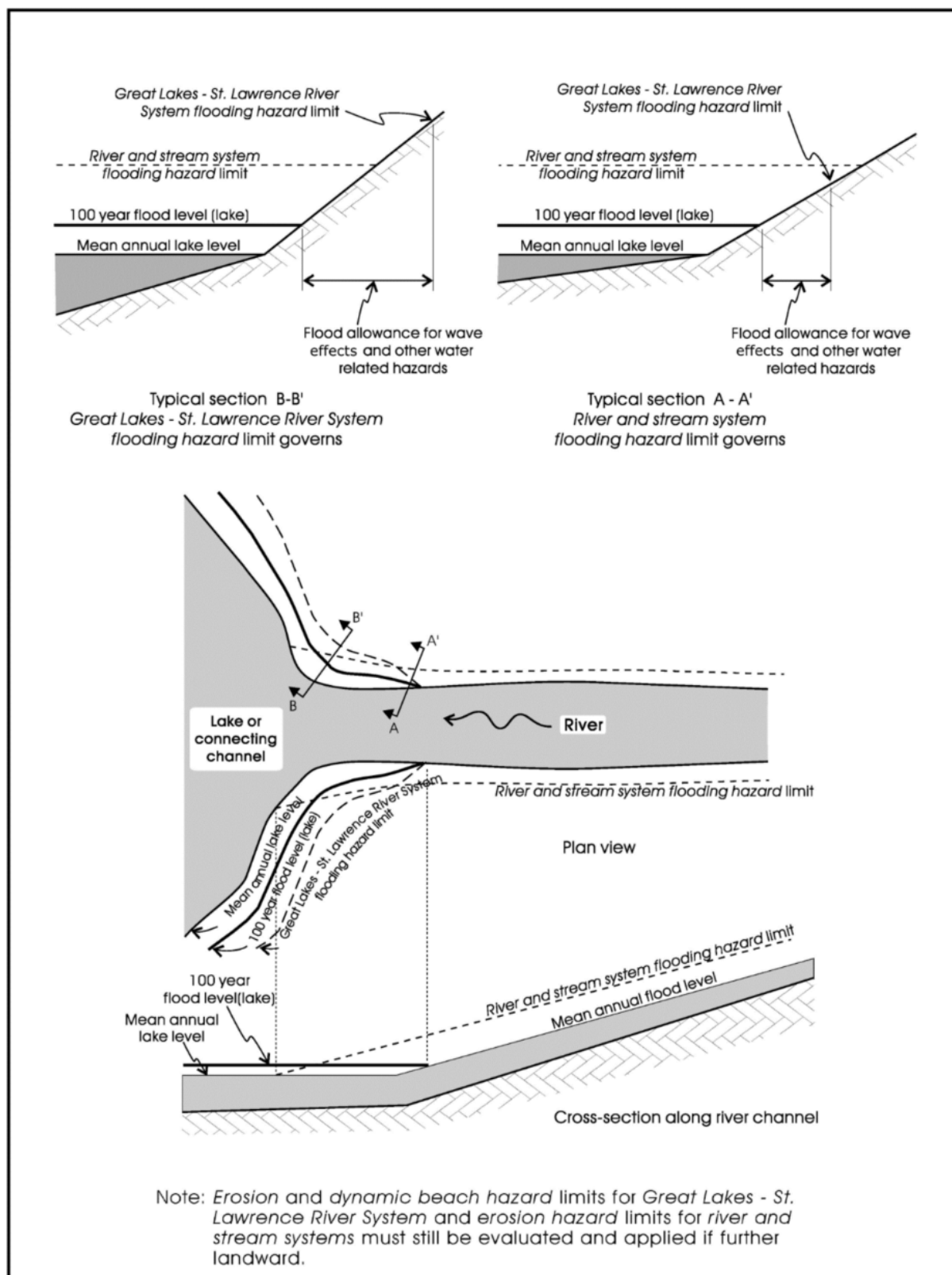


Figure 5-1 Selection of flood standards at the confluence of lake, rivers and streams

It is usually sufficient to include only those bridges and waterway crossings that would significantly affect the waterway. Based upon field inspection, maps and air photo interpretation, many small walkways, trestles, and at-grade roadways may be neglected, depending upon the magnitude of the design flow. All culvert embankments, however, should be carefully considered.

Waterway encroachments, due to urban developments, roadway embankments and structures, may also significantly affect flood plain hydraulics. Many existing hydraulic models have built-in capability to include special routines for this purpose. The encroachment area may affect one or several cross sections. Where numerous buildings have been constructed at-grade, but no actual filling of the flood plain has occurred, the engineer may choose to modify the cross section and vary the flood plain roughness coefficient to represent the encroachment.

A consideration with respect to crossings such as rail and roadway embankments, with small openings (e.g., culverts), includes storage effects on upstream flooding hazard limits. If the embankment is high, with a small opening and the upstream storage is significant, steady state hydraulic modelling alone may overestimate upstream flood elevations and flooding hazard limits. The limited hydraulic capacity through the embankment may cause the steady state hydraulic model to predict overtopping of the embankment to convey the flooding hazard standard flood flow. A flood routing analysis may be needed to confirm the hydraulic model estimates of upstream flood elevations. The flood routing analysis will determine if there is sufficient volume in the flood hydrograph to flood the area upstream of the embankment. It is appropriate to establish flooding hazard limits upstream of an embankment based on flood routing analysis described in the above, when it confirms there is sufficient volume in the runoff hydrograph to flood to the limits of the flooding hazard upstream of the embankment.

The technical convention is to base flood profile computations, for flood hazard mapping purposes, upon existing structural and hydraulic conditions along the river. Where a structure may be replaced in future, or upon inspection of preliminary hydraulic results, a structure may prove to be unstable or breach on overtopping in such cases, additional hydraulic analyses of the resulting downstream flood wave should be carried out.

Flooding hazard limits (i.e., flood lines) upstream of bridges and culverts shall make allowance for backwater effects caused by these structures. Downstream of a culvert or bridge the natural, unimpeded flood line shall be used for delineating the flooding hazard limit, making no allowance for the upstream temporary ponding.

5.4.4 Bridges and Culverts

Bridges, culverts and associated embankments cause discontinuities in the flood profile; therefore, a model should extend far enough upstream and downstream to ensure that boundary conditions are not influencing model results.

Any software used must normally include provisions for computing bridge losses under three possible conditions. The first is a low flow condition when the water surface is below the bottom chord of the bridge. The second is that a pressure flow condition that exists when the surface is above the bottom chord, and the third is a combination of weir and pressure flow when the bridge is overtopped. No allowance should be made for potential scour of the stream bed during a flood in the assessment of hydraulic losses.

If the hydraulic analysis is carried out on a small stream that flows through culverts, the capability to compute culvert losses is required in the program. In most cases, where the flood is considerably larger than the design discharge for a culvert, the conditions would be similar to that of a bridge under a combination of weir and pressure flow.

No allowance should be made for potential scour of the stream bed during a flood in the assessment of hydraulic losses.

5.4.5 Split Channel Flow

This type of flow occurs when the discharge is separated into two or more channels by the presence of islands within the flood plain. In this situation it is necessary to determine the proper division of the flow in each of the channels and the corresponding water surface elevation.

5.4.6 Dykes

Area behind a dyke is regarded as flood fringe area if dykes are high enough to provide protection against the flood standard for the area. Development in this area is subject to flood proofing requirements to flood standard, unless designated as a Special Policy Area.

Dykes and floodwalls may act to hold back water during a flood, but few have been purposefully designed to hold back or contain flood waters during a regulatory event. All floodwalls and dykes will fail at some point depending on the magnitude of the flood event, poor design, lack of maintenance, or because of other complicating factors such as ice jams or flow events larger than their design. Modes of failure can include overtopping, erosion, seepage, and settlement.

A special problem arises where dykes have been constructed in the flood plain for protective purposes. If the dykes are too low and would be overtopped by the flood standard selected for flood plain definition, the land behind the dykes would be in the flood plain and, therefore, within the flood risk area. If the dykes are of sufficient height to contain the flood standard and are appropriately designed, constructed and maintained, the dykes can delineate the extent of the floodway, with the area behind the dyke regarded as flood fringe. For recommended minimum requirements to determine if dykes are appropriately designed, refer to *Technical Bulletin - Special Flooding Hazard Conditions in River and Stream Systems*.

A structural assessment of dykes would not normally be considered as part of the investigation but would likely be specified prior to the commencement of any hydraulic analysis.

Mapping of the flooding hazard behind the dyke or floodwall would use the constrained flood elevation between the floodwall or dyke to map the flooding hazard behind the floodwall or dyke. While dykes and floodwalls alleviate a portion of the flood risk, a residual risk of flooding remains behind, on the dry side of the structure. Dykes and floodwalls would not be regarded as permanent flood control structures and development behind the structures should continue to require floodproofing to the water surface elevation under the applicable flood event standard.

5.4.7 Flood Spills from the Channel

In case of ill-defined channels or top of banks combined with high flows, flood levels can overtop the banks and spill over-land. Frequently, this spill will move into another watershed or join the same watercourse at a distance downstream of where it spilled over land.

The effect of spills moving into another watershed should be assessed to determine the potential flood risks. Alternative measures should be investigated to prevent the spill moving into the adjacent watershed. If the amount of spill is relatively small (i.e., less than 10% of the peak flow), flood hazard mapping for the watercourse should be based on the original flow, without any deduction for the spill. For larger spills, allowance for the reduced flow should only be made where the review of alternatives proves that the spill cannot be prevented, either because there are no feasible alternatives or the costs, when compared to the potential benefits, are too high.

Where the spill re-joins the watercourse further downstream, the identification of the full extent of affected hazard areas related to spills is required. As 1D models may not be capable of identifying the flood path and ultimate extent of flooding once a spill occurs, the use of 2D or integrated 1D-2D models can aid in defining overland flow paths and accurately calculate any loss, and possible recovery, of flow in the channel downstream of the spill locations. No reduction should be made for the spill in the downstream flooding hazard computations.

Flows within the 2D model can be overwritten and adjusted based on engineering judgement to ensure that no flow reduction is accounted for at the spill. Inflows, negative or positive, can be used to force the 2D model to the required flow values.

- If attenuation is occurring (e.g., upstream of a rail embankment), an additional inflow can be added downstream to offset the attenuated flow.
- When flow spills from the channel an inflow can be added to the channel so that there is no flow reduction in the channel.
- Similarly, if spilled flow returns to the channel, a negative flow input can be added, effectively subtracting the theoretical spill, or a portion thereof, as appropriate.

This addition and subtraction of flows, works in both quasi-steady and unsteady flow simulations. Of course, it is much simpler when using quasi-steady state. The manual addition and subtraction of flows will require site specific knowledge and good engineering judgement. Detailed documentation of the flows being added/subtracted and associated reasoning should be included with the model files for future users.

Depending on the route of the spilled flows, the associated hazard area can be quite large, particularly if the flow does not reconnect with the same flood plain. In these cases, the use of unsteady flow analysis may be applicable as the available storage along the spill route could far exceed the total spill volume.

Additional specific details on modelling spills from the channel are provided in [Section 4.2.2 Solution Methods – Steady vs. Unsteady Flow](#) and [Section 4.2.3 One-Dimensional, Two-Dimensional and Quasi- and Coupled 1D-2D- Approaches](#)

5.4.8 Other Specific Considerations

There are several other considerations that should be evaluated in the selection of hydraulic software for computing water surface profiles. It should be fully documented so

that the methodology and the algorithms can be followed, assumptions verified, and changes made where necessary. It is essential that the user fully understands the model's representation of the area rather than treating it as "black box". It may be necessary to add routines for special circumstances that may exist in a particular reach but are not provided for in the original program. This can be done through the use of external calculations or specialty software to develop rating curves etc. which is then entered into the main model to represent a specific condition. Further to this, a range of considerations that the model must be able to consider and quantify are outlined in this section.

i. Floodway

In addition to the points mentioned above, most of which will be common to all streams investigated, there are two other important items that may have to be considered in some parts of Ontario. The first of these items is the delineation of the floodway on the flood hazard maps. Under the option of a two-zone concept, as described in MNR (2002), a floodway has to be separated from flood fringes of a flood plain. The floodway in this case is defined as the stream channel and that part of the flood plain required to convey the majority of the flood. Being that the extent of the floodway is smaller in area than the flooding hazard limit, there must be an increase in water surface elevation. Therefore, the maximum increase will have to be specified when defining the floodway. At first glance this concept appears to offer no advantages as it increases the area of the flooding hazard. However, in cases where the floodway is designated, some types of low-risk development may be permissible within the designated flood fringe area, consistent with the PPS.

Should it be necessary to delineate a floodway under a two-zone concept, as described in MNR (2002), software must be selected that has the capability to carry out the necessary calculations. Normally trial limits of the floodway would be specified, based on the geometry of the channel and the flood plain, and a trial-and-error technique used to adjust the floodway width until the increase in water surface elevations is maintained within the specified maximum increase.

ii. Ice Jams

The second consideration is one that is encountered in some parts of Ontario, in particular an increase in water levels caused by ice jams. Such an increase can be considerable, leading to more widespread flooding than would be experienced with a flood of much larger magnitude without the jam occurring. Ice jams generally occur at a specific location, where there is a constriction in the channel, either natural or artificial. It would be normal, therefore, to survey a cross-section at that point, which

would enable a backwater program to take account of such an event. Should flooding in the entire reach being mapped due to an ice jam, the backwater computations should be conducted upstream of the location of the jam. If the jam occurs within the reach, the procedure must be split into two parts. Downstream of the jam, the normal procedure will be followed for the specified flood. At the location of the jam, a new initial water surface elevation must be specified to compute the profile for the upstream reach. Thus, it is a fairly simple matter to account for ice jams in the hydraulic analysis, if the stage resulting from the jam can be determined.

However, estimating the effects of such jams on a flood of a given magnitude is not a simple problem. Estimates should be based on the history of jams at the particular location, taking any relevant facts into account. It may be possible to develop estimates for the resulting water level directly on a probability basis or by adding the stage effect of a jam to that resulting from a flood of a given probability. It is probable that this type of decision will be made prior to the commencement of an investigation. Additional guidance regarding water level computations in relation to ice jams can be found in Section 6.

5.5 Hydraulic Model Calibration

Having selected an appropriate hydraulic model and set it up to represent a particular reach of river, it is essential that a process of model calibration and testing be carried out. It is likely that errors due to data uncertainties will be greater than computational errors. Since much of the input data (including Manning's 'n', loss coefficients, reach selection) depends upon hydraulic engineering experience and judgement, the model should be compared against observed flows and water levels in the study reach.

Model calibration refers to the process of adjusting the appropriate coefficients and parameters so that the model will replicate an observed flood event as closely as possible. Ideally, sufficient flood records will be available so that the "split-sample" technique may be employed, in which the model is calibrated against one set of flood levels, and verified against a different set.

The following general procedures are suitable for calibration of most hydraulic river models:

- Suitable streamflow and water level measurements are collected by field survey along the watercourse.
- The sensitive hydraulic model parameters are identified and estimated in order to simulate the documented flood event. Normally, these would include

Manning's 'n', expansion and contraction coefficients, and peak flows, but may be extended to include starting water levels, cross section data, and the presence of ice, sedimentation, and other debris

- The ultimate goal of the hydraulic analysis (i.e., flood hazard mapping) should be considered when selecting roughness parameters. Refer to [Section 4.2.8](#). Care should be taken to not over-calibrate to a specific event as the resulting model needs to be representative of the overall conditions of the system.
- The parameters are varied as required, through a process of iteration, until a satisfactory comparison between computed and measured water levels is obtained.
- The model is then verified by comparing computed and measured water levels for a flood event not used in the calibration process.

By this means, a satisfactory computer model can usually be established. If this is not the case, the engineer should examine the study reach for significant factors not included in the model (such as ice blockage) or reconsider the choice of hydraulic model itself.

Some municipalities and conservation authorities have developed standardized hydraulic parameters, including Manning's 'n' roughness coefficients, for use in their jurisdiction. The intent of a standard set of values ensures consistency is applied across the watershed and simplifies the model development for practitioners. These standardized values are established assuming full vegetation growth in flood plains and represent the seasons which flooding potential is highest in our jurisdiction (i.e., late spring to early summer).

Water levels for calibration purposes must usually be measured in the field, but several sources of useful data may be present including historic high-water levels which may be obtained from landowners, residents and local officials (see [Section 4.3.2](#)). The developed model will almost certainly be used for events larger than the calibration data when the primary goal of the model development is flood hazard mapping. For flood hazard mapping the primary focus is on accurately modelling water surface elevations and associated inundation under peak, regulatory flood magnitudes. As such, bias should be towards larger calibration events to reduce the errors of extrapolation.

Flows and water levels may be obtained from stream gauging stations operating in the study area. Environment and Climate Change Canada (ECCC), municipalities, conservation authorities, and various other agencies may operate suitable gauges in the

area. If more than one gauge happens to exist along the study reach, water surface profile information on many floods can be obtained. With the advent of robust autonomous water level gauges and associated loggers, surveying to an established benchmark monument, these technologies can provide useful information to fill spatial gaps in water level information, where required.

Maximum water levels may also be obtained by surveying high water marks in the field after the passage of a flood. In many cases, these marks persist for several weeks after the event, from which a reasonably accurate profile can be surveyed. Measurements along both banks should be obtained and compared, in this case. Flow estimates may be available from an upstream or downstream gauge and adjusted to the reach of interest.

A field monitoring program may be carried out to directly measure water levels during the passage of the flood. Simple crest water level gauges can be installed at intervals along the study reach and read after the flood. Flow records may be made in the field using the Slope-Area method (Brown & Metcalfe, 2014), but it is usually more practical to resort to a nearby stream gauge on the same river, where available. Acoustic Doppler Current Profilers (ADCPs), including bottom mounted ‘uplooking’ and unmanned survey boat hydroacoustic current meters, represent a robust and safe way to capture flow and level information during floods. When equipped with a Global Navigation Satellite System (GNSS), ADCPs can provide detailed bathymetric cross sectional transect information, to support hydraulic analyses under flood flow conditions.

In setting up such a monitoring program, gauge durability, convenient access, vandalism, and hydraulically unobstructed location should be borne in mind.

Hydraulic model calibration in the traditional sense is more difficult for 2D modelling due to a frequent lack of measured flow and/or depths of inundation and therefore users must rely on anecdotal information and photographic evidence of severe flood events to estimate flood extents and depths. The calibration process for 2D modelling is generally the same as that for 1D modelling in which model parameters including Manning’s “n” of the surface, grid cell size, and boundary conditions can be adjusted to achieve model calibration.

5.6 Verification

When insufficient data exists to support a calibration, model performance can be checked using verification. When undertaking a verification of a model, it is important to use all available reliable data to check the model results.

For example, it would be suitable to use a water level gauge on a study reach with no flow data. In this case, for instance the water level gauge could be used to calculate median annual flood levels and check them against modelled two-year return period events (i.e., 50% AEP events) which can provide some level of confirmation of frequent events, but not necessarily those being modelled to define the flooding hazard.

It is essential that a sensibility check be undertaken on any anecdotal evidence. This evidence often reflects small, regular floods rather than extreme floods, and can be less informative. However, anecdotal data collected during past flood events can result in changes being made to models. For instance, early model versions may show flooding of properties at low magnitude events that is not supported by local data. While adjusting parameters to prevent this does not constitute calibration, these changes can help better reflect reality.

The verification process can give valuable insight into model performance when you cannot complete full calibration. It's often the only option to undertake a reality check on results.

5.7 Testing and Sensitivity

The purpose of model testing and sensitivity analysis is to assess the impact of variations or uncertainties in the various calibrated model parameters on flood profiles. The relative importance of the variables is determined by changing one variable, within prescribed limits, and conducting simulations with all other variables held constant. The following parameters are usually considered:

- peak flood discharge;
- channel and flood plain roughness;
- expansion and contraction coefficients;
- starting water levels, tidal conditions, or control gate operations;
- channel configuration, including the spacing, location, and definition of cross sections;
- ice-jamming and debris blockage; and
- sedimentation and sand bars.

Peak discharge and roughness (i.e., Manning's 'n') are usually found to be the most sensitive parameters. However, in some cases, one or more of these other variables may prove to be significant, in which case additional consideration should be given to the field monitoring, calibration, and delineation of the ultimate flood lines on the flood hazard map. For example, Alipour and colleagues (2022) determined that DEM resolutions, mesh resolution and the floodplain roughness coefficient were key factors in the maximum water level prediction in the application of a 2D hydraulic model.

The calculations behind hydraulic models, including HEC-RAS, include sets of equations that are solved repeatedly for each timestep in the simulation. Conditions within the model at the previous timestep can dictate which equation is used in the previous timestep. The software's selection of equation set is based on the combination of tailwater and headwater elevations, including if pressure flow is present. The abrupt change from one equation to another can cause incongruous results. A sensitivity analysis can help to identify and rectify these incongruous results.

If testing and sensitivity analyses identify specific uncertainties, the effect of these uncertainties should be further tested and explored. For instance, if testing identifies sensitivity to a particular uncertainty, it may be important to collect more data such as installing a flow gauging station.

In particular, results from sensitivity runs should be used to:

- establish any change in flood extents
- establish change in peak water levels and flows
- identify if the flood mechanism is changed
- note any increased model instability
- identify areas where changes are predicted and confirm if these align with main sites of interests (e.g., specific flood receptors)
- establish if any findings will affect project conclusions.

6 Ice-Affected Flooding - Water Level Derivations and Computations

Ice can significantly impact flooding at specific locations or reaches, in some drainage basins in Ontario. Background information regarding ice related processes including ice jams and associated guidance, including a description of procedures and methods for considering ice jam effects in hydraulic analyses presented in this section aim to provide the practitioner with necessary tools to consider the role of ice in flooding for specific watersheds. This includes procedures to support the evaluation and determination of ice influenced flooding hazard limits, facilitating comparisons to open-water flood conditions; and a delineation of the extent of ice jam flood hazard limits (i.e., ice-related flood hazard mapping), in a manner consistent with the PPS.

This includes guidance on determining where ice affected floods are the primary cause of flooding hazards, and particularly where these floods have been observed to exceed flood elevations produced by the flooding hazard standards applied in open water conditions (see [Section 5 Water Level Computations – Open Water Hydraulic Modelling](#)). Assessing the potential for ice-affected flooding, including ice jam flooding is an essential step in protecting people and property. A failure to consider ice affected floods under such circumstances could lead to unwise land use planning decisions, and associated consequences, particularly at impacted sections of streams and rivers.

6.1 Evaluating Ice Induced Flooding as a Flooding Hazard

When deciding on an approach to evaluate and estimate the role and magnitude of ice jam flooding hydraulic modelling it is useful to:

- consider the goal of modelling
- identify the appropriate level of detail
- define the components that will be required to adequately evaluate and model ice jams and associated flooding hazards for the river or stream in question.

For ice induced flooding, this includes considering the overall role of ice related flooding in the catchment being considered, as well as the relative magnitude of recent and more historical ice-jam related flooding, relative to floods under open water conditions. Like open-water flooding, in some instances it may be possible to quantify ice related flood potential at a particular location by developing a stage-frequency relationship. However, several characteristics of ice events make them less than amenable to traditional stage-

frequency analyses. The complex processes that lead to ice jam formation, progression and release are highly site-specific. Regional equations, such as those developed for some open-water flood frequency studies, and transposing of data from gauged to ungauged sites, for instance, do not apply (see Kovachis et al., 2017). The definition of flooding hazard in the PPS allows for consideration of floods greater than those produced by Hurricane Hazel, the Timmins flood or the 100-year flood, for example those affected by ice, however it is specified that they must be based on floods *actually experienced* in a watershed or portion thereof, and the use and application of this flood must be approved by the Minister of Natural Resources. This places some limitations on the data, analyses and modelling that can be used to define these observed floods and associate levels.

Ice-related flooding is usually associated with the formation or release of ice jams. Ice jams are blockages to open channel flow that cause a temporary rise in water levels that are higher in stage than floods during ice free (i.e., open water) conditions under equivalent flow conditions (Lindenschmidt et al., 2018). Ice jam flooding can be more severe than open-water flooding as an ice jam can raise water levels two to three times higher than an ice-free or open-water flood event (Beltaos, 2008a), under the same or lower discharge; with a breakup ice jam typically producing greater flood levels than a freeze-up ice jam ([Figure 6-1](#)). Ice effects should be considered when conducting hydrologic and hydraulic assessments to support flooding hazard delineation along river and stream systems. This analysis should include river reaches that have a documented history of ice-related flooding through an assessment of the impacts of ice jams on water levels including relative to the defined flooding hazard standard for the areas of interest.

Ice related floods are highly variable events that are notably difficult to evaluate, particularly with respect to the likelihood of site-specific re-occurrence and analyses of statistical frequency (Lindenschmidt et al., 2018). Foundational information and field data required to support the modelling of ice related flood events are often unavailable, or less readily available, and while flood hazard delineation during open-water conditions is widely applied and accepted in conventional flood hazard assessment and delineation, methodologies for ice-jam flood hazard delineation are comparatively less developed (e.g., Lindenschmidt et al., 2018). This, however, should not preclude further investigation into ice-related flood events to define the flooding hazard and associated flood hazard mapping, where historical data and analyses indicate that water surface elevations under an ice-related flood event have exceeded other applicable flood event standards (i.e., Hurricane Hazel, Timmins Storm, 100-year flood, which ever is greatest). For additional information on planning concepts applied to open-water situations, refer to Section [4.1 Planning for Hydraulic Modelling](#).

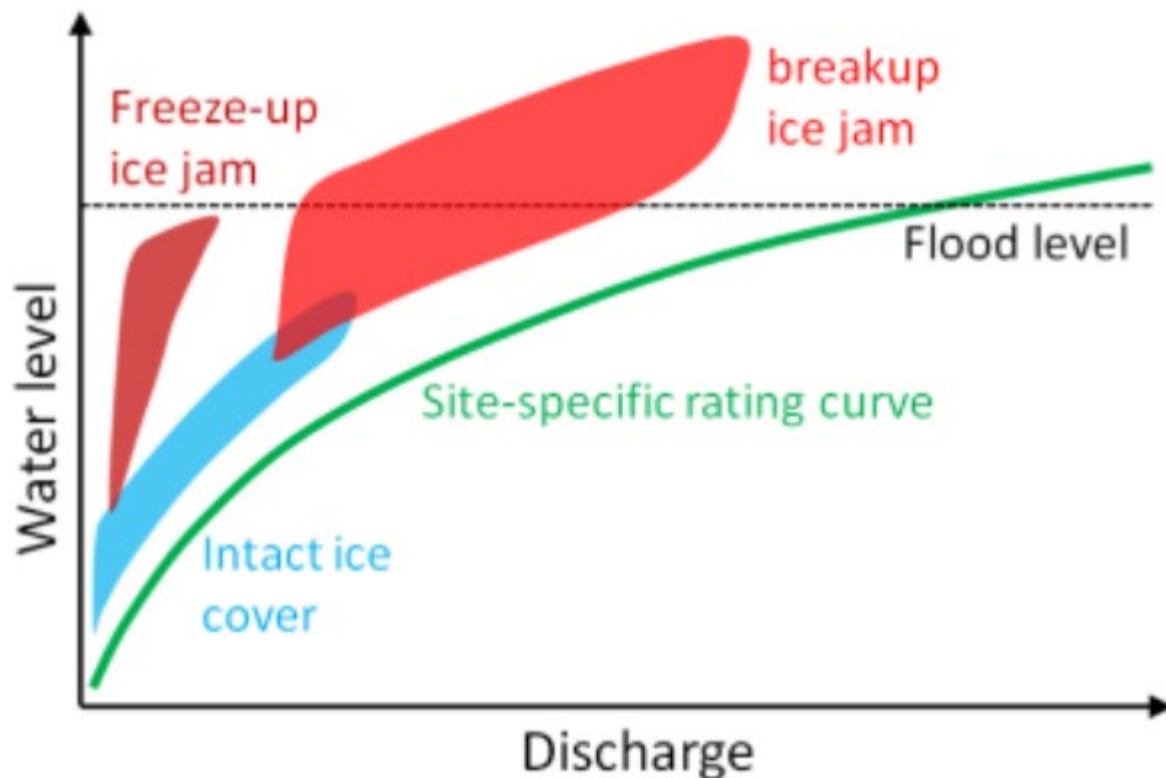


Figure 6-1 Relationship between discharge and stage along a river reach, for open-water conditions (green line – “Site-specific rating curve”) and for ice conditions (coloured polygon areas). The segmented horizontal line defines the stage sufficiently high to cause flooding (Adapted from Barrette and Gaiqui (2023) after Turcotte et al. (2019).

Suitable contemporary methods supporting ice jam flood hazard estimation, corroboration and delineation are outlined further in proceeding sections.

6.2 River Ice Processes and Influence on River Stage

Ice has a significant influence on the hydraulic conditions of rivers and streams in Ontario. On gauged rivers and streams, streamflow is not directly measured, but rather estimated through a stage-discharge relationship that has been established from direct measurements of discharge and water level under open water conditions at the streamflow gauging station; however, under ice covered conditions in a river and stream they are estimated and flagged as such, by assigning a “B” flag by the Water Survey of Canada, to identify ice conditions. Observed relationships of streamflow data, illustrate that under open water conditions there is a deterministic relationship between water surface elevation and discharge, in contrast, under ice conditions, a more scattered distribution exists (Das and Lindenschmidt, 2021b).

Ice-related floods are complex physical processes that occur in many rivers and streams in Ontario. The primary cause of ice-related flooding in Ontario is ice jams, which can occur at freeze-up, at breakup or during a mid-winter thaw. Generally, spring break up ice jams are more common and have the highest potential for flooding and associated destruction, owing to larger natural discharge present at these times and the associated rapid ice break up and large ice volumes. Nevertheless, freeze-up jams have the potential to cause flooding, often occurring in regulated rivers where generation of electricity via waterpower and associated dam operations can result in higher discharges in colder months than experienced under natural conditions, promoting frazil ice and ice floe formation (Andres, 1995; Beltaos, 2021; Barrette and Lindenschmidt, 2023). Typically, freeze-up ice jams are more common in rivers where discharge during freeze-up is high, and often there is a large lake or reservoir in the watershed (Burrell et al, 2015). The potential for ice jam flooding is present in Ontario anytime that ice cover is present. During this period, flooding may result from accumulations of frazil slush, or fragmented sheet ice or both.

Spring-breakup ice-jam floods are particularly common and destructive, as they are associated with high flows, rapid ice breakup, and large ice volumes. A synthesis of relevant Canadian research suggests that in most cases the annual peak ice-influenced water level occurs during the breakup event (e.g., Beltaos, 2021). Content presented herein focuses primarily on ice breakup jam dynamics and estimations but also acknowledges the role of freeze-up (and midwinter melt) jams that form by the same force balance that control the thickness of breakup jams (see Beltaos, 2021). In Ontario's Far North and other northern Ontario rivers, river ice breakup is typically triggered by spring freshet, driven by snowmelt, which can be enhanced by rainfall and rain on snow contributions. In more southerly areas of the province, mid-winter breakup events can be more common resulting from brief thaws and associated rainfall (Beltaos, 2021). These mid-winter ice jams have the potential to be even more extreme than spring ice jams (see Burrell and Tang, 2009; Curi et al., 2019).

These processes and ice jam potential are typically summarized according to the three primary lifecycle stages of river ice, 1) formation, 2) thickening and 3) breakup ([Figure 6-2](#)). Ultimately controlled by antecedent and prevailing hydro-meteorologic conditions, the ice break up on any river reach can be classified as either mechanical or thermal (see [Figure 6-2](#)). Thermal breakup is typically associated with lower flows that are insignificant to displace and mobilize winter ice cover, which typically remains in place and decays by thermal inputs. Significant ice jams are considered to only occur during mechanical break-up events (Beltaos, 2012). As subfreezing winter weather breaks with the arrival of spring and associated warmer temperatures, runoff increases, and progressive weakening of river ice cover begins. Increasing discharge and associated water levels

lead to fracturing ice cover and reducing ice attachments to riverbanks until entire ice sheets from the original ice cover become mobilized. Once in motion these ice sheets can quickly break into small blocks which may become stalled at places where ice cover has remained intact, creating breakup jams (Beltaos, 2008b).

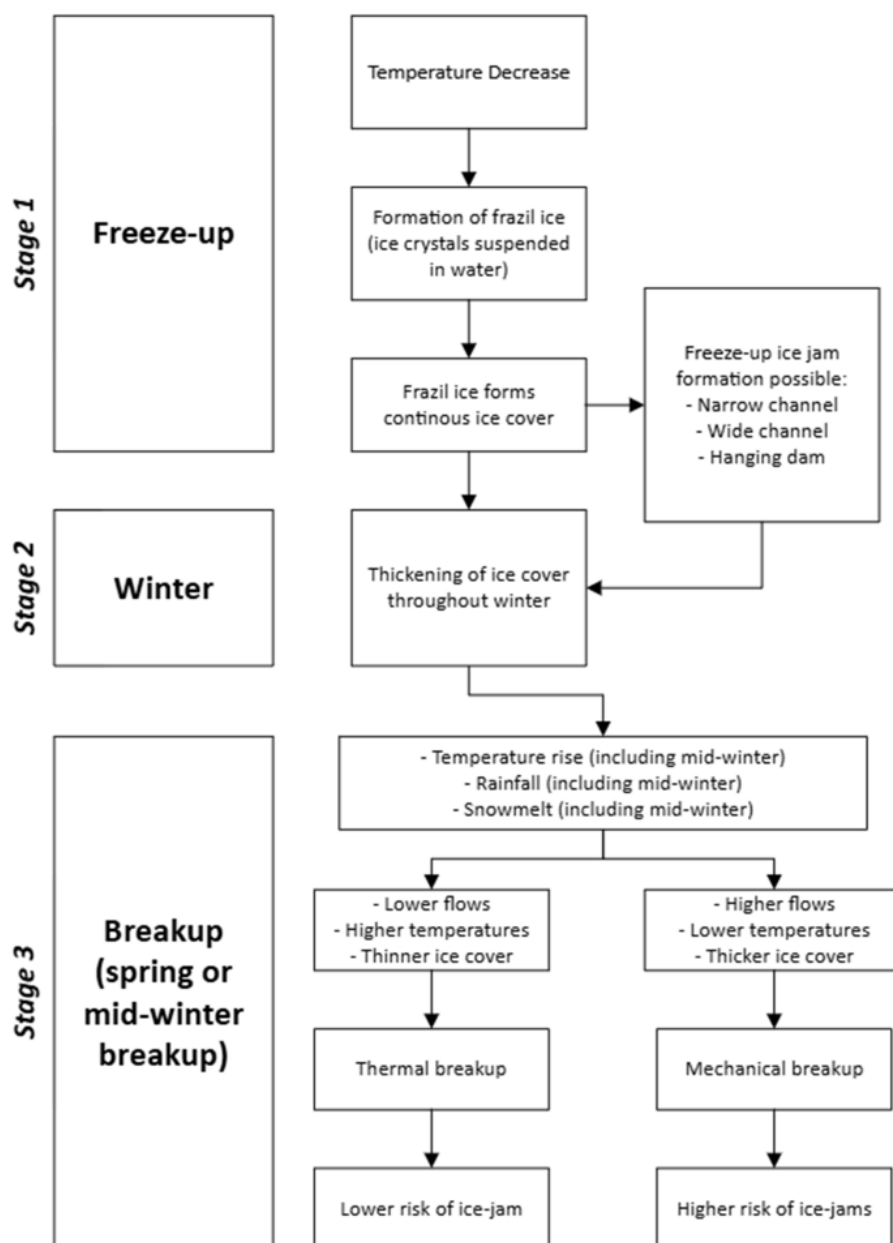


Figure 6-2. Typical processes contributing to ice jams (source: NRCan, 2023)

Numerous ice-jam prone river reaches have been identified in Ontario. In many cases, the ice jam flood levels have exceeded 100-year open-water flood levels, even when combined with relatively frequent (e.g., 5-year) floods. These ice-related events often result in higher water levels and more extensive damage than for open-water events. Most flooding events have arisen from ice jams during break-up. In some cases, frazil ice blockages, culvert icing, and aufeis (flow on top of the ice) accumulations have also resulted in flooding.

Ultimately the main factors impacting the severity of flooding from an ice jam are summarized below (adapted from Beltaos, 1988):

a) Discharge

- It affects both the jam thickness and flow depth under the jam, having a strong effect on jam flood stage. The effect of discharge extends to the surge or jave caused by the release of the jam, as surge characteristics are dependent on the initial water level profile.

b) Channel Width and Slope

- These are important factors controlling the thickness of 'wide' type ice jams.

c) Hydraulic Resistance

- The roughness of the underside of the jam and of the stream or riverbed, influence flow depth and jam thickness.

d) Ice Volume

- The amount of available ice to form a jam can affect the jam stage, especially if it is less than that needed to develop an 'equilibrium section'.

e) Strength Characteristics of the Jam

- The internal friction and cohesion of a jam influences its ultimate thickness. All things being equal, breakup jams, being practically cohesionless, should be thicker than freeze-up jams. In addition, discharge is much larger during breakup jams than for freeze up jams, which explains why breakup jams usually govern peak ice-affected water levels.

f) Water Temperature and Heat Transfer

- Other than the possible effects on freeze up jam strength, melting of a breakup jam can be significant.

g) Strength and Thickness of Ice Cover During Breakup

- Competent ice cover will cause more persistent and frequent jams than a highly deteriorated one. Over time, this could translate to high water levels.

The various ways in which an ice jam may affect the peak water level at a specific river reach, is illustrated in [Figure 6-3](#) and elaborated upon below, after Beltaos (2021). A range of effects on water surface elevations may be observed in the vicinity of an ice dam. In [Figure 6-3](#), the ice jam is located a significant distance downstream of location 1, with no backwater effect observed (i.e., peak stage = open water stage). At location 2 ([Figure 6-3](#)) the jam is located downstream but not far from this location and ice effects are observed (i.e., peak discharge < than the equilibrium jam stage). The ice jam fully affects the site at location 3 ([Figure 6-3](#)) (i.e., peak stage $\leq$ equilibrium ice stage). A jam may be too short to attain the equilibrium condition, depending on the volume of the available ice rubble; in such a case it will not match the full jam effect.

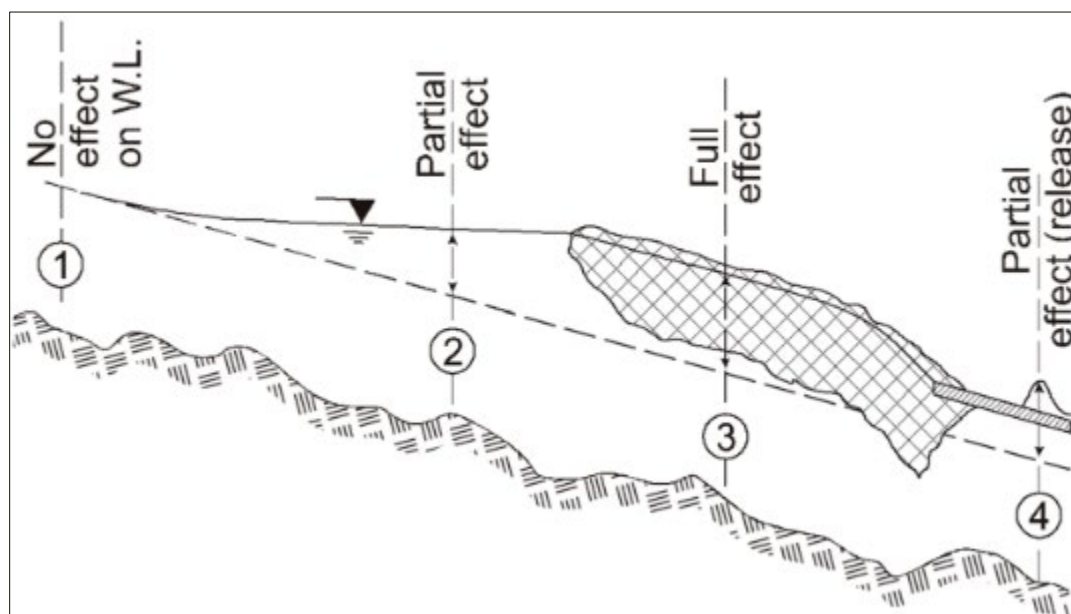


Figure 6-3. Peak Water Level (W.L.) caused by and ice jam as dependent on the location of interest and relative to the location and extent of the ice jam. The dashed line represents the open-water surface profile under the same discharge (Source: Beltaos, 2012).

At location 4 ([Figure 6-3](#)) the jam is located upstream, where the peak stage will equal the peak of the 'jave', defined as the sharp ice-jam wave ([Figure 6-4](#)) that is generated upon ice jam release (Beltaos, 2012). The height of the jave is related to, and smaller than, the difference between the total water depths caused by the parent jam and the usually ice-covered water or open water flow downstream of the toe (Beltaos, 2013). This difference increases with discharge and is enhanced by channel width and channel slope (Beltaos, 2012).

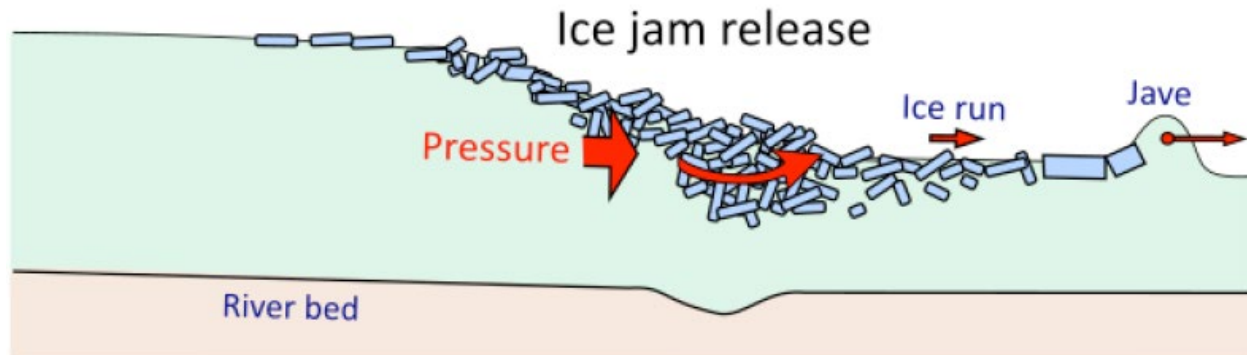


Figure 6-4 Ice jam release with associated wave (or 'jave') and ice run. (Source: adapted from Barrette et al, 2025).

6.2.1 Classification of Thickened Ice Jam Type and Elaboration on Equilibrium Ice Jam Conditions:

Three types of thickened jams are typically reported, differentiated and classified in ice jam literature, two based on the 'width type' characteristics of the jam and the other based more on depth accumulation characteristics, with all including consideration of the forces and formation processes acting on ice jams. Based on this evaluation, thickened accumulation jams can be referred to as hanging dams, "narrow channel", and "wide channel" type jams.

A "hanging dam", with its main formation processes including, submergence transport and deposition, can form during freeze-up and breakup, with its creation and accumulation governed by prolonged transport of frazil ice under an existing ice sheet. The thickness of hanging dams can vary from metres to tens of metres, and are too thick to shove, achieving significant dimensions in deeper river sections. Hanging dams often occur where there is an abrupt reduction in river slope (Beltaos, 2019) and can reside in place for months causing substantial water level rise and associated damage. Often the source of ice inflow is a steep reach, that may contain rapids, which remains open for a much of or the entire winter period (Beltaos, 2021). Therefore, hanging dams can cause

seasonal peak water level in river reaches where no wide-channel type jams form, and the peak water level caused by a hanging dam is not as simply related to discharge as in the wide-channel jam case. The occurrence of hanging dams has been documented across southern and northern Ontario (Beltaos et al., 2007a; 2007b; Wazney & Carson, 2021), and Beltaos (2013) has presented a comprehensive bibliography on hanging dams including a detailed discussion of the relevant physical processes affecting their distribution.

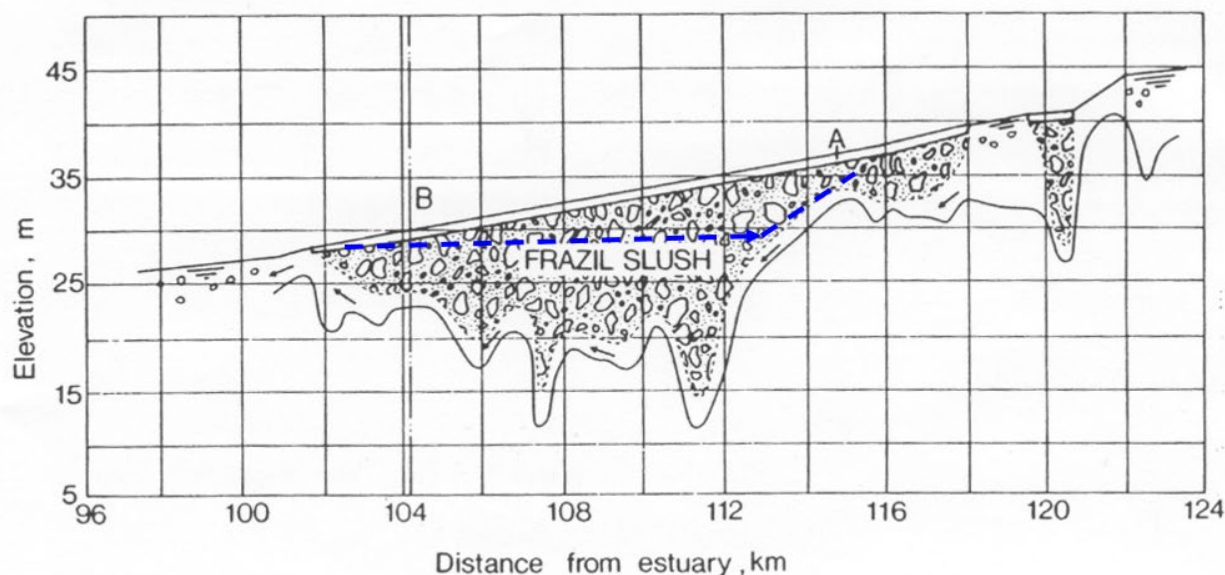


Figure 6-5. Longitudinal profile of hanging dam in the LaGrand River (Source: Michel and Drouin, 1981, adapted by Beltaos, 2019). Blue line depicts solid ice cover in flat portion of reach, downstream of rapids section.

In the “narrow-channel” jam case, where submergence and deposition are the main formation processes, the internal stresses acting within the jam decrease in the downstream direction (i.e., thickness is controlled by hydraulic conditions at the upstream end). No thickening by shoves takes place and the thickness is governed by hydrodynamic constraints at the head of the jam. This narrow condition is very unlikely in break-up ice jams but is common during the freeze-up process. Unless the narrow jam is strengthened by freezing effects, it generally collapses to form a ‘wide-channel’ accumulation jam. The “narrow” type of jam is considered not as severe as the “wide” variety and is not expected to form during breakup in streams with width-to-depth ratios over approximately 10 (Beltaos, 1980). While a narrow channel jam can conceivably form in wider channels (i.e., width-to-depth ratio > 10) during freeze up if thermal effects make it stronger than expected from internal friction alone (Beltaos, 1980), it is uncommon.

With the more common “*wide channel*” jam case, where shoving and collapse are the main formation processes, the final thickness of the ice-jam is governed by the flow shear stress and fragment strength considerations. The internal stresses increase in the downstream direction in response to upstream forces (e.g., shear on the bottom and cover weight component) and reach a maximum value equal to the compressive strength of the jam within a few river widths from the head of the jam. A “wide” channel jam will remain in place as long as the downstream acting forces of water shear and gravity, and possibly wind shear, can be resisted by the jam’s strength and resistive shear stresses acting between the ice and the banks (Zufelt and Ettema, 1997). Beltaos (1987) illustrated that, during breakup, internal friction is practically the sole source of jam strength and therefore collapse-type, wide-channel jams should be the rule for all but very small streams. Wide channel jams are considered more common, are rougher and thicker than narrow jams and more strongly interfere with river flows compared to narrow jams; however, an ice jam can have both narrow and wide regions.

The accumulation thickness of the floating ice jam at this location prevails throughout the equilibrium reach of the jam. Equations derived from the equilibrium ice jam theory are used to predict maximum water levels and flood risk by calculating the stable or steady state thickness based on river ice conditions and discharge. It must be noted that the equilibrium jam concept is most typically applicable to the ‘wide channel’ jam case. Most empirical equations are based on the assumption of equilibrium, particularly under wide channel type jams.

6.2.2 Ice Affected Flooding in Large Rivers Relative to Smaller Rivers and Streams

Much of the ice jam research and applied case studies in northern temperate climates, and polar regions, including those in Canada, have focused on large northern rivers (e.g., Prowse et al., 2007; Rokoya et al., 2018a, 2018b; de Rham et al., 2020). While these rivers are generally similar in size and comparable in latitude to those in Ontario’s Far North (see Grover et al., 1999; Hossein, 2018, Wazney and Carson, 2021), they can be driven by notably different processes, cryographic and hydroclimatic factors, and the relative impacts to people, property and infrastructure may differ compared to more moderately and smaller sized drainage basins within more densely developed, populated catchments in central and southerly areas of the Province. Nonetheless, the important and readily transferrable research, data and analyses, remain applicable to more southerly (Curi et al., 2019) moderately sized and smaller rivers and streams (e.g., Wong, 2021a, 2021b; Curi and Mitchel, 2021) in Ontario affected by ice-jam flooding.

Researchers and practitioners (see Turcotte et al. 2017) have suggested that ice related processes in streams and rivers of small to moderate drainage basins (e.g., $\leq 2000 \text{ km}^2$ among other discriminating parameters) can involve different fundamental processes and characteristics that can influence associated ice-jam related flooding; relative to larger (e.g., $>2000 \text{ km}^2$) river basins. [Table 6-1](#) presents a list of quantitative (hydrological, geomorphic and geometric) parameters that serve to differentiate streams from smaller, ephemeral headwater channels as well as from larger rivers and associated larger channels. These ranges generally correspond to what Meybeck et al. (1996) considers a small river, a stream, or a small stream in their classification of rivers based on discharge characteristics, drainage area and river width. The corollary being those rivers exceeding these parameters ([Table 6-1](#)), are classified as larger rivers, herein. The characteristics, or common classification of jam types, namely narrow and wide, lends itself well to a further classification of ice jams based on catchment area

Table 6-1 Lower and upper limits of parameters that discriminate streams and smaller rivers from larger rivers (after Turcotte et al., 2017a)

Parameter	Lower Limit	Upper Limit
Drainage area	1 km ²	2000 km ²
Bankfull Width	1 m	100 m
Bankfull Depth	0.5 m	5 m
Bankfull Discharge	1 m ³ /s	1000 m ³ /s
Channel Gradient	0.1%	N/A
Stream Order	2	6

To better understand these processes and characteristics and their influence on assessing ice jam flood hazard delineation, further elaboration is provided below and should be considered in the context of the river or stream where flooding hazards are being assessed. A range of different hydraulic, morphologic and cryologic conditions promote ice jamming and these conditions and dominant processes can differ between

larger rivers (e.g., catchments > 2000 km²) and smaller rivers and streams (e.g., catchments <2000 km²) and are strongly influenced by channel morphology.

6.2.2.1 Ice Jam Processes in Larger Rivers

Larger river basins (>2000 km²) characterized by wider channels and typically lower velocity flows in winter under natural conditions, often form more extensive and uniform ice cover. In these large channels, ice related flooding is most often caused by hydro-mechanical breakup events associated with the fracturing and movement of ice as river discharge increases.

Ice accumulations may be formed by ice pieces drifting slowly downstream and coming to rest against the front of an existing cover. If the surface velocity is sufficiently low, these pieces abut against each other without overturning or otherwise being carried beneath the cover. A thin fragmented ice field is thereby formed and the front progresses upstream at a rate dictated by the supply of ice.

At slightly higher velocities, the incoming ice may be entrained at the front of the accumulation and pass beneath the cover and deposit downstream. The incoming ice may also be entrained and stack at the front of the accumulation causing the cover to progress upstream. This depends upon the size and quality of ice reaching the front of the accumulation.

If the ice is entrained into the flow and passes beneath the cover it will deposit in low velocity areas, generally progressing in a downstream direction. On large rivers, the frazil slush has been documented to deposit along the thalweg of the channel. Hanging dams often form as the result of frazil deposition.

The thickening of the ice cover front by ice floes under turning and accumulating by stacking, or packing, etc. is often referred to as hydraulic thickening. The cover progresses upstream at the prescribed thickness, and no further thickening occurs. In this case, the internal forces are sufficiently balanced by the shear at the banks.

Overall, these processes of entrainment, deposition and hydraulic thickening progressively form an ice accumulation which is thicker than that of the individual pieces forming it, and this increases the resistance to flow and raises water levels.

This can be seen from the following equation, which describes the depth of gravity flow under a floating ice cover in terms of the open-water depth for the same discharge (e.g. see Beltaos and Prowse, 2001):

$$\frac{Y_{cover}}{Y_{open}} \approx \left[1 + \left(\frac{n_i}{n_b} \right)^{\left(\frac{3}{2} \right)} \right]^{\frac{2}{5}} + 0.92 \frac{h}{Y_{open}} \quad (6.1)$$

in which Y_{cover} is water depth under ice-covered conditions, while Y_{open} is water depth under open-water conditions, respectively; h is ice-cover thickness; and n_i and n_b denote Manning roughness coefficients for the bottom of the cover and the riverbed, respectively. The first term on the right-hand-side of equation 6.1 expresses the ratio of the under-ice depth of flow to Y_{open} while the second term is the ratio of the submerged thickness of the cover to Y_{open} . It is not uncommon for ice jams to have Manning coefficients two to three times as much as those of typical riverbeds, and a thickness comparable to, or even greater than, Y_{open} . Substitution in equation 6.1 indicates that the presence of a jam can easily raise the water level so high as to attain 2.5-3 times the open-water depth required to pass the same flow. The latter depends primarily on discharge as well as channel slope and width (Lindenschmidt et al., 2018; Beltaos, 2021). When the discharge is relatively large, as is the case in unregulated rivers during the spring breakup period, the backwater can amount to several or many metres depending on channel morphology and basin hydrology (Beltaos, 2021).

The shear force of the water flowing beneath the accumulation and the cover weight acting in the direction of the slope of the water surface increase as the accumulation grows upstream. These forces are transferred through the accumulation to the riverbanks or to the original obstruction. If the thickness of the cover is not sufficient to sustain these forces, the cover will thicken by internal collapse or “shoves” ([Figure 6-6](#)). This thickening process continues within the accumulation until the forces acting on it are sustained by shear at the banks. At that time, no additional load is transferred to the downstream obstruction/accumulation and the thickness of the accumulation reaches a maximum which is called the equilibrium thickness (see [Figure 6-8](#)). This process continues as the cover advances upstream, with thickness varying from reach-to-reach in response to the governing forces. The thickness and roughness of this type of accumulation can be quite substantial and are shown in the walls of fragment ice floes (shear walls) left along the riverbanks after the release or melt of an ice jam event.

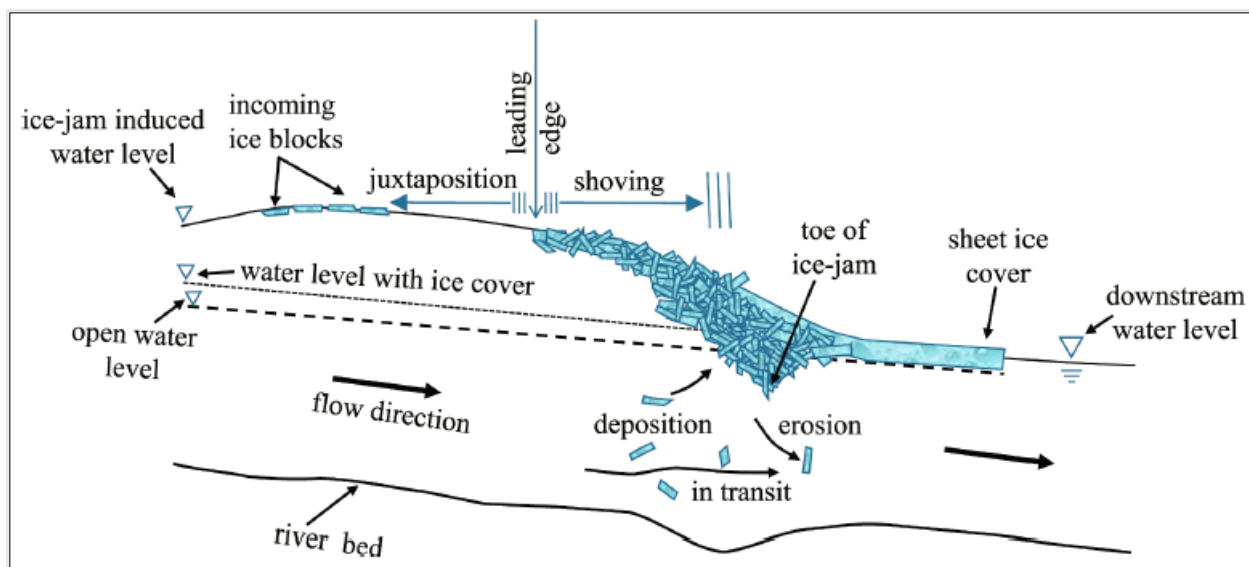


Figure 6-6 Schematic Illustration of a river ice jam and associated backwater level (Source: Rokaya et al., 2022).

This idealized description of the hydraulic and structural factors may often be seen during the regular accumulation of frazil slush or floes during cover formation. Once the accumulation freezes to form a solid cover, the water levels are higher than the open water case for equivalent flows because of the addition of the surface boundary and the displacement effect of the ice ([Figure 6-7](#)).

During ice break-up, a regular accumulation and upstream progression of an ice accumulation which commonly occurs during freeze-up is highly variable, and the process is typically more violent and unpredictable. Irregular blocks of ice and large pieces of sheet ice are swept along with the current to collide with other stationary pieces, crushing them into smaller fragments. Small blockages form and release to affect a general downstream movement of a mass of ice debris. This mass may eventually come to a halt after considerable crushing, abrasion, breaking and piling to form a jam such as that shown in [Figure 6-6 Schematic Illustration of a river ice jam and associated backwater level](#). During this period of time, the pack remains unconsolidated.

Some contact with the bed (grounding) is common for a short section near the toe of the jam. In a grounded or “dry” ice jam, this blockage extends to the bottom across the full width of the channel. The flow obstruction is almost complete, and exceptionally high-water levels can result with the majority of flow occurring in the flood plain. In a “floating” ice-jam, which seems to be the most common type of jam, it is not unusual for a short section near the toe to be grounded across part of the channel width. Save for this short

section near the toe, however, the jam is floating, and a relatively unobstructed flow path is maintained beneath its full length.

Other than additional complexities affected by dynamic phenomena or by hanging dams, the peak ice-induced stage occurring in any one ice season ranges between those water surface elevations generated by sheet ice cover and an equilibrium jam (see Beltaos, 1995) ([Figure 6-7](#)). Barring significant grounding or surges resulting from ice jam re-formation, it has been shown that the highest flood stage which can be caused by floating ice jams occurs within the fully developed or 'equilibrium' section of the jam ([Figure 6-8](#)). The forces acting on ice jams have been evaluated by a variety of investigators to determine the equilibrium thickness. Break-up ice jams between the transitions at the head and toe will attain a constant thickness (equilibrium thickness). Uniform flow will exist in this section, and flood depths are the maximum that can be caused by a floating ice jam. This constant thickness supports a quasi-steady state type condition that allows for modelling of ice jam flood potential and associated water levels, by determining where ice jam thickness (affected by water depth, slope and river width) match relevant stability criteria, preventing further thickening.

The upstream edge of a stationary ice sheet ([Figure 6-6](#)) is the most common site for the formation of break-up ice jams. In ice jam years, this sheet may be more resistant to movement because of bed or bank attachment or thickness, or simply unaffected by the wave of water released by the breakup of upstream jams. In non-jamming years, the cover may melt in place or be significantly weakened by thermal decay or simply be carried away with the passage of water and ice from upstream. This simple break-up process without significant jamming is typical for the province.

Although initiation of an ice jam commonly occurs at a stationary ice cover, other possible sites where stationery ice exists include bends, narrows, islands, bridge piers, bars, and slope transitions. At any of these sites, the downstream movement of the break-up ice may stall and form an ice jam.

A more comprehensive assessment of river ice processes and their mathematical modelling can be found in Shen (2010).

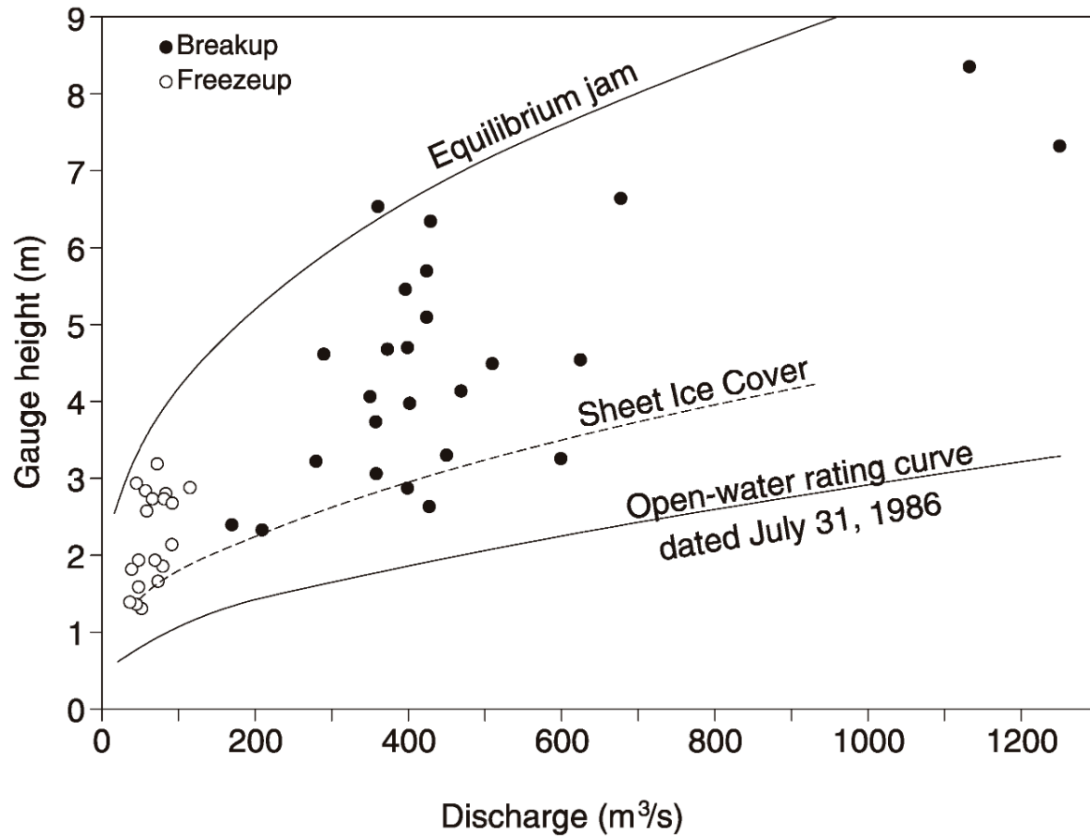


Figure 6-7 Example peak ice-influenced water levels plotted versus associated discharge, as recorded at the Water Survey of Canada hydrometric station on the Restigouche River near Rafting Ground Brook, Canada, 1972-1988. (Source: Beltaos, 2021)

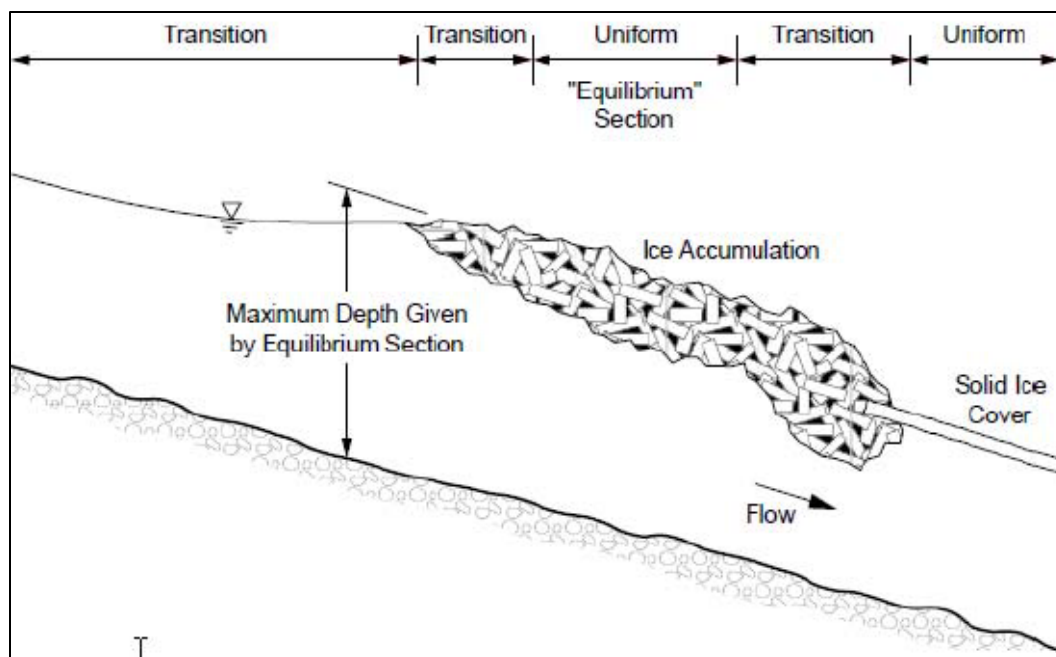


Figure 6-8 Schematic Illustration of an Equilibrium Ice Jam (Source: USACE, 2011)

6.2.2.2 Ice Jam Processes in Smaller Rivers and Streams

In smaller rivers and streams (i.e., <2000 km²), ice jams may be caused by similar or dissimilar processes to those affecting larger rivers as outlined in [Section 6.2.2.1 Ice Jam Processes in Larger Rivers](#). In smaller riverine environments, localized physical characteristics of the channel (i.e., channel morphology) become more important as well as thermally driven ice process such as ice dam formation, which can combine to represent a dominant source of ice-induced flooding (Turcotte et al., 2017). Freeze up jams can also be more prevalent in smaller rivers and streams, influenced by channel shape (Buffin-Belanger et al., 2013), as well as by width-depth ratios, particularly in smaller rivers with a width-depth ratio less than 10 (Beltaos, 1983). Knowledge of these processes and their hydraulic effects is important in assessing the potential impact they may have on representing a dominant event that may define the flooding hazard limit. Particularly relevant ice processes in smaller rivers and streams include, anchor ice, and frazil ice flooding, ice dams, freeze up jams, thickened freeze up jams, hanging dams, aufeis and icing, in addition to break-up jams. The localised occurrence of many of these phenomena function to reduce flow conveyance that can generate a significant backwater impact due to channel blockage. Effects can be particularly significant in shallower stream segments downstream of steep reaches. In these smaller rivers and streams, ice jams can form very quickly contributing to rapid (i.e., flash flood type), localised flooding with little to no warming time.

Channel characteristics where ice jams are common, and typical locations for ice jams in smaller rivers and streams include (after Turcotte et al. 2017):

- At slope reductions, including large pools, reservoir or lake entrances,
- Entering multichannel reaches (bifurcations, secondary and/or anastomosed channels that evacuate water),
- At locations of low and wide flood plains (becoming a floodway; (e.g., Calkins, 1983),
- At sharp bends and constrictions causing flooding upstream (see [Figure 6-9](#)),
- Downstream of confluences (especially when tributaries breakup first),
- At channel enlargements (where the ice run energy is dissipated over a large surface), and
- Against large instream obstacles such as boulders, culverts or in-channel structures

Generally, the presence of intact ice cover is required to initiate ice jamming, hence other than in reaches situated downstream of lakes or large reservoirs, ice jams can occur along any size of small stream or river, regardless of morphological characteristic (Lind et al., 2016; Turcotte and Morse, 2016; Turcotte et al., 2017).

Under these conditions dramatic increases in stage can be observed at relatively low discharge, compared to open water stage discharge curves. The location specific nature of ice jams can lead to more localized ice jam flooding that can result in significant flood related damage resulting from substantial localized ice accumulation.

Channel and hydraulic characteristics in small rivers and streams can be considerably different from those observed in larger rivers. For instance, in small streams and rivers ice jams are more likely to be of the “narrow type”, resulting in lower water levels than the wide channel type jam, and remaining stable only in very short lengths (Beltaos, 1980). In addition, ice jams may be largely grounded (ice in contact with channel bed), and equilibrium may not be met, even if the channel is relatively wide (Turcotte et al., 2017). ‘Narrow type’ ice jams, also referred to as hydraulically thickened jams, are formed by sinking, submergence and entrainment of fragmented floating ice detained at the upstream edge of the jam, whereby ice is forced under existing ice cover (Dow-Ambtman and Hicks, 2012). Ice processes in smaller rivers and streams, particularly characterised by steeper channels, can differ significantly from those in large, particularly low gradient

ivers, and cannot be extrapolated using research results from larger rivers (Turcotte et al., 2011). For instance, overbank flooding associated with ice in small, steep rivers can involve factors such as aufeis build-up or bottom ice, ice dams caused by growth of anchor ice (Daly et al, 2013; Turcotte et al., 2017).

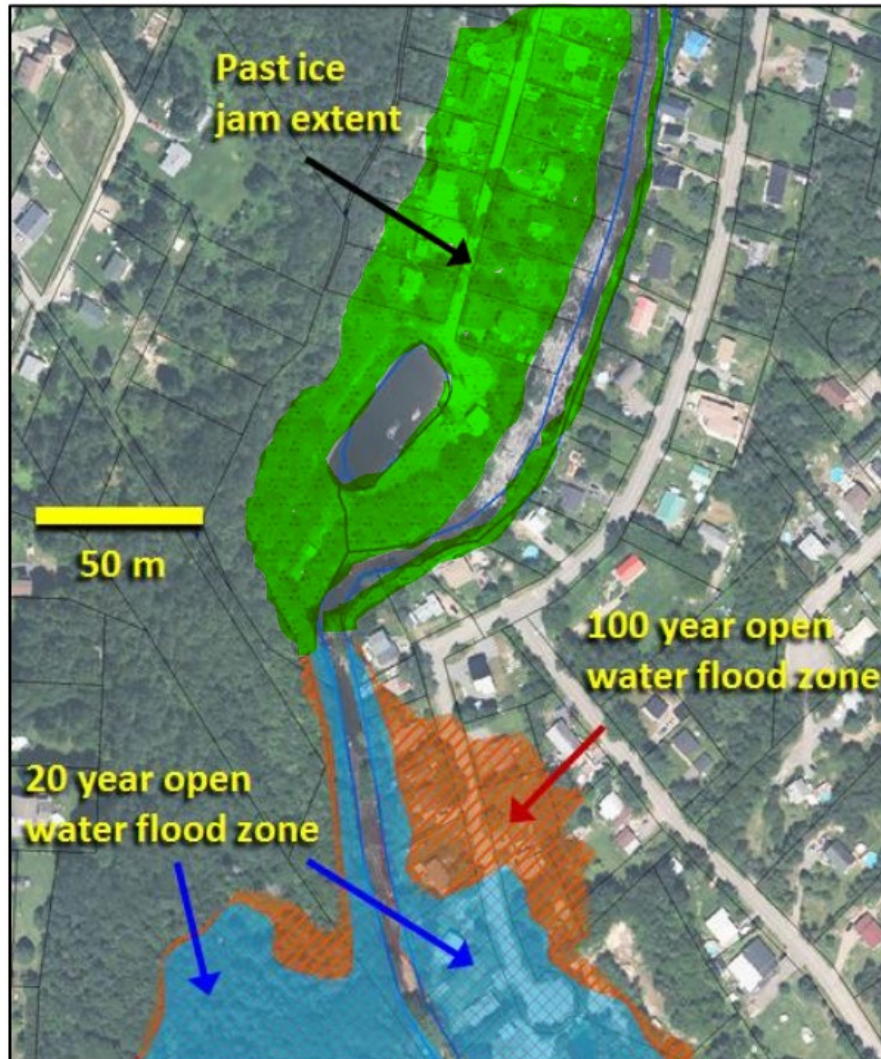


Figure 6-9 Flood map showing open water flood zones and the extent of a past ice-jam flood event (highlighted in green) along the Des Hurons River, a small stream draining a catchment of approximately 75 km² in Stoneham-Tewksbury, north of Quebec City, Canada. Streamflow direction is toward the bottom of the image (adapted from Turcotte et al., 2017).

Overarching hydraulic conditions and channel characteristics in streams and small rivers can be substantially different from those observed in larger rivers. As summarised by Turcotte et al. (2017), these include:

- The ratio of ice thickness (t) to channel width (W) or depth (H) is larger.
- The width to depth ratio is smaller and it cannot be assumed that the hydraulic radius is approximately equal to the mean flow depth.
- The channel width (W), channel gradient (S_o) and channel depth (H) can vary significantly over short distances.
- The average open water velocity can exceed 2 m/s, for instance in rapids sections and supercritical flow conditions are common, especially at high flows.

While it can be observed from the discussion so far that ice jams in streams and small rivers form at similar or comparable locations to those of large rivers, they are more likely to remain in situ where the floodplain is low, bankfull heights are limited or where secondary channels exist, as these act to release pressure on ice accumulations and promote ice grounding (see Turcotte et al., 2017). Consequently, these conditions impose a limit on the jam thickness and maximum water level (H), as reported by Calkins (1983), and while this concept applies to larger rivers, bankfull is not as readily achieved as in streams and small rivers where ice jams are present.

6.3 High Level Assessment of Ice Jam Levels and Ice Jams as a flooding Hazard

In general, it is quite possible that many of the observed ice-effect stages at a site may not have been caused by ice jams in equilibrium. For example, the historical flood levels may have arisen from conditions under the following circumstances:

- Flood peaks from the break-up of an upstream jam (attenuated effects or surge effects from a jam).
- The backwater from a distant downstream ice jam.
- Evolving ice jams which release before building to their maximum flood depth at equilibrium.
- Ice jams which cannot be sustained beyond a given flow rate (i.e., an ice clearing discharge discussed by Beltaos (2008)).
- Ice jams which result in spillage over dykes or banks into the floodplain behind these structures.

- Ice jams which have their development limited by a limited supply of ice from upstream.
- Ice affected flood stages impacted by other forms of ice (narrow type) hanging dams, grounded ice or aufeis, which may be particularly relevant in streams and smaller rivers.

These conditions and their effect on water levels can be accounted for by careful field inspection, data review and analysis, hydraulic models, and by some of the analytical models outlined above.

The importance of accurate historical and field data from the area of interest cannot be overstated. Ice jam water levels are local phenomena which result from site-specific conditions (i.e., a constriction). Unlike hydrologic data which can often be transposed from site to site or generated by regional analyses, the ice jam flood level data at one site have been shown to be distinct at another site, even on the same river, and are not amenable to such transposition. These characteristics differentiate ice jam flood hazard delineation from open water conditions profiled so far in this Guide.

6.4 Methods to Estimate Ice Influenced Flood Stage at Specific Areas of Interest on Rivers and Streams

A range of possible methods exist to support efforts to better understand the influence of ice affected flooding on flood elevations in the reach of interest. This includes for flooding hazard identification, as well as supporting an assessment of flood jams as a flooding hazard, relative to the established flooding hazard standards for the relevant flood criteria zone. Ice jams are known to be much more site specific than open-water flooding, as ice affected flooding can occur in one specific location of a river that remains flood free a few kilometres upstream or downstream of this location (Gerard and Davar, 1995). Some of the initial questions that must be asked by the practitioner undertaking the study are:

- a) are ice jams or ice affected flooding known to occur in vicinity of the area to be mapped? and;
- b) if so, are there data to support, and assist in defining the stage and /or extent of the flooding?

Many different approaches have been summarised to support ice affected flood hazard delineation, by way of defining the depth and extent of these flood events. These approaches and their classifications have been consistently advocated by well

established Canadian researchers and practitioners in the field of ice influenced flooding (Burrell et al., 2015; Kovachis et al., 2017; Lindenschmidt et al., 2018; Das & Lindenschmidt, 2020; Beltaos, 2021, Burrell et al., 2024). Approaches are organized into four basic categories and essentially focus on the range of data sources and methods to inform ice jam flood stage, including the:

1. Biophysical/environmental approach, using physical characteristics (e.g., vegetation, soils, debris lines) of topography and ecology to identify high water levels and the extent of flooding.
2. Historical/past flood extent approach, utilizing field (e.g., high-water marks) photographic (aerial photo, satellite imagery) and personal account-based information.
3. Flood envelope approach, integrating data from multiple past flood events.
4. Hydrotechnical approach, typically involving the use of applied hydrology to derive design floods and applies this design flood information to a hydraulic or hydrodynamic modelling analysis to obtain ice influenced flood profiles.

While the basic intent of these approaches is to derive suitable data to support the development of stage-frequency distributions of ice-related flood events, they arrive at those estimates using different and disparate data sources. Furthermore, while it may be useful to rank ice affected flood events, in particular to gain an appreciation of maximum flood stage, a consideration of ice affected floods in Ontario does not require knowledge of specific ice jam return period floods (i.e., 1:50 yr, 1:100 yr or 1:200 yr etc.); making the development of stage-frequency plots and assignment of return periods or AEPs to these stages less vital. Especially since it is not necessary to use the frequency analysis or associated plot to extrapolate to values beyond the range of observed values. Rather, the focus is on the maximum observed ice affected flood *actually experienced* in the portion of the watershed that comprises the area of interest for the flood hazard study. These approaches are profiled in more detail in the following paragraphs, particularly for the purposes of application in Ontario. Furthermore, a hybrid approach is advocated for deriving and developing data for ice affected flood stages that are actually observed, but that may not have been captured or documented by a hydrometric gauging station.

6.4.1 Biophysical Approach

The *biophysical approach* uses physical characteristics (e.g., vegetation, soils, and debris lines) of topography and ecology to identify the flood extent areas. To determine the ice-jam flood extent, this includes vegetation trim lines, debris lines and ice scars are used

(Lindenschmidt et al. 2018) and frequent aufeis accumulation on riparian vegetation may also be discernible (Turcotte et al., 2017). Most often the biophysical approach is used as a preliminary step in flood hazard delineation (Gerard, 1989). It can also be used to delineate final flood hazard areas exclusively if sufficient biophysical data are available to support that determination. The other approaches benefit outlined above and below benefit from the complementary data collected via biophysical approaches. For instance, using dendrochronology or dendrogeomorphology, researchers can collect tree scar data including physical samples that allow back calculation of when the flooding *actually* occurred as well as how high the flood was that the given location where the tree was located (Lubinieki et al., 2022) (e.g., [Figure 6-10](#), [Figure 6-11](#)).

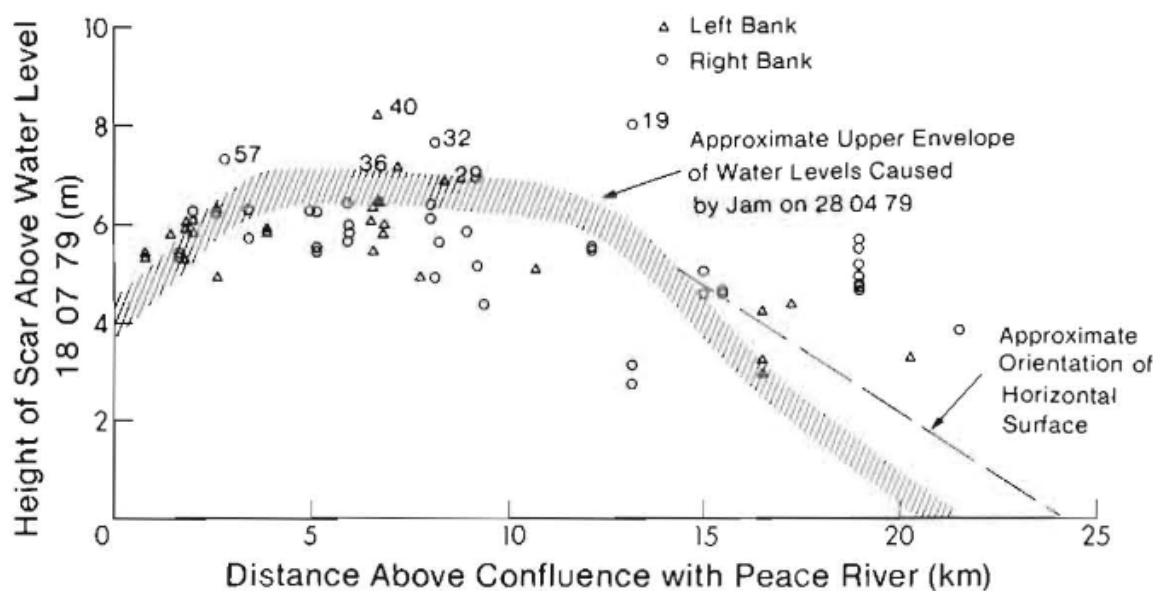


Figure 6-10 Water elevation affected by ice jam on April 27-29, 1979 and elevations of ice scars caused by this event (Source: Gerard, 1981).

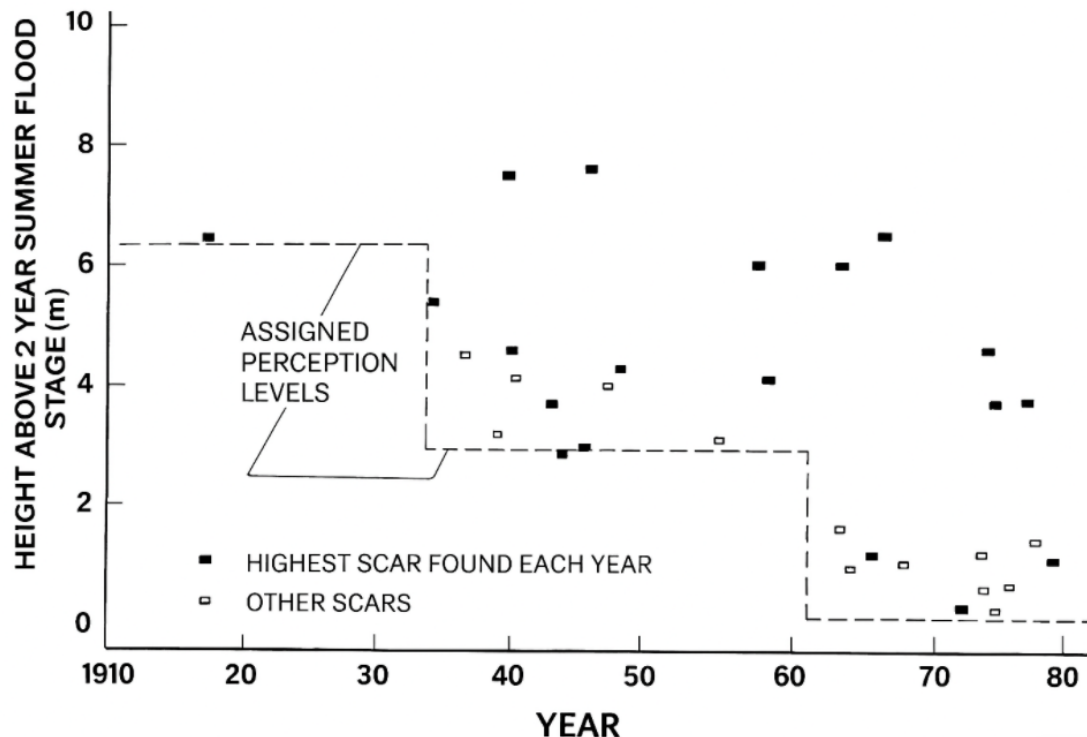


Figure 6-11 Ice scar record with assigned 'perception levels' for three separate time periods, Peace River near Fort Vermillion, AB. (Source: Modified from Gerard, 1989).

6.4.2 Historical Flood Approach

One of the most convenient approaches for identifying flood hazard areas is historical flood extent. In this approach, field (e.g., high water marks) and photographic (aerial photo, satellite imagery) information from a major past flood event are used to estimate flood extent and depth against river distance (Burrell et al. 2015).

Accurate flood-hazard-area delineation does however, depend upon the information on flood extent and (or) depth having been collected near peak floodwater levels. Flood profiles are prepared by plotting flood elevations obtained by levelling water levels during the flood and high-water marks in the field or shown in photographic records or residents' recollections. Aerial extent of ice affected floods can be determined from aircraft or satellite imagery collected during the flood event, and by plotting flood profiles against topography. This approach has the advantages of being easily understood and defensible, as the event has already occurred, and it is applicable to ice-affected floods. Past flooding, even if its probability of occurrence is not known is a good indication of potential future flooding and associated consequences including potential harm to people and property, as flood hazard potential can be estimated based on the location and occupancy of the area that was flooded. Other recent examples of the use of the approach

(e.g., Pagneux et al., 2010) include to enhance time series data of historical and contemporary events obtained by discharge monitoring, including taking full advantage of the spatial references mentioned in newspapers and dwellers' diaries, and using a GIS to link those to topographic elevations.

Residents' recollections of past ice jam flood elevations and timing, referred to as, Historical Information and Traditional Knowledge (H-TK) have been evaluated post hoc, by Beltaos (2025). Overall, the H-TK record of large ice jam floods was shown to be very reliable, in the sense that all or nearly all the reported events, did actually occur. Because ice jam studies that include interviews and querying of local inhabitants rely on accurate information about the magnitude and year of occurrence of large floods, validation of the H-TK record of major events by Beltaos (2025) enhances confidence in assessments of future flood regimes and the use of similar approaches in Ontario.

In this way the historical flood approach can actively incorporate components of the biophysical and flood envelope approach and is further discussed in the following sections.

6.4.3 Flood Envelope Approach

The flood envelope approach consists of surveying evidence of past flooding, regardless of cause, and plotting the obtained elevations against river distance to derive an envelope curve of maximum flood levels. The envelope curve is an assumed flood profile that can be applied to topographical information to determine the extent of the flood hazard. The flood envelope approach is similar to the mapping of a single historic flood event, except that the flood profile may be based on several different flood events as defined the elevations that are used. Therefore, it is particularly applicable in areas of limited data availability for application of other approaches to flood-hazard-area delineation. There may be past accounts of historical flooding, ice scars and other physical evidence, and personal accounts that could be used to ensemble enough information for evaluation of flood hazard and for delineation of a flood-prone area based on methods such as described by Burrell et al. (1988) and Stanley and Gerard (1992). This approach involves applying an assumed flood profile elevation or series of elevations that can be applied to topographical maps or DEMs to delineate the extent of the flooding hazard.

6.4.4 Hydrotechnical Approach

The hydrotechnical approach involves the application of hydraulic or hydrodynamic models for deriving flooding hazard limits (Burrell et al., 2015). The approach is suited for situations in which information on past flood stages is limited, and can play an important

role in corroborating and/or elucidating results from the application of approaches 1) to 3) outlined above, or where detailed analyses are required about a specific ice related phenomena such as with hanging dams (e.g., Beltaos et al., 2007a; KGS Group, 2007; Beltaos, 2012). In such instances, the available information is used to calibrate an analytical or numerical model that links ice jam stage to river discharge. Discharge is a variable that is much more readily available than ice jam stage and can be transferred from other locations. Once a model is calibrated, it can be applied to all years of record to synthetically reconstruct annual values of peak ice-influenced stages. However, it must be acknowledged that peak ice influenced stages reconstructed in that manner do not equate to an actually observed flood, as is required in Ontario as per the PPS. Therefore, the use of the hydrotechnical approach in Ontario should be used more to synthetically reconstruct and corroborate, check or confirm actually observed flood stages derived from the other classes of methods outlined above.

Despite the above outlined restrictions or limitations regarding the use of the hydrotechnical approach at defining an *actually observed* ice influenced flood, where no direct evidence is available in the context of Ontario's PPS, there is value in further highlighting some details regarding these models. While the, hydrotechnical approach can delineate flooding hazards even if the information on past flooding is insufficient or unavailable, this approach requires extensive field data collection and analyses, mathematical computational resources and in-depth knowledge regarding the processes; as such it can be more expensive than other approaches, and involves specific expertise beyond that of open-water modelling, requiring involvement of experts on the topic (Kovachis et al., 2017, Lindenschmidt et al., 2018; Burrell et al., 2024).

A variety of non-proprietary, open-source river ice models have been developed over the past several decades. For a description of the variety of these models described in the literature, see Barrette et al. (2024), and for a summary of these models for consideration by practitioners for the purposes of flood hazard delineation, refer to Das and Lindenschmidt (2021b). A thorough review by Carson et al. (2011) has compared the performance of some existing models for documented ice jams (for both calibrated and non-calibrated modes). Complementary to this, White (1999) presented a comprehensive overview of physical factors that affect the water levels caused by ice jams, including their numerical values, as they have been deduced from model calibrations (Burrell et al., 2024).

6.4.5 Summary

Respective subsections serve to highlight recommended approaches for use by practitioners in Ontario, and are affected by the available data, and the overall intent or need of the study.

Furthermore, and as previously indicated, to meet the intent of the PPS definition of 'flooding hazard', an event larger than Hurricane Hazel, the Timmins Storm or the 100 year flood (depending on where the site of interest is located in the province), such as from ice affected floods, including ice jams, this flood must have been actually experienced in a particular watershed or portion thereof. Hence, while application of analytical or hydrotechnical modelling approaches may be used to corroborate observed (i.e., "actually experienced") data, they cannot be used to predict what possibly, "could occur" at the site of interest, under the right conditions. Such modelling, and subsequent application of modelled flood elevations not actually observed, would not be consistent with the PPS.

Many models, including hydrodynamic models exist to predict and describe many components of ice jams, including for ice jam formation and break-up, the purpose of methods advanced herein are to calculate, predict or estimate ice jam flood stage for assessing the ice affected flooding hazard elevation, to support comparisons to flood elevations under open water flooding hazard standards (i.e., Hurricane Hazel, Timmins storm, 100-year Flood stages). All models employ basic physical understanding that has been gained over several decades of relevant research and informs the formulation of relevant equations (e.g., see Pariset et al. 1966; Beltaos, 1983; Beltaos 1995; Shen 2010). It must also be emphasized that all methods for evaluating and estimating ice influenced flood stage, have uncertainties that affect the delineation of the flooding hazard.

Ultimately, the purpose of applying one or more of the following methods is to estimate the magnitude of an ice jam flood (i.e., flood stage) under the ice jam related flooding hazard. For rivers and streams where ice jam flooding has been identified as a risk, results from these analyses will support a comparison to flood magnitudes for the other flooding hazard standards applicable to the area of the province, to support delineating the flooding hazard.

6.4.6 Empirical Ice Jam Models for Determining Potential Ice Jam Stage

One of the simplest ways to gain an appreciation of ice affected flood stage is through the application of established empirical formulas to support calculation of the upper envelope of peak breakup stages under a 'wide'-type equilibrium jam scenario. This can support

quantification of maximum possible stage under an equilibrium ice jam, which can be improved by consideration of additional physical constraints (Calkins, 1983; Gerard and Calkins, 1984; Tuthill et al., 1996). Methodical inspection of the site of interest may reveal extensive flood plains where water and ice can escape, limiting the maximum stage to a constant rather than a flow dependent value (Beltaos, 2011) (see [Section iii Equilibrium Ice Jam Analysis with Floodplain Flow \(Calkins, 1983\)](#)).

The same is true where an ice clearing discharge can be defined, such that no ice jam can remain in place beyond that discharge value (see Tuthill et al., 1996) ([Figure 6-21](#)). Under such conditions any temporal changes in ice jam parameters would yield little increase in ice-jam thickness or water depth at a given discharge. Hence, such conditions may be used to represent the upper-envelope, or maximum ice-affected stage hydrograph. This upper threshold has been described as most critical, as it limits the maximum ice-affected stage that can occur at a given location (FEMA, 2023). Ultimately, higher discharges will lead to ice-jam release and prohibit additional downstream ice-jam formation, driven by local flow hydraulics, channel morphology, or a combination of the two (FEMA, 2023; USACE, 2011).

These equilibrium jam stage values should be quantified, compared and cross referenced to flooding hazard elevations derived using the methods presented in [Section 5 Water Level Computations – Open Water Hydraulic Modelling](#) for the relevant flooding hazard criteria zone in which you are working. This should include queries such as:

- i) Does the flood stage produced under an equilibrium break-up jam exceed that of the flood elevation produced by the Hurricane Hazel storm?
- ii) If so, additional reconnaissance and data collection is warranted; however, if not, there is no need to investigate further.

Where initial site reconnaissance suggests that ice affected flood stage may be impacted by other types of jams, such as hanging dams, an alternative process may be warranted, where information suggests that stages exceeding those potentially observed under an equilibrium ice jam may be observed at the site of interest. Floods caused by hanging dams are described as relatively rare but have been reported on small rivers (Turcotte et al., 2017; Beltaos, 1997).

Furthermore, Tuthill et al (1996) also evaluated the minimum flow that can mobilize and break up a sheet ice cover [$P(J) = 0$ below this threshold] and the maximum flow above which a sheet-ice cover cannot remain in place (i.e., H_m reverts to the open water stage) ([Figure 6-22](#)). The possibility of these occurrences and associated refinements at a local scale should be kept in mind when conducting in situ reconnaissance and resident interviews in relation to ice affected flooding (e.g. Beltaos, 2011).

Several methods have been advanced to calculate the ice jam stage within the equilibrium reach of these “wide” channel jams. With these empirical formulas, in general, the following assumptions are made:

- 1) The break-up ice jam is a floating, granular mass and is formed with no cohesion between the ice pieces.
- 2) Flow conditions are reasonably steady in the reach of interest
- 3) The channel geometry can be adequately described by its average geometry (mean width, depth, and roughness), and that flow depth and jam thickness do not vary across the equilibrium accumulation.
- 4) By-pass channels or low banks with wide flood plains are not present to interrupt stage increases by providing a form of relief through spillage.

The analytical models are widely applied to derive an upper envelope to potential ice jam related flooding for wide jams, under equilibrium conditions, at a particular location for any given discharge. Strictly speaking, as outlined by Beltaos (2013b) the equilibrium concept applies only to prismatic channels but may be extrapolated to natural streams by replacing the bottom slope of the prismatic channel with the open water surface slope, provided that:

- a) the reach under consideration is long enough to permit meaningful averaging of channel characteristics; and
- b) the flow is free to assume a relatively uniform condition, i.e. there are no significant flow control influences. It is convenient to refer to this type of river flow as quasi-uniform.

As noted by Gerard (1989), while assumptions applied in the development of the empirical relationship (i.e., Beltaos, 1983a) appear rather restrictive, a comparison of the empirical curve ([Figure 6-13](#)) with the actual field measurements ([Figure 6-14](#)) indicates that it fits rather well in a wide range of field study circumstances.

The results of the theoretical calculations expressed below are considered to produce conservative equilibrium water depths because many jams do not attain the equilibrium condition, therefore can and should be used as a means of estimating or approximating historical data of lower reliability; and assessing the relative magnitude of those data to the theoretical upper envelope equilibrium curve; where applying an equilibrium curve is appropriate. This includes historical data gathered through archives, newspaper reports, photographs, environmental evidence as well as the recollection of residents. Where

historical data are completely absent, theoretical data should be supplemented by a minimum of one- or two-years field observations during breakup to corroborate theoretical calculations (see Beltaos, 1980). Where these observations do not confirm the occurrence of ice jams, but it is anticipated that ice affected flooding may play a role in flooding additional efforts to install hydrometric monitoring equipment and surveillance should be implemented.

The following analytical approaches are all based on the “wide” channel equilibrium jam assumption and governed equations and are applicable to Large Rivers including those with a width-depth ratio greater than 10. However, their use for the purposes of this guidance, and as a coarse level screening approach, can be applied to smaller streams, regardless of whether they experience equilibrium jams or not.

6.4.6.1 Basic Upper Envelope Equilibrium Ice Jam Curve Estimation

Furthermore, from a dimensional analysis perspective and in a more detailed manner, dimensionless discharge (ξ) and dimensionless ice jam depth (η) can be further determined as follows (after Beltaos (1983; 2008b) and further to Lindenschmidt et al. (2024, 2025)):

$$\xi = \frac{\left(\frac{q^2}{gS}\right)^{1/3}}{WS} \quad (6.2)$$

$$\eta = \frac{H}{WS} \quad (6.3)$$

where g is the gravitational acceleration (9.81m/s^2), H is the backwater staging depth due to an ice jam (in metres), q is the unit discharge = Q/W (m^2/s), S is the slope (-) and W is the river width (in metres) as taken under the ice jam – the approximate distance between resultant shear walls upon ice-jam release (see [Figure 6-15](#)).

Rearranging the equation 6.3 such that H is a function of η , W and S , yields the following:

$$H = \eta \cdot W \cdot S \quad (6.4)$$

The relationship between the parameters ξ (dimensionless discharge) and η (dimensionless water depth or ice jam stage) as well as the component variables that are incorporated by these parameters, are defined and shown graphically in [Figure 6-13](#), including a +/- confidence band. These functions are based on data from ice observations at several sites including the Thames and Grand Rivers in Ontario, whereby [Figure 6-14](#) plots the data, and [Figure 6-13](#) plots the monotonic function derived from the data.

Ranges of equilibrium ice jam values for dimensionless discharge (ξ) and dimensionless water depth or ice jam stage (η) can be estimated for ungauged rivers flowing adjacent to communities, from which flood depths (H) can be back calculated.

The simplified procedure for applying this analytical method is as follows:

- a) Gather input data including discharge (Q), river width (W) and channel slope (S)
- b) Calculate specific discharge $q = \frac{Q}{W}$
- c) Using equation 6.2, calculate dimensionless discharge, ξ
- d) Use dimensionless discharge value, ξ calculated in step c) to derive dimensionless stage η using [Figure 6-13](#)
- e) Using dimensionless stage, η derived in step d), use equation 6.4 to solve for the backwater depth (H)

Steps a, b and c are universally applicable to the methods outlined in [Section 6.4.1.2 Upper Envelope Equilibrium Ice Jam Curve Estimation](#) and [Section 6.4.1.3 Extended Equation for Upper Envelope Ice Jam Curve Estimation](#).

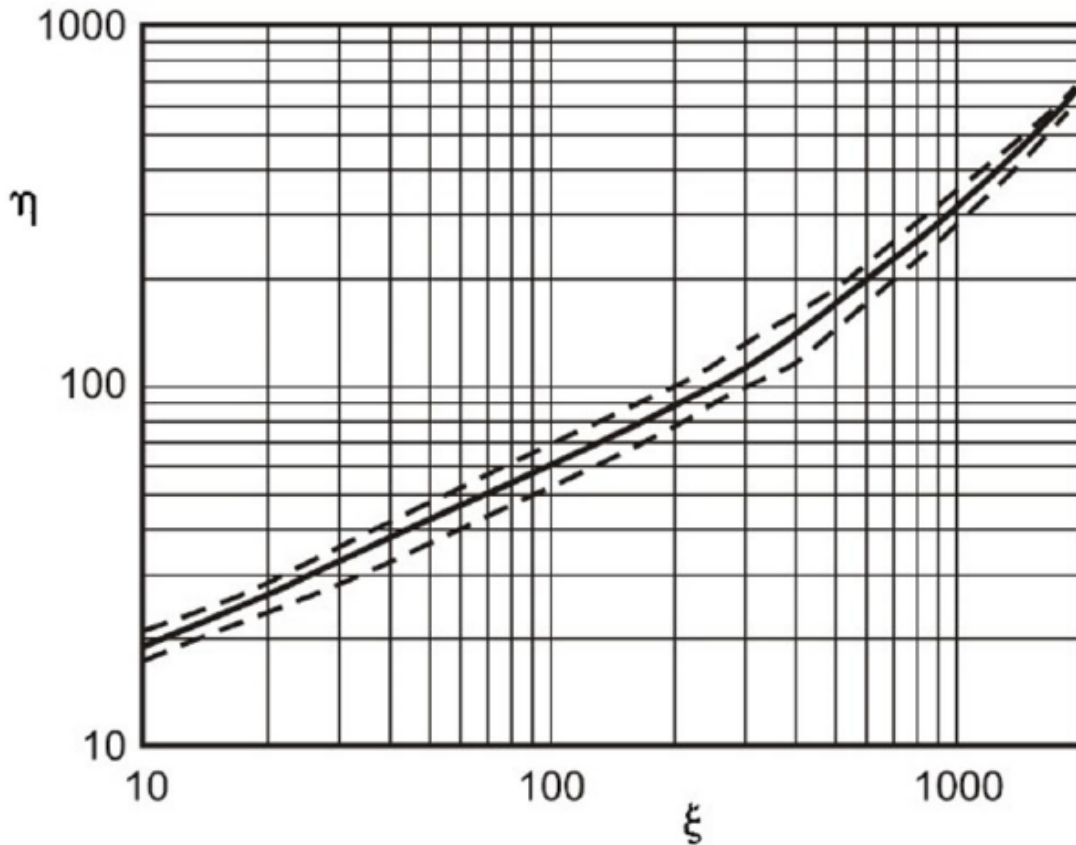


Figure 6-12. Summary of non-dimensional discharge (ξ) versus non-dimensional ice-jam stage (water depth) (η) established from field data from breakup case study relationships for wide-channel equilibrium jams, including confidence bands. Source (Beltaos, 2013; after Beltaos, 1995b with edits)

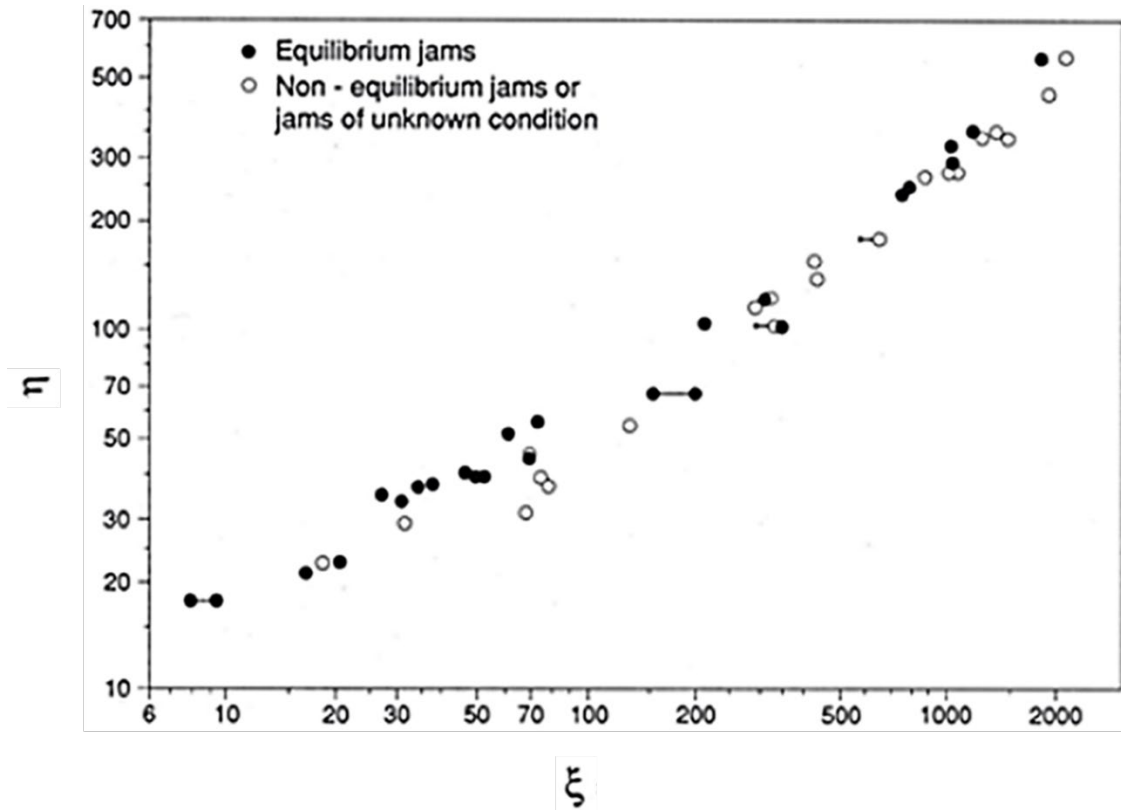


Figure 6-13 Dimensionless discharge (ξ) and dimensionless ice jam stage (η) (water depth) relationship for breakup ice jams (Source: Burrell et al., 2015, after Beltaos (1983) with changes)

6.4.6.2 Intermediate Upper Envelope Equilibrium Ice Jam Curve Estimation

Equilibrium ice jam depths are primarily dependent on channel width, slope and discharge, as defined by the following empirical relationship, based on a semi-empirical fit to a large data set derived from case studies of breakup jams ([Figure 6-13](#)) in Canadian Rivers (Beltaos, 2008b, 2010), that can be used to quickly estimate the water level of equilibrium jams for a given discharge:

$$\eta \approx 0.96\xi^{0.82} + 4.79 \left[1 + \sqrt{1 + 0.22\xi^{0.82}} \right] \quad (6.5)$$

Using equation 6.5 it is therefore possible to evaluate empirically (for a reach of relatively uniform gradient and geometry) the potential breakup water level or ice induced flood level using the water depth (H) of an equilibrium ice jam, without requiring the use of the graphic relationship between ξ and η , expressed in [Figure 6-12](#).

In this way, applying equation 6.5 supplies an upper envelope curve to potential flooding at a particular location for any given discharge. This supports fitting a regression line through the historical events plotted in [Figure 6-12](#) and using this method to derive an empirical curve that can be utilized to estimate ice jam affected water level of equilibrium jams for a given discharge.

After calculating specific discharge, solve for dimensionless discharge ξ using equation 6.2. The result of η , obtained from equation 6.5 is subsequently used in equation 6.4 to solve for H , to derive the ice jam backwater depth (m).

6.4.6.3 Extended Equation for Upper Envelope Ice Jam Curve Estimation

Empirical values can be further refined by using an extended equation for η as follows [after Beltaos (1993) and extended by Lindenschmidt (2020)]:

$$\eta = \frac{H}{WS} = 0.63 f_o^{1/3} \xi + \frac{5.75}{\mu} \left(1 + \sqrt{1 + 0.11 \mu f_o^{1/3} \left(\frac{f_i}{f_o} \right) \xi} \right) \quad (6.6)$$

Where f_o is a composite friction factor calculated from the average of ice friction factor, f_i and the riverbed friction factor f_b , and μ is a friction coefficient. More specifically the parameter μ is the friction coefficient that represents the internal strength of the ice cover, and is a lumped variable representing the effect of several difficult to determine or unknown factors, including ice on bank friction, lateral stress within the ice cover, porosity and possibly cohesion, and the angle of internal resistance. These values are substituted using the typical range of parameters for various riverbed slopes based on the work of Beltaos (1993, 1995) and Lindenschmidt et al. (2024; 2025) and shown in ([Table 6-2](#)). In this way, application of equation 6.6 includes additional parameter components, and inherently more details than the simplified equation 6.2 and to an extent, equation 6.5.

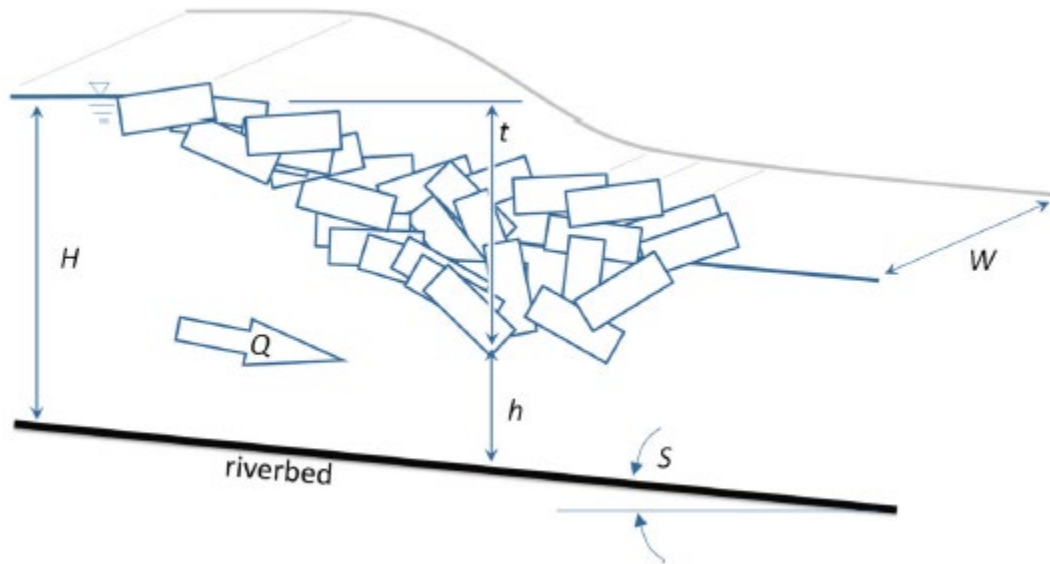


Figure 6-14 Equilibrium ice jam of thickness t along a river reach of slope S and width W for a discharge Q with an overall backwater staging depth due to an ice jam H and depth of flow (h) under the ice jam (Source Lindenschmidt et al. (2025).

Table 6-2 Typical ranges of f_o , f_i/f_o and μ for different riverbed slopes, compiled by Beltaos (1993a) and Lindenschmidt (2024) as reported in Lindenschmidt et al. (2024; 2025).

Slope	f_o	f_i/f_o	μ	Example Sites from Field Data
0.0001–0.0003	<0.1	1.4–1.5	1.2–1.3	Thames River; Churchill River (Labrador)
0.0003–0.0004	0.3–0.4	1.3–1.7	0.8–1.6	Athabasca River near Fort McMurray; upper Dauphin River; Red River
0.0007–0.0010	0.1–0.7	0.6–1.5	0.6–1.2	Smoky River; Athabasca River upstream of Fort McMurray; lower Dauphin River

The basic procedure assumes that the width of the accumulation W does not change with stage, but an iterative approach may be used to account for limited variations of W . More explicitly the basic procedures involve,

- determining dimensionless discharge ξ using equation 6.2
- solving for dimensionless stage (η) using equation 6.6

- c) using the resulting dimensionless stage value (η) to solve for the ice jam backwater depth (H) employing equation 6.4.

The calculation of break-up ice jam stage may be conducted quickly by using this approach. In contrast to the Beltaos' (1983a) simplified method, it is further assumed that the composite friction factor, f_o calculated from the average of the ice friction factor f_i , and the riverbed friction factor f_b , do indeed vary from site to site, but their range can be defined based on those values observed in various field studies, and defined by river slope ([Table 6-2](#)). Furthermore, application of the extended equation for η does not require the use or inspection of the graph in [Figure 6-12](#).

6.4.6.4 Equilibrium Ice Jam Analysis with Floodplain Flow (Calkins, 1983)

The ice jam equilibrium thickness equation has also been used by Calkins (1983) in an approach which is generally similar to Beltaos (1983a) and those empirical relationships outlined above, but is focused on smaller streams and rivers. It includes a limiting discharge concept for the main channel of rivers having significant overbank spillage into the flood plain, or floodplain flow. In this way the floodplain essentially serves as a floodway when the water level rises above bankfull stage.

For ice floes to travel onto the floodplain, the total river stage must rise to a value greater than

$$H_o + S_i t_i \quad (6.7)$$

where t_i is the initial ice cover thickness and H_o is the bankfull depth and $S_i = \rho_i/\rho =$ specific gravity of ice = 0.92.

It is assumed in this derivation that most of the broken ice floes moving into the floodplain have a minimum thickness dimension of t_i . The assumption which has been confirmed reasonable in using field data, asserts that when the stage reaches $H_o + S_i t_i$ the ice jam will not shove to thicken the cover.

The total stage for a combined floodplain and channel flow, with the thickness of ice in the floodplain equaling the channel ice cover thickness just prior to breakup, which will reach the minimum height of

$$2Rh + S_i t = H_o + S_i t_i \quad (6.8)$$

after some substitutions to eliminate Rh (hydraulic radius beneath the ice cover) and assuming c , the cohesion of ice cover is equal to 0, gives the equilibrium ice jam thickness in channels with floodplain flow

$$\mu S_i(1 - S_i)t^2 = BS_w \left[\frac{H_o + S_i t + S_i t_i}{2} \right] \quad (6.9)$$

where μ is an internal ice frictional resistance coefficient, assumed to be 1.2, t is the ice jam thickness, B is the width of the river surface, and S_w is the slope of the water surface gradient. Physically, this assumption means that the depth of the channel at bankfull stage is increased by the quantity $S_i t_i$, for computation of the ice thickness.

The maximum flow in the channel in these situations (and the maximum ice jam thickness) is defined as that which will raise the stage sufficiently to carry oncoming ice pieces into the floodway. As highlighted above, at that point the channel ice jam will no longer thicken by shoves.

Not all flood plains provide complete flow relief of this sort, and oncoming ice may also be confined to the main channel without entering the flood plain. In addition, flow partitioning for the flood plain and channel may be necessary outside of the model to compute the final stage once overbank flooding occurs. Hence, it is a useful concept if applied with caution and supported by evidence from field reconnaissance.

6.4.7 Direct Methods Including Data Sources

Direct methods are those which employ data derived from hydrometric stations or other historical sources, as opposed to synthetic approaches

6.4.7.1 Direct Methods for Establishing Ice Affected Stage-Frequency

The presence of hydrometric gauging stations in the vicinity of the area historically affected by ice dams may or may not be helpful in defining ice affected flooding. For instance, gaps in hydrometric records are common for ice-related events as sensors and orifices at gauging stations are often damaged or fouled by ice, and/or established stage-discharge relationships are impacted and estimates of stage or discharge may be unavailable. These facts often make ice affected stage data shorter or sparser than open-water data for the same site but also result in a significant amount of missing data under ice conditions; including those data that would otherwise have been incorporated into frequency analyses of ice affected extreme events (NHC, 2023). In the event that ice

influenced data are available for your site of interest, and these data are reliable, as confirmed by an analysis of the data (see Beltaos, 1990; Turcotte and Nafziger, 2021), and confirmation and corroboration from local residents and photographs (for instance), the most simple direction is, to use them to derive maximum ice affected stage information, through establishing an ice affected stage frequency analysis (USACE, 2016; FEMA, 2023).

6.4.7.1.1 Ice Affected Stage Frequency Analysis

Under these ideal circumstances an ice affected stage frequency analysis will be based on a large set of observed annual peak ice influenced water levels measured by a WSC station at a particular site over an adequate period of record (Beltaos, 2010; USACE, 2011; White and Beltaos, 2008). Where such data exists, a stage frequency relationship can be developed using a graphical procedure. When the distribution of the population is unknown and there is no basis for assuming a particular distribution, as is often the case with ice affected stream and river stages, a graphical (or nonparametric) statistical method can be employed. This is achieved by ordering the observations in descending order and ranking data from largest (1) to smallest (N), where N is the total number of robust observations available (White and Beltaos, 2008). An exceedance probability p_m is computed for each value using a plotting position formula:

$$p_m = \frac{m - a}{N + b} \quad (6.10)$$

where m is the rank of the particular observation. The constants a and b range from 0 to 1 and are dependent on the probability distribution that is assumed to best apply to the data set being summarized. In practice, however, standard probability distribution functions rarely fit ice-related stages (Gerard, 1989) and it may be preferable to use a distribution-free pair ($a = 0.25$, $b = 0.5$) (Adamowski, 1981) (White and Beltaos, 2008).

Once the data are plotted, a curve is visually fit to the data, and from this, the probability of occurrence can be determined. Plotting practices include showing ice affected peak stage versus exceedance probability, graphically either on linear axes or using standard probability distributions (i.e., normal or log-normal), depending on personal preference and final plotting appearance. Perhaps even more so than in open water flood frequency analysis, extrapolation below the lowest, but even more so, above the highest value is discouraged. Inaccuracies from extrapolating from these rare events arise from a variety of factors such as ice jam release, ice jam release flows, and out of bank or riparian flows which can act to limit the maximum ice jam stages (USACE, 2011).

The culminating graph showing open-water versus ice-jam breakup data plotted on a probability scale is shown in [Figure 6-15](#).

An appreciation of the overall annual stage probability distribution can be important for delineating the extent of the flooding hazard limit in the province, or for determining the location/elevation of development to be located outside of effects of high-water levels. An appreciation of the frequency distribution will provide valuable information on relative return periods of ice affected flows and maximum stage

6.4.7.1.2 Combined Distributions

To evaluate ice-affected water surface elevations for a particular river reach, the open-water and ice-related stage-frequency distributions can be directly combined as with other mixed populations, as the distributions are reasonably independent. Following obtaining a satisfactory series of open water and ice related annual peak water levels, the combined probability of a given stage being exceeded in any given year can be calculated as:

$$P(H) = P_i + P_o - P_i P_o \quad (6.11)$$

where

- $P(H)$ is the probability that the stage H will be equaled or exceeded in any one year, either by an ice influenced peak or by open-water flooding, or both, and
- P_i and P_o are exceedance probabilities of ice influenced peaks and open water peaks, respectively.

The resulting plot can look like [Figure 6-16](#), showing that ice affected stages can cause substantially higher peak levels than open-water floods at return periods of approximately 10 years or more, at a river site in western Canada that experiences occasional ice jam flooding.

6.4.7.1.3 Separate Distributions

An alternative to combining the separate distributions is to derive an annual curve by choosing the peak stage (ice-related or open-water) for each year for which information is available for both. The annual distribution can then be plotted directly. However, for years without both open water and ice-related data, useful information from a limited database is wasted. Where time series of open-water and ice-influenced annual peaks are available, they can be ranked, and respective frequency curves developed.

However, standard probability distribution functions seldom fit ice-related stages (Gerard, 1989) and it may be preferable to use a distribution-free pair ($a = 0.25$, $b = 0.5$; Adamowski, 1981). The largest observed stage associated with ice-related conditions is then identified and flagged for use in comparison to relevant flooding hazard standards for applicable to the zone the area of interest is located.

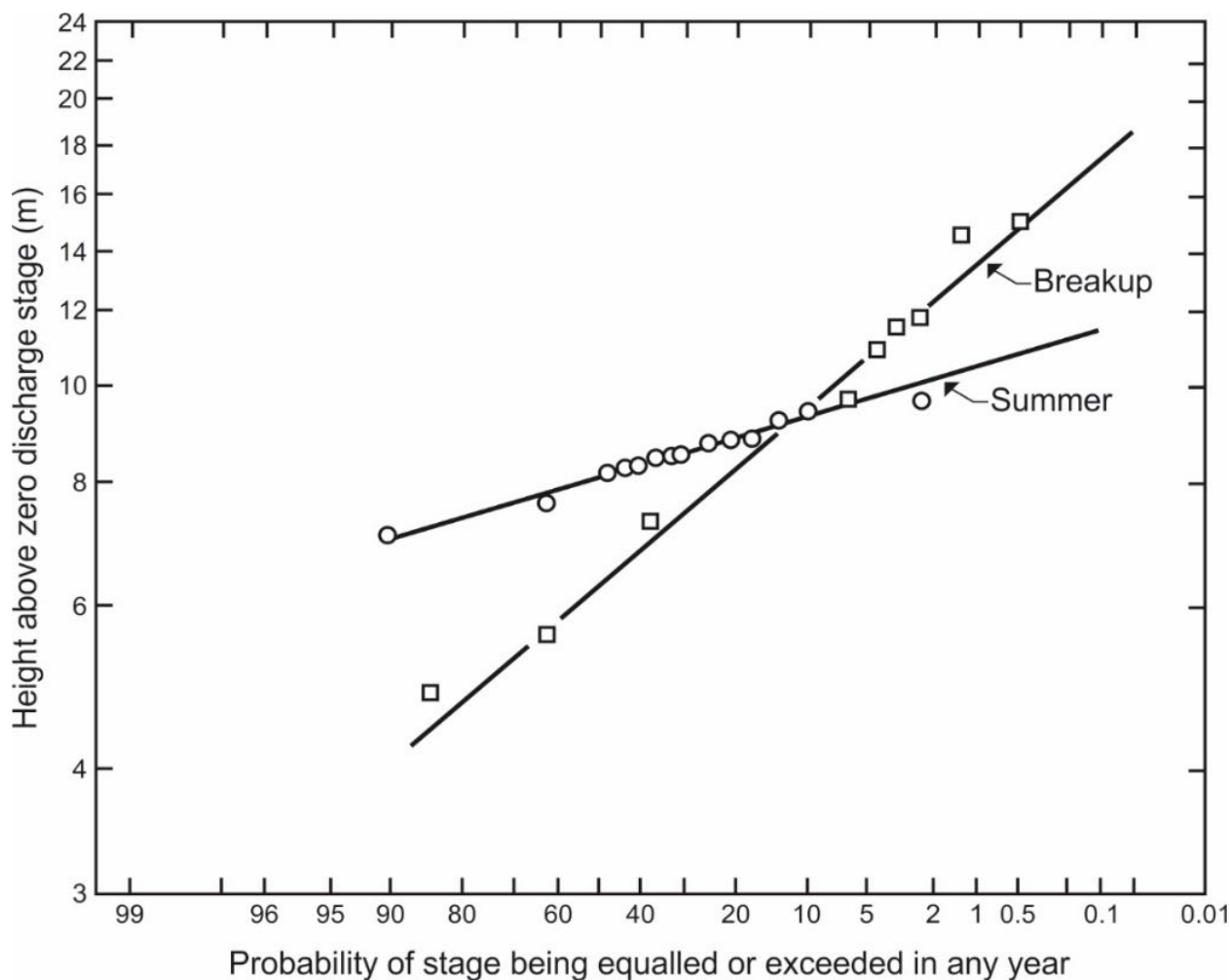


Figure 6-15 Example cumulative probability distribution for ice-influence (open square) and open-water (open circle) annual peak stages, where stages above the intersection of the two lines are due to ice jams (Source: Beltaos, 2021, adapted from Gerard and Karpuk, 1979).

6.4.7.2 Historically Based Direct Flood Stage Estimates

6.4.7.2.1 Integrated Direct Historically Based Approach

This method recognizes that different sources of historical flood levels may be associated with different stage thresholds above which an event would be noticed and provides guidelines on how to estimate the frequencies of water levels to which different perception stages apply. (Burrell et al., 2024).

Several direct historical flood level observations will usually be found in the review of historical data at sites which have experienced ice jam flooding. The most recent flood will likely be reasonably well documented, as will those floods which occurred in the last 5 to 20 years. There will be substantially less information on lesser floods or those occurring in the more distant past, but in view of the local nature of ice jam floods and the general paucity of data, it is important to include these events as well.

An approach which makes full use of all the historical observations in a probability analysis for ice affected jam flooding was originally documented by Gerard and Karpuk (1979). The methodology is as follows and has been further highlighted by Gerard (1989) and Beltaos (2021) and further modified by Kriwoken and Brown (1988).

- 1) Collect and combine a complete history of ice jam flood stages from all possible sources (i.e., surveys, environmental evidence, interviews, among others) and relate them to a reference stage, such as the zero-discharge stage.
- 2) Examine topographic mapping, field surveys, air photography, satellite imagery, Google Earth, Forest Resource Inventory (FRI) and OWIT Data, among others, to identify the exact site to which the data applies and features of the area which could affect ice jam flood levels (i.e., relief channels).
- 3) Establish a perception stage for each data source (see Gerard and Karpuk, 1979). This is the stage below which each source of data would not likely have provided information on the maximum ice related stage, (for example, this could be close to bank-full stage for local residents or be the minimum gauge reading which could be recorded by hydrometric instrumentation, where one exists).
- 4) Organize the information to display the stages identified from each source and their perception stages (see [Figure 6-16](#)).

- 5) Establish a record length, rank and confidence limit associated with each flood stage. A summary diagram which gives the lowest perception stage for each year of record to be used in the analysis can assist in this evaluation (for example diagrams see [Figure 6-16](#) and [Figure 6-17](#)). The record length is the sum of all years having perception levels which do not exceed the given flood level. The rank of each flood level is determined by comparing that level to others in the group with a perception stage equal to or lower than the peak itself. This process may give the same rank for several peaks and different record lengths for different peaks.

Optional Component(s)

- 6) Determine the cumulative probability for the maximum break-up stages, employing a plotting position formula and fitting a cumulative distribution curve to the data. This fitted curve may follow one of the standard statistical distributions, but this is unlikely. This is because the ice-related stages are likely to be near the upper bound of possible flood levels from ice jamming and are quite likely limited to a maximum stage by physical constraints at the site (i.e., flood plain spill, diversion). For details on frequency distributions and testing, refer to [Section 3.2 Flood Frequency Analysis](#).
- 7) Use a hydrodynamic model to corroborate, field validate and reconstruct historical events (refer to [Section 6.4.3.5 Hydrodynamic Modelling of Ice Affected Stage](#)).

An example of the plots produced using this method is shown on [Figure 6-18](#), which highlights that this approach leads directly to a probability distribution of ice jam flood stages without a requirement to have direct hydrometric gauging station data or to simulate ice jam configurations, flood levels or winter flows. This would be an advantage at sites where severe grounding or aufeis accumulations have been identified as the cause of flooding problems, and for other forms of ice affecting flooding where no recorded hydrometric data are available. It requires that a reasonable number of years of observed levels be available at a flood-prone site and that the ice regime has been reasonably stationary in the past, as the ice regime will change if there has been a significant change in the flow regime. The availability of observed levels, however, does not mean that hydrometric gauging station data are required. Given that these constraints are met, it provides a probability distribution which is consistent with other approaches for estimating ice stage frequency.

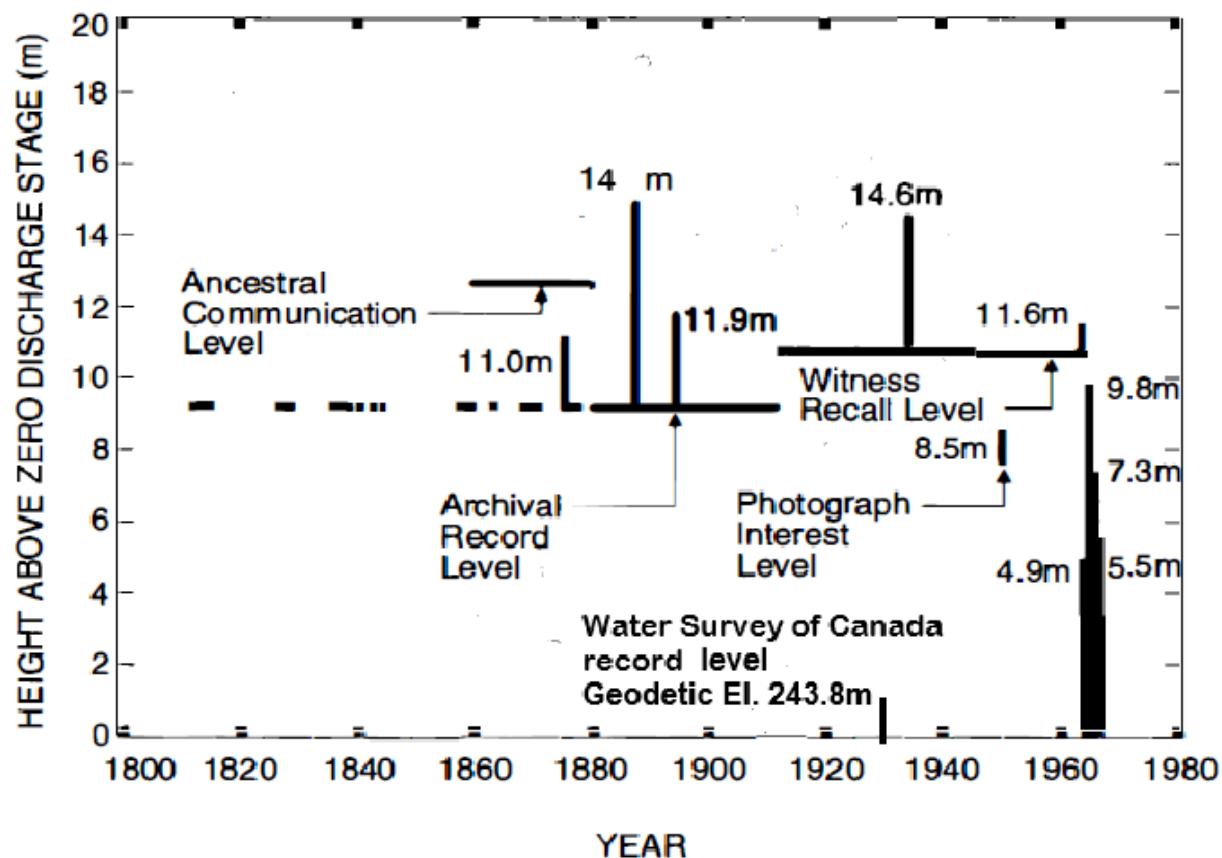


Figure 6-16 Illustration of various perception stages and their utility in assigning suitable frequencies to stages reported by different data sources (adapted from Gerard and Karpuk, after White and Beltaos, 2008).

This approach requires the practitioner to conduct a very careful examination of ice jam flood stages and the flood prone area. It cannot be completed without on-site investigation and interviews, both of which provide necessary insight and valuable supporting details which might otherwise be missed in a desktop study.

The same methodology applies to sites which have a long record of gauged water levels through the break-up period. At such sites, it may be possible to replace significant portions of the more qualitative information derived from other sources with the gauged data or employ the monitored information to validate the perception stage information. Overall, such records may reduce the data acquisition phase, but as mentioned above, should not replace the essential requirement for the practitioner/modeller to evaluate the physical characteristics of the site and their implications on ice-effect stages.

With regards to the application of the perception stage and integration of quantitative and qualitative historical data from the site versus local accounts which may be varied in detail

and relative to the amount of pertinent information, which is often fragmented. It is imperative to appreciate the relative quality of these data. Some studies (Kriwoken and Brown, 1988) have elected to rank these sources of information according to data reliability, in the following order from most reliable to least:

- 1) physical evidence
- 2) photographs
- 3) historical documents (i.e. newspapers)
- 4) personal recollection.

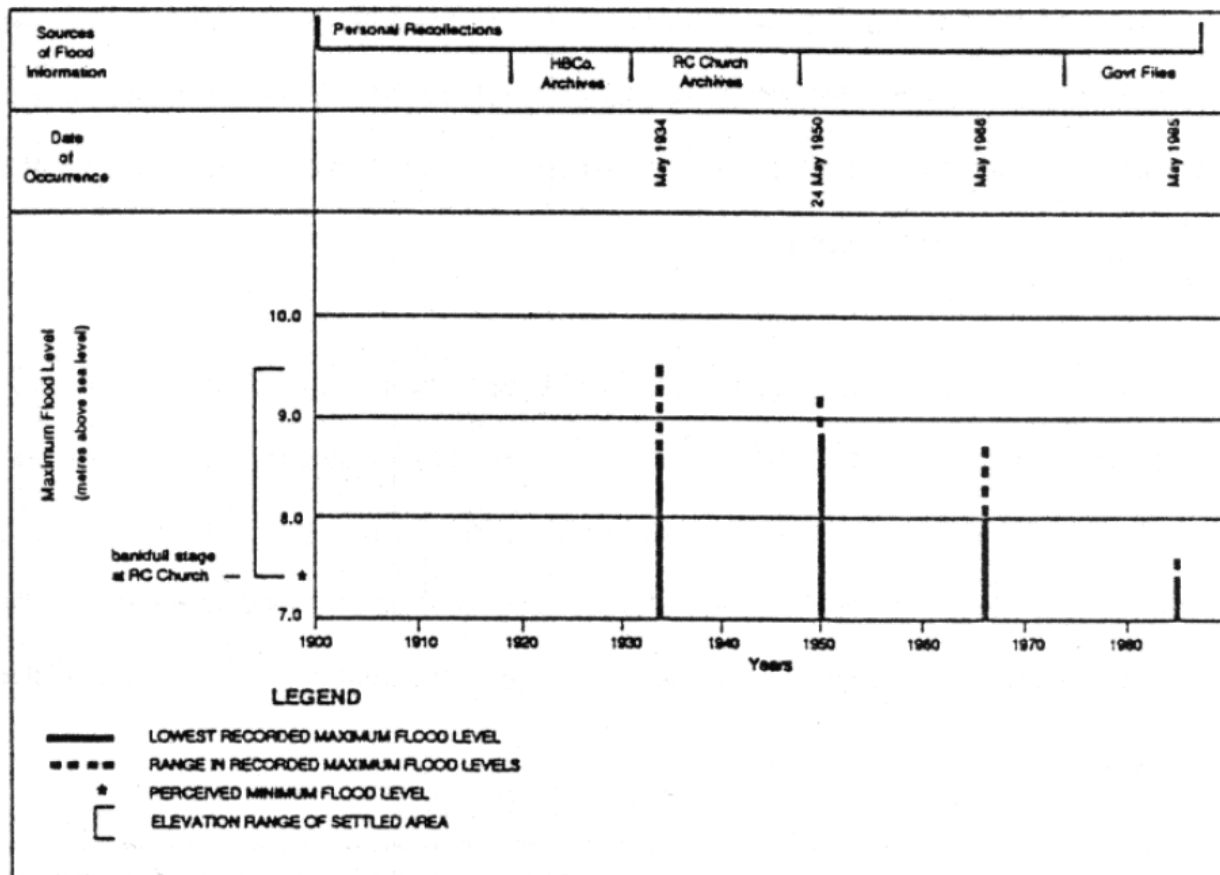


Figure 6-17 Chronology of historical ice related flood levels summarised on a chronological chart producing a flood summary for Attawapiskat, Ontario (Source: Kriwoken and Brown, 1988)

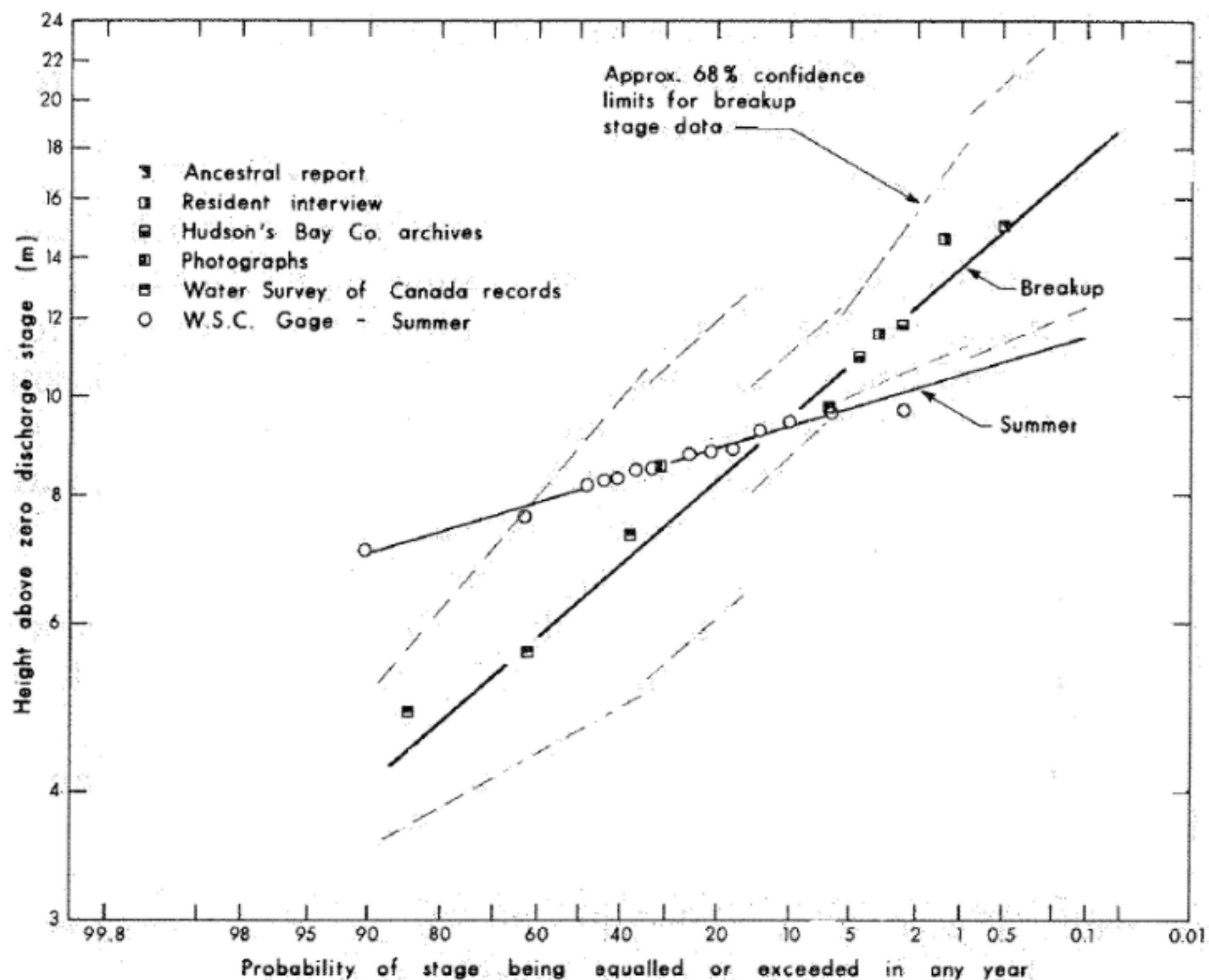


Figure 6-18 Stage-frequency curves for open-water and ice-covered peaks showing range of sources of data for various data points (Source: Gerard and Karpuk, 1979). A cleaner, revised version of [Figure 6-18](#) without multiple lines or the depiction of various separate data sources as points, is shown in [Figure 6-16](#).

6.4.7.2.2 Maximum Expected Ice Affected Stage Approach

Indirect methods, including the derivation of reliable synthetic ice affected stage frequency distributions that will be outlined later in this document (see [Section 6.4.4.2 Synthetic Frequency Analysis - Discrete Outcomes Method](#) and [Section 6.4.4.3 Synthetic Frequency Analysis - Distributed Function Method](#)) can be time consuming, involve a high level of judgment and expertise and results may be inherently uncertain. As a counter

approach, in a review of ice jam floods, Gerard (1989) outlined a sequence of methods for determining the maximum expected stage(s) that may result from ice-affected flooding including the utilization of historic data.

In circumstances where only a reasonable estimate of the maximum expected stage is required, such as in an initial assessment of the role of ice affected flooding relative to Ontario's established flooding hazard standard(s), a suitable procedure that may include the collection of additional field data, employing the methods outlined above, (see [Section 6.4.3.1 Direct Historically Based Approach](#)) is further described below:

- 1) In the field, based on site reconnaissance, determine the maximum elevation of evidence of ice action, such as vegetation damage, tree scars and bank scour;
- 2) Collect direct historically based information on ice-related stages, ice jams, and other unusual ice phenomena from various sources (for instance, local newspapers, long term residents, government files);
- 3) Determine the elevations of flood plains and any relief channels
- 4) Define the mean channel geometry in the reach or reaches of interest;
- 5) Locate any available hydrometric data for the reach of interest and from spatially relevant and hydrologically similar nearby river and stream systems (see [Section 3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins](#));
- 6) Ascertain from topographic maps, aerial photographs, FRI imagery (GeoHub), OWIT (MNR, 2024) Google Earth and DEMs (e.g., MNRs GeoHub) the geomorphic features of the study area/reach, including evidence of ice action and locations likely susceptible to ice jamming.
- 7) Using the data gathered from steps 1 to 6, estimate the maximum historical ice-related stage, and if supported by available data, the lower and upper bounds of physically possible ice-related stages.
- 8) Check estimations against calculated stages using the method outlined in [Section 6.4.1.1 Basic Upper Envelope Equilibrium Ice Jam Curve Estimation](#), and/or equations 6.5, and 6.6, as well as [Figure 6-13](#).

With the collection of the above data and based on the results of the above investigations, it should be possible to reasonably estimate the maximum historical ice-related flood stage, as well as the lower and upper bounds of physically possible ice-related flood

stages. Estimations can additionally be checked against calculated stages, anticipated ice jam break up discharges and channel geometry. Additional corroboration can be done using hydrodynamic models.

6.4.8 Indirect Methods and Approaches to Ice Affected Stage Derivation and Computation

Often, flood risk and hazard assessments are required at sites where there are few or no historical data on past ice-influenced flood peaks. Under these circumstances, the only alternative is to develop synthetic flood-frequency relationships (Gerard, 1989; White and Beltaos, 2008; FEMA, 2023; USACE, 2011). The primary issue for an indirect method is how to come up with the correct stage-discharge relationship under ice conditions (White and Beltaos, 2008) as well as deriving an appropriate and representative source of discharge data.

Contributing to the uncertainty and complexity is the fact that unlike under open-water flows, the stage-flow rating function under ice conditions is no longer unique. The water level is dependent not only on the flow, but also on the site-specific channel configuration impacting the ice resistance to flow. The cold season weather during the year affects the ice formation and breakup inconsistently. In addition, probabilistic components, such as the ice supply, type of ice cover, hydraulic characteristic of the ice cover, and thickness of the ice, influence the water level.

Open water and certain ice-influence historical flow data are far more readily available than peak ice-influenced stages because the spatial variability of ice-jam stages is much greater than that of flows. As a result, the flow at an ungauged site may often be deduced from records of upstream and downstream hydrometric gauges, regional estimates, proration or modeled estimates (see [Section 3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins](#)); whereas ice-jam stage data cannot be transposed. The frequency of peak ice-influenced flows is therefore much easier to establish and the frequency of corresponding stages can be established by other synthetic means.

Synthetic frequency curves can be used to generate estimates of ice-jam flood stages under circumstances where ice jams are known to occur and are an anticipated flooding hazard, but where existing recorded ice-jam data is not of appropriate length or quality to extrapolate to high return period/low probability events. These synthetic approaches combine flow frequency estimates with a synthetic stage-discharge relationship for ice affected flow conditions (White and Beltaos, 2008). It is important to acknowledge that synthetic frequency curves involve a high level of judgment and expertise and can be an

inherently uncertain process. As such, in their use and application the practitioner must be cognizant of these limitations and consider the results accordingly.

6.4.8.1 Synthetic Generalized Estimation of Annual Maximum Stages

This approach is suitable for river and stream reaches at a site of interest where no hydrometric data exist, and where there are no observations of any ice-related processes available, but historically based observational data are needed if a cumulative probability distribution is desired.

The generalized procedure for estimating annual maximum stages due to ice including (as described by Gerard (1989))

- 1) Compute stage discharge curves for
 - a. open water
 - b. a solid ice cover
 - c. a fragmented ice cover or jam in equilibrium
- 2) Estimate the upper limit of stage that could be caused by a jam, for example as imposed by overflow onto the floodplain or by maximum equilibrium jam (See methods and equations included in [Section 6.4.1 Empirical Ice Jam Models for Determining Potential Ice Jam Stage](#))
- 3) Estimate the maximum ice-related stage for each year for which sufficient regional data are available or other simulated streamflow sources:
 - a. Estimate the winter period when the solid ice sheet and fragmented ice sheet conditions apply (on the basis of regional break-up information and associated relevant meteorological information).
 - b. Using synthesized streamflow data, estimate the maximum discharge for the 'solid ice sheet' period and the break-up or 'fragmented ice sheet' period.
 - c. Using 3a) and 3b) estimate the maximum ice related stage for each winter using the developed rating curves in Step 1 above, and the estimated upper limits (Step 2 above).
- 4) Once the annual ice-related peak stages are established, a cumulative probability distribution can be constructed (but is not necessary) following the Integrated

Direct Historically Based Approach ([6.4.6.2.1 Integrated Direct Historically Based Approach](#)) characterised by Gerard and Karpuk (1979) and employed by Kriwoken and Brown (1988), including the collection of observational information from local residents, as outlined in [Section 6.4.3.1 Direct Historically Based Approach](#). As highlighted by Beltaos (2008a; 2012) while it is unlikely that value of Q_{ic} and other threshold flows dependent on stream bathymetry and morphology as well as antecedent conditions such as freeze-up stage and sheet ice thickness will be available at sites where there are no historical stage data, it is emphasized that an experienced river ice specialist may be able to estimate threshold clearing and breakup flows in circumstances where there is comprehensive observational data provided by local residents. When undertaking site reconnaissance and resident interviews, practitioners undertaking such work should keep the possibilities of such refinements top of mind.

The resulting frequency distribution or probability of ice jam stages must be combined with the open water stage distribution to arrive at the combined distribution (Gerard, 1989), which is of assistance for understanding the magnitude of ice jams versus open water flooding, required for flooding hazard limit delineation.

6.4.8.2 Synthetic Frequency Analysis - Discrete Outcomes Method

Once the frequency of peak ice-influenced flows is established, the frequency of corresponding stages can be determined by introducing an empirically assessed local probability function of jam occurrence in any one year

The Discrete Outcomes Method (DOM), also known as the “G-C method”, after Gerard and Calkins (1984), involves generating stage-frequency relationships, where historical stage data are sparse or non-existent. It is described as the simplest hydrotechnical method available to determine the exceedance probability of ice-influenced flood peaks. The DOM assumes that the annual ice-influenced peak stage is caused by either sheet ice cover prior to its thermal decay or a fully developed local ice jam and determines the probability of ice affected stages by combining breakup flow frequency with conditional probabilities of ice jams.

As with other indirect approaches, this approach combines flow frequency estimates with two synthetic stage flow rating functions for ice affected conditions. The rating functions represent an upper limit and a lower limit such that stage can equal either a discharge dependent upper value or a discharge dependent lower value (Beltaos, 2012). Certain rivers and streams in Ontario have been or will be significantly modified by channelization, dyking, reservoir construction or ice retention works. As a result, the ice regime will not be statistically stationary and deterministic estimates rather than historically based ones

will be required to estimate ice jam stages. Such estimates may also be desirable to augment the historical base, or to provide a second estimate of design levels.

Whether an ice jam forms at a particular location as the ice cover breaks up has a direct impact on the peak stage during breakup. As such, the exceedance probability of an ice-jam stage during any one ice season must account for the probability of an ice jam occurring near the site of interest in any one year (Gerard and Calkins, 1984). The result is a combined probability considering both jam and no-jam conditions.

Historical flow data are far more readily available than peak ice-influenced stages because the spatial variability of flow is much smaller than that of ice-jam water levels. Therefore, the flow at an ungauged site may often be derived from records of upstream and/or downstream gauges, regional estimates, or by transposing data from drainage basins posing similar physiographic and landscape-based attributes. However, ice-jam stage data cannot be meaningfully transposed from other sites. Following the establishment of the frequency of peak ice-influenced flows is, the frequency of corresponding stages can be determined by introducing the empirically assessed local probability of jam occurrence in any one year, $P(J)$ (Beltaos, 2011).

As postulated by Gerard and Calkins (1984), if a jam does form, the peak breakup stage H_m can be assumed to be equal to the stage caused by an equilibrium jam that fully affects the site of interest at the prevailing discharge, Q_m . If a jam does not form, H_m is assumed to be equal to the stage associated with flow under a sheet-ice cover (Beltaos, 2011). The probability of H_m is then calculated using [Figure 6-19](#) combined with separately established probability of the discharge, and the probabilities of jam occurrence, $P(J)$, and non-occurrence, $P(NJ) = 1 - P(J)$. The approach has been termed the “discrete-outcomes method” by Beltaos (2011; 2012) since at any given flow, H_m can only take one of two possible values.

The analytical models described in [Section 6.4.1 Empirical Ice Jam Models](#) provide ice jam stage estimates as a function of streamflow. These flow data (unlike ice stage data) can be transposed to ice jam sites from non-jam areas and can be used to develop a probability distribution of ice jam stage. The methodology for conducting this work has been outlined by several authors, most notably, Gerard and Calkins (1984) and as summarized by Beltaos (2010; 2011, 2012; 2021).

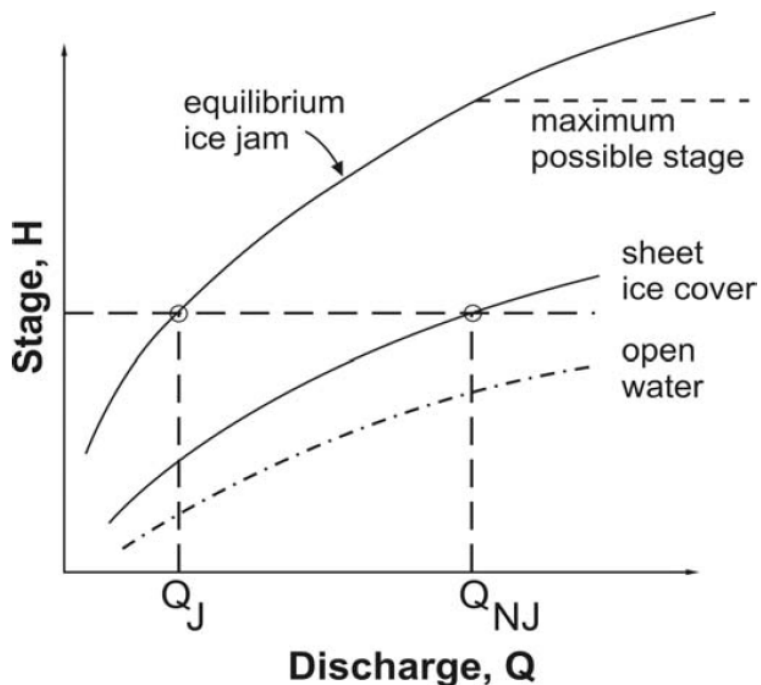


Figure 6-19 Schematic illustration of discharge-dependent upper and lower envelopes of peak ice jam breakup stages, depicting conceptual rating curves for open water, sheet ice cover and an equilibrium ice jam scenarios; where Q_J and Q_{NJ} , relate to 'jam' and 'no-jam'.

The procedure can be used with one or more of the analytical equations outlined earlier in, [6.4.1 Empirical Models](#), and is as follows:

- 1) Develop stage discharge curves for ice-affected conditions. This includes:
 - the stage effect of a solid ice cover (a lower bound on ice stage)
 - the equilibrium ice jam stage (an upper bound on ice stage) calculated by using the analytical approaches described in [Section 6.4.5 Empirical Ice Jam Models for Determining Potential Ice Jam Stage](#).
 - open water conditions ([See Section 5 Water Level Computations – Open Water Hydraulic Modelling](#)).

For the purposes of this approach the ice jam stage rating curve can be computed using equation 6.6 and [\(after Beltaos, 1995b\)](#).

- a) The suggested procedures include
 - i) determine the stream slope
 - ii) plot curves of mean depth and surface width as a function of stage
 - iii) estimate bed roughness
 - iv) estimate expected hydraulic roughness of the ice cover
- b) Then the following
 - i) select a stage for the bottom of the ice cover
 - ii) read of the mean depth and surface width of this stage
 - iii) calculate the water level stage.
- 2) Evaluate stage and discharge limits associated with each ice cover condition and incorporate the results in the stage-discharge curves considering the following details:
 - a) In river reaches with wide flood plains, environmental evidence (i.e., ice abrasion scars, shear lines), as well as analytical estimates (Calkins, 1983), will indicate an approximate upper limit defined by leakage or overflow.
 - b) If information is available from gauge or other records, site observations or local observers, an estimate of the range of stages or discharges associated with the solid ice cover prior to the break-up ice run should be made.
 - c) The volume of ice required to develop an ice jam of sufficient length and thickness may also limit the stage (Calkins, 1983).
- 3) Derive estimates of the maximum discharges at the site during ice covered periods.

Where no gauge exists in the reach, ice periods will have to be estimated from meteorological data and discharge data transposed from elsewhere on the stream or river of interested or other suitable streams in the vicinity by reviewing ice affected water level data, dates, and locations or by estimating using precipitation runoff models or other suitable modelled data (also See [Section 3.2.4 Regional Transfer of Location Techniques for Deriving Flood Peaks and Flow Time Series in Ungauged Basins](#))

- 4) For the limited number of years for which actual historical ice stage data are available or collected using the historical approach, derive stages from the synthetic rating curves and compare the observed stages to the synthesized stages and adjust the synthesized curve (or establish a compromise) if necessary. The degree of compromise should be based on the quality of the historical data.
- 5) For each year, determine, a) the maximum solid ice cover stage (from the stage-discharge curve for solid ice conditions) using the maximum ice-effect flow estimate. Similarly, determine b) the maximum ice jam stage (from the ice jam stage-discharge curve) using the maximum flow during the break-up period.
- 6) Rank the stage for each of these two sequences for the period of study and determine the cumulative probability distribution for each sequence.

Note: For discharges greater than the estimated breakup/ ice-run discharge, the ice jam stage distribution will be an upper bound on the true ice-related stage distribution in that it assumes the site is affected by an *equilibrium jam* each year. The solid ice stage provides a lower bound in that it assumes no jams form each year. If, for example, the peak discharge in a year is less than the breakup/ice-run discharge then there will only be a solid ice cover stage assigned to that year. This assumes it to have been a year in which the ice effectively melted in place.

- 7) Estimate the frequency of ice jam formation in the vicinity of the site from available historical information (See [Section 6.4.3.1 Direct Historically Based Approach](#)) and use this to deduce a final compromise probability distribution between the upper and lower bounds described above using the following information.

The exceedance probability, $P_J(H)$, of an ice-jam stage, H , during any one ice season can be calculated from:

$$P_J(H) = P(Q_{H,J})P(J) \quad (6.12)$$

in which $P(Q_{H,J})$ is the exceedance probability of the discharge ($Q_{H,J}$) that corresponds to H under ice jam conditions; and $P(J)$ is the probability of a jam occurring near the site of interest in any one year. The combined probability, $P_i(H)$, for ice affected stage under both jam and no-jam conditions (considered mutually exclusive events) near the site of interest and the exceedance probability of the discharge that corresponds to the water level under and ice jam and solid ice cover, respectively is obtained from:

$$P_i(H) = P(J)P(Q_{H,J}) + P(NJ)P(Q_{H-IC}) \quad (6.13)$$

where $P(NJ)$ is the probability of jam non-occurrence, and $P(Q_{H-IC})$ is the exceedance probability of the discharge that corresponds to the water level under solid ice cover.

Please note that this method of Gerard and Calkins (1984) assumes a constant $P(J)$, equal to the number of jam years divided by the total number of years of record (Beltaos, 2011, 2012).

The final product of this work will be a stage probability distribution curve similar to that shown by the solid curve in [Figure 6-20](#). Also shown in [Figure 6-20](#), are actual ice-related stages measured at the site during 35 ice jam events in the 58 years of record. The agreement is quite good and demonstrates that this analytically based deterministic approach (i.e., discrete function) can be used with some confidence.

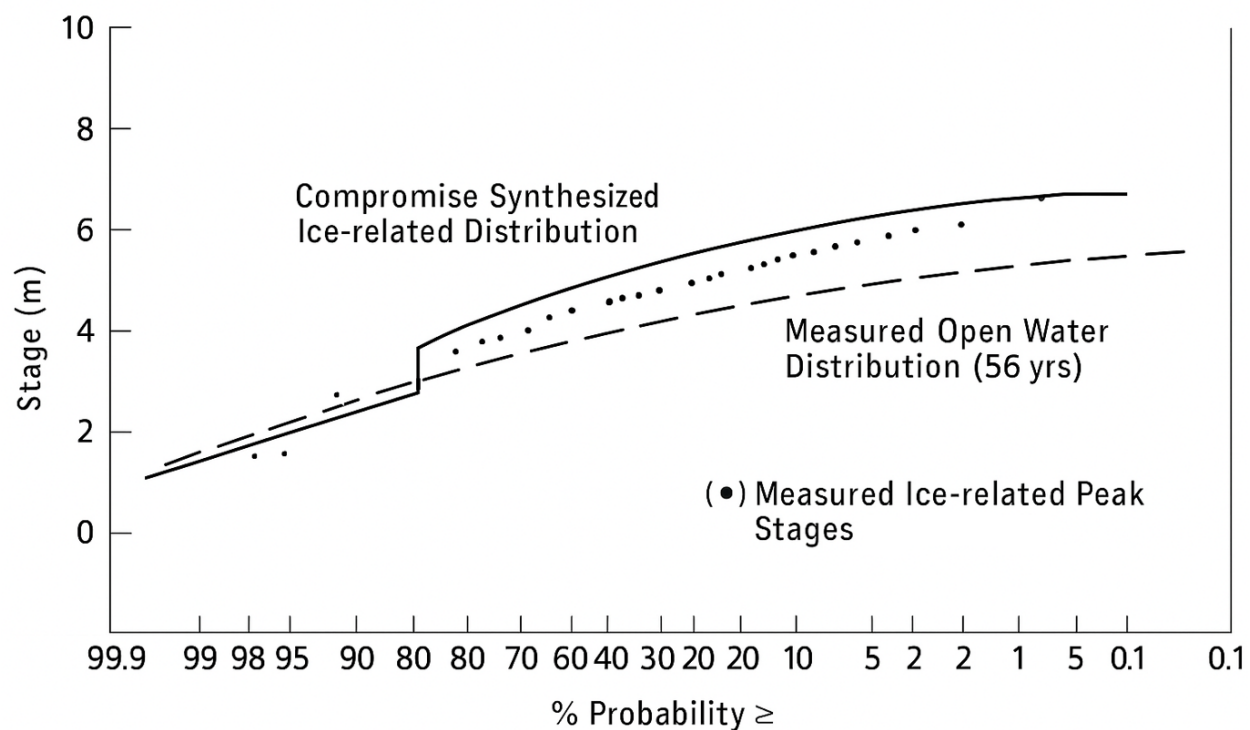


Figure 6-20 Comparison of synthesised and measured ice-related and open water stage probability distributions (Source: after Gerard and Calkins (1984).

As in the Integrated Direct Historically Based Approach (), this technique requires the practitioner to evaluate the physical characteristics of the site (and their implications on ice-effect stages) and draw together important information from interviews and environmental evidence at the site.

6.4.8.2.1 Caveats and Considerations

The probability of jamming is a central parameter in this analysis. The G-C method uses a constant value of $P(J)$ in its approach, which is equal to the number of jam years divided by the total number of years of record. However, this may not always be observed, as higher flows can reduce the chance of ice jam occurrence as suggested by the fact that no jams can form beyond a threshold discharge (White and Beltaos, 2008). This means that $P(J)$ could decrease with increasing discharge, becoming zero when discharge exceeds the ice-clearing flow value.

These concepts can be improved by consideration of additional physical constraints, including those introduced by Tuthill et al (1996) regarding the role of ice related threshold flows. This includes the minimum flow that can mobilize and break up sheet ice cover (i.e., $P(J) = 0$ below this threshold), as well as the maximum flow above which, sheet-ice cover cannot remain in place, often referred to as the ice clearing discharge (i.e., H_m reverts to the open water stage). The possibility of making such informed refinements should be kept in mind when conducting in situ reconnaissance and resident interviews in relation to ice affected flood history in the area of interest for flood hazard mapping.

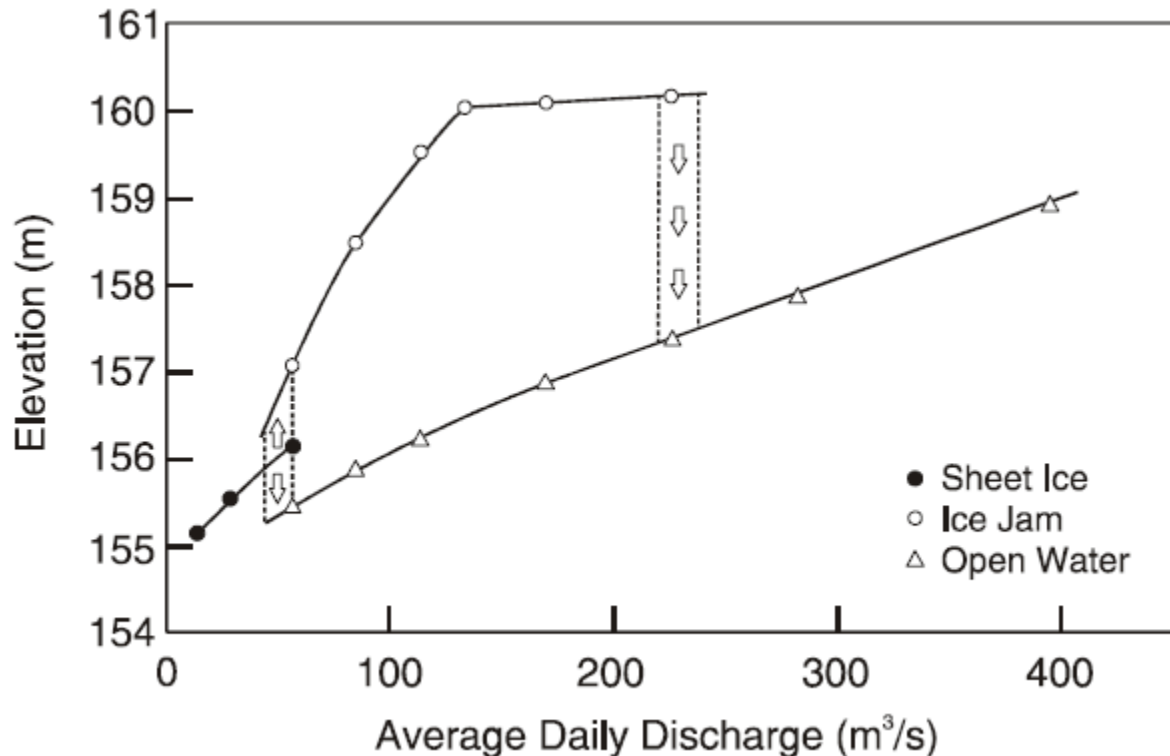


Figure 6-21 Ice affect on a portion of the stage discharge curve taking into account non-jam conditions at low and high flows (Source: Tuthill et al., 1996; with changes, from Beltaos, 2010)

6.4.8.3 Synthetic Frequency Analysis - Distributed Function Method

The distributed function method (DFM) can be used to generate synthetic probability distributions of ice jam flood levels, when historical data are scarce, based on the breakup discharge. The DFM (Beltaos, 2011; 2012; 2021) differs from conventional synthetic approaches for developing probability relationships in that it does not assume that the peak ice influenced stage is either equal to the maximum stage caused by an equilibrium jam or the minimum stage associated with the flow under ice sheet cover. Rather than assuming that peak ice influenced flow can only take one of two different values at a given flow, the DFM is considered to improve on the G-C method by treating peak breakup stages as a continuous distribution between discharge-dependent maximum and minimum envelopes, rather than discrete outcomes, by way of accounting for the conditional probability of stage. It was noted that in review of plots of peak breakup states versus prevailing discharge, that the data points do not tend to cluster around either the upper or lower envelopes, making them less discrete (Beltaos, 2012). Rather, data points tend to fill the enclosed area between the curves ([Figure 6-22](#)).

In this way, a peak ice influenced state (H_m) is actually considered a distributed function that can take on any value between the highest possible ice influenced stage (i.e., jam), H_{max} , and lowest possible ice influenced stage (i.e., no jam), H_{min} , defined by the upper and lower envelopes shown in [Figure 6-22](#). The DFM was developed to address this fact and uses a cumulative probability distribution while normalizing the water level based on site specific coefficients to determine the overall susceptibility of the site or area of interest to ice jam formation. In this way the DFM recognizes that stages within the envelope lines of water levels for sheet ice and fully developed ice jams are possible, considering the probability of peak discharge distribution as a function of the prevailing discharge.

The current state of science regarding breakup and jamming processes, verifies these observations and the example data plotted in [Figure 6-22](#). Acknowledging that a breakup event at any given site on any particular river can be either mechanical or thermal in nature (See [Section 6.2 River Ice Processes and Influence on River Stage](#)), and that significant ice jams only occur during mechanical breakup events, minor accumulations of ice flows may occur during a thermal event but they cause less significant observed impacts on stage. Therefore, the stage discharge relationship best describing a thermal event is that of ice sheet cover, depicted by the lower envelope (H_{min}) line in [Figure 6-22](#). As stressed by many (Beltaos, 2010, 2011, 2012; Burrell et al., 2024.), the probability (P) of jamming, $P(J)$, is not a precise concept, particularly with reference to ice jam stage. As highlighted in the discussion in [Section 6.2 River Ice Processes and Influence on River Stage](#) and in relation to [Figure 6-3](#), it is clear that jams formed by mechanical events are not located in exactly the same spot each year, and that the effect of the jam on stage can vary in relation to the site of interest. In the absence of knowledge of the jam length and proximity to the site of interest for flood hazard mapping, formation of a jam only implies a range of possible values of H_{min} , with only (H_{max}) being consistent with the discrete outcomes method of Gerard and Calkins (1984).

The DFM approach avoids these issues by recognizing that all values between H_{min} and H_{max} are possible, as long as the discharge is within the range observed during breakup flows, referred to here as the breakup discharge (Q_m). Unless an ice clearing discharge value can be determined from local bathymetric and morphological evidence, the practitioner must select a peak flow for the break up discharge event, Q_{peak} , so as to be conservative. The relative difference between Q_m and Q_{peak} affects the synthetic probability distribution. For the applicable flow, Q_m , the values will be consistent with a continuous cumulative probability distribution, so that $P(H_m < H)$ will vary between 0 and 1, as H varies from H_{min} to H_{max} . Furthermore, for $H < H_{min}$, $P(H_m < H) = 0$ and for $H > H_{max}$, $P(H_m < H) = 1$.

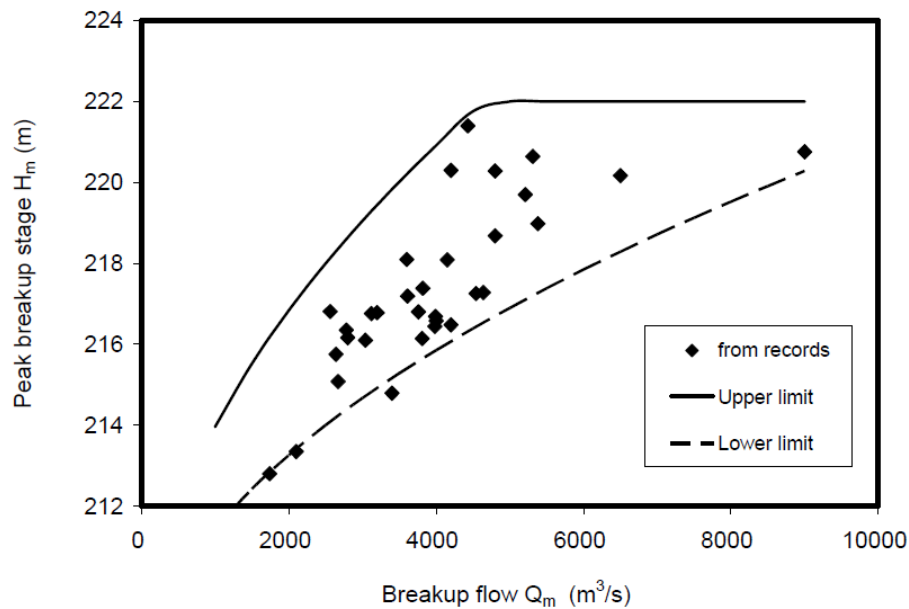


Figure 6-22 Plot of peak breakup stage (H_m) versus breakup flow (Q_m), along with upper and lower ice related envelopes, for Peace River at Peace Point, Partially based on Water Survey of Canada archived data (Beltaos, 2011).

The DFM approach does not make assumptions regarding the cause of peak stage, rather it relies on an empirically established similarity function, referred to as the probability density function $\varphi(\eta)$, that describes the conditional non-exceedance probability of the maximum breakup stage. Based on data and observations from four Canadian river research sites, $\varphi(\eta)$ can be expressed as,

$$\varphi(\eta) = (k + 1) \eta - k\eta^2 \quad (6.14)$$

where k is a site-specific factor related to ice jamming and η the dimensionless similarity variable, can be described as

$$\eta = \frac{(H - H_{min})}{(H_{max} - H_{min})} \quad (6.15)$$

Note that $\varphi(\eta)$ varies from 0 to 1 as η varies from 0 to 1, while $\varphi = 0$ for negative values of η , while $\varphi = 1$ when $\eta > 1$.

The probability distribution of the breakup flow, P_i , can be determined by assessing

$$P_i = P(H_m < H) = \int_0^1 \varphi(\eta) dP_Q \quad (6.16)$$

where dP_Q is the probability distribution of the breakup flow.

Equation 6.16 shows that P_i is controlled by two probability distributions:

- a) the probability distribution of the dimensionless stage variable, η
- b) the probability distribution of the flow Q_m .

Any plotted pair (Q_m , H_m) implies a value of η , where the corresponding H_{max} and H_{min} are used to calculate $\eta = (H_m - H_{min}) / (H_{max} - H_{min})$. Undertaking this operation will generate as many η -values as there are years of record. These can subsequently be ranked and probabilities assigned using a plotting position formula. Beltaos (2012) adopted the Weibull equation:

$$P = \frac{r}{n + 1} \quad (6.17)$$

where r is the rank in the ascending series order and n is the total number of available values.

The steps in operationalising the DFM approach are outlined below. Where data are limited it is possible to derive data from proxy basins, modelling or a combination of that include the Direct Historically Based Approach, see [Section 6.4.2.2 Historically Based Direct Flood Stage Estimates](#)

- 1) Collect data and undertake site characterisation
 - a. Undertake field reconnaissance to measure stream widths, bathymetry, gain an appreciation for channel morphology
 - b. Compile available historical data including if available, water levels, breakup dates, ice thickness, and discharge records. Where data are unavailable or sparse, collect from nearby gauged basins or via hydrologic models, which may be supplemented by meteorological information (e.g., air temperature, solar radiation among others).
 - c. Define specific reach boundaries based on hydraulic controls and available information on location of previous ice affected flooding.
- 2) Derive discharge dependent envelope curves

- a. Define maximum stage (H_{max}) by determining the equilibrium ice jam stage using equation 6.4, equation 6.5 or equation 6.6, representing the highest possible stage for a wide channel jam at a given flow (using data collected in 1b)
- b. Define the minimum stage (H_{min}) for sheet ice cover conditions at various breakup discharges (using data collected in 1b)
- c. Items 2a and 2b create or define the discharge dependent envelope within which any actual breakup discharge (H_m) will fall
- d. Determine or define the ice clear discharge value as determined from local bathymetric and morphological evidence where possible, if not select peak flow for the breakup discharge event Q_{peak} , to be conservative.

3) Normalize the flood stage data

Using equation 6.15 to derive the dimensionless parameter η to represent where a specific stage (H) resides between the two limits, where $\eta = 0$ corresponds to the lower envelope and $\eta = 1$ to the upper envelope; use this procedure to convert water levels into a normalized scale between 0 and 1.

4) Calculate conditional probability of the probability of stage

Rank the calculated η values in by ordering the observations in descending order and ranking data from largest (1) to smallest (n), where n is the total number of robust observations available by applying equation 6.17. Then, fit the distribution using the similarity function $\varphi(\eta)$ to assume the distribution of η is independent of the discharge (Q) using equation 6.14, where the parameter k is adjusted based on the susceptibility to jamming (See Beltaos, 2011; 2012).

To calibrate the DFM at a particular river site, historical ice-influenced stage data are necessary. If such data are not available, the DFM can be applied to examine different scenarios using plausible values of the coefficient k . Results from four river sites studied to-date suggest a range of 0.6 (jam-prone) to 0.9 (infrequent and/or minor jams) is a good approximation of k .

5) Quantify probability distribution and stage frequency relationship

Following the calculation of the similarity function $\varphi(\eta)$ use equation 6.15 which has been programmed into an Excel spreadsheet (see Beltaos, 2011; 2012) to fit the data. The exceedance probability (P_i) calculated using equation 6.16 is used to predict the probability that a specific ice jam flood stage will be exceeded in any given year. Any one value of H_m is treated as a constant, and upon changing this constant a new P_i is computed in the aforementioned spreadsheet.

6) Validation/Data Check

Estimate maximum possible stage, using local morphology and historically based data, as well as calculation check assuming there is no maximum possible stage, and assess relative to the maximum end of the probability curve.

For additional information, please consult Beltaos (2011, 2012) including details for using Excel to complete the computations by means of equation 6.16 once the function φ (equation 6.14) is at hand.

Site inspection and characterisation may identify extensive floodplains where water and ice can spill and escape, thereby limiting $H_{\max}$ to a constant rather than a flow dependent metric (see dashed line segment depicting maximum possible stage in [Figure 6-19](#) as well as the flattening of the upper limit line in [Figure 6-22](#)).

The final result of applying the DFM is shown in [Figure 6-23](#), where the simulated stage frequency distribution is shown to be in good agreement with the data points derived from historical gauge records. It is apparent that the vertical scale of [Figure 6-23](#) is unsuitable for determining water levels associated with peak ice influenced events. To accomplish this, the y-axis may be adjusted to the range 0.9 to 1, including tick marks corresponding to 0.91, 0.92, . . . , 0.99 (Beltaos, 2021). Stages can then be extracted from the modified graph and applicable return periods calculated as $1/(1 - P_{i,n})$. For example, the 20-, 50-, and 100-year stages would correspond to P_i values of 0.95, 0.98, and 0.99, respectively (Beltaos, 2021).

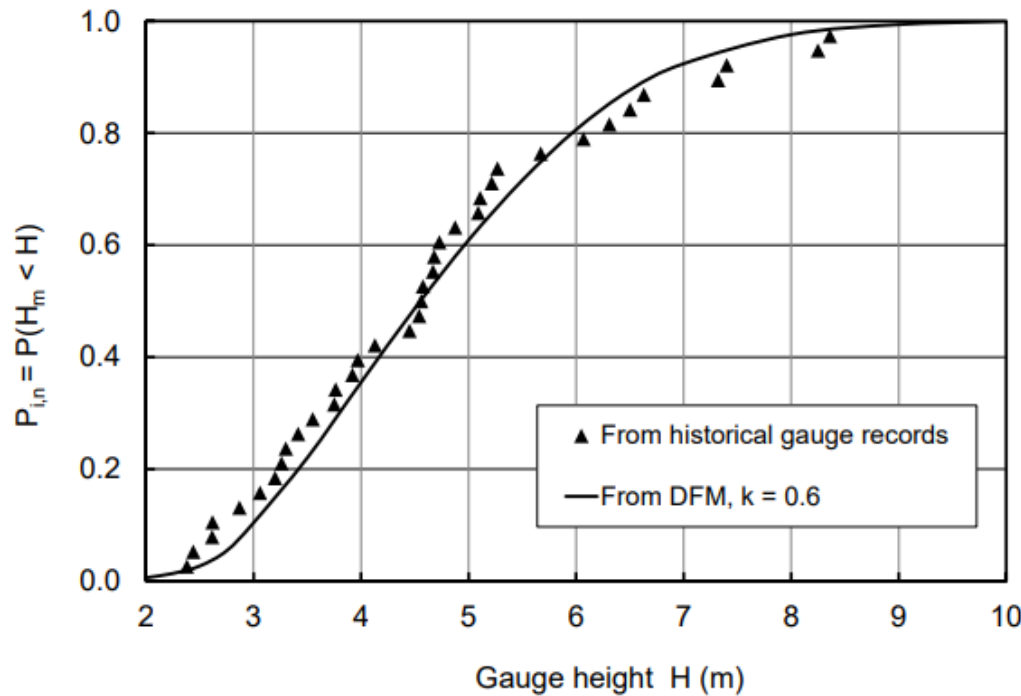


Figure 6-23 Probability distribution of breakup stage, for WSC gauge 01BJ007, Restigouche River above Grounding Brook, NB, Canada (Source: Beltaos, 2021, adapted from Beltaos, 2012).

While the focus of this work has been breakup jams, the DFM can be adapted to other ice affected flooding including freeze-up jams, as long as upper and lower thresholds can be defined for stage discharge plots. Additional work is required to better understand if the DFM can be applied to hanging dams, although information does suggest that flood stages of hanging dams are controlled by discharge.

6.4.8.4 Ice Jam Flood Frequency Analysis using a Monte-Carlo Analysis Framework

Advancements in approaches that combine modelling and statistical analysis such as those employed by Lindenschmidt (2015, 2016, 2024, 2025) across a range of Canadian rivers, involve employing a stochastic modelling approach to simulate the processes of ice-jam formation and associated flooding along the river during freeze-up or break up scenarios. The first approach (Lindenschmidt et al., 2015, 2016) involves the use of a deterministic river ice hydraulics model, whereas the second (Lindenschmidt et al., 2024, 2025) involves an empirical approach to calculate equilibrium backwater levels, with both separately being run repeatedly within a Monte-Carlo framework. The empirical equations specifically include the application of equation 6.4 and equation 6.6 as well as the

relationship between dimensionless discharge and dimensionless ice jam stage for equilibrium ice jams ([Figure 6-13](#)). The Monte-Carlo simulations involve running the model hundreds of times (or more), with each run having a different set of boundary conditions and parameter values randomly chosen from frequency distributions. An ensemble of backwater levels is then produced from which profiles of exceedance probabilities are calculated. Exceedance probabilities are chosen according to desired return periods (e.g., 1:100, 1:200 etc.) and water level profiles derived from the Monte-Carlo simulations are extrapolated into the floodplain to determine flood depths and extents under the various return period ice jam conditions.

The Monte-Carlo analysis provides a framework for accounting for the inherent uncertainty in ice-jam hazard assessment by simulating a range of possible ice jam scenarios (Lindenschmidt et al., 2025). Water surface elevations are then mapped to demarcate the ice-jam flooding hazard resulting from ice-induced river flooding. This approach supports a more comprehensive understanding of potential ice affected flooding hazards by considering the full spectrum of possible scenarios, rather than relying on a single model output. The opportunity to employ either an empirical model or a calibrated hydraulic model also allows one to cater to the available data and means of the study.

While applying these approaches is not necessarily appropriate for determining the actually experienced ice affected flood, they may be useful for corroborating or comparing to the elevation of the observed flood and to better understand the return period of that flood relative to the flood elevations of the established flooding hazard standards (i.e., Hurricane Hazel, Timmins flood and/or 1:100 year open water flood).

6.4.8.5 Hydrodynamic Modelling of Ice Affected Stage

At sites where ice-related information on past flood stages is limited or absent, the equations applied for use in the direct method (see [Section 6.4.2 Direct Methods - Including Data Sources](#)) cannot be used directly. Rather, available data are used to calibrate an analytical or numerical model that links ice-jam stage to river discharge, a variable that is much more readily available than ice-jam stage. Once the model is calibrated, it can be applied to all years of record synthetically to reconstruct annual values of peak ice-influenced stages. Input data to river ice models normally include the bathymetry of the channel and topography of the floodplain, the location and extent of the ice jam, a range of simulated discharges, realistic boundary conditions, channel bed and bank roughness, and historical water levels used for model calibration. The total amount of ice within the simulated jam (calculated by the model) can also be used for validation (the volume of ice in the jam can only be smaller than the volume of ice of the contributing

reach as defined by the ice thickness, channel width and reach length). The ice-jam routines of the models require the user to specify several parameters that describe hydro-mechanical properties of ice jams. This introduces a degree of subjectivity, especially where limited site-specific calibration data are available.

Enhanced accuracy can be attained using numerical ice-jam models, which can also compute water levels caused by non-equilibrium jams. The latter typically result from local constraints to the ice volume that can be available to form an ice jam. Several ice-jam models have been developed to date, ranging from simple one-dimensional, steady-state formulations to complex two-dimensional, dynamic algorithms. Beltaos (2008) described some of these models, while new ones have been developed since then (She et al., 2009; Nolin et al., 2009). The performance of various models in actual case studies has been investigated by a Task Force of the CGU-HS Committee on River Ice Processes and the Environment (Carson et al., 2005, 2007).

The ice options in hydraulic models can be extremely flexible in determining the stage when the discharge and ice jam thickness causes water to flow onto the flood plain. Using one of the techniques to determine reach averaged ice jam thickness at or near bankfull conditions, these thicknesses can then be used as input in the models to evaluate the conditions when flood plain flow occurs.

Furthermore, hydrodynamic models can prove invaluable for quantifying and better understanding the configurations of ice related phenomena, beyond exclusively ice jams, including for instance flooding and causative mechanisms surrounding hanging ice dams. For instance, a 1D model ([Figure 6-25](#)) was used in the lower reaches of the Kaministiquia River in northwestern Ontario to examine a hanging ice dam, quantifying and corroborating frazil ice generated hanging ice dam evolution and ice depth and associated flood elevations including calibration to observed water surface elevations (KGS, 2007).

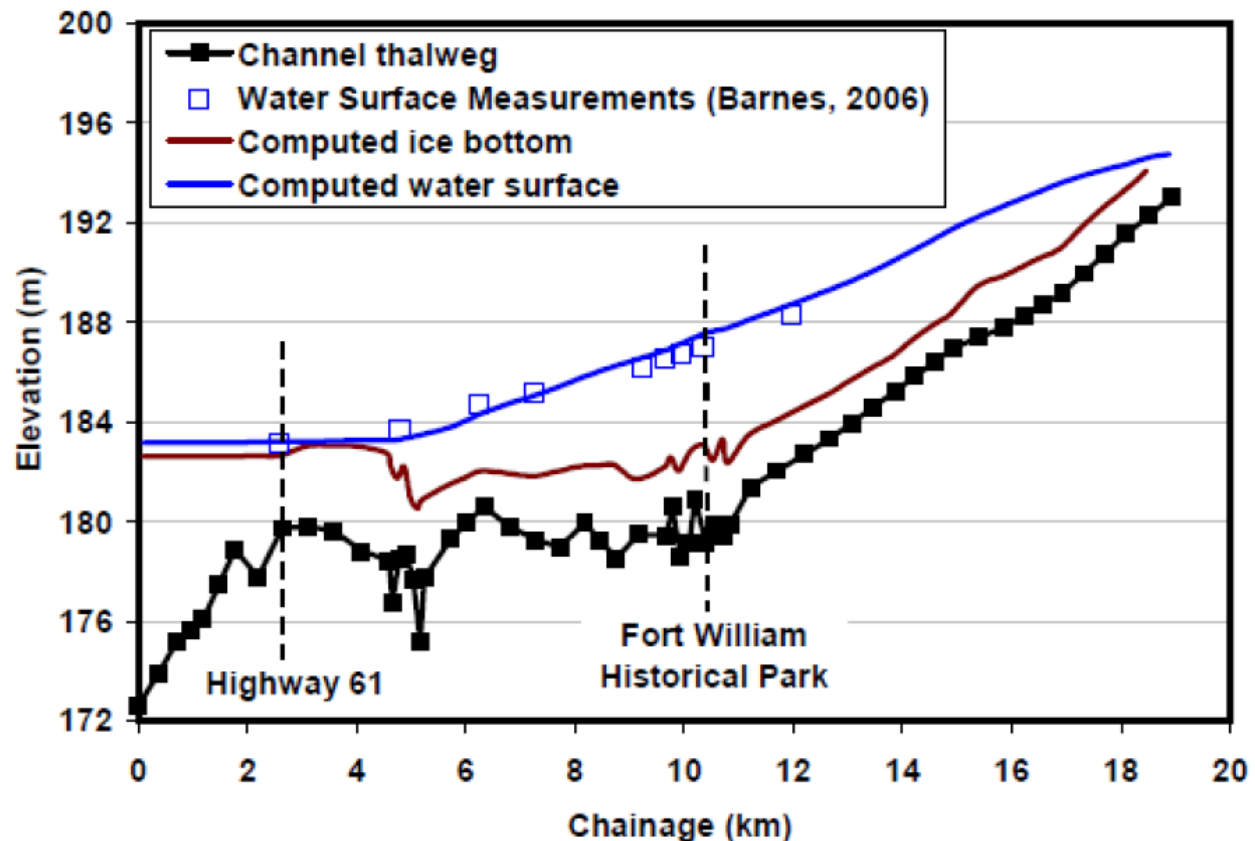


Figure 6-24 1D model of a hanging dam configuration (shown in burgundy) in the lower section of the Kaministiquia River in northwestern Ontario (KGS, 2007).

Hydrodynamic models can play a key role in corroborating historical ice jam information, whether it be biophysical, historical flood, flood envelope, or a combination thereof, by reconstructing past events, allowing practitioners to validate observed water levels, determine ice volumes and using these data, map flood extents that were not fully documented at the time of the ice affected flooding. Use and implementation of these models can provide important corroborative evidence in the reconstruction of historical events.

6.5 Procedure for Estimating the Ice-Affected Effect Stage and Comparison to Open-Water Flooding Hazard Standards

The first decision in the procedure follows a preliminary field reconnaissance and data review and is based on historical flooding at the site of interest. If none of the past flooding is affected, then ice-related levels need not be considered. If ice affected flooding has caused high water levels, the following steps are recommended to better quantify the impacts of ice on these levels and support a comparison to the open water flooding hazard standards.

- 1) Collection of relevant data is one of the most important elements in the evaluation of ice-related stages. The type of information required/available from the site is important and this key research is best undertaken by the engineer who will eventually complete any subsequent modelling or estimating of stage relationships.
 - a) Detailed river geometry can be as important or even more important in ice analyses as it is in open-water flood assessments. Information from on-site investigations may suggest that surveys be extended to higher elevations than for open water cases, for example, or include surveys of high-level diversions. If such cross-section data are not available, additional field surveys may be needed (or proposed surveys for open-water studies may require some extension). An ice reconnaissance survey is appropriate as well.
- 2) Once field data are assembled, reviewed and prepared in a reference report, the occurrence of ice impacted flood levels at the site can be estimated for subsequent use. This work is followed by several decision steps to determine which approach may or may not be used for estimating ice affected flood levels and their frequency as appropriate, but firstly (i.e., item a) below) it includes an estimate to evaluate whether the additional steps outlined below are required:
 - a) Using one or multiple empirical models listed in [Section 6.4.6 Empirical Ice Jam Models for Determining Potential Ice Jam Stage](#), estimate the equilibrium ice jam flood elevation for the site of interest and compare this elevation to the open-water flood stage calculated using the flooding hazard standards for area of interest (Figure 1-1). If the open water flood elevation exceeds the equilibrium jam stage, no additional work (other than documentation of this fact in report) is required. If the equilibrium jam stage

exceeds the water surface elevation under the flooding hazard standard, proceed through the listed items below.

- b) Where there is a long history of gauged ice affected flood stage data, and the data are robust and the ice regime has been statistically stationary during the period of these observations, and is anticipated to remain so for the foreseeable future), it is possible to employ these gauged data directly using the methods outlined in [Section 6.4.7.1.1 Ice Affected Stage Frequency Analysis](#).
- c) Where there is a long history of ice affected flood stage observations and the ice regime has been statistically stationary during the period of these observations (and will remain so for the foreseeable future), and gauged data are sparse or not available, it is possible to go directly to the Direct Historically Based approach for estimating ice-influenced stages outlined in [Section 6.4.7.2 Historically Based Direct Flood Stage Estimates](#).
- d) If these qualifications are not met, an analytical procedure leading to an Indirect Synthetic Generalized methods may be required ([see Section 6.4.8.1 Synthetic Generalized Estimation of Annual Maximum Stages](#)). However, it must be emphasised to remain consistent with the PPS, it is required that data must rely on actually observed ice induced flood phenomena, rather than estimates that synthetically produce data/results. This approach may also be used for stationary regimes with a long history of data to derive the best compromise estimates for upper and lower bounds on ice stage, to check, confirm or corroborate data gathered using Directly based methods outlined above.
- e) The study may only require a preliminary estimate of the ice jam stage-probability relationship. If there are some reliable data and the ice/river regime is statistically stationary, then the Direct Historically Based approach may be used for this estimate including corroboration with additional Indirect Methods as is suitable.
- f) Indirect Synthetic Approaches are used where there is limited ice-stage data, or the river regime is statistically non-stationary (or is anticipated to be in the near future). The analytical modelling procedures involved require measured or synthetically derived discharge data for the ice-related period, and it may be convenient to begin to assemble or generate the data at this point. Again these can be used to reconstruct or corroborate actually observed data outlined in items b) and c) above.
- g) The next step is to develop ice-related stage discharge curves and determine the best approach to this work. An evaluation providing very rapid development of ice-stage relationships for the site may be adequate. If so the Maximum Expected Ice Affected Stage may be suitable.

If a detailed evaluation is deemed necessary (to account for channel width variations, for example) employing one of more detailed approaches would be appropriate.

- 3) At this juncture, the above developed relationships must be adjusted on the basis of field evidence and historical data (which may indicate flood plain spillage, diversion flow, or other influences setting physical limits on the stage reached by ice-related flooding).
- 4) If appropriate, the adjusted stage discharge relationships are converted into frequency relationships using the discharge data assembled earlier; these are then converted into probabilistic relationships using the estimated frequency of significant ice jams developed from historical data or societal sources.
- 5) Overall, these procedures give a deterministic estimate of the ice jam stage-probability distribution paralleling that of the Direct Historically Based approach. These relationships may then be combined with open-water flood stage probability to determine an annual probability distribution of peak stage or used in separate flood damage analyses if seasonal flood damages are significantly different. Alternatively, an ice-related design flood level may be taken directly from the ice-related stage probability relationship.

In relative terms, the inclusion of an ice-related analysis with an open-water assessment will require a greater level of effort than an open-water assessment alone. A portion of this additional effort is taken up by the key requirement for collection and detailed study of environmental/societal ice data from the site. The remainder is taken up by the analysis approach. The Direct Historically Based approach is the most rapid because the analytical modelling procedures of the Indirect Synthetic approach are not required.

An important consideration and key decision point in applying a flood that is ice affected, involves the comparison of the water surface elevation or stage, observed or estimated to be observed, under that situation, to water surface elevation computations for the open-water flooding hazard conditions (i.e., Hurricane Hazel, Timmins Flood, 100-year) and assessing which is greater. The ice influenced stage can only be applied where it exceeds the established flooding hazard, and via approval from the Minister of Natural Resources.

As a result of the site-specific nature of ice-induced flooding, it is anticipated that the use and application of ice impacted flood stages will be localized and spatially integrated into broader flood modelling and mapping outside of the river reaches impacted by ice related floods. In this way, flooding hazard delineation will respect and delineate the ice-influenced flooding hazards in the context of the broader flooding hazards for the flood criteria zone where the area of interest is situated.

6.6 Historical Documentation and Collection of Ice-Effect Level Data

Investigation and documentation of historic, observed, recorded, and recent ice jam floods in the study catchment and in particular the reach(s) planned for flood hazard mapping is required to provide insight into potential ice jam flood mechanisms and to support ice jam assessment and modelling. Depending on the data available, an exhaustive narrative of a particular flood may not be required unless it directly supports estimates of ice jam flood elevation and hazard delineation tasks.

Local observations can help define the extent and character of previous flood events. It is important to investigate a variety of sources of additional information, including WSC data, the Canadian River Ice Database from then National Hydrometric Program archives (de Rham et al., 2020), the Canadian Disaster Database, maintained by Public Safety Canada, Ontario's Emergency Management Historical Events data layer (Ontario GeoHub), showing the location of past emergency events in Ontario and associated metadata. Specifically, the Canadian River Ice Database (de Rham et al., 2020) represents a significant amalgamation of river ice data summarizing key river ice parameters including timing of, and water level at, freeze-up, ice cover duration, water level at the initiation of breakup and spring breakup peak water levels, and contains valuable information if a specific project happens to be located near one of the 196 stations that have been analyzed (Turcotte and Nafziger, 2021; Barrette and Khan, 2024). For a description of the 15 key river ice parameters and their abbreviations summarized in the Canadian River Ice Database as well as others mentioned above, refer to Barrette & Khan (2024).

Historical information on ice-related floods, field observations and associated data are required for anticipating potential ice problems, completing ice management studies, modelling ice processes, and conducting stage frequency analysis of ice related flood levels. Some of the data needs for the latter are collected as standard procedure for open-water flood assessments, but many of the data needed for ice studies are additional to typical requirements for assessments under open-water conditions. Of particular importance can be gathering and reviewing original WSC paper and pencil/pen-based water level records identifying real-time recorded gauge height data as collected before the transition to digital records. Combing through data series is useful to establish knowledge on the link between watershed or gauging station specific river ice process and their signatures observed in water level time series data (see Turcotte and Nafziger, 2021). Additional guidelines have been established by Beltaos (1990) related to ice jam

case study and analysis and extraction of ice jam related data from hydrometric station records ([Figure 6-24](#)).

Relative to the definition of “flooding hazard” in the PPS, specifically item b) 3 “which was actually experienced in a particular watershed or portion thereof, for example as a result of ice jams”, highlights that the purpose of the consideration of ice jams or ice influenced floods is to consider documented floods including their flood stage in comparison to flood stages under the other established flooding hazard standards. As such, while it is important to understand the theories and mathematical models available to estimate maximum flood stage under the equilibrium ice jam condition, or to develop historical probability analysis plots that aim to define various recurrence interval stages for ice-influenced floods, this may not directly inform the requirements outlined at the beginning of this paragraph. However, these various methods may prove valuable in corroborating, confirming or establishing the ice influenced flood stage historically observed in the watershed, at a site of interest. It is with regard to these comparative foci that these methods are presented; and their value will be relative to amount of local data available in relation to historical flood events in the area of interest.

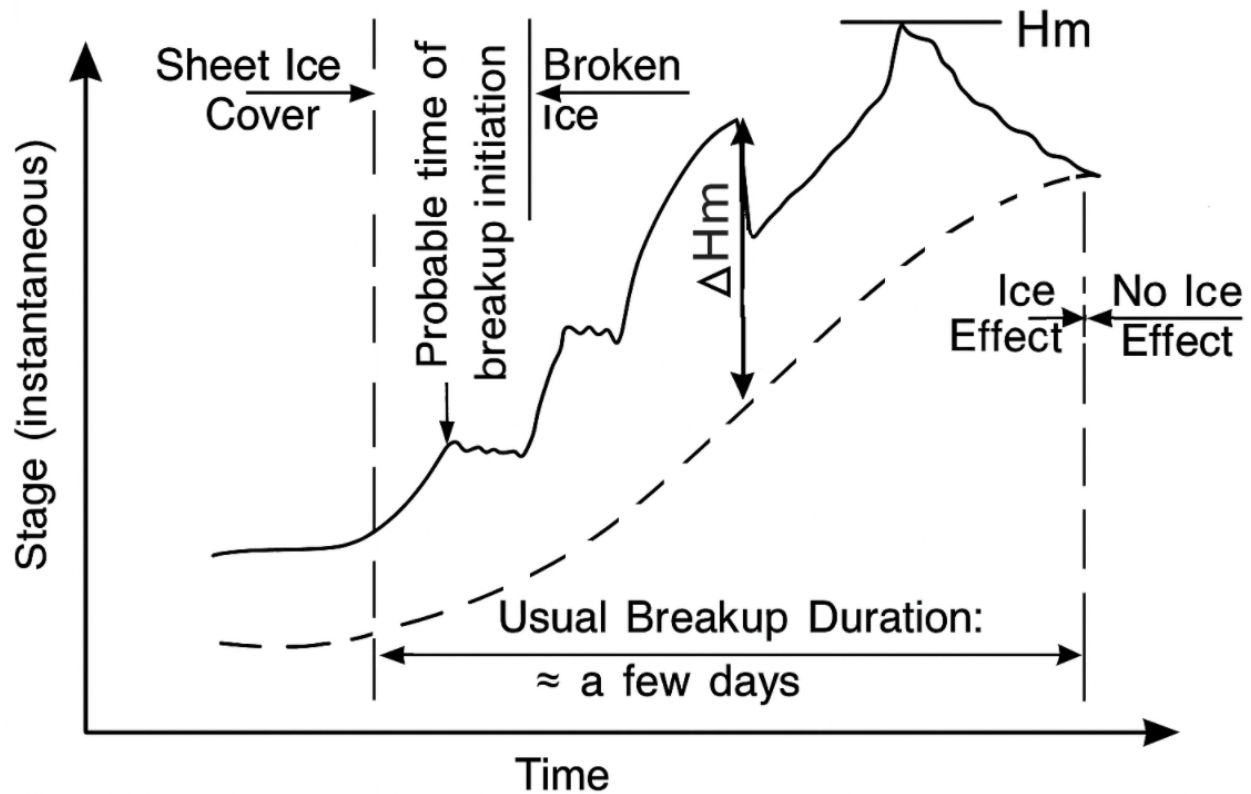


Figure 6-25 Annotated Illustration of instantaneous stage with time during ice breakup, where H_m is equal to maximum breakup stage and ΔH_m is the difference between the actual stage and the effective stage (i.e., ice effect on stage) (After Beltaos, 1990).

6.6.1 Historical Flood Level Data

As ice jams are reach-specific phenomena, historical flood level information is needed from the specific reach under study. This data may be obtained from environmental evidence or other sources. Information from the latter includes recorded data in:

- diaries, logs, letters, journals, local and regional records, archives, conservation reports, and previous studies
- photograph albums, television news records, video coverage
- remotely sensed data (aerial and satellite imagery, UAV imagery)
- stream gauge records and Water Survey of Canada ice condition notes (e.g., HYDAT HYDEX remarks)
- ice jam databases (Canadian River Ice Database from then National Hydrometric Program archives (de Rham et al., 2020), the Canadian Disaster Database, maintained by Public Safety Canada, Ontario's Emergency Management Historical Events data layer (Ontario GeoHub), showing the location of past emergency events in Ontario and associated metadata).

Information and techniques profiled in [Section 4.3 Criteria and Associated Data to Support Hydraulic Analysis in Delineating the Flooding Hazard](#), may prove particularly useful in relation to ice affected flooding also.

Valuable data also resides with streamside residents, traditional environmental knowledge, community newspaper archives and local libraries, former Hudson's Bay Company trading post archives (Hudson's Bay Company Archives records are extensive from 1670 onwards see Archives of Manitoba, including maps of HBC fur trade posts in Ontario to better understand proximity to the site of interest), public works officials and government agencies. From these data sources and interviews with these observers, important data can be derived on:

- ice jam locations
- ice-related flood levels and extent
- dates, frequency and the relative timing or sequence of events during ice-related floods
- velocity, depth, ice thickness and dimensions
- antecedent and post-jamming conditions upstream and downstream
- water level stages perceived to be indicative of developing ice jam problems.

The location and effect of ice jamming may vary from location to location within even a short study reach and hence it is useful to interview a number of people within the study area. Information provided by each should be carefully screened to ensure that flood levels described by each can be identified with a specific location.

Environmental evidence of historical ice-affected flood levels is given by:

- ice scars or abrasion marks (on trees, structures, drainage outlets or other features that may be scarred by ice), or by large scale vegetation damage
- disturbed bank material or ridges caused by ice push.

Scars on trees may be dated by tree ring analysis to provide the year as well as the elevation of the flood, and the profile of ice scar levels derived along the study reach gives substantial information for use in establishing envelopes of flood stages occurring in the past. These data assist in deriving ice jam elevation data and also corroborating or reconciling or calibrating flood accounts by local inhabitants to these environmental indicators.

Ice scars—wounds on trees caused by moving ice are highly reliable indicators of past maximum ice-breakup water levels. When analysed via dendrochronology and dendrogeomorphology including tree-ring dating, that can provide precise annual records of ice-push events, often acting as the only available data for remote, ungauged locations.

Studies show that ice scars often correlate strongly with the highest water level elevations recorded during specific ice-jam events, making them effective for reconstructing flood history (Gerard, 1981; Lubiniecki et al., 2022). Researchers use tree-ring analysis (dendrogeomorphology) to date the scars, which allows them to determine the exact year of an ice jam event occurred (Lubiniecki et al., 2022).

Ice scars are used to map historical flood hazards, estimate the frequency of ice-jam events, and identify the maximum height reached by ice piles along rivers and lakes.

These scars act as a natural gauge that allows specialists in the field of dendrogeomorphology to reconstruct long-term, high-resolution datasets for flood hazard analyses.

While useful, ice scars must be collected from appropriate, similar sites to avoid overestimating flood levels; factors like shore exposure, wind direction, and tree location can influence the height of the scar.

6.6.2 River Geometry

Cross sections and profiles required for open-water analyses are also one of the key requirements for assessing factors which control ice levels. For ice studies, physical survey requirements include:

- Sufficient cross sections to adequately describe the reach averaged geometry and any atypical features, and that these all be tied to a reference datum. A reach length in the range of 10 times the width may be sufficient to adequately define the geometry; however optimal spacing for stable model results may be closer to 1 channel width (i.e., 1W) (Beltaos and Tang, 2013). It is important that cross sections capture representative planform channel and floodplain characteristics, changes in discharge, slope changes, structure locations, and potential overland flow routes (NHC, 2023).
- Bed and water level profiles extending upstream and downstream of the site particularly if ice jams in these areas are the cause of flooding at the site of interest.
- Identification of overbank spill areas or diversions, or obstructions to such diversions in the likely ice jam reach.

Detailed elevation datasets including lidar-derived digital terrain models (DTMs) are a definite asset, particularly if there is a possibility of spills or diversions around the ice jam site. With the extensive use of detailed digital elevation data, many software packages have effective tools for calculating where water will go when an ice jam is introduced. As ice jam stages may be several metres above even the 100-year open-water level, it is worthwhile including these high levels in establishing the vertical limits of the field survey program. Additional useful information regarding the collection of river geometry information for ice studies including for developing hydraulic models, can be found in NHC (2023).

6.6.3 Ice Monitoring

Ice observations at freeze-up, break-up and mid-winter provide valuable insight into the processes that may shape the break-up regime during ice jam years. In most years, such observation will not coincide with significant ice jam flooding since such floods are generally infrequent. Nevertheless, where ice-jamming causes flooding, establishing a regular program will assist in interpreting cause-effect relationships in ice jam years and provide an understanding of ice processes in the reach and quantitative stage data for use in stage frequency analysis.

The level of effort involved in an annual monitoring program will depend on the site, the nature and significance of the ice problem, the frequency of significant floods and budget

constraints. At a minimum, field observation which will be useful for projecting ice-related stages should be taken over a long reach upstream and downstream of the site of interest and should include:

- stage measurements at freeze-up and break-up
- documentation of Freeze-up and break-up dates
- descriptive observations at break-up (sequences, jam locations, thickness and strength)
- photographic records of ice conditions (surface texture, anomalies).

At some sites, the general understanding of ice processes might be aided by:

- periodic stage measurements from freeze-up to post break-up
- periodic ice thickness and thickness cross section surveys
- records of open water areas, leads
- stage-discharge measurements
- additional photographic surveys
- aerial and/or near real time satellite monitoring; UAV monitoring.

Also, it may be advantageous at some locations to:

- install new or additional level gauges
- establish benchmarks to assist in level monitoring in ice jam years.

6.6.4 During an Ice Jam

If ice jamming does occur, as much quantitative work as can be conducted with the resources at hand will be invaluable for later study. Particular focal points should include:

- water surface profile along the jam
- nature of the ice comprising the jam
- location of the ice jam (toe) affecting the site and cause of the jam
- evolution of the ice jam
- flow estimates (channel, overbanks, in diversions)
- complete photographic coverage (from ground and air)
- time record of observations; location of upstream or downstream jams
- reconstruction of break-up sequence that led to the jam.

6.6.5 Post Ice Jam Observations

Post-jam observations and data collection can also provide highly valuable information:

- associated meteorological conditions
- ice floe thickness along riverbanks, ice strength (qualitative)
- shear wall thickness, location, elevation
- high water mark elevations along the jam.

6.6.6 Report and Documentation

The final and most important result of this work is a summary document drawing all the observations together. It has proven most beneficial to prepare such a report immediately after the observations are completed, so that recent recollections can be recorded while they are still fresh in the observer's mind. Further to this, in the event that the observed ice jam stage does indeed exceed the respective flooding hazard standard(s) for the area of study, solid documentation will be required to inform an application by the municipality and/or conservation authority to support the Ministers review and any subsequent approval that may be granted to an ice jam stage value as the flooding hazard standard for that specific area of river.

6.6.7 Ice-Affected Flooding: Influence of a Changing Climate

To assist in preparing for the impacts of a changing climate and potential increases in the risk associated with flooding, as outlined in the PPS, it is important to recognize that with a changing climate, it is anticipated that mid-winter thaw related ice jamming may become a more common occurrence (Das and Lindenschmidt, 2021b). Trend analyses of ice jam flooding for streamflow gauges across Canada (Royaka et al., 2018b) have highlighted distinct shifts in the magnitude and timing of observed ice influenced floods. For southeastern Canada, including Ontario, this suggests a tendency towards the occurrence of earlier ice jamming, with the largest shifts observed in unregulated rivers and in streams with smaller drainage basins. A study of trends in ice conditions in Ontario rivers (Curi et al., 2021) identified a decreasing trend in the number of ice-influenced daily flows and water levels, including the number of days with ice-influenced conditions in southern and central portions of the province, and showed a consistent trend towards earlier occurrence of maximum flows under these conditions. The same study showed that the trend for maximum water levels occurring during these ice-related conditions was undefined, being approximately evenly split between increasing and decreasing trends. Under increasing trends, ice jams may have the potential to produce higher water levels than the existing regulatory event defining the flooding hazard limit (i.e. 100-year flood, Hurricane Hazel, Timmins Flood). This will be dependent on a range of factors, including, but not limited to, local reach conditions, the history of ice-related issues, including ice

jamming. Trends in and propensity of ice-effect related flooding should be monitored closely to ascertain any noteworthy shifts that may affect local flooding hazards, and the associated water surface elevations and inundation under observed or projected ice-related flooding hazard conditions.

An example decision tree is included (see [Figure 6-25](#)) highlighting a simplistic representation of how the probability of seasonal ice jam flooding may change under different climatic conditions based on air temperature and discharge. with differing predicted future changes in air temperature and streamflow. For rivers and streams currently affected by ice jams, application of this decision tree can be useful in better understanding how ice jam occurrence and timing may change. This information can be used to inform how such conditions may affect the magnitude of flooding and associated regulatory flow events influenced by or defined by ice jam flooding.

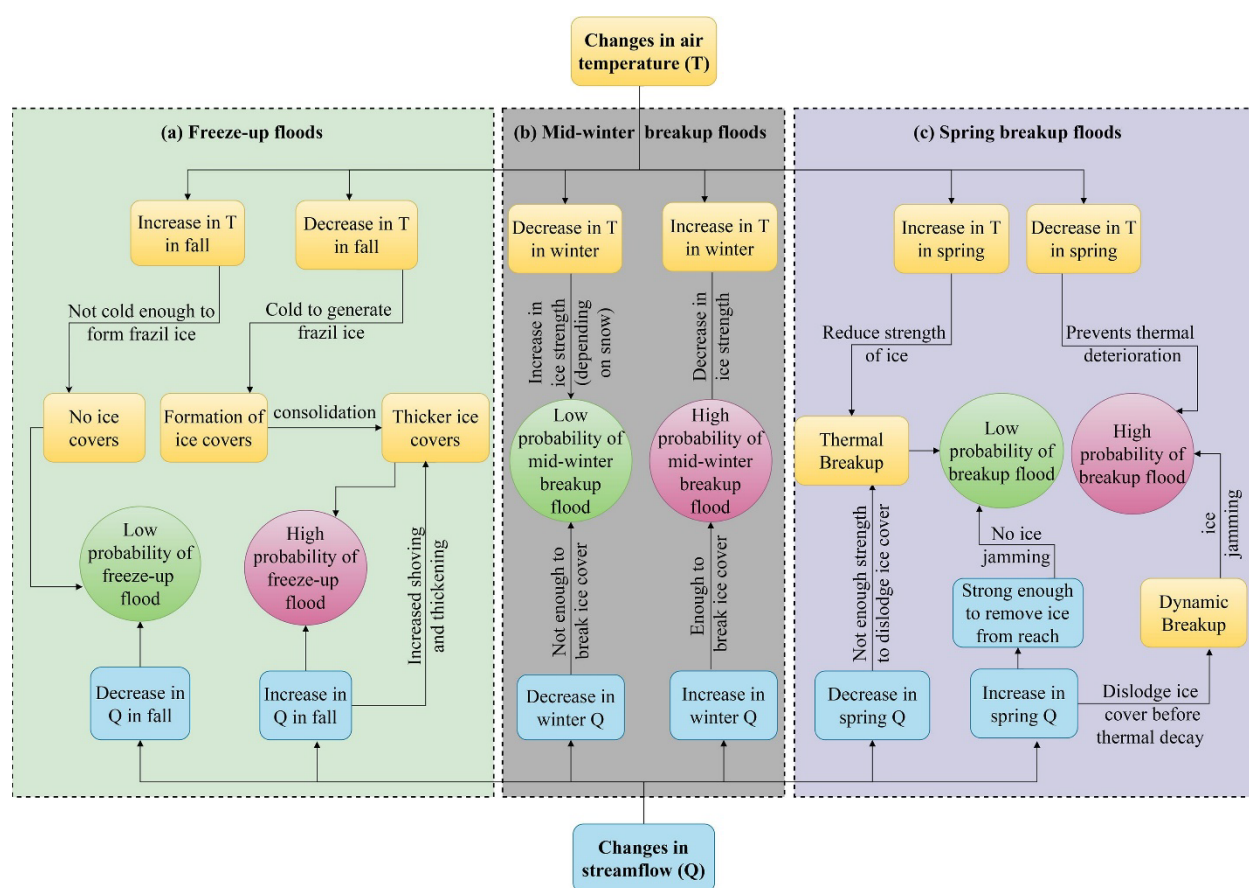


Figure 6-26 Decision tree for probability of seasonal ice affected flooding occurrence given future changes in air temperature and streamflow. For information purposes only (Source: Rokaya et al., 2022)

6.7 Conclusions

The potential exists for floods affected by ice to produce water surface elevations that exceed those experienced under the flooding hazard standards across all three flooding hazard zones in Ontario ([Figure 1-1](#)); particularly at historically ice-jam flood prone locations along large and small rivers in the province. Where history (recurrent and more recent) has shown that ice related flooding represents a substantive flood generating mechanism, further investigations are warranted to compare the flood stages under ice-related flood events, to water surface elevations computed for the established flooding hazard standard(s). To limit flood hazard modelling (and mapping) to only open-water events is to ignore a type of riverine flooding that can have severe consequences in Ontario.

It is evident from the above discussions that a substantial commitment can be required to develop ice-related frequency distributions which can be employed with reasonable confidence. The approaches require:

- detailed site reconnaissance
- collection of historical and environmental data
- careful evaluation of ice data.

River ice monitoring relies on multiple data sources, including aerial assessments, that can involve flying planes (e.g., typical manned aircraft or unmanned aerial vehicle (i.e., drone)) over the rivers or streams get an aerial overview.. Use of near real-time satellite data and new methods of river ice detection helps to monitor, confirm, and classify river ice in an early stage. A roll-up of these data that assist in flood forecasting can be invaluable for informing ice jam computations and assessing ice jam flood levels for flood hazard assessments and flood hazard mapping, where appropriate. This includes for areas where ice jams have historically been an issue, or recent evidence suggests they may become an issue in the future.

Although some of this work is an additional requirement, much of it can be efficiently combined with that which is undertaken for more standard open-water flood studies.

At sites which lack a reasonably long-term history of ice observation, or for which changes have or will be made to the stream and/or streamflow, or where the influence of a changing climate is anticipated to result in an increase in ice-related flooding potential it will be necessary to:

- generate peak winter streamflow data

- prepare ice-related stage-discharge curves.

Significant progress has been made over the last 25 years in the discipline of ice jams and ice processes research, which has been integrated into this guidance document; however, it is anticipated that additional refinements will continue to be made. Additional experience gained via site assessments in Ontario, and as the ice-related site information and modelling approaches continue to be refined, it is anticipated that this process will become more streamlined and similar to open water assessments in their ease of implementation and application.

7 Technical Report and Documentation

7.1 Technical Report

An important step towards meeting the goals of flood hazard management is the completion of flood hazard studies, which will provide communities, planning authorities and administering agencies with the necessary database of clearly presented technical information. The following description summarizes the technical information recommended to be included for those undertaking the work in house or sourcing it from service providers, as required by administering agencies. Technical staff involved in the review process may periodically need advice on modelling or statistical analysis. Such expertise may be available with private consultants and academics.

Upon completion of the hydrologic and hydraulic investigations, and prior to the compilation of the flood risk maps, a draft technical report should be prepared. Technical Advisory committees can be a useful vehicle for planning authorities (i.e., municipalities) to coordinate with partner agencies.

The report should present the studies in sufficient detail so that specialists in this field can determine the adequacy of the work and its conformance to the procedures outlined in this document, as well as reproduce any results.

A review of the draft report will be undertaken at this stage and then, subject to the acceptance of the work described in it, the flood hazard maps will be compiled. Third party reviews should also be conducted throughout development of the report, or at completion of the report.

The technical report should include the following material:

1. Introduction:

Include a brief statement on the purpose of the study and a summary of past flooding problems. Any past flood plain studies and mapping should also be briefly referenced.

2. Study Area:

Describe study scope and identify study limits on small scale maps. Communities affected and any existing structural and non-structural flood protection and

emergency measures should be presented. List future land use plans, and any proposed flood mitigation measures.

Describe the area to be mapped, the watercourse and its tributaries, watershed characteristics, climatic conditions and flood-generating mechanism.

3. Hydrological Analyses:

Flood Standard:

Describe the flood standard to be used for rivers and lakes, and any past recorded flood events within or in adjacent watersheds. If a *Two Zone* or *Special Policy Area* scenario is considered as an alternative to the one zone concept, provide detailed description of the alternative and any limitation on the increase in water levels.

Data:

All the data used in the investigation, whether measured or estimated, should be fully described.

The source of the data should be given and background information must be provided for any assumed values to enable an assessment of their validity. Tables, maps, and graphs should be used to illustrate data such as streamflow records, historical storms, stage-discharge relationships, cross-sections, and surveyed profiles. Reproductions of relevant aerial photographs should also be presented.

The supporting documents used for crediting any purpose-built *constructed storage* or constructed flood structures must be provided. This includes studies, structural assessments, calculations and checklists as appropriate. See MNR (2026-Draft) for additional details.

Flood Computations – Flood Frequency Analysis:

Describe streamflow and rainfall groups and records. Information relevant to the conversion of regulated streamflow to natural conditions and the stationary nature of the data series should be fully described. If the effects are not sufficiently large to warrant adjustments to the data, this should be explained with quantitative estimates of their significance at the point in question.

For a single station frequency analysis, the data used should be shown and the choice of probability distribution and method of parameter estimation explained. Frequency curves with plotted data points and computed confidence limits are essential. When the flood standard is based on the 100-year flood, the expected probability adjustment should be shown as well. If a historical flood is included in the analysis, the techniques used must be explained and the effects of its inclusion on the frequency curve and the variance of the estimate should be shown. Should a joint probability analysis be required, all relevant information including basic assumptions should be described.

For a regional flood frequency analysis, the extent of the region and streams included must be described along with the records used at each station. The single station analysis should be covered as above and, depending on the method of regionalization used, homogeneity tests, variables used in multiple variable regression analysis and their statistical significance, regression equations, correlation coefficients, and standard errors of estimate must be described.

Should a transfer of location of the estimated flood be required, the method and the underlying assumptions must be explained. Reconversion of natural flood estimates to regulated conditions should be described when appropriate along with an explanation of the operating procedures assumed and the basis for the assumptions.

Flood Computations – Rainfall/Runoff Modelling:

The input in this case must be fully documented and any storm transposition and change in orientation explained. A brief description of the hydrologic model employed must be included, outlining the basic methodology and assumptions, and the history of its previous use. The data required to operate the model, both measured and estimated, should be shown with a background explanation for the estimated values.

Calibration and Validation:

Describe the method of calibration and validation, the events used, and errors. Compare computed flows with flows obtained from other analyses, such as the MTO, Regional Frequency or for small areas, the Rational Method.

Presentation of Flows:

Present flows in tabular form for different events and locations. Compare results with previous estimates and recorded events.

4. Hydraulic Analyses:

Describe topography of the flood plain and the methods used for taking cross-sections. Specify the datum used for the study and the conversion of any data inputs from other datums. Provide a summary of structures, dimensions and photos. For the river system describe the roughness coefficients and starting water levels used.

Describe river crossings with significant storage effects.

Extrapolation of stage-discharge curves should be shown with an explanation of the methods employed. A description of the backwater program should include a brief explanation of each aspect of its operation that is significant to the particular stream. It should be clear which coefficients were estimated and which were obtained by direct or indirect measurement. An executable digital version of all study models should be provided. This will allow review of plots of all cross-sections, as well as water surface profiles for the entire reach for each of the flood events considered.

If the floodway is delineated, the specified increase in the water levels should be explained and the limits of the floodway shown on the cross-section plots. The water surface profile computed for the floodway condition must also be included. In cases where ice or log jams are taken into account, a complete description of the techniques used should be given.

If structures (dykes, berms, flood walls, flood protection landforms) have been purpose built for flood management, the floodway and flood fringe should be explained and the limits and a summary of the impacts of the structure on the flooding hazard limit and associated water levels, should be included.

A summary of the backwater computation should be provided, listing water levels for different flows and locations. Previous backwater analyses and observed past flood levels should be incorporated in the summary.

Calibrate the backwater model, or in absence of historical observed flood observations, undertake a sensitivity analysis to test the changes in water levels caused by changes in roughness coefficients and other appropriate parameters.

This shall include a report documenting results of calibration and sensitivity analyses.

Spill areas should be delineated, and the effect of spill should be commented on.

5. Flood Line Delineation:

Describe mapping used, date of aerial photos, scale, contour intervals, accuracy and checking, topographic data sources and datum specified. Explain the method used to plot flood lines. Where the study included the floodway and flood fringe zones, describe each and the effect of encroachment on the flood plain. Similarly, describe the effect of dykes or dams on the flood lines. See MNR (2023) for additional information regarding data survey and mapping specifications.

6. Other Studies:

Should special cases arise that necessitate the use of procedures outside the scope of this document, a complete description of the techniques used must be given. It should not be necessary for the reviewer to search through a list of references to fully understand the investigation; the report should stand as a self-explanatory document to personnel who are experienced in this field of study.

7. References

8. Exhibits:

- location maps to identify study area;
- land use and soil map/data used;
- identification of historically flooded areas;
- identification of location of major structures, bridge data sheets;
- historical photos, if available;
- tables and hydrographs to provide summary of discharges, elevations and mean velocities with a cross section reference;
- flood frequency curves;
- where the two-zone application was used, computed width, depth and velocities across the flood fringe areas at both sides of the section; and

- hydraulic model outputs including plot of flood profile.

9. Appendices:

Input data and output summary should be presented in Appendices.

7.2 Third party technical reviews

Study components of technical reports including hydrological and hydraulic analyses should be reviewed to determine correctness, completeness and appropriateness of procedures used to develop information for the report.

Reviews can be undertaken near the end of a study, or throughout the study. Both approaches have limitations. For example, there may not be sufficient budget to correct any deficiencies found in a review if the review is conducted near the end of a study. Also, a reviewer's independence may become jeopardized if reviews are conducted throughout the study, as the reviewer may act as an advisor rather than a reviewer. The preference is to have the reviewer involved throughout the study.

A formal process should be followed to review hydrological and hydraulic studies, and each review should be documented. Reviews may be performed by the conservation authority or other organization soliciting the work or can be undertaken by third parties.

Reviewers must have sufficient expertise in the area of practice to properly assess the analysis. Reviewers must be capable of conducting the study, and reviewers must have a thorough understanding of methodologies and knowledge of relevant accepted engineering practice. A reviewer should have access to any computer program being used in the study. A reviewer should be a Professional Engineer licensed in the Province of Ontario.

The reviewers report should identify the following:

- The purpose of the review
- The basis under which the review was conducted
- A brief description of the technical report under review
- A summary of documentation provided to the reviewer
- Communications made during the review

- A description of the reviewer's methodology for conducting the review
- The reviewer's findings that include information upon which the opinions are based.

8 References

- Alberta Environment and Parks. (2022). *Alberta's Flow Estimation Tool for Ungauged Watersheds – Users Guide* (AFETUW v. 2.0). Government of Alberta.
- Alfieri, L., Laio, F., and Claps, P. (2008). A simulation experiment for optimal design hyetograph selection. *Hydrological Processes: An International Journal*, 22(6), 813-820.
- Alipour, A., Jafarzadegan, K., Moradkhani, H. (2022). Global sensitivity analysis in hydrodynamic modelling and flood inundation mapping. *Environmental Modelling Software*, 152, 105398.
- Anderson-Nichols and Co. (1984). *Application Guide for Hydrological Simulation Program - FORTRAN (HSPF)* (EPA-600/3-84-065). U.S. Environmental Protection Agency: Environmental Research Laboratory. Athens, Georgia, USA.
- Andres, D. (1995). Frazil generation and ice floe formation on a regulated river. In *Proceedings of the 8th Workshop on the Hydraulics of Ice Covered Rivers* (pp. 59-75), Kamloops, British Columbia, Canada.
- Annis, A., Nardi, F., Volpi, E., and Fiori, A. (2020). Quantifying the relative impact of hydrological and hydraulic modelling parameterizations on uncertainty of inundation maps. *Hydrological Sciences Journal*, 65(4): 507-523.
- Barrette, P., and C. Gaiqui. (2023). *Overview of water level, discharge and ice in the St. Lawrence Seaway at Montreal* (Report No. NRC-OCRE-2022-TR-036). National Research Council of Canada, Ocean, Coastal and River Engineering Research Centre. Ottawa, Ontario, Canada.
- Barrette, P., Ghobrial T. and Kolerski T. (2025). *On floods caused by river ice: An overview of mitigation measures* (NRC-OCRE-2024-TR-027). National Research Council of Canada. Ottawa, Ontario, Canada.
- Barrette, P.D. and Lindenschmidt, K.E. (2023). Frazil ice events: assessing what to expect in the future. *Hydrology Research*, 54(6): 770–781.

- Bartlett, M.S., Parolan, A.J., McDonnell, J.J., and Porporato, A. (2016). Beyond the SCS-CN method: A theoretical framework for spatially lumped rainfall-runoff response. *Water Resources Research*, 52(6): 4608-4627.
- Barrette, P.D., Khan, A.A. and Lindenschmidt, K.-E. (2024). A glance at 25 process-based hydraulic models for river ice. 27th IAHR International Symposium on Ice, Gdansk Poland – June 9-13, 2024.
- Beaton, et al. (2019).
- Beaton, A.D., Han, M., Tolson, B.A., Buttle, J.M., Metcalfe, R.A. (2024). Assessing the impact of distributed snow water equivalent calibration and assimilation of Copernicus snow water equivalent on modelled snow and streamflow performance. *Hydrological Processes*, 38(2), e15075.
- Beltaos, S. (1983a). River ice jams: theory, case studies, and applications. *Journal of Hydraulic Engineering*, 109(10), 1338-1359.
- Beltaos, S. (1983b). Ice freeze up and breakup in the lower Thames River: 1980-81 Observations. National Water Research Institute, Hydraulics Division, Unpublished Report for Environment Canada, Burlington, Ontario, 83 pp.
- Beltaos, S. (1984). *Ice Jams*, Lecture Notes, Short Course on River Ice Engineering. Fredericton, N.B., Canada.
- Beltaos, S. (1987). Ice Jams. IAHS Workshop on River Ice Vancouver, August 12, 1987
- Beltaos, S. (1988). Configuration and properties of a breakup jam. *Canadian Journal of Civil Engineering*, 14(4): 685-697.
- Beltaos S. 1990. Guidelines for extraction of ice break-up data from hydrometric station records. In Working Group on River Ice Jams: Field Studies and Research Needs. NHRI Science Report No. 2, National Hydrology Research Institute, Environment Canada, Saskatoon, SK; 37-70
- Beltaos, S. (1995a). (Ed.) *River Ice Jams*. Water Resources Publications, Highlands Ranch, CO. USA.
- Beltaos, S. (1995b). Chapter 4: Theory. In: *River Ice Jams* (S. Beltaos, ed.), Water Resources Publications, Highlands Ranch, Colorado, USA.

- Beltaos, S. (2008a). Challenges and opportunities in the study of river ice processes. 19th IAHR International Symposium on Ice, Vancouver, British Columbia, Canada July 6 to 11, 2008.
- Beltaos, S. (2008b). River Ice Breakup. Water Resources Publications, Highlands Ranch, Colorado, USA
- Beltaos, S. (2010). Assessing Ice-Jam Flood Risk: Methodology and Limitations. 20th IAHR Symposium on Ice, Lahti Finland, June 14-18.
- Beltaos, S. (2012). Distributed function analysis of ice jam flood frequency. *Cold Regions Science and Technology*, 71, 1-10.
- Beltaos, S. (Ed.) River Ice Formation; Committee on River Ice Processes and the Environment, Canadian Geophysical Union: Edmonton, AB, Canada, 2013.
- Beltaos, S. (2021). Assessing the frequency of floods in ice-covered rivers under a changing climate: review of methodology. *Geosciences*, 11(12), 514.
- Beltaos, S. (2025). Ice Jam Flooding of the Drying Peace-Athabasca Delta: Hindsight on the Accuracy of the Traditional Knowledge and Historical Flood Record. *Environments*, 12, 376
- Beltaos, S., Boyle, P., Hryciw, K. (2007a). 2005-06 ice-jam flooding, Kaministiquia River near Fort William Historical Park. 14th River Ice Workshop, Québec City, CGUHS CRIPE, paper No. a1.
- Beltaos, S., and Prowse, T. D. (2001). Climate impacts on extreme ice-jam events in Canadian rivers. *Hydrological Sciences Journal*, 46(1), 157-181.
- Beltaos, S., Hulley, M., Keene, B. & Watt, E. (2007b). Frazil-ice flooding and potential mitigation: Moira River at Belleville. In: Proceedings of the 14th Workshop on the Hydraulics of Ice Covered Rivers Quebec City.
- Beltaos, S. and Tang, P. 2013. Applying HEC-RAS to simulate river ice jams: snags and practical hints. *Proceedings of the 17th Workshop on River Ice*, CGU HS Committee on River Ice Processes and the Environment, Edmonton, AB.
- Berton, R. and Rahmani, V. (2024). Differences in flood quantiles estimate of disturbed and undisturbed watersheds in the United States. *Hydroecology and Engineering*, 1(1), 17.

- Brown, S., and Metcalfe, R.A. (2014). *The slope-area method for estimating continuous discharge*. Ontario Ministry of Natural Resources.
- Bruce, J.P. (1957). *Hydrometeorological Analysis of the Storm of August 28- 30, 1956 in Ontario* (CIR 2886, TEC 246). Meteorological Branch.
- Buffin-Belanger, T., Bergeron, N.E., Dube, J. 2013. Chapter 11: Ice Formation in Small Rivers. Beltaos, S., Ed.; Committee on River Ice Processes and the Environment, Canadian Geophysical Union: Edmonton, AB, Canada, 2013; pp. 385-409.
- Burn and Whitfield. (2016).
- Burrell, B.C., Comfort, G., Beltaos, S. (2024). Considerations in the planning and design of bridges in ice-affected rivers: a review. *Canadian Journal of Civil Engineering*. 51(2): 109-124.
- Burrell, B.C., Huokuna, M., Beltaos, S., Kovachis, N., Turcotte, B., and Jasek, M. (2015). Flood hazard and risk delineation of ice-related floods: Present status and outlook. *Proceedings of the 18th Workshop on the Hydraulics of Ice Covered Rivers*, Quebec City, Quebec, Canada.
- Burrell, B.C. and Tang, P. (2009). River-ice management implications of midwinter jamming. In *Proceedings of the 15th Workshop on River Ice* (pp. 209–218). St. John's, NL, Canada.
- Burges, S. J., Lettenmaier, D. P., and Bates, C. L. (1975). Properties of the three-parameter log normal probability distribution. *Water Resources Research*, 11(2), 229-235.
- Calkins, D. J. (1983). Ice jams in shallow rivers with floodplain flow. *Canadian Journal of Civil Engineering*, 10(3), 538-548.
- Canadian Dam Association (CDA). (2007). *Technical Bulletin: Hydrotechnical Considerations for Dam Safety*. CDA.
- Carson, R., Beltaos, S., Groeneveld, J., Healy, D., She, Y., Malenchak, J., Morris, M., Saucet, J-P., Kolerski, T., Shen, H.T. (2011) Comparative testing of numerical models of river ice jams. *Canadian Journal of Civil Engineering* 38: 669–678.

- Chen, B., Krajewski, W. F., Liu, F., Fang, W., and Xu, Z. (2017a). Estimating instantaneous peak flow from mean daily flow. *Hydrology Research*, 48(6), 1474-1488.
- Chen, X., Shao, Q., Xu, C.Y., Zhang, J., Zhang, L., and Ye, C. (2017b). Comparative Study on the Selection Criteria for Fitting Flood Frequency Distribution Models with Emphasis on Upper-Tail Behavior. *Water*, 9(5):320.
- Chen et al. (2018).
- Chow, V., Maidment, D., and Mays, L. (1988). *Applied Hydrology*. McGraw-Hill Book Company, New York.
- Crawford, N.H., and Linsley, R.K. (1966). *Digital Simulation in Hydrology: Stanford Watershed Model IV* (Technical Report No.39). Stanford University: Department of Civil Engineering. Palo Alto, California, USA.
- Conservation Authorities Act*, R.S.O. 1990, CHAPTER C.27.
- Conservation Halton. (2024). *Conservation Halton Spill Flood Hazard Policy: Technical Companion Document [DRAFT]*. Accessed on February 5, 2026, from Conservation Halton Web Site: <https://www.conservationhalton.ca/wp-content/uploads/2024/09/Attachment-2-CH-Spill-Technical-Companion-Document-24-09-19.pdf>
- Curi, F., Beaton, A., Beltaos, S. (2021). Recent Trends in Ice Conditions in Rivers in Ontario, Canada. *Proceedings of the 21st Workshop on the Hydraulics of Ice Covered Rivers*, Saskatoon, Saskatchewan, Canada.
- Curi, F., Boyd, D., Carson, R., and Rungis, G. (2019). Brantford 2018 Ice Jam. *Proceedings of the 20th Workshop on the Hydraulics of Ice Covered Rivers*, Ottawa, Ontario, Canada.
- Curi, F. and Mitchell, C. (2021). Bolton 2019 Ice Jam. In *CGU HS Committee on River Ice Processes and the Environment, 21st Workshop on the Hydraulics of Ice Covered Rivers*, Saskatoon, Saskatchewan, Canada.
- Cumming Cockburn Ltd. (1986). *Regional Flood Frequency Study of Ontario*. Report elaborated for the Ontario Ministry of Natural Resources.

- Cumming Cockburn Ltd. (2000). *Flood Regionalization for Ontario Streams*. Prepared for the Ontario Ministry of Natural Resources.
- Daly, S.F. Aufeis. Chapter 6. In *River Ice Formation*; Beltaos, S., Ed.; Committee on River Ice Processes and the Environment, Canadian Geophysical Union: Edmonton, AB, Canada, 2013; pp. 159–179
- Dalrymple, T. (1960). *Flood Frequency Analysis, Manual of Hydrology: Part 3* (Paper 1543-A). U.S. Geological Survey Water-Supply.
- Das, A., and Lindenschmidt, K.E. (2020). Current status and advancement suggestions of ice-jam flood hazard and risk assessment. *Environmental Reviews*, 28(4), 373-379.
- Das, A., and Lindenschmidt K.E. (2021a). Evaluation of the sensitivity of hydraulic model parameters, boundary conditions and digital elevation models on ice-jam flood delineation. *Cold Regions Science and Technology*, 183, 103218.
- Das, A., and Lindenschmidt, K.E. (2021b). Modelling climatic impacts on ice-jam floods: A review of current models, modelling capabilities, challenges, and future prospects. *Environmental Reviews*, 29(3), 378-390.
- de Munck, S., Gauthier, Y., Bernier, M., Chokmani, K., and Legare, S. (2017). River predisposition to ice jams: a simplified geospatial model. *Natural Hazards and Earth System Sciences*, 17: 1033-1045.
- de Rahm, L., Dibike, Y., Beltaos, S., Peters, D., Bonsal, B., and Prowse, T. (2020). A Canadian River Ice Database from the National Hydrometric Program Archives. *Earth System Science Data*, 12: 1835-1860.
- Devak, M., and Dhanya, C.T. (2017). Sensitivity analysis of hydrological models: review and way forward. *Journal of Water and Climate Change*, 8(4): 557-575.
- DHI Water and Environment Inc (DHI). (2018). *MIKE FLOOD Modelling Approaches for Floodplain Mapping* (DHI Project #: 42801316). Toronto and Region Conservation Authority.
- Di Baldassarre, G., and Claps, P. (2011). A hydraulic study on the applicability of flood rating curves. *Hydrology Research*, 42(1), 10-19.
- Di Baldassarre, G., Laio, F., and Montanari, A. (2009). Design flood estimation using model selection criteria. *Physics and Chemistry of the Earth*, 34: 606-611.

- Dickinson, T., Rudra, R., and Panjabi, K. (2018, March 7). *Floods in Southern Ontario Have Changed*. Presentation at Ontario Flood Plain Mapping Knowledge Transfer Workshop, Vaughan, Ontario, Canada.
- Dillon, M.M. (1994). *Snow Pack Water Equivalent Study*. Ontario Ministry of Natural Resources.
- Doherty, C., Adams, B., and Haley, D. (2017). *Technical Guidelines for Flood Hazard Mapping*. Environmental Water Resources Group Ltd. and University of Toronto. Toronto, Ontario, Canada.
- Dow Ambtman, K. and Hicks, F. (2012). Field estimates of discharge associated with ice jam formation and release events. *Canadian Water Resources Journal*, 37(1): 47-56. 10.4296/cwrj3701868
- Emergency Management and Civil Protection Act*, R.S.O. 1990, CHAPTER E.9.
- England, Jr. J.F., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, Jr. W.O., Veilleux, A.G., Kiang, J.E., and Mason, Jr. R.R. (2019). *Guidelines for determining flood flow frequency—Bulletin 17C: Book 4 -Hydrologic Analysis and Interpretation, Chapter 5 of Section B -Surface Water (Techniques and Methods 4-B5, Version 1.1)*. U.S. Department of the Interior: U.S. Geological Survey. <https://doi.org/10.3133/tm4B5>
- Environmental Protection Agency (EPA). (2009).
- Faghfour, A., Hentati, A., Fortin, G., Germain, D. (2023). A novel statistical model for flood prediction in the Eel River watershed, New Brunswick, Canada. *Water Science*, 37(1).
- Faulkner, D., Griffen, A., Hannaford, J., Sharkey, P., Warren, S., Shelton, K., Vesuviano, G., Mastrantonas, and Stewart, L. (2020). *Development of interim national guidance on non-stationary fluvial flood frequency estimation - Technical Report (FRS18087/IG/R1)*. UK Environment Agency.
- Faulkner, D., and Warren. S. (2020). *Development of interim national guidance on non-stationary fluvial flood frequency estimation – practitioner guidance (FRS18087/IG/R2)*. UK Environment Agency: Flood and Coastal Erosion Risk Management Research and Development Programme.
- Faulkner, D., Warren, S., and Burn, D. (2016). Design Floods for all of Canada. *Canadian Water Resources Journal*, 41(3): 398-411.

- FEMA. (2023). Guidance for flood risk analysis and mapping - Ice-Jam Analyses and Mapping. Guidance Document No. 94. November 2023. U.S. Department of Homeland Security.
- Fenton, J.D., and Keller, R.J. (2001). *The Calculation of Streamflow from measurements of stage* (Technical Report 01/6). Cooperative Research Centre for Catchment Hydrology.
- Fill, H.D and Steiner, A. A. (2003). Estimating instantaneous peak flow from mean daily flow data. *Journal of Hydrologic Engineering*, 8: 365-369.
- Fread, D.L. (1993). Chapter 10: Flow routing. In D.R. Maidment (Ed.), *Handbook of Hydrology*, pp. 10.1-10.30. McGraw-Hill, INC., New York, NY, USA.
- Gaur et al. (2018)
- Gerard, R. (1981). Ice scars: Are they reliable indicators of past ice breakup water levels? Proceedings of the International Association of for Hydro-Environment Engineering and Research, Ice Symposium, Quebec City: 847-859.
- Gerard, R., and Karpuk, E. W. (1979). Probability analysis of historical flood data. *Journal of the Hydraulics Division*, 105(9), 1153-1165.
- Gerard, R., and Calkins, D.J. (1984). Ice Related Flood Frequency Analysis: Application of Analytical Estimates. In *Proceedings of Cold Regions Engineering Specialty Conference*, CSCE, Montreal, Quebec, Canada.
- Gerard, R. (1989). Chapter 10: Ice jam floods. In: *Hydrology of Floods in Canada*; Watt, W.E., Ed.; National Research Council of Canada: Ottawa, ON, Canada; pp. 169–184.
- Gerard, R.L., and Davar, K.S. (1995). Introduction. In: S. Beltaos (Ed.), *River Ice Jams* (p.p 1-28). Water Resources Publications.
- Ghate, S. R., and Whiteley, H. R. (1982). GAWSER—A Modified HYMO Model Incorporating Areally-Variable Infiltration. *Transactions of the ASAE*, 25(1), 134-142 & 149.
- Glaves, R. and Waylen, P.R. (1997). Regional flood frequency analysis in Southern Ontario using L-moments. *Canadian Geographer*, 41(2): 178-193.

- Goodell, C. (2016). *1D? 2D? or 1D/2D? How Should I Build my Model? The RAS Solution, The Place for HEC-RAS Modelers*. <https://www.kleinschmidtgroup.com/ras-post/1d-2d-or-1d-2d-how-should-i-build-my-model/>.
- Grover, P., Vrkljan, C., Beltaos, S., and Andres, D. (1999). Prediction of ice jam water levels in a multichannel river: Fort Albany, Ontario. In *Proceedings of the 10th Workshop on River Ice* (pp. 15 - 28), Winnipeg, Manitoba, Canada.
- Gupta, A., Herrera, W., Wijesekara, G.N., Valeo, C., Marceau, D.J., and Moreno, N. (2014). Prediction of near real time natural flows in gauged and ungauged watershed of south Saskatchewan River Basin – What are our options? In J.W. Pomeroy, P.H. Whitfield, and C. Spence (Eds.), *Putting Prediction in Ungauged Basins into Practice* (pp. 221-230). Canadian Water Resources Association.
- Hamed, K.H., and Rao, A.R. (1999). *Flood Frequency Analysis*. CRC Press, Boca Raton, Florida, USA.
- Hirose, M. and Okada, K. (2018). A 0.01° resolving TRMM PR Precipitation Climatology. *Journal of Applied Meteorology and Climatology*. 57: DOI: 10.1175/JAMC-D-17-0280.1
- Hogg, W.D. (1980). Time Distribution of Short Duration Storm Rainfall in Canada. In *Canadian Hydrology Symposium: 80* (pp. 56-63). National Research Council Canada, Ottawa, Ontario, Canada.
- Hogg, W.D., and Carr, D.A. (1985). *Rainfall Frequency Atlas for Canada*. Environment Canada: Atmospheric Environment Service, Ottawa, Ontario, Canada.
- Holtan H. N. (1961). *A Concept for Infiltration Estimates in Watershed Engineering* (USDA-ARS Volumes 41–51). USDA: Agriculture Research Service. Washington, DC, USA.
- Holtan, H. N., and Lopez, N. C. (1971). *USDAHL-70 Model of Watershed Hydrology* (Technical Bulletin No. 1435). US Department of Agriculture. Washington, DC, USA.
- Holtan, H.N., Stiltner, G.J., Henson, W.H., and Lopez, N.C. (1975). *USDAHL-74 Revised Model of Watershed Hydrology* (Technical Bulletin No. 1518). USDA: Agriculture Research Service. Washington, DC, USA.
- Horton, R. E. (1941). An approach toward a physical interpretation of infiltration-capacity 1. *Soil science society of America journal*, 5(C), 399-417.

- Hosking, J.R.M. (1990). L-Moments: Analysis and estimation of distributions using linear combination of order statistics. *Journal of the Royal Statistical Society. Series B (Methodological)*, 52(1): 105-124.
- Hosking, J.R.M. (2024). *Package 'lmom'* (Version 3.2). R functions for use with the method of L-Moments. <https://cran.r-project.org/web/packages/lmom/lmom.pdf>
- Hosking, J. R. M., and Wallis, J. R. (1993). Some statistics useful in regional frequency analysis. *Water resources research*, 29(2), 271-281.
- Hosking, J., and Wallis, J. (1997). *Regional Frequency Analysis: An Approach Based on L-Moments*. Cambridge, Cambridge University Press.
- Hosseini, B. (2018). *A brief review of numerical modelling of ice jam flooding and studies relevant to coastal communities in Ontario's Far North* (Technical Report OCRE-TR-2018-007). National Research Council of Canada: Ocean, Coastal and River Engineering. Ottawa, Ontario, Canada.
- Hu, L., Nikolopoulos, E.I., Marra, F., and Anagnostou, E.N. (2020). Sensitivity of flood frequency analysis to data record statistical model, and parameter estimation methods: An evaluation over the contiguous United States. *Journal of Flood Risk Management*, 13: e12580.
- Huber, W.C., Heaney, J.P., Nix, S.J., Dickinson, R.E., and Polmann, D.J. (1982). *Storm Water Management Model User's Manual, Version III*. U.S. Environmental Protection Agency: Municipal Environmental Research Centre: Office of Research and Development. Cincinnati, Ohio, USA.
- Huff, F. A. (1967). Time distributions of rainfall in heavy storms. *Water Resources Research*, 3: 1007-1019.
- Hughes, D.A. and Smakhtin, V.U. (1996). Daily flow data time series patching or extension, A spatial interpolation approach based on flow duration curves. *Hydrological Sciences Journal*, 41(6): 851-871.
- HYDROTEK. (1985). *Flowfreq/PC Frequency Analysis of Hydrologic Maxima: Notes and Users Manual*. HYDROTEK Water Resource Consultants, Richmond Hill.
- International Association of Hydrological Sciences (IAHS). (2023). *Welcome to the PUB-Hub!* IAHS. Retrieved April 14, 2023, from <https://iahs.info/pub/index.php>

- Jain, S.K. and Singh, V.P. (2023). *Water Resources Systems Planning and Management* (2nd Ed., Vol. 51). Elsevier Science.
- Johanson, R.C., Imhoff, J.C., and Davis, Jr. H.H. (1980). *User's Manual for Hydrological Simulation Program - Fortran (HSPF)* (EPA-600/9-80-015). US Environmental Protection Agency: Office of Research and Development: Environmental Research Laboratory. Athens, Georgia, USA.
- Keifer, C.J., and Chu, H.H. (1957). Synthetic storm pattern for drainage design. *Journal of the Hydraulics Division*, 83(4): 1332.1-1332.25.
- KGS Group. 2007. Fort William Historical Park, Thunder Bay, Ontario: Conceptual study of flood mitigation. Interim report prepared for Ontario Ministry of Tourism, Winnipeg, Manitoba, Canada.
- Khaliq, M.N. (2024). Estimation of instantaneous peak flows in Canadian rivers: an evaluation of conventional, nonlinear regression, and machine learning methods. *Water Science & Technology*, 89(9): 2225-2239.
- Klemes, V. (1986). Operational testing of hydrological simulation models. *Hydrological Sciences Journal*, 31(1): 13-24.
- Knack, I.M., Shaw, J., Groeneveld, J., McHenry, J., Lavender, S.T., and Friday, W.L. (2024). Numerical modelling of ice control on the Albany River, Ontario, Canada. *Canadian Journal of Civil Engineering*. 51: 898-907.
- Koenig, T.A., Bruce, J.L., O'Connor, J., McGee, B.D., Holmes, Jr. R.R., Hollins, R., Forbes, B.T., Kohn, M.S., Schellekens, M.F., Martin, Z.W., and Pepler, M.C. (2016). *Identifying and preserving high-water mark data*, Book 3 - Applications of Hydraulics, Chapter 24 of Section A - Surface-Water Techniques (Techniques and Methods 3-A24). U.S. Department of the Interior: U.S. Geological Survey.
- Kousar, S., Khan, A.R., Hassan, M.U., Noreen, Z., Bhatti, S. (2020). Some best-fit probability distributions for at-side flood frequency analysis of the Ume River. *Journal of Flood Risk Management*, 13 (3), e12640
- Kovachis, N., Burrell, B. C., Huokuna, M., Beltaos, S., Turcotte, B., and Jasek, M. (2017). Ice-jam flood delineation: Challenges and research needs. *Canadian Water Resources Journal/Revue canadienne des ressources hydriques*, 42(3), 258-268.

- Kriwoken, L.A. and Brown, D.W. (1988). Using historical data to determine flood risk from ice jams. Proc., 5th Workshop on Hydraulics of River Ice/Ice Jams, Winnipeg, Manitoba, Canada, 97-113.
- Ladouceur, J-R., Morse, B., and Lindenschmidt, K.E. (2023). A comprehensive method to estimate flood levels of rivers subject to ice jams: A case study of the Chaudiere River, Quebec, Canada. *Hydrology Research*, 54(9): 995-1016.
- Lind, L., Alfredsen, K., Kuglerová, L., and Nilsson, C. (2016). Hydrological and thermal controls of ice formation in 25 boreal stream reaches. *Journal of Hydrology*, 540, 797-811.
- Lindenschmidt, K.E. (2020). *River ice processes and ice flood forecasting — a guide for practitioners and students*. Springer Nature, Switzerland AG.
- Lindenschmidt, K.E. (2023). Extension and refinement of a stochastic modelling approach to assess ice-jam flood hazard. *Hydrology Research*, 54(2): 149-164.
- Lindenschmidt, K.-E. (2024) *River Ice Processes and Ice Flood Forecasting—A Guide for Practitioners and Students*, 2nd ed.; Springer Nature, Switzerland AG: Cham, Switzerland.
- Lindenschmidt, K.E., Coles, A., and Saade, J. (2024). Reach-based extrapolation to assess the ice-jam flood hazard of an ungauged river reach along the Mackenzie River, Canada. *Water*, 16(11), 1535:
- Lindenschmidt, K.E., Coles, A., and Saade, J. (2024). Reach-based extrapolation to assess the ice-jam flood hazard of an ungauged river reach along the Mackenzie River, Canada. *Water*, 16(11), 1535:
- Lindenschmidt, K.E., Das, A., Rokaya, P., and Chu, T. (2016). Ice-jam flood risk assessment and mapping. *Hydrologic Processes*, 30: 3754-3769.
- Lindenschmidt, K.E., Das, A., Rokaya, P., Chun, K., and Chu, T. (2015). Ice jam flood hazard assessment and mapping of the Peace River at the Town of Peace River. *Proceedings of the 18th Workshop on the Hydraulics of Ice Covered Rivers*, Quebec City, Quebec, Canada.
- Lindenschmidt, K.E., Ghoreishi, M., and Khan, A. (2024, June, 9-12). Freeze up ice-jam flood hazard assessment and mapping. In the *International Symposium on Ice*. International Association for Hydro-Environment Engineering and Research (IAHR), Gdansk, Poland.

- Lindenschmidt, K.E., Huokuna, M., Burrell, B.C. and Beltaos, S. (2018). Lessons learned from past ice-jam floods concerning the challenges of flood mapping. *International Journal of River Basin Management*, 16(4): 457-468.
- Louie, P.Y.T., and Hogg, W.D. (1980). Extreme Value Estimates of Snowmelt. In *Canadian Hydrology Symposium: 80*. National Research Council Canada, Toronto, Ontario, Canada. pp. 64-78.
- Mackey et al. (1994).
- MacLaren Plansearch Inc. (1982). *Technical Brief on an SCS/HYMO Based Continuous Simulation and Real-Time Forecast Model QFORECAST*. Toronto, Ontario.
- MacLaren Plansearch Inc. (1984). *Show Hydrology Study, Phases I and II: Study Methodology and Single Event Simulation*. Elaborated for the Ontario Ministry of Natural Resources. Toronto, Ontario, Canada.
- Makkonen, L. (2006). Plotting positions in extreme value analysis. *Journal of Applied Meteorology and Climatology*, 45(2), 334-340.
- Maria, D., Sasmito, A., Sushama, L., and Khaliq, M.N. (2024). Non-stationary analysis of future floods using physical covariates and implications for dams across Canada. *Results in Engineering*. 24, 103645.
- Mason, A.H., Thomas, M.K., and Boyd, D.W. (1955). *The October 15-16, 1954 storm, "Hurricane Hazel" in Ontario* (Technical Circular No. CIR-2606 TEC-210) [Internal report]. Canada Department of Transportation: Meteorological Division. Ottawa, Ontario, Canada. 49 pp.
- McMullen, D. N. (1962). *Timmings Flood, August 31 – September 1, 1961: A Design Storm for Ontario* (Circular No. 3746, Tech. No. 428). Department of Transport: Meteorological Branch.
- McCuen, R. H., Johnson, P. A., and Ragan, R. M. (2002). *Highway Hydrology: Hydraulic Design Series Number 2*, Second Edition. US. Department of Transportation: Federal Highway Administration.
- Mesnard, F. and Sauvageot, H. (2003). Structural characteristics of rain fields. *Journal of Geophysical Research*, 108(D13), 4385, doi:10.1029/2002JD002808.

- Metcalfe, R. A., Chang, C., and Smakhtin, V. (2005). Tools to support the implementation of environmentally sustainable flow regimes at Ontario's waterpower facilities. *Canadian water resources journal*, 30(2), 97-110.
- Metcalfe, R.A., English, J., and Luce, J.J. (2018). *Variability and trends in seasonal snow cover in Ontario from 1980 to 2010 detected using remote sensing* (Climate Change Research Report CCRR-50). Ontario Ministry of Natural Resources: Science and Research Branch. Peterborough, Ontario. pp. 27.
- Metcalfe, R.A., Mackereth, R.W., Grantham, B., Jones, N., Pyrcce, R.S., Haxton, T., Luce, J.J., and Stainton, R. (2013). *Aquatic Ecosystem Assessments for Rivers*. Ministry of Natural Resources: Science and Research Branch. Peterborough, Ontario. pp. 210.
- Metcalfe, R.A., Smakhtin, V.Y., and Krezek, C. (2003). *Simulating and characterising natural flow regimes* (Waterpower Project Science Transfer Report 1.0). Ontario Ministry of Natural Resources. pp. 14.
- Millington, N., Das, S., and Simonovic, S.P. (2011). *The Comparison of GEV, Log-Pearson Type 3 and Gumbel Distributions in the Upper Thames River Watershed under Global Climate Models* (Water Resources Research Report. 40). The University of Western Ontario: Department of Civil and Environmental Engineering. London, Ontario, Canada.
- Ministry of the Environment, Conservation and Parks (MECP). (2003). *Stormwater Management Planning and Design Manual*. Ontario Ministry of the Environment.
- Ministry of Natural Resources (MNR). (1985). *Snow Surveying Manual – Standards and Procedures*. Ontario Ministry of Natural Resources: Conservation Authorities and Water Management Branch. Pp. 12.
- Ministry of Natural Resources (MNR). (1989a). *Great Lakes System Flood Levels and Water Related Hazards*. MNR: Conservation Authorities and Water Management Branch.
https://www.greysauble.on.ca/wp-content/uploads/2020/05/Great_Lakes_System_and_Water_Related_HazardsMNR_1989.pdf.
- Ministry of Natural Resources (MNR). (1989b). *Snow Hydrology Guide*. Queen's Printer for Ontario.
- Ministry of Natural Resources (MNR). (2001). Technical Guide for Great Lakes – St. Lawrence River System. In MNR, *Great Lakes- St. Lawrence River System and*

large inland lakes. Technical guides for flooding, erosion and dynamic beaches, in support of natural hazards policies 3.1 of the provincial policy statement [CD-ROM]. Ontario Ministry of Natural Resources.

Ministry of Natural Resources (MNR). (2002). *Technical Guide – Rivers and Stream Systems: Flooding Hazard Limit*. Ontario Ministry of Natural Resources: Water Resources Section.

Ministry of Natural Resources (MNR). (2013). *Extreme Flow Statistics: Flood Flow and Low Flow Statistics for the Southwestern Hudson Bay and Nelson River Watersheds*. Ontario Ministry of Natural Resources: Corporate Management and Information Division: Spatial Data Infrastructure. <https://www.publicdocs.mnr.gov.on.ca/mirb/Extreme%20Flow%20Analysis%20-%20Technical%20Release.pdf> .

Ministry of Natural Resources (MNR). (2014). *Flood Flow Statistics for the Great Lakes Watershed System*. Ontario Ministry of Natural Resources: Corporate Management and Information Division: Spatial Data Infrastructure. <https://www.publicdocs.mnr.gov.on.ca/mirb/Flood%20Flow%20Statistics%20-%20User%20Guide.pdf> .

Ministry of Natural Resources (MNR). (2019). *Ontario Snow Surveying Manual – Addendum: Frequently Asked Questions relating to the 1985 Snow Surveying Manual*. Ontario Ministry of Natural Resources: Watershed Hydrology Lab, Aquatic Research and Monitoring Section. pp. 19.

Ministry of Natural Resources (MNR). (2020). *User Guide for Ontario Flow Assessment Tool (OFAT)*. MNR/MNR: Corporate Management and Information Division: Provincial Mapping Unit. <https://www.sdc.gov.on.ca/sites/MNR/MNR-PublicDocs/EN/CMID/OFAT%20-%20User%20Guide%20-%20eng.pdf>.

Ministry of Natural Resources (MNR). (2023). *Technical Bulletin - Delineating Flooding Hazards: Data, Survey and Mapping Specifications (Version 1.0)*. Ontario Ministry of Natural Resources.

Ministry of Northern Development and Mining. (1997).

Ministry of Transportation Ontario (MTO). (1986). *MTC Drainage Manual Chapter H: Design Flood Estimation for Medium and Large Watersheds*. Ontario Ministry of Transportation and Communications: Drainage and Hydrology Section. Downsview, Ontario.

- Ministry of Transportation Ontario (MTO). (1996). *Modified Index Flood Method*. Ontario Ministry of Transportation.
- Ministry of Transportation Ontario (MTO). (1997). *MTO Drainage Management Manual*. Ontario Ministry of Transportation: Quality and Standards Division, and Quality and Standards Division.
- Ministry of Transportation Ontario (MTO). (2016a). *Ontario IDF Curve Lookup* (Version 3.0). Ontario Ministry of Transportation: IDF Curve Lookup. <https://idfcurves.mto.gov.on.ca/terms.shtml>
- Ministry of Transportation Ontario (MTO). (2016b). Provincial Engineering Memorandum: Implementing the New Unified Ontario Flood Method for Calculating the Design Flow Rates for Water Crossings (DCSO #2016-03). Ontario Ministry of Transportation: Memorandum Design and Contract Standards Office, Highway Standards Branch.
- Mohamoud, Y.M. (2002). Prediction of daily flow duration curves and streamflow for ungauged catchments using regional flow duration curves. *Hydrological Sciences Journal*, 53(4). 706-724.
- Mohanty, M.P., Nithya, S., Nair, A.S., Indu, J., Ghosh, S., Bhatt C.M., Rao, G.S., and Karmakar, S. (2020). Sensitivity of various topographic data in flood management: Implications on inundation mapping over large data-scarce regions. *Journal of Hydrology*, 590: 125523.
- Moin, S.M.A., and Shaw, M.A. (1985). *Regional Flood Frequency Analysis for Ontario Streams: Volume 1, Single Station Analysis and Index Method*. Developed for Environment Canada and the Ontario Ministry of Natural Resources as part of the Canada/Ontario Flood Damage Reduction Program. Environment Canada: Inland Waters Directorate. Burlington, Ontario. Canada.
- Moin, S.M.A., and Shaw, M.A. (1986). *Regional Flood Frequency Analysis for Ontario Streams: Volume 2, Multiple Regression Method*. Developed for Environment Canada and the Ontario Ministry of Natural Resources as part of the Canada/Ontario Flood Damage Reduction Program. Environment Canada: Inland Waters Directorate, Burlington, Ontario. Canada.
- Mondal, K., Bandyopadhyay, S., and Karmakar, S. (2023). Framework for global sensitivity analysis in a complex 1D-2D coupled hydrodynamic model: Highlighting

- its importance on management over large data-scarce regions. *Journal of Environmental Management*, 332: 117312.
- Moore, S. (2019). *Hydrometric Field Manual: Measurement of Stage* (qSOP-NA008-02-2019). Environment and Climate Change Canada: Water Survey of Canada.
- Morgan, J. L., Gergel, S. E., and Coops, N. C. (2010). Aerial photography: a rapidly evolving tool for ecological management. *BioScience*, 60(1), 47-59.
- Musa, Z. N., Popescu, I., and Mynett, A. (2015). Sensitivity analysis of the 2D SOBEK hydrodynamic model of the Niger River. In *Proceedings of the 36th IAHR World Congress* (Vol. 28), The Hague, the Netherlands.
- Myers, V.A., Zehr, R.M., 1980. A Methodology for Point-to-Area Rainfall Frequency Ratios, NOAA Technical Report NWS 24, National Weather Service, Silver Spring Maryland.
- Nash, J.E. (1957). The Form of the Instantaneous Unit Hydrograph. *International Association of Scientific Hydrology*, 45(3), 114-121.
- Natural Resources Canada (NRCan). (2023). *Federal Hydrologic and Hydraulic Procedures for Flood Hazard Delineation* (Version 2.0. - General Information Product 113e). Government of Canada: Natural Resources Canada & Public Safety Canada.
- Natural Resources Conservation Service (NRCS). (2004). Chapter 10: Estimation of Direct Runoff from Storm Rainfall. *National Engineering Handbook: Part 630 – Hydrology*. United States Department of Agriculture.
- Natural Resources Conservation Service (NRCS). (2021). Chapter 2: Estimating Runoff Volume and Peak Discharge. *Part 650 Engineering Field Handbook – National Engineering Handbook* (2010-650-H, 2nd Ed.). United States Department of Agriculture.
- Neelz, S., and Pender, G. (2009). *Desktop review of 2D hydraulic modelling packages* (Science Report: SC080035). UK Environment Agency: Science Department.
- Nguyen, T.H., El Outayek, S., and Lim, S.H. (2017). A systematic approach to selecting the best probability models for annual maximum rainfalls – A case study using data in Ontario (Canada). *Journal of Hydrology*, 553, 49-58.

- Northwest Hydraulic Consultants Ltd. (2023). *Northwest Territories Ice Jam Flood Mapping Guidelines V1.0* (NHC Reference 1007306). Prepared for Natural Resources Canada and Government of Northwest Territories. Pp. 32.
- Paixao, E., Auld, H., Mirza, M. M. Q., Klaassen, J., and Shephard, M. W. (2011). Regionalization of heavy rainfall to improve climatic design values for infrastructure: case study in Southern Ontario, Canada. *Hydrological Sciences Journal*, 56(7), 1067-1089.
- Pappenberger, F., Beven, K.J., Ratto, M., and Matgen, P. (2008). Multi-method global sensitivity analysis of flood inundation models. *Advances in Water Resources*, 31(1): 1-14.
- Parnas, F. E., Abdalla, E. M., and Muthanna, T. M. (2021). Evaluating three commonly used infiltration methods for permeable surfaces in urban areas using the SWMM and STORM. *Hydrology Research*, 52(1), 160-175.
- Pathiraja, S., Westra, S., and Sharma, A.J.W.R.R. (2012). Why continuous simulation? The role of antecedent moisture in design flood estimation. *Water Resources Research*, 48(6).
- Pilgrim, D. H., and Cordery, I. (1975). Rainfall temporal patterns for design floods. *Journal of the Hydraulics Division*, 101(1), 81-95.
- Pinos, J. and Timbe, L. (2019). Performance assessment of two-dimensional hydraulic models for generation of flood inundation maps in mountain river basins. *Water Science and Engineering*, 12(1): 11-18.
- Planning Act*, R.S.O. 1990, CHAPTER P.13.
- Pool, S., Vis, M., and Seibert, J. (2021). Regionalization for ungauged catchments – lessons learned from a comparative large-sample study. *Water Resources Research*, 57(10), e2021WR030437.
- Prowse, T.D., Bonsal, B.R., Duguay, C.R., and Lacroix, M.P. (2007). River-ice break-up/freeze-up: a review of climatic drivers, historical trends and future predictions. *Annals of Glaciology*, 46, 443–451.
- Quick, M.C., and Pipes, A.U.B.C. (1977). UBC WATERSHED MODEL/Le modèle du bassin versant UCB. *Hydrological Sciences Journal*, 22(1), 153-161.

- Rahman, A.S., Rahman, A., Zaman, M.A., Haddad, K., Ahsan, A., and Imteaz, M. (2013). A study on selection of probabilistic distributions for at-site flood frequency analysis in Australia. *Natural Hazards*, 69(3), 1803–1813.
- Razavi, T., and Coulibaly, P. (2013). Classification of Ontario watersheds based on physical attributes and streamflow series. *Journal of Hydrology*, 493, 81-94.
- Reinders, J.B., and Munoz, S.E. (2024). Accounting for hydroclimatic properties in flood frequency analysis procedures. *Hydrology and Earth System Sciences*, 28: 217-227.
- Robson, A.J. and Reed, D.W. (2008). *Statistical Procedures for Flood Frequency Estimation*. Volume 3 of the Flood Estimation Handbook. Centre for Ecology and Hydrology. Wallingford, Oxfordshire, UK.
- Rokaya, P., Budhathoki, S., and Lindenschmidt, K.E. (2018a). Ice-jam flood research: A scoping review. *Natural Hazards*, 94(3): 1439-1457.
- Rokaya, P., Budhathoki, S., and Lindenschmidt, K.E. (2018b). Trends in the Timing and Magnitude of Ice-Jam Floods in Canada. *Scientific Reports*, 8(1): 5834.
- Rokaya, P., Lindenschmidt, K. E., Pietroniro, A., and Clark, M. (2022). Modelling of ice jam floods under past and future climates: A review. *Journal of Hydrology X*, 100120.
- Rossman, L., and W. Huber. (2015). *Storm Water Management Model Reference Manual Volume I – Hydrology* (EPA/600/R-15/162A). U.S. EPA, Office of Research and Development. Washington, DC, USA.
- Rowney, A.C., and Wisner, P.E. (1984). *QUALHYMO: Users Manual* (Release 1.0). University of Ottawa: Department of Civil Engineering. Ottawa, Ontario, Canada.
- Samuels, P.G. (1990, September 17-20). *Cross-section location in 1-D models* [Paper presentation]. International Conference on River and Flood Hydraulics at Hydraulics Research, Wallingford, U.K.
- Sangal, B.P. (1981). *A Practical Method of Estimating Peak from Mean Daily Flows with Application to Streams in Ontario* (NHRI Paper No. 16, IWD Technical Bulletin No. 122). National Hydrology Research Institute: Inland Waters Directorate. Ottawa, Ontario, Canada.

- Sangal, B.P., and Biswas, A.K. (1970). The 3-parameter lognormal distribution and its applications in hydrology. *Water Resources Research*, 6(2), 505-515.
- Sangal, B.P., and Kallio, R.W. (1977). *Magnitude and frequency of floods in southern Ontario* (Technical Bulletin Series No. 99). Fisheries and Environment Canada: Inland Waters Directorate, Water Planning and Management Branch.
- Sehgal, K. and Drake, J. (2016). *Unified Ontario Flood Method: Regional Flood Frequency Analysis of Ontario Streams Using Multiple Regression*. Prepared for the Ontario Ministry of Transportation. University of Toronto: Toronto, Ontario, Canada.
- Shao, Q., Dudda, D., Karim, F. and Petheram, C. (2018). A method for extending stage-discharge relationships using a hydrodynamic model and quantifying the associated uncertainty. *Journal of Hydrology*, 556: 154-172.
- Shen, H. T. (2010). Mathematical modeling of river ice processes. *Cold Regions Science and Technology*, 62(1), 3-13.
- Shu, C., and Ouarda, T. B. (2012). Improved methods for daily streamflow estimates at ungauged sites. *Water Resources Research*, 48(2).
- Sifalda, V. (1973). Development of a design rainfall for the calculation of channels. *GWF Wasser/Abwasser*, 114: 435–440.
- Sitterson, J., Knightes, C., Parmar, R., Wolfe, K., Muche, M., and Avant, B. (2017). *An Overview of Rainfall-Runoff Model Types* (EPA/600/R-17/482). US Environmental Protection Agency. Washington, DC, USA.
- Slater, L. J., Anderson, B., Buechel, M., Dadson, S., Han, S., Harrigan, S., Kelder, T., Kowal, K., Lees, T., Matthews, T., Murphy, C., and Wilby, R. L. (2021). Nonstationary weather and water extremes: a review of methods for their detection, attribution, and management. *Hydrology and Earth System Sciences*, 25: 3897–3935.
- Smakhtin V.U. (1999). Generation of natural daily flow time-series in regulated rivers research using a non-linear spatial interpolation technique. *Regulated Rivers Research and Management*, 15(4): 311-323.
- Smakhtin V.Y., Creuse-Naudin, E. and Hughes D.A. (1997). Regionalisation of daily flow characteristics in part of the Eastern Cape, South Africa. *Hydrological Sciences Journal*, 42: 919 - 936.

- Smakhtin, V.U., and Masse, B. (2000). Continuous daily hydrograph simulation using duration curves of a precipitation index. *Hydrological Processes*, 14(6): 1083-1100.
- Smith, M.J., Asal, F.F.F., and Priestnall, G. (2004). The use of photogrammetry and lidar for landscape roughness estimation in hydrodynamic studies. In *Proceedings of International Society for Photogrammetry and Remote Sensing XXth Congress* (pp. 12-23), Istanbul, Turkey.
- Smith, A. A., and Falcone, J. P. (1984). MIDUSS: A design program for storm water sewerage. *Canadian Journal of Civil Engineering*, 11(3), 530-541.
- Song, X., Zhang, J., Zhan, C., Xuan, Y., Ye, M., and Xu, C. (2015). Global sensitivity analysis in hydrological modeling: review of concepts, methods, theoretical framework, and applications. *Journal of Hydrology*, 523: 739-757.
- Stainton, R.T. (2010). *Evaluation of Hydrological Models in the Ontario Flow Assessment Techniques (OFAT) Software Tool: Assessment of Model Suitability for Ontario's Far North* (Report No. 0209). Prepared for, and with the collaborative support of, the Ontario Ministry of Natural Resources. Trent University: Institute for Watershed Science.
- Stanley, S.J. and Gerard, R. 1992. Probability analysis of historical ice jam flood data for a complex reach: a case study. *Canadian Journal of Civil Engineering*, 19: 875-885.
- Swain, R.E., England, J.F.Jr., Bullard, K.L., Raff, D.A. (2004). Hydrologic Hazard Curve Estimating Procedures. Dam Safety Research Program, Research Report DSO-04-08. U.S. Department of the Interior Bureau of Reclamation,. June 2004.
- Terzi, R.A. (1981). *Hydrometric Field Manual: Measurement of Streamflow*. Fisheries and Environment Canada: Inland Waters Directory, Water Resources Branch. Ottawa, Ontario, Canada.
- Tetford, P.E., and Desloges, J.R. (2023). Modelling flood frequency and magnitude in a glacially conditioned, heterogeneous landscape: testing the importance of land cover and land use. *Hydrology and Earth System Sciences*, 27(21), 3977-3998.
- Tiwari D, R. A. Metcalfe and J. J. Luce. 2025. State of Ontario's Biodiversity - Alterations to streamflow. Ontario Ministry of Natural Resources, Science and Research Branch, Peterborough, ON. Science and Research Technical Report TR-74. 36 p. + appendices

- Turcotte, B., Alfredsen, K., Beltaos, S., and Burrell, B.C. (2017, July 10th-12th,). *Ice-related floods and flood delineation along streams and small rivers*. In CGU HS Committee on River Ice Processes and the Environment, 19th Workshop on the Hydraulics of Ice Covered Rivers, Whitehorse, Yukon, Canada.
- Turcotte, B., Burrell, B. C., Beltaos, S., and She, Y. (2019, May). The impact of climate change on breakup ice jams in Canada: State of knowledge and research approaches. In *Proceedings of the 20th Workshop on the Hydraulics of Ice Covered Rivers*. Ottawa, Ontario, Canada.
- Turcotte, B., and Morse, B. (2016). A simple 1-D model to simulate frazil jam formation and mitigation strategies. In *Proceedings of the 23rd IAHR International Symposium on Ice*. Ann Arbor, Michigan, USA.
- Turcotte, B., Morse, B., Bergeron, N. E., and Roy, A. G. (2011). Sediment transport in ice-affected rivers. *Journal of hydrology*, 409(1-2), 561-577.
- U.S. Army Corps of Engineers (USACE). (1956). *Snow Hydrology, Summary Report of the Snow Investigations*. USACE: North Pacific Division. Portland, Oregon, USA.
- Turcotte, B. and Nafziger, J. (2021). Detailed interpretation of river ice processes in water level time series. 21st Workshop on the Hydraulics of Ice Covered Rivers, Saskatoon Saskatchewan, Canada, August 29-September 1. CGU HS Committee Workshop on River Ice Processes and the Environment
- Tuthill, A.M., J.L. Wuebben, S.F. Daly, White, K.D.. (1996). Probability distributions for peak stage on rivers affected by ice jams. *Journal of Cold Regions Engineering*, 10(1), 36-57.
- U.S. Army Corps of Engineers (USACE). (1977). *STORM: Storage, Treatment, Overflow Runoff Model, User's Manual* (CPD-7). USACE: Institute for Water Resources: Hydrologic Engineering Center. Davis, California, USA. [https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/S/TORM_UsersManual_\(CPD-7\).pdf](https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/S/TORM_UsersManual_(CPD-7).pdf).
- U.S. Army Corps Engineers (USACE). (2011). *Ice-Affected Stage Frequency. Engineering and Design* (Technical Letter No. 1110-2-576). Washington, D.C., USA. pp. 20.
- U.S. Army Corps of Engineers (USACE). (2023a). *HEC-RAS River Analysis System: Boundary Conditions*. (Version 6.3). USACE: Hydrologic Engineering Center.

- U.S. Army Corps of Engineers (USACE). (2023b). *HEC-RAS River Analysis System: HEC-RAS 2D User's Manual* (Version 6.3). USACE: Hydrologic Engineering Center.
- U.S. Army Corps of Engineers (USACE). (2023c). *Hydrological Modeling System: Modeling Rain-on-Snow Events*. USACE: Hydrologic Engineering Center.
- U.S. Department of Agriculture – Soil Conservation Service (USDA-SCS). (1972). Section 4: Hydrology. In *National Engineering Handbook* (NEH). Washington, D.C.: USDA Soil Conservation Service.
- U.S. Department of Agriculture – Natural Resources Conservation Service (USDA-NRCS). (2021). Chapter 2: Estimating Runoff Volume and Peak Discharge. In *Part 650 Engineering Field Handbook 2nd edition* (Title 2010-Engineering Handbooks: National Engineering Handbook). USDA-NRCS.
- Vansteenkiste, T., Tavakoli, M., Van Steenberg, N., De Smedt, F., Batelaan, O., Pereira, F., and Willems, P. (2014). Intercomparison of five lumped and distributed models for catchment runoff and extreme flow simulation. *Journal of Hydrology*, 511, 335-349.
- Wang, T., Hamann, A., Spittlehouse, D., and Carroll, C. (2016). Locally Downscaled and Spatially Customizable Climate Data for Historical and Future Periods for North America. *PLoS one*, 11(6), e0156720.
- Watt, W.E. (1994). *Development of Hydrology Method for Medium-Sized Watersheds in Northern Ontario*. Queens University: Department of Civil Engineering, Hydrologic Modelling Group. Kingston, Ontario, Canada.
- Watt, W.E. (Ed.). (1989). *Hydrology of Floods in Canada: A Guide to Planning and Design*. National Research Council Canada: Associate Committee on Hydrology.
- Watt, W.E., and Associates. (1979). *Calibration of the NWSRFS for Two Southern Ontario Basins*. Elaborates for the Ontario Ministry of Natural Resources, Toronto, Ontario, Canada.
- Watt, W.E., Chow, K.C.A., Hogg, W.D., and Lathem, K.W. (1986). A 1-h urban design storm for Canada. *Canadian Journal of Civil Engineering*, 13(3): 293–300.
- Wazney, L., and Carson, R. (2021, August 29-September 1). *The 2019 Freeze-up Ice Jam on the Severn River and Flooding of Bearskin Lake First Nation*. CGU HS

Committee on River Ice Processes and the Environment, 21st Workshop on the Hydraulics of Ice Covered Rivers, Saskatoon, Saskatchewan, Canada.

Wenzel, Jr.H.G. (1982). Chapter 2: Rainfall for Urban Stormwater Design. In D.F. Kibler (Ed.), *Urban Stormwater Hydrology* (Water Resources Monograph 7), pp. 35-67. American Geophysical Union, Washington, D.C., USA.

White, K., and Beltaos, S. (2008). Chapter 9: Development of ice-affected stage frequency curves. In S. Beltaos (Ed.), *River ice breakup*. Water Resources Publications, LLC, Highlands Ranch, Colorado, USA.

Whiteley, H.R., and Ghatge, S.R. (1977). GAWSER (Guelph Agricultural Watershed Storm Runoff) Model: Users Manual (Tech. Rep. No. 126-37). University of Guelph: School of Engineering. Guelph, Ontario, Canada. pp 91.

Williams, B., Das, A., Luo, B., and Lindenschmidt, K.E. (2021). An ice jam flood hazard assessment of a lowland river and its terminus delta. *Natural Hazards*, 105(3), 2799-2817.

Williams, J.R., and Hann, R.W. (1973). *HYMO, A Problem-Oriented Computer Language for Hydrologic Modelling, User's Manual* (ARS-S-9). USDA: Agricultural Research Service: Southern Region.

Wisner, P.E., and P'ng, E.E. (1984). *OTTHYMO: A Model for Master Drainage Plans* (Part III), IMPSWM Urban Drainage Modelling Procedures (2nd edition). University of Ottawa: Department of Civil Engineering. Ottawa, Ontario, Canada.

Wong, J.C.W. (2021a, August 29-September 1). *An attempt to Forecast the Mobilization of a Grounded Ice-Jam*. In CGU HS Committee on River Ice Processes and the Environment, 21st Workshop on the Hydraulics of Ice Covered Rivers, Saskatoon, Saskatchewan, Canada. pp12.

Wong, J.C.W. (2021b, August 29-September 1, 2021). *Forecasting and Management of Ice Break-Ups with the Inclusion of Snowmelt in the Degree Day Method and the Use of HEC Products*. In CGU HS Committee on River Ice Processes and the Environment, 21st Workshop on the Hydraulics of Ice Covered Rivers, Saskatoon, Saskatchewan, Canada. pp 26.

World Meteorological Organization (WMO). (1973). *Manual for Estimation of Probable Maximum Precipitation* (Operational Hydrology Report No. 1, WMO – No.332). Secretariat of the World Meteorological Organization. Geneva, Switzerland.

- World Meteorological Organization (WMO). (2020). *Guide to Hydrological Practice Volume 1: Hydrology – From Measurement to Hydrological Information*, 2008 edition updated in 2020 (WMO No. 168). WMO.
- Yen, B. C. and Chow, V. T. (1968). *A study of surface runoff due to moving rainstorms: A contribution to the International Hydrological Decade* (Hydraulic Engineering Series No. 17). University of Illinois: Civil Engineering Studies. Urbana, Illinois, USA.
- Yen, B. C., and Chow, V. T. (1980). Design hyetographs for small drainage structures. *Journal of the Hydraulics Division*, 106(6), 1055-1076.
- Yue, S., and Wang, C. (2004). Determination of regional probability distributions of Canadian flood flows using L-moments. *Journal of Hydrology (New Zealand)*, 59-73.
- Zaghloul, M., Papalexiou, S.M., Elshorbagy, A., and Coulibaly, P. (2020). Revisiting flood peak distributions: A pan-Canadian investigation. *Advances in Water Resources*, 145, 103720.
- Zhang, Z., and Stradnyk, T.A. (2020). Investigation of attributes for identifying homogeneous flood regions for regional flood frequency analysis in Canada. *Water*, 12(9): 2570.
- Zhang, Z.; Stadnyk, T.A.; and Burn, D.H. (2020) Identification of a preferred statistical distribution for at-site flood frequency analysis in Canada. *Canadian Water Resources Journal*, 45(1), 43–58.
- Zufelt, J.E., and Ettema, R. (1997). *Unsteady Ice Jam Processes* (CRREL Report 97-7). USACR: Cold Regions Research and Engineering Laboratory.

9 Glossary

In compliance with the commitments established in the Flooding Strategy, MNR's is updating its existing series of natural hazard technical guides. In this context, and in order to avoid duplication or repetition of the terminology as much as possible, this glossary focuses mainly on terms that have not been defined in the PPS, and/or defines them emphasising a hydrological perspective.

Annual Exceedance Probability (AEP): Refers to the probability, of a flood (or storm) event of a specific magnitude occurring in any given year expressed as percentage values. Flood frequency analysis provides information regarding the magnitude and frequency of flood events, based on an analysis of historical data (e.g., water level, streamflow data). Statistical analyses are carried out on observed records with the aim of assessing future probabilities of exceedance, where the probability of flooding for a given magnitude and its frequency of recurrence or return period is provided. For example, a large flood calculated to have 1% AEP is a flood that has 1% chance of occurring in any one year and is also referred to as the 100-year (1:100) flood. In this sense, a 20% AEP flood event is equivalent to a 1 in 5-year flood event, and a 10% AEP flood is the same as a 1 in 10-year (1:10) flood.

Base map: A foundational, georeferenced map (often digital, such as raster tiles or vector layers) that provides essential spatial context for water-related data. It is used as an underlying map with geographical features(e.g., rivers, lakes, contours, relief, roads, streets, municipal boundaries) which are essential to display, analyze, and delineate flood hazard information and conduct hydrological modeling..

Calibration: The systematic process of adjusting the appropriate coefficients and parameters so that the model will replicate an observed flood event (e.g., water levels, flow volumes, timing) as closely as possible, aimed at reducing uncertainty and ensuring accuracy in the representation.

Catchment: Also known as a drainage area, drainage basin or watershed. It is the area of land defined by terrain slopes that collect, drain and direct all precipitation and surface runoff, channeling it to a single, low-level point, such as a river, lake, wetland or reservoir. It includes the upstream drainage area of the main waterway as well as any tributary streams.

Coupled 1D-2D model: A hydraulic simulated approach that combines one-dimensional (1D) and two-dimensional (2D) modelling methods in which the channel and structures are represented in a 1D domain, while flood plain flow is modelled in a 2D domain, and the two components are dynamically linked.

Cross-Section: A survey string or vertical profile of a channel and flood plain elevations that is taken perpendicular (i.e., right angle) to the main flow direction in a river.

Design Storm: A hypothetical rainfall event defined by specific characteristics (e.g., duration, intensity, temporal distribution) used to estimate peak runoff and volume for designing infrastructure like sewers, culverts, and dams. It represents a specific return frequency (e.g., 100-year or 1% AEP, 25-year or 4% AEP design storm) to assess flood risk.

Drainage Area: See **Catchment**.

Drainage Basin: See **Catchment**.

Encroachment: Any activity, structure (e.g., fences, sheds, retaining walls, pools) or construction within the floodway of the floodplain, including fill material, new construction, substantial improvements, or other development that alters the natural topography in ways that disrupt the capacity of the floodway or floodplain to carry floodwaters. Encroachment has the potential to result in increases in upstream flood levels, increase in downstream flows, increase in downstream velocities, and change in the timing of flows.

Fetch: The uninterrupted, horizontal length of water surface exposed to wind. It is a critical factor in wave generation (along with wind speed and duration) as longer fetch distances allow wind to transfer more energy, resulting in larger, more powerful waves.

Flood Fringe Area: the outer portion of the floodplain, situated between the high-hazard floodway and the flooding hazard limit. This outer less hazardous part of the floodplain is where flood depths, flow velocities, or wave energies are relatively low and less severe than those experienced in the floodway. Some types of development and site alteration could be permitted within the flood fringe, provided that such development is compatible with the flood hazard and subject to appropriate floodproofing measures are kept for the designated flooding hazard elevation or another flooding standard approved by the Ministry of Natural Resources.

Flood Hazard Management: a comprehensive, multi-layered approach of corrective and preventative measures aimed towards minimizing the loss of life, property damage, and

social disruption caused by riverine flooding. It combines, but not limited to, prevention (e.g., land-use planning and regulations), emergency response measures (e.g., emergency preparedness plans, forecasting and early warning systems), and protection (e.g., structural flood-control works), to manage riverine flood risk.

Flood Hazard Mapping: relies on hydrologic and hydraulic modeling to map the extent, depth, and velocity of water along rivers, streams or shorelines, and to define, delineate areas susceptible to inundation during specific regulatory flood events (e.g., 100-year or 1% AEP flood). They are often used as regulatory maps for: land use planning, riverine flood hazard management, and flood mitigation.

Flooding Standard: is the maximum flood level used to define the maximum flooding hazard limit of a floodplain along rivers and stream systems, and is applied in land-use planning, to regulate development, and to protect against riverine flood risk. Regulatory floods are used to calculate the flood standard, which is typically greater than 100-year (1% AEP) storm or a specific regional storm event (e.g., Hurricane Hazel or Timmins flood events).

Floodway (for river and stream systems): is composed by the channel of a watercourse (river or stream) and the interior portion of the floodplain where flood depths and velocities are highest and most destructive. It represents the area required for the safe passage of regulatory flood flows and is the high-risk inner portion of the floodplain where development and site alteration would cause a danger to public health and safety or property damage. Exceptions could be made for uses which by their nature must be located within the floodway, such as flood and/or erosion control works, or where appropriate, minor additions or passive, non-structural uses which do not affect flood flows.

The 'two zone' concept is applied in Ontario:

Where the 'one zone' concept is applied, the floodway is the entire contiguous flood plain.

Where the 'two zone' concept is applied, the floodway is the contiguous inner portion of the flood plain, representing that area required for the safe passage of flood flow and/or that area where flood depths and/or velocities are considered to be such that they pose a potential threat to life and/or property damage. Where the 'two zone' concept applies, the outer portion of the flood plain is called the flood fringe.

Natural Hazard: A naturally occurring physical process such as flooding, erosion and slope failure that pose a potential threat to life, property and/or infrastructure, and may cause social and economic disruption, or environmental degradation.

Light Detection and Ranging (LiDAR): An instrument that measures distance to a reflecting object by emitting timed pulses of light and measuring the time difference between emissions of a laser pulse and the reception of the pulse's reflection(s). The measured time interval for each reflection is converted to distance, which when combined with the position and altitude information from GPS, IMU, and the instrument itself, allows the derivation of the 3D-point location of the reflecting target's location.

One-Dimensional (1D) Model: A computational approach that simulates water flow along a single axis (longitudinal direction), along the centreline of the channel. It represents rivers, channels and floodplains as a linear series of cross-section-averaged values for water surface elevation, velocity and flow rate. It is commonly applied for floodplain mapping and stormwater management.

One Hundred Year Precipitation: it is a statistical term representing an extreme, rare, and high-intensity precipitation event that has a 1% probability (AEP) of being met or exceeded in any given year, designed for flood risk management, stormwater planning, and infrastructure design. It is often referred to as a 100-year storm and is commonly misunderstood as only occurring once in 100 years. However, experiencing a 100-year storm does not reduce the probability of another 100-year storm occurring again within a short period of time; it is possible for a 100-year storm to occur multiple years in a row. Normally, the data is based on extrapolation of recorded events, as none of the Ontario rainfall stations have 100 years of record.

Risk: The combination of the likelihood or probability of a specific, dangerous hydrological event occurring and the adverse consequences (potential impact) of the event on human life, property, infrastructure and the environment.

Significant: Important in terms of amount, content, representation, or effect.

Thalweg: Line connecting the deepest points of successive cross-sections along a stream channel.

Topography: The arrangement of natural and artificial surface features of the land, including elevation, terrain, slope, and landforms that dictate how water moves across the landscape. It is a critical driver of surface runoff, infiltration rates, and the delineation of watershed boundaries.

Two-Dimensional (2D) Model: A hydraulic model that calculates water flow direction, velocity and depth in both the horizontal (x and y) directions across a defined spatial grid or mesh. They are used in complex areas where water spreads out, such as floodplains, urban environments, and at the confluence of rivers

Watershed: See **Catchment**.