Ontario

Technical Guide

River & Stream Systems: Flooding Hazard Limit

Ontario Ministry of Natural Resources
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A. Preface

A new efficient and effective land use planning process was introduced in Ontario to replace the complex planning system used in the past and to ensure the environmental, economic and social well-being of the people of Ontario.

The 1997 Provincial Policy Statement issued under the authority of Section 3 of the Planning Act provides policy direction on matters of Provincial interests related to land use planning and development. According to the Planning Act, planning authorities "shall have regard to" policy statements issued under the Act. One important principle behind the Policy statement to achieve long-term economic prosperity, environmental health and social well-being is the reduction of public cost and risk to Ontario's residents by directing development away from areas where there is risk to public health or safety or risk of property damage. Accordingly, the Province of Ontario adopted policies under Public Health and Safety policy areas addressing natural and human-made hazards. Part of the new approach empowers municipalities in protecting the environment and streamlining an ecosystem based planning process.

Flood plain studies form an important part of watershed management and planning. Proper planning requires the balancing of a wide range of public and private interests. Ecosystem based planning within a watershed or a sub-watershed, involves an up-front evaluation of numerous, and often competing, land use and natural resources interests. It provides a means of examining and developing planning strategies that balance local as well as community-based needs. Through ecosystem based planning processes, environmental concerns such as flood and erosion hazard lands can be identified and incorporated into the land use planing process.

The Ministry of Natural Resources (MNR) through its refocussed water management program will ensure that the program is effectively aligned with the Ministry's vision of sustainable development and its mission to achieve ecological sustainability. MNR objectives to achieve ecological sustainability include the continuation of Provincial programs for the protection of life and property from flooding and other water-related hazards, and the provision of direction and input through policy, information and science support.

The Provincial government's role in the planning and management of flood risk areas is to protect society, including all levels of government, from being forced to bear unreasonable social and economic burdens due to unwise individual choices.

The Province sets minimum standards to ensure that these risks and costs to society are reduced. There are instances where local conditions dictate that the minimum standards may not be sufficient and a higher standard may be more appropriate. The Province has empowered municipalities to assume responsibilities for the management of flood risk areas, the associated liability and the risk relative to planning for new land uses in and around these areas. The municipality making these decisions should ensure that flood plain management studies are undertaken by a Professional Engineer and resulting flood lines, flood proofing standards are examined and approved by a Professional Engineer using accepted engineering principles.

The management of flood susceptible lands involves a combination of three main program components:

i) Prevention, by land use planning and regulation of development;
ii) Protection, by applying structural and non-structural measures and acquisition; and
iii) Emergency response, by flood forecasting/warning and flood/erosion disaster relief.

Over the long-term prevention is the preferred method for the management of flood plain lands.

The Ministry of Natural Resources, as a participant in the Province of Ontario's planning reform initiatives, is responsible for providing policy leadership and directions specific to natural hazards, including flood hazards along river and stream systems within the Province. This Rivers and Streams Technical Guide: Flooding Hazard Limit document has been prepared to assist in the understanding of the latest Provincial Policy Statement and to describe approaches which have been determined to be consistent with the new policies. The enclosed document is based on the 1996 Provincial Policy Statement and it updates the original 1986 Flood Plain Management in Ontario, Technical Guideline publication. It incorporates the previously released Provincial Flood Plain Planning Policy Statement Implementation Guidelines, (1988), new technology and findings of additional studies in the field of flood plain investigations and court decisions.

This guide serves in an advisory role and is not intended to introduce any changes from the Policy Statement. Instead, it should be read in conjunction with the Provincial Policy Statement as well as other flood related implementation guides.

In preparing the Rivers and Streams Technical Guide, the Ministry intends to document standardized approaches to manage flood susceptible lands across the Province. Designers and review agency staff will find the Guide helpful in their work as it is based on a standard methodology. The material presented will go a long way to avoid and/or reduce the duplication in flood plain calculations and, as a result, should represent a cost saving for flood plain management projects. Also, the Guide will assist in the approval process and in explaining, or if necessary defending, the methodology when challenged.

The past practice of rapid conveyance of floodwaters away from urban and rural areas has affected our natural watercourses. River valleys and stream corridors are constantly adjusting to natural and human induced processes. In addition to providing hydraulic conveyance to flows, these corridors perform other hydrological functions, such as the discharge and recharge of groundwater, and the erosion, transport and deposition of sediment. Channels lined with hard surfaces have been constructed to carry the floodwaters, replacing the natural streams and valleys. These changes in the river systems have resulted in a loss of fishery and wildlife habitat, and deterioration of water quality and aesthetic features associated with natural streams. The recent changes in Provincial policies and raised environmental concerns have resulted in changes to our approach to watershed management, and have forced us to regard channels as important multi-functional links within our ecosystem.

This document describes, in general terms, an important component of watershed management; it presents the hydrologic and hydraulic work needed to conduct flood plain analyses. It is not intended to be a list of mandatory instructions on technical methodologies to be rigidly applied in all circumstances, rather, it serves to assist technical staff experienced in water resources in the selection of the most appropriate computational method and flexible implementation measures, provided the decisions made are consistent with the latest Provincial Policy Statement. The Guide cannot replace good engineering and environmental judgement in adopting the most appropriate procedures required to achieve the amount of detail and effort involved, and in determining the practical degree of accuracy achievable when adopting a flood related study program.

The following seven steps outline the proposed flood plain study tasks to be followed, indicating the relevant Technical Guide chapters which provide more detailed descriptions on the recommended procedures.

Step 1.Select flood plain standard (Chapter B)

- Identify study area on Figure B-1 to determine Zone and corresponding flood standard
- Select flood standard from: Historical Storm (Hazel, Timmins), 100 year flood, or a historical storm observed in the area provided it exceeds the 100 year flood

Step 2.Review data requirements, methods of hydrologic and hydraulic calculations (Chapter C)

- Data requirement: streamflows, 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, (see Chapters D and E)
- Hydraulic modelling, type of flow, cross-section data, roughness, bridge and culvert losses, plotting (see Chapters F and G)
- Select mapping (for detailed specification see Chapter J)

Step 3.Select hydrologic modelling parameters (Chapter D)

- Select rainfall input to modelling of flood standard: Hazel, Timmins, 100 year storm depth, duration, distribution, snowmelt
- Select soil data
- Select land use

Step 4.Select methods of computing flows (Chapter E)

- Hydrologic models: single and continuous models
- Computational procedures: snowmelts, infiltration, soil moisture account, base flow, watershed routing
- Recommended model selection
- Model calibration

Step 5.Select method of computing water levels for open water conditions, (Chapter F)

- Recommended models
- Flood routing
- Reservoir routing
- Effect of lakes and reservoirs
- Waterway crossings and encroachments
- Model calibration and sensitivity

Step 6.Compute ice jam levels, where appropriate, (Chapter G)

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

Step 7.Prepare technical report, (Chapter H)

The last two chapters describe:

- **Implementation and agency roles (Chapter I)** and
- **Survey and mapping specification, (Chapter J)**.

The six Appendices which provide information are listed below:

Appendix 1	Bibliography
Appendix 2	Glossary
Appendix 3	Application to change the flood standard
Appendix 4	Application of the two zone concept
Appendix 5	Special Policy Areas, factors to be considered
Appendix 6	**Floodproofing**.

1. BACKGROUND

The Provincial government's role in the planning and management of flood risk areas is to protect society, including all levels of government, from being forced to bear unreasonable social and economic burdens due to unwise individual choices. The management of flood susceptible lands involves the combination of three main program components: i) prevention, by land use planning and regulation of development, ii) protection, by applying structural and non-structural measures, and acquisition, and iii) emergency response, by flood forecasting/warning and flood/erosion disaster relief. Over the long term, prevention is the preferred method for the management of flood plain lands.

The Provincial Policy Statement (May 1996) issued under the authority of the Planning Act provides policy direction on matters of provincial interest related to land use planning and development. According to the Provincial Policy Statement, the Province's long-term economic prosperity, environmental health and social well-being depend on reducing the potential for public cost and risk to Ontario's residents by directing development away from areas where there is a risk to public health and safety or a risk of property damage. The policy on public health and safety states that development will generally be diverted to areas outside of hazardous lands adjacent to river and stream systems which are impacted by flooding and/or erosion hazards.

The guiding Principles behind the Provincial Policy Statement are as follows:

- Proper flood plain management requires flood/erosion hazards to be simultaneously recognized and addressed in a manner that is integrated with land use planning and maintains environmental ecosystem integrity;
- Effective flood plain management can only occur on a watershed basis with due consideration given to development effects and associated environmental impacts;
- Local conditions vary along the flood plain and, accordingly, must be taken into account in the planning and managing of flood plains;
- New development which is susceptible to flood/erosion hazards or which will cause or aggravate flood/erosion hazards to existing and approved uses, or which will cause adverse environmental impacts must not be permitted to occur unless the flood/erosion hazards and environmental impacts have been addressed; and
- Flood plain management and land use planning are distinct yet related activities that require overall co-ordination on the part of municipalities, Conservation Authorities, the Ministry of Natural Resources and Ministry of Municipal Affairs and Housing.

The following sections describe the flood plain policies and standards to be used for delineating flood hazard lands across the Province.

2. PROVINCIAL NATURAL HAZARD POLICIES

2.1 Natural Hazard Policies

The Provincial Policy Statement on Public Health and Safety describes the Natural Hazards policies (Provincial Policy Statement, Section 3.1) relating to flood plains as follows:

Policy 3.1.1 Development will generally be directed to areas outside of:

a) hazardous lands adjacent to shorelines of the Great lakes - St. Lawrence River System and large inland lakes which are impacted by flooding, erosion, and/or dynamic beach hazards;
b) hazardous lands adjacent to river and stream systems which are impacted by flooding and/or erosion hazards; and
c) hazardous sites

Policy 3.1.2 Development and site alterations will not be permitted within:

a) defined portions of dynamic beach;
b) defined portions of the one hundred year flood level along connecting channels (the St. Mary's, St. Clair, Detroit, Niagara and St. Lawrence Rivers); and
c) defined floodways (except in those exceptional situations where a Special Policy Area has been approved).

Policy 3.1.3 Except as provided in policy 3.1.2, development and site alteration may be permitted in hazardous lands and hazardous sites, provided that all of the following can be achieved:

a) the hazards can be safely addressed, and the development and site alteration is carried out in accordance with established standards and procedures;
b) new hazards are not created and existing hazards are not aggravated;
c) no adverse environmental impacts will result;
d) vehicles and people have a way of safely entering and exiting the area during times of flooding, erosion and other emergencies; and
e) the development does not include institutional uses, essential emergency services of the disposal, manufacture, treatment or storage of hazardous substances.

2.2 Provincial Interests - Flooding Hazards

The overall interests and expectations of the Province of Ontario in flood plain management along river and stream systems are described here:

- All land use planning and resource management bodies within the Province will have regard to the implications of their actions respecting the creation of new or aggravation of existing flood plain management problems;

- Municipalities and planning boards will recognize the flood and erosion susceptibility and environmental integrity of flood plains at the various stages of the land use planning process for which they have jurisdiction; and

- Municipalities and planning boards will direct new development to areas outside of the flood and erosion hazard areas.

2.3 Flood Standards for River Systems

Flooding hazards means the inundation, under the conditions specified below, of areas adjacent to a river system (that are not ordinarily covered by water). The flooding hazard limit is the greater of:

i) the flood resulting from a 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;

ii) the one hundred year flood; or

iii) a flood which is greater than i) or ii) which was actually experienced on a particular watershed or portion thereof, for example as a result of ice jams and which has been approved as the standard for that specific area by the Minister of Natural Resources; and

The exception is where the use of the 100 year flood or actually experienced event as the flood standard for a specific watershed, even though it does not exceed the Hazel or Timmins event, has been approved by the Minister of Natural Resources, (where past history of flooding supports the lowering of the standard).

For those watersheds with a flood standard greater than the minimum acceptable for the area (see Figure B-1), the option exists for municipalities and planning boards to apply to the Minister of Natural Resources, in accordance with procedures established, to change the standard, subject to the following overriding conditions.

(a) Changes to the existing flood standard will only be considered with the support of a significant majority of municipalities and/or planning boards within the watershed, in consultation with the local Conservation Authority or Ministry of Natural Resources, where Conservation Authorities do not exist; and

(b) The lowering of the existing flood standard where the past history of flooding reveals that a higher level is more appropriate will not be considered.

The method of applying to change the flood plain standards are described in Appendix 3.

Where flooding is experienced in excess of the existing flood standard, the Minister of Natural Resources may require the flood standard to be modified to reflect the observed flood event. The 100 year flood is the minimum acceptable flood standard for flood plain delineation.

2.4 Flood Hazard Limits for Lakes < 100 sq. km.

The basic principles of defining flood standards along lakes are similar to the principles adopted for rivers. However, there are additional factors, such as the effect of wind, which have to be considered when selecting standards for Lake Levels.

Smaller lakes, less than 100 km^2 in size may or may not respond to a single runoff event. Similarly riverine lakes and large rivers (i.e., Ottawa River) which, due to their size and fetch (>3 km), may be subject to wind set up, generating water levels higher than the flood standard selected for the river.

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

2.5 Official Plan Flooding Hazard Limit Policies

Municipalities and planning boards should show or describe flood plain lands in their official plans and incorporate policies to address new development consistent with the policy statement.

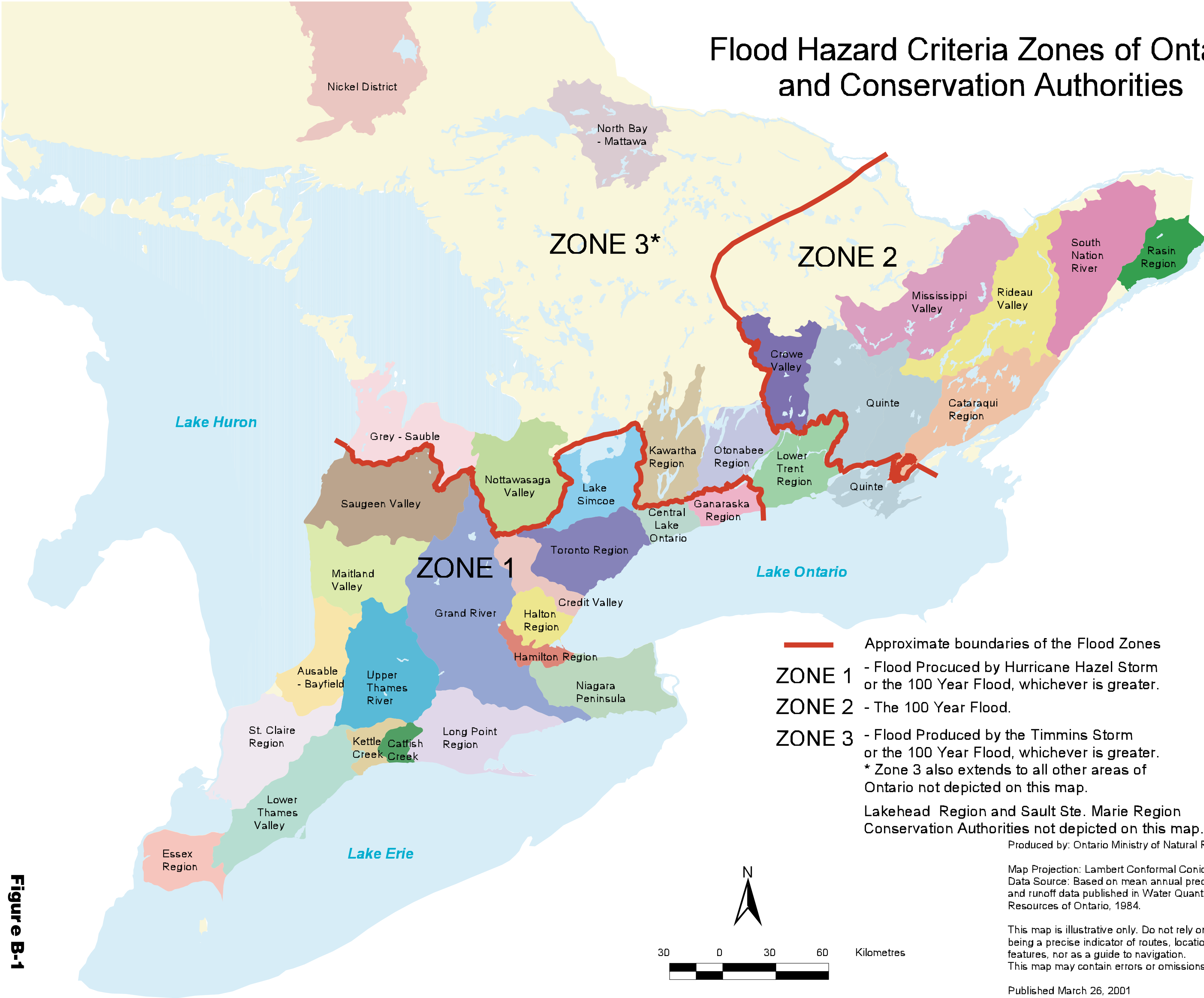
Municipalities and planning boards, in consultation with the local Conservation Authority or Ministry of Natural Resources, where no Conservation Authority exists, should include in their official plans:

(a) policies where by uses permitted in flood plains are cognizant of flood susceptibility and flood risk;

(b) policies whereby no new buildings or structures are permitted which are susceptible to flood related damages or will cause adverse impacts to existing upstream or downstream development or lands;

(c) policies addressing additions or alterations to existing buildings or structures and replacement of buildings or structures located in flood plains; and

Flood Hazard Criteria Zones of Ontario and Conservation Authorities



Approximate boundaries of the Flood Zones

ZONE 1 - Flood Procuced by Hurricane Hazel Storm or the 100 Year Flood, whichever is greater.

ZONE 2 - The 100 Year Flood.

ZONE 3 - Flood Produced by the Timmins Storm or the 100 Year Flood, whichever is greater.
* Zone 3 also extends to all other areas of Ontario not depicted on this map.

Lakehead Region and Sault Ste. Marie Region Conservation Authorities not depicted on this map.

Produced by: Ontario Ministry of Natural Resources

Map Projection: Lambert Conformal Conic
Data Source: Based on mean annual precipitation and runoff data published in Water Quantity Resources of Ontario, 1984.

This map is illustrative only. Do not rely on it as being a precise indicator of routes, locations of features, nor as a guide to navigation.
This map may contain errors or omissions.

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Figure B-1

(d) policies addressing such public and private works that must 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 flood plain policies of the Province. Where no official plan exists, the zoning document affecting the area should contain provisions to reflect this policy statement.

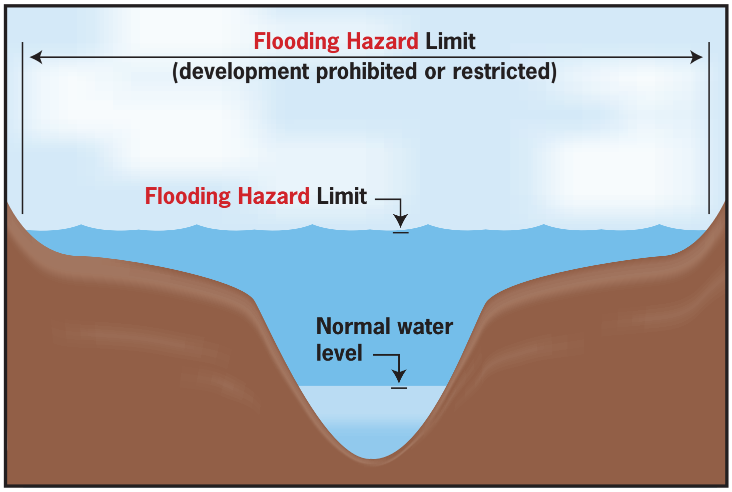
2.6 One Zone Concept

Generally, the flood plain will consist of one zone, defined by the selected flood standard (see Figure B-1). New development in the flood plain is to be prohibited or restricted. Where the one zone concept is applied:

i) Municipalities and planning boards should include policies in their official plans that explain the intent of the one zone concept;

ii) The flood plain should be appropriately zoned in conformity with the official plan designation to reflect its prohibitive or restrictive use; and

iii) The entire flood plain should be treated as the floodway.



(NOT TO SCALE)

Figure B-2 - One Zone Floodplain Concept

2.7 Two Zone Concept.

The two zone concept recognizes the fact that the flood plain can often be divided into two zones: the floodway, where the majority of the flow is conveyed and flood fringes which exist on both sides of the floodway.

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

Floodway for a river system means the portion of the flood plain 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 flows may be permitted. New development in the floodway is to be prohibited or restricted.

For outer portions of flood plains that could potentially be safely developed with no adverse impacts, the Conservation Authorities in Ontario, or where no Conservation Authorities exist, the Ministry of Natural Resources, in co-operation with the watershed municipalities, have the option of selective application of the two zone (floodway - flood fringe) concept (see Figure B-2).

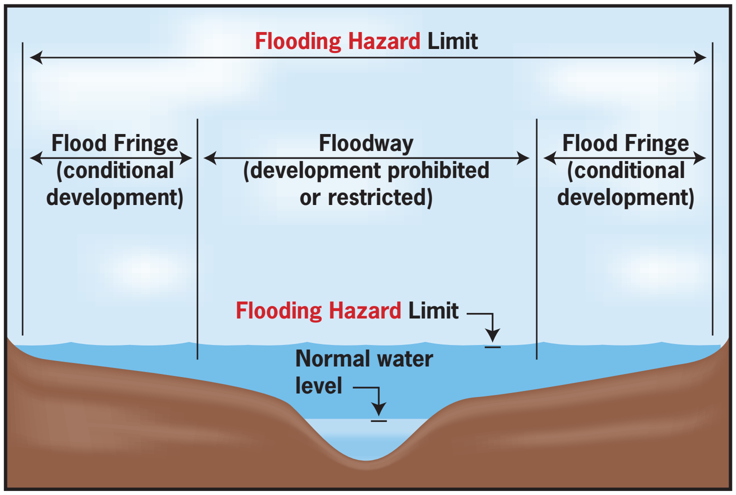


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Figure B-3 - Two Zone Floodway - Floodfringe concept

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 or downstream impacts. Generally, flow depth in excess of 1 m and/or flow velocities above 1 m/s can create significant hazards for developments.
The two zone approach adopted in 1982 used the magnitude of flood as the design criteria to identify the floodway and flood fringe areas. The Floodway was based on the 100 year flood, while the flood fringe was based on the Hazel or Timmins flood. The benefits of this approach was that the 100 year flood represents a sufficiently extreme event to identify a portion of the river that carries the majority of the flow, and it is relatively easy to identify the limits of the floodway. However, in some instances it resulted in a narrow strip of flood fringe land, and it did not reflect the actual risks involved in filling the flood fringe areas.

The present practice, introduced in 1988 for the selection of the floodway and flood fringe areas, replaced the rigid 100 year criterion with a more flexible approach based on the critical flood depth and velocity values. It recognizes the fact that the impact of encroachment caused by filling and/or development in a flood fringe area can result in:

- increases in upstream flood levels
- increases in downstream flows
- increases in downstream velocities
- change in the timing of flows.

This present approach appears to be more equitable than the rigid flood frequency criterion, however, it is vulnerable to political interference. Instead of a frequency criterion, a hydraulic criterion is introduced with consideration to the upstream and downstream impacts on the existing and proposed developments.

A rigorous approach based on the present hydraulic criterion to identify the floodway and flood fringe boundary was developed by Moin and Shaw (1989) using the DWOPER model. Multiple regression equations are used to undertake a sensitivity analysis to tests the changes in flood levels, flows and velocities caused by various degrees of flood fringe encroachment. The method is based on equations which relate the topographic features and degree of encroachment to the hydraulic changes in the flood plain. Generally, five different levels of encroachment are modelled for at least three different flows ranging from a 25 year to the Hazel or Timmins floods. The final selection is based on the proposed development scenario with the least upstream and downstream impacts. New development that may be permitted in the flood fringe should be protected to the level of the flood 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 floodway.

Where the two-zone concept is applied, the flood fringe should be zoned in conformity with the 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.

The factors to be considered and the application procedures for a two zone designation are presented in Appendix 4.

2.8 Special Policy Area Concept

Special Policy area means an area within a community that has historically existed in the flood plain where site specific policies apply, approved by the Ministries of Natural Resources and Municipal Affairs and Housing, which are intended to address the significant social and economic hardships to the community that would result from strict adherence to provincial policies concerning development.

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

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 and the application procedures for a Special Policy area designation are presented in Appendix 5.

2.9 Access and Floodproofing

Any new development permitted in the flood plain, in accordance with the policy statement, should be protected by acceptable floodproofing actions or measures. However, the new development does 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/egress for new buildings be such that vehicular and pedestrian movement is not prevented during times of flooding. The potential for risk to life along the flood hazard lands are the highest during storm events. Access involving both ingress and egress should be addressed prior to approval developments near flood hazard lands. Ingress and egress to and from the development should be such that movement of people and vehicles are not be prevented during high floods. 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.

A more detailed description on floodproofing methods are presented in Appendix 6.

2.10 Public Safety

Notwithstanding the above flood plain policies, new development will not be permitted to locate in the flood plain where the use is:

- associated with the manufacture, storage, disposal and/or consumption of hazardous substances or the treatment, collection and disposal of sewage, which would pose an unacceptable threat to public safety if they were to escape their normal containment/use as a result of flooding or failure of floodproofing measures;
- associated with institutional services, such as hospitals, nursing homes and schools, which would pose a significant threat to the safety of the inhabitants (i.e., the sick, the elderly, the disabled or the young), if involved in an emergency evacuation situation as a result of flooding or failure of floodproofing measures; and
- associated with services such as those provided by fire, police and ambulance stations and electrical sub-stations, which would be impaired during a flood emergency as a result of flooding or failure of floodproofing measures.

Where new development identified in Public Safety Policy is not considered to pose an unacceptable risk to public safety, a higher level of flood protection and/or additional floodproofing precautions above the flood standard level, may still be required due to the sensitive nature of the development.

3. DEFINITIONS OF THE FLOODING HAZARD STANDARDS

3.1 Introduction

The group of flood standards referred to in the Natural Hazard Policy is the basis by which flood plains are delineated. It is designed to accomplish the main objectives of flood plain management: to prevent loss of life and to minimize property damage and social disruption.

There are three types of flood events used in defining the flood standard: i) Synthetic storms developed from the two large historical events (Hurricane Hazel and Timmins storm), ii) Observed and documented historical events (if larger than the 100 year event), and iii) Statistically derived 100 year events.

The magnitude of the flood which defines the flood plain limits in a particular area of the Province is largely dependent upon the susceptibility of that area to tropical or thunderstorms, rainfall, snowfall or a combination of these meteorological events. Figure B-1 shows the areas of the Province, subdivided into three zones.

3.2 Selecting the Flood Standards

The applicable flood standards for each of the three zones are described below.

Zone 1

For all watersheds within Zone 1 the flooding hazard limit is the greater of:

i) the flood resulting from a rainfall actually experienced during Hurricane Hazel storm (1954), transposed and centred over the watershed and combined with the local conditions;

ii) the one hundred year flood; or

iii) a flood which is greater than i) or ii) which was actually experienced on a particular watershed or portion thereof, for example as a result of ice jams and which has been approved as the standard for that specific area by the Minister of Natural Resources.

Alternatively, if approved by the Minister of Natural Resources, the 100 year flood level could replace the Hazel criterion, if no observed floods (see iii) exceeded the 100 year flood level in the watershed.

Zone 2

For all watersheds within Zone 2 the flood standard is the 100 year flood level if there are no records of observed and documented flood levels exceeding this criterion within the same watershed.

However, where recorded and documented flood levels are found in the same watershed within Zone 2 which exceeded the computed 100 year flood levels, the use of the 100 year criterion should be reviewed. As a guide, it is suggested that if the observed event is at least 0.1 m higher than the computed 100 year event, and the watershed characteristics have not changed since the historical observation, then the historical event should be considered for the flood plain standard.

Zone 3

For all watersheds within Zone 3 the flooding hazard limit is the greater of:

i) the flood resulting from a rainfall actually experienced during Timmins storm (1961), transposed and centred over the watershed and combined with the local conditions;

ii) the one hundred year flood; or

iii) a flood which is greater than i) or ii) which was actually experienced on a particular watershed or portion thereof, for example as a result of ice jams and which has been approved as the standard for that specific area by the Minister of Natural Resources.

Hazel and Timmins events have been extensively analyzed and the hourly distribution and areal reduction factors for different drainage areas have been developed for each storm (Chapter D). The resulting storms can be transposed within their respective zones to the watershed in question.

3.3 Historical Storms

The extreme total rainfall produced by Hurricane Hazel in 1954 in Southern Ontario, which is used as a flood standard, is not the highest observed event on record; the Hazel event was exceeded by the Harrow (Essex) storm in July 1989. Over a small area of 10 km^2, the centre of the storm produced 450 mm of rain in 30 hours, which is far in excess of the 285 mm rain produced by the 48 hour Hazel storm. Despite this, the Harrow storm resulted in no fatalities and it created only relatively small amount of flood damages compared to Hazel. For storm areas 1,000 km^2 or larger, the Hazel storm still dominates. Therefore, although the extreme Harrow storm is not used as a flood plain standard in the Essex county area, developers and designers of hydraulic structures should be cognizant of the potential effects if such a storm would occur in the Essex region.

Information on observed floods can come from surveyed high water marks, reliable reports, photographs or data publications. The event could be caused by rainfall, with or without snowmelt, which could be extended for the entire watershed. While the transposition of the Hazel and Timmins storms within their zones has been found to be valid, the transposition of observed historical events may not be valid. Therefore, a historical event should be used within the watershed where the observation has taken place. Where evidence suggests that the flood event could have potentially occurred over an adjacent watershed, the observed event can be transposed to the adjacent watershed. If the event was caused by an ice jam, the event cannot be extended within or transposed to an adjacent watershed. The investigation of an observed historical event requires a careful assessment before it can replace the 100 year criterion. When considering the use of a past historic event as a flood standard, it is important to consider the changes in watershed characteristics which took place since the observed event. A calibrated hydrological computer model should be used to estimate the flood events for watershed and land use characteristics selected for the flood plain mapping study.

For very small areas (generally <1 km^2) usually forming part of the headwaters of a stream characterized, due to short times of concentration, the 100 year flood event can exceed the Hazel flood. In such a case, the 100 year standard should be used for defining the flood plain for the headwaters. Consequently, for large watersheds more than one flood standard may apply; 100 year flood for the small headwaters area and the Hazel or Timmins storm for the rest of the watershed. However, there are also large watersheds where the 100 year flood is greater than the Timmins flood.

TABLE B-1
DESIGN RETURN PERIOD IN YEARS (T)
FOR DIFFERENT DESIGN LIFE AND RISK VALUES

Risk (r)	Design Life in Years (L)	
	50	100
1%	4977	9953
10%	475	950
22%	200	400
25%	174	348
39%	100	200
50%	73	145
64%	50	100

Figure B-1
Examples of risk value and corresponding design criterion for the two most frequently used design life categories are presented in this table.

Applying the Flood Standards

When applying flood standards, the Flooding Hazard Limit (or the “Regulatory Flood Line”) is the **greater of** the regional storm, the 100-year, a documented maximum observed flood event including ice jams. Flood elevations greater than those generated by the noted standardsmay be approved by the Minister of Natural Resources on an individual basis. Reductions in the flood standards may also be approved by the Minister after receiving documented support of the majority of municipalities in the watershed. Appendix 3, Application to Change the Flood Standard Within A Watershed outlines the process for approvals.

3.4 Return Period Floods.

Hazel and Timmins storms represent extreme storm events within their respective regions. A comparison of these extreme events with other observed historical rainfall values showed that neither storm fitted the historical rainfall data distribution. Hence no statistical analysis could be carried out to establish the return periods for Hazel or Timmins. The only conclusion derived from the analysis is the fact that each of the two storms was in excess of the computed 100-year storm. When calculating risk of flood damages based on Hazel and Timmins storms, the designer should assume a return period in excess of 100-year, (i.e., 200 years or more).

The use of predicted return period storm and return period flood require careful interpretation. For example, a 100-year flood does not mean that flood conditions will occur only once every 100 years, but that flood conditions will occur on average once every 100 years, and that during any one year, there is a 1% probability of occurrence. Although the return period has been used widely as a risk criterion, it has grave disadvantages unless it is translated in terms of other criteria. Given a particular return period, it is not at all clear what risk is being undertaken in a specified engineering operation.

In order to provide the complete picture, three components are required to describe the design criterion:

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

These three are related by the equation:

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

The table clearly demonstrates the magnitude of risk that the 100 year flood could be exceeded within a 50 or 100 year design life of a project. A 100 year design return period would provide a 39% risk of being equalled or exceeded for an expected project life of 50 years, and a 64% risk for an expected project life of 100 years.

Even a 1000 year return period flood has a 5% risk of being equalled or exceeded in a 50 year period. Risk in practice cannot be eliminated, it can only be reduced to an acceptable level.

4. SPECIAL FLOOD HAZARD CONDITIONS

4.1 Flood Hazard Standards Downstream of a Control Structure

4.1.1 Dams

Dams and dykes can reduce flood risk downstream or behind a dyke, but they do not eliminate the risk. The purpose of a dam or a dyke is to protect existing development, but not to free up additional land and allow for new development.

A number of flow management approaches concerning dams and downstream flood hazards are available: 1) Use reduced regulated flow, 2) Use unregulated flow and 3) Use flow resulting from failure.

Reduced peak flows based on the operation of the dam is not always in the public interest, since funds to maintain and replace the structure in the future cannot be assured. Also, projected flood peak attenuation may not be achieved as a result of ice, debris or sediment accumulation that affect storage, operating problems that alter discharge capacity, or floods that vary from the design event in terms of timing, volume and hydrograph shape. The use of peak flows resulting from a dam failure is the most conservative option, and the recommended option where public safety is the issue. The preferred approach is the use of unregulated flow to identify flood hazard limits downstream of a dam.

Various types and designs of flood control dams exist. Some are passive in that there is no means of controlling discharge (i.e., earthen dams); some have basic means of control such as stop logs, and while others have electrically operated flood gates with full-time operators. Many dams are considered multi-purpose; providing recreational use, irrigation or flow augmentation in addition to flood control. Flood control dams are normally designed to control to the flood standard, although others control to higher levels (i.e., maximum probable).

It must be remembered that the function of a flood control dam is to hold back upstream flood waters. Whatever the type or design of dam, these structures are not a floodproofing option for downstream developments. Through construction, design and operational errors, and deterioration of the structure, dams cannot be completely relied upon and new construction in flood hazard areas should not be permitted through reliance on control works. Similarly, stormwater management facilities should not be relied upon in the establishment of flood hazard limits. It is suggested that dam break analysis be undertaken, specifically downstream of major impoundments/dykes to determine the flood hazard.

The local Conservation Authority, or the Ministry of Natural Resources where no Conservation Authority exists, will have to be consulted as to their treatment of flood control dams within their overall approach to flood plain management.

For additional details concerning dam management in the Ontario, please consult the *Lakes and Rivers Improvement Act.*

4.1.2 Dykes and Flood Walls

Where a dyke has been properly designed and constructed to the flood standard, and a suitable maintenance program is in place, the area behind the dyke can be considered as flood fringe. As such, new development would still be required to be floodproofed to the flood standard. The floodway would be considered to be contained within the dyke area. If new development in the flood fringe cannot be floodproofed to the flood standard, then special policy area status may be requested, subject to the appropriate requirements.

As a precaution, certain areas immediately behind a dyke may be considered too hazardous for any use or certain types of uses if failure of the dyke was ever to occur. Also, the area immediately behind the dyke may be required for maintenance purposes.

The establishment of no development or limited development zones behind a dyke will be dependent on local conditions (i.e., flood depth and velocity) and local approaches to flood plain management. Construction of these flood control structures may result in an increase in flood levels at the site and along downstream reaches of the river. Dykes and flood walls protect existing areas located behind, but do not provide additional flood benefits.

Dykes and flood walls are not regarded as permanent flood control structures and the land behind the dykes and flood walls should continue to require protection to the revised (increased) flood standard.

4.2 Bridges and Culverts -

Bridges and culverts are primarily designed based on economic consideration. Roadway crossings are not intended to act as dams although the design often has to accommodate temporary ponding behind the structure. This could increase the flood plain limits upstream and reduce the flood peaks; hence, the flood hazard downstream may be reduced to some extent. When the structure is enlarged or removed, the temporary backwater ponding is reduced or eliminated, thereby potentially changing both the upstream and downstream flood lines. It is recommended, that the upstream flood line should make allowance for the backwater effects caused by the structure. Where this assumption results in unacceptable conditions the culvert should be replaced, or alternatively, where feasible, the two zone concept should be introduced. Under the two zone concept minor filling would be permitted in shallow areas, provided the filling would create no adverse flooding or environmental impacts upstream or downstream.

Downstream of the culvert or bridge, the natural flood line should be used for delineating the flood hazard, making no allowance for the temporary upstream ponding.

4.3 Confluence of Lakes, Rivers and Streams

In rivers flowing into large lakes (Figure B-4), 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 flood level in the river and/or stream and the flood hazard limit in the connecting channel, (See The Great Lakes - St. Lawrence River System and Large Inland Lakes Technical Guide.)

ii) the flood hazard limit (Hurricane Hazel, Timmins Storms, observed or the 100 year event) in the mean monthly levels in the connecting channel or lake.

Where the 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 km^2) 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 flood standard at the junction should be based on flood standard of the lake and river.

4.4 Confluence of 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 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.

4.5 Floodproofing of Buildings

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, Design of the Building Code, 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 (three stories high or less). This section details waterproofing, surface and subsurface drainage.

4.6 Stormwater Management Ponds

Stormwater management facilities may not be used to provide any reduction in flood flows.

5. DESIGN STANDARDS FOR DAMS

Floods and flood levels described in the previous section are used to define flood plain limits and to protect new developments along rivers and lakes.

The design criteria for sizing water control structures may differ from the flood plain standard. The factors which most influence the selection of the design criterion are the risks of loss of life and of property damage, the consequences of using lower criteria and economic considerations.

Dams and Reservoirs are frequently used to retard and store flood runoff during high flows, in order to protect the downstream environment. The Ministry of Natural Resources requires Ontario dams to safely accommodate the minimum inflows shown on Table B-2. The magnitude of design flood depends on the size of the structure, storage volume and the hazard caused by a potential dam failure due to floods in excess of design capacity (see Table B-2). The top of dam elevation must include a freeboard above the computed design flood elevation.

TABLE B-2 CLASSIFICATION CRITERIA AND INFLOW DESIGN FLOODS FOR DAMS

HAZARD POTENTIAL	SIZE OF DAM: SMALL Height: < 7.5m or < 25 ft. Storage: < 100 x 10^3 cu.m or < 80 ac-ft	INTERMEDIATE Height: 7.5 to 15m or 25 to 50 ft. Storage: 100 x 10^3 to 1000 x 10^3 cu.m or 80 to 800 ac-ft	LARGE Height: > 15m or > 50 ft. Storage: > 1000 x 10^3 cu.m or > 800 ac-ft
Damage to dam only	25-year return period flood to 50-year return period flood	50-year return period flood to 100-year return period flood	100-year return period flood to Regional Flood**
LOW **Property Damage:** Minimal to agriculture,other dams or structures not for human habitation. None to residential, commercial, industrial or land to be developed within 20 years. **Loss of Life:** None	25-year return period flood to 50-year return period flood	100-year return period flood to Regional Flood**	Regional Flood** to Probable Maximum Flood
SIGNIFICANT **Property Damage:** Appreciable to agriculture , operations other dams or residential, commercial, industrial development or land to be developed within 20 years.* **Loss of Life:** None expected	100-year return period flood to Regional Flood**	Regional Flood** to Probable Maximum Flood	Probable Maximum Flood
HIGH **Property Damage:** Extensive to agricultural operations, other dams or residential, commercial, or industrial development **Loss of Life:** One or more	Regional Flood** to Probable Maximum Flood	Probable Maximum Flood	Probable Maximum Flood

* where any land to be affected is developed for residential, commercial or industrial use or is to be developed within 20 years use Regional Flood or greater

** Regional Flood: (see section 2.3 Flood Standards for River Systems and Figure B-1 Flood Hazard Criteria Zones of Ontario

Table B-2 taken from Guidelines and Criteria for Location and Plans and Specifications Approvals, Lakes and Rivers Improvement Act, 1977, Ontario Ministry of Natural Resources

6. DESIGN STANDARDS FOR STRUCTURES

Waterway openings for culverts, bridge crossings and other drainage facilities for provincial highways should be designed in accordance with the pertinent policies and guidelines established by the Ministry of Transportation. Commonly used flood frequencies for sizing bridges, culverts, storm drainage systems and related stream channels are shown in Table B-3. Municipal and private stream crossings should also be designed to similar criteria.

TABLE B-3
DESIGN FLOOD FOR ROAD CROSSINGS

Road Classification	DESIGN FLOODS	
	Total span up to 6.0 m.	Total span over 6.0 m.
Freeway, Urban Arterial	50 Year	100 Year
Rural Arterial Collector Road Local (paved)	25 Year	50 year
Local (unpaved) Resource Access Road	10 Year	25 year
Temporary Detours	1 to 5 year	1 to 10 year

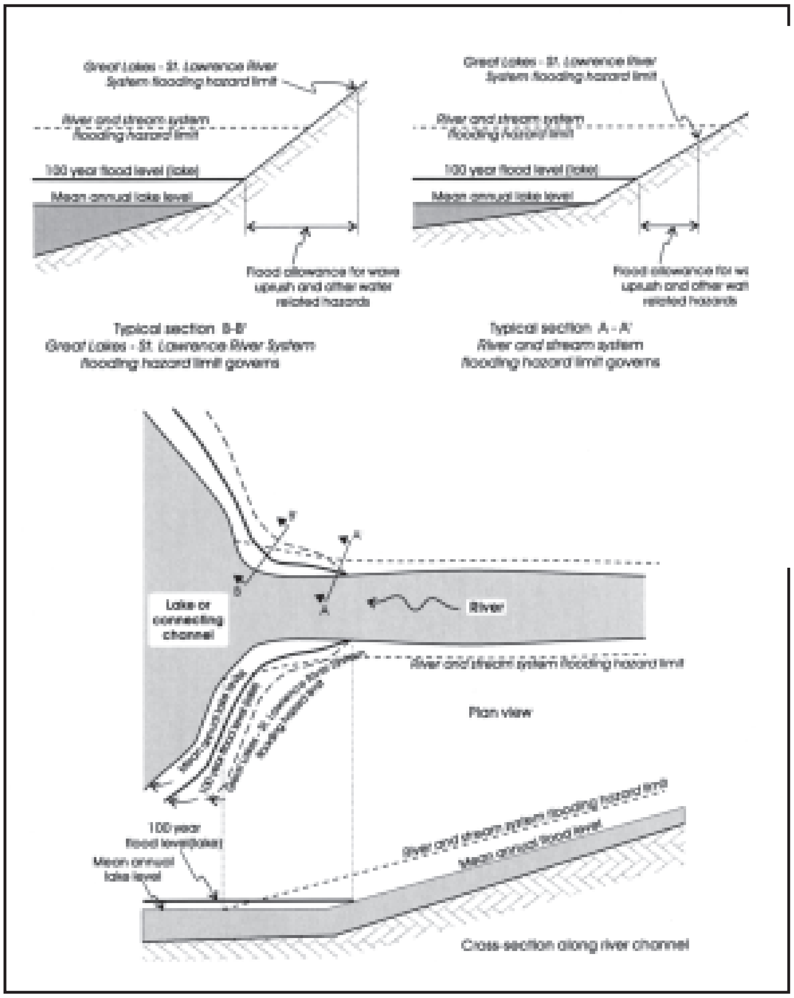


Figure B-4 Selection of flood standards at Confluence of Lakes, Rivers and Streams

C - HYDROLOGIC AND HYDRAULIC PROCEDURES

1. FLOODS

1.1 General

The criteria to be used to define limits of flood hazards across Ontario are described in Chapter B. This Chapter describes the general hydrologic and hydraulic procedures recommended for flood plain delineation in Ontario. A brief description is provided on data requirements, flood flow calculations and water level analysis. Basically, the criteria refer to floods based on probability (100 year) or on a specified meteorological event. Detailed descriptions on specific methods of computing flood flows and water levels in Ontario are given in Chapters D to G.

1.2 Floods Based on Exceedance Probability of Occurrence

A flood based on probability must be determined by a frequency analysis of recorded flood peaks 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.

1.3 Flood Produced by a Specified Meteorological Event

This type of flood is that produced by determining either the effects of a large storm over the basin or the runoff from a specified combination of snowmelt and precipitation. A specific probability is not attached to such a flood; for example, the peak flow resulting from the 100 year storm is not necessarily the 100 year flood. Whether or not the 100 year storm and the resulting flood have the same exceedance probability of occurrence depends on the antecedent moisture condition of a watershed when the storm event occurs.

Examples of floods produced by a specific meteorological event are Hurricane Hazel and the Timmins storm.

1.4 Observed Floods

Observed floods greater than the 100 year flood level may be used to define the flood risk area. Observed floods may be the result of precipitation, snowmelt, ice jams, other such causes, or any combination thereof. Observed floods must meet acceptable documentation standards as outlined in Section B.

Observed floods are useful in illustrating to the general public the severity and extent of the flood selected for the flood plain definition. The limits of the observed flood should be shown on the mapping for comparison, even if they are not the selected flood.

2. DATA REQUIRED

To determine the peak flows of the 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 flood to be mapped, the methodology to be used and the location. Much of the information will be common to all cases.

2.1 Streamflow Records

The first and most important data required are any streamflow records that may exist for the watercourse in question and any tributaries involved. 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 (i.e. MNR, MOE, etc.), conservation authorities and power 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, data should be retrieved directly from the latest version of HYDAT CD-ROM rather than from annual publications.

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

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.

2.2 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 an historical flood from information on high water marks. Such information may be found in newspaper files and public archives.

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 flood plain definition.

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

2.3 Specified Meteorological Event

In Zones 1 and 3, as defined in the Provincial Policy Statement, the flood standards are based on specified meteorological events. The rainfall volumes and their time distributions for the Hurricane Hazel and the Timmins regional storms are defined in Chapter B.

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

2.4 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 of course 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 is usually done for gauging stations maintained by Environment Canada staff of the appropriate District Office.

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 normally occur in the spring, this would entail taking measurements over a period of several months to establish a reasonable range of the rating curve. Streamflow measurements should be as accurate as possible; this can be achieved by following the procedures described in the "Hydrometric Field Manual - Measurement of River Discharge", published by the Water Resources Branch, Canada Department of the Environment.

2.5 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 takes into account 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.

2.6 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.

The best way to obtain this 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 Environment Canada, and, if available, this information should be collected.

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.

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.

2.7 Aerial Photographs

In several cases where large floods have occurred in the last few years, 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, the information can serve as a check on water levels, roughness coefficients, etc. 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.

2.8 Cross-Section

In order 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 flood plain which is accomplished by surveying cross-sections at various locations. Cross-sections can also be obtained from 1:2000 or larger scale topographic maps, by photogrammetric methods, and from digital terrain models where available.

Cross-sections are required at all representative locations throughout the channel reach. Such locations are where changes occur in slope, cross-sectional area or channel roughness; locations where levees begin and end; and at bridges and other channel restrictions. Where an abrupt change occurs, several cross-sections should be used to describe the change regardless of the distance between them.

It is impossible to specify the interval at which cross-sections should be surveyed, but two points should be kept in mind. First, sufficient sections should be obtained to adequately define the river geometry and second, the interval between them should be such that the assumption of gradually varied flow within a section should be reasonably valid.

Surveyed cross-sections must include the entire flood plain of the main channel and any tributaries that are likely to experience backwater effects. Sufficient points should be established to accurately define the geometry of the cross-sections and they must be tied in horizontally to permanent structures and vertically to established bench marks.

2.9 Regulated Flows

If the stream under consideration is subject to a significant artificial regulation by dams, diversions, etc., that have significant effects on peak flows, it is necessary to obtain data on the effect of such regulation to enable a conversion of streamflows to natural conditions prior to undertaking a flood frequency analysis.

2.10 Meteorologic and Physiographic Data

These types of data will be necessary for two areas of study, regional flood frequency analysis and hydrologic modelling.

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

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, as well as mean annual runoff, precipitation and snowfall. Other variables may also be of significance depending upon the region, its topography and climate.

For hydrologic modelling purposes, many of the above variables are used as well as information on soil types, forest cover, groundwater, land use, infiltration rates and soil moisture conditions. To operate a model, a great deal of meteorologic data is required including rainfall, temperature and snowfall records, radiation data, snowmelt coefficients and lapse rates. 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 2.5 above.

Other methods under development 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. Current research to improve these models and increased radar and meteorological data availability will eventually provide accurate, year-round flow estimates which in some instances could replace hydrometric data.

The use of remote sensing to obtain hydrologic or water level information also presents potential opportunities. Satellites which monitor 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 the satellite information will only be available at daily intervals at best, this method would be more applicable for large and remote watersheds. At present the level of accuracy and the cost prohibits the use of remote sensing data to estimate flows for flood plain mapping purposes.

2.11 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 flood plain.

3. FLOOD MAGNITUDES

The two main steps in the mapping of a flood plain are: (1) to determine the flood criteria and the corresponding flood flow; and (2) to delineate the area inundated by the flood flow. Whether the selected flood is based on a flood frequency analysis or the resultant runoff of a specified meteorologic input, 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 when legally challenged.

3.1 Flood Frequency Analysis

In some cases, the floods to be mapped will be determined by frequency analysis. While it may be difficult to specify some of the aspects of flood frequency analysis, there are some rules that must be followed.

(a) Conversion of Regulated Flows to Natural 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 determine the influence of regulation on the streamflows. If necessary, the conversion of regulated streamflows to natural conditions is achieved by removing the effect of dams and diversions.

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 are the most likely to affect peak flows, but 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.

If regulations were significant and their effects have not been removed, the results from single station flood flow analysis are applicable only to the watercourse between the control structure and the next major confluence with other tributaries. They should not be included in regional flood flow frequency analysis.

(b) Non-Stationary Record

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 the flood regime. Forestry operations, urbanization, agricultural drainage and irrigation can have a considerable effect on streamflows. If this effect is significant, it is necessary to assess the changes that have occurred with time in order to develop a stationary record for analysis.

(c) Extension of Streamflow Records

It has been the practice in the past, 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.

(d) Single Station Flood Frequency Analysis

A single station flood frequency analysis for the stream in question will be adequate only if the record is long and reliable. If the record is not of sufficient length to calculate an extreme event, such as a 100 year flood, (i.e., 30 to 40 years) or there is some doubt of its reliability, a regional flood frequency analysis should be carried out, in which case several single station analyses are combined.

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 is not available, mean daily values have to 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 would lead to an erroneous discharge value.

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 include: (1) Extreme Value Distribution (Gumbel 1); (2) Lognormal Distribution; (3) Three-Parameter Lognormal Distribution; and (4) Log Pearson Type 3 Distribution. The computer program Consolidated Frequency Analysis CFA version 3.1 developed by Environment Canada is a user friendly interactive flood frequency analysis program which is capable of analyzing streamflow data using all the probability distributions mentioned above.

The preferred method of estimating distribution parameters is that of maximum likelihood, since minimum variance estimates are obtained. If no maximum likelihood solution can be found, 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 significance tests available, but with the small sample sizes usually found in hydrology, they are of little value. Although graphical estimates should not be used alone, it is always worthwhile to plot the data and the computed frequency curve to give a visual indication to confirm it is the best fit. 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, the confidence limits.

Documented computer programs are available for statistical computation. These and other programs 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 Ontario, the preferred distribution is found in previous studies to be the three-parameter lognormal. When the skew of an annual flood series is negative, the 3-parameter lognormal has no solution. In this case and for regional flood frequency analysis, the skew is assumed to be zero, ie., the 2-parameter lognormal is used instead, as discussed in Subsection (e) below.

Another factor which should be considered is the possibility of including an 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 taken into account 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. At the present time, this type of analysis can only be carried out using the Extreme Value Distribution with parameters estimated by maximum likelihood. A documented computer program of this analysis is available from the Engineering Division of the Water Planning and Management Branch, Environment Canada. An admittedly limited amount of testing shows that, although the inclusion of an 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.

As was stated earlier, it is usually good practice to plot the observed peaks at empirical plotting positions and the computed frequency curve on the relevant probability paper 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 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.

(e) 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 a hydrologically similar region. Recently, OMNR updated the Regional Flood Frequency Analysis, with data up to 1997. A program that will calculate the 1:100 yr to 1:20 yr flows for unguaged watersheds has been produced and is available from the Water Resources Section, MNR.

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 nor data for the stream, the only solution is a regional analysis; it should also be used when a stream is gauged at some distance from the reach to be mapped.

The first problem in carrying out a regional analysis is to determine the extent of the homogeneous region. This must be based initially on knowledge of meteorological and physiographic conditions so that all streams used would appear to have similar runoff characteristics. For each stream in the region with sufficient length of record (at least 10 years), a single station frequency analysis is carried out, taking into account all the factors described earlier. Obviously 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.

(i) 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. A check on the homogeneity of the region can be made using a test such as that described by Dalrymple (1968). It should be noted that the confidence limits derived in the Dalrymple study were developed for the Extreme Value Distribution; if other distributions are used applicable 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.

The Regional Flood Frequency Analysis using the Index Flood method prepared by Moin and Shaw (1985) presented in the previous Flood Plain Management Technical Guideline document will require updating to include post -1983 data before using it for flood plain analysis. The same applies to the Regional Frequency Study based on the regression method. This was prepared by Cumming Cockburn and Associates Ltd in 1985, and the results were incorporated in the previous Flood Plain Management Technical Guideline document.

(ii) Estimating Floods of Various Return Periods:

A second technique is to estimate floods of specified return periods, including the mean annual flood, directly by 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 expected to be similar.

(iii) Estimating Distribution Parameters:

Multiple regression techniques can also be used 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.

(f) Transfer of Location

In many cases there will be no gauge on a stream at the location where mapping of the flood plain 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.

(g) Change to Natural Conditions

As previously mentioned, streamflows that include the effect of artificial regulation need to be converted to natural conditions prior to undertaking a frequency analysis. Once an estimate has been made of the natural flood magnitude, it is necessary to reconvert the flow back to regulated conditions. As well, 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 flood plain.

3.2 Runoff Simulation From a Specified Meteorologic Event

In the event that the flood plain definition is to be 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 meteorologic 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.

The Hazel and Timmins storms are defined in the Flood Plain Policy and summarized in Chapter B. 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 of these models 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 a model. 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, ie., 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, etc., 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 take into account the 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 groundwater influence. Furthermore, the model should have the ability to incorporate those types of artificial regulation that may exist in the basin under study.

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, computations on a daily basis 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. 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. It is not adequate for the purposes of the flood risk mapping program to blindly apply any model to a watershed without adequate testing for both calibration and verification 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 until an adequate reconstitution is developed. This procedure is followed for several historical events 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 must be calibrated and tested on a similar adjacent watershed where the variable parameters can be assumed to match those of the basin under study.

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, (ie., 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.

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 verification 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 the streamflow simulation are not calculated. Streamflow simulation to establish floodplain limits depends on the use of computer models, that represent mathematically the complex physical process inherent in flood generation. Thorough understanding of the flood characteristics of the river and the watershed and the calibration/verification of the model is essential to derive credible results.

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.

When flood flows and levels are computed for different land use scenarios (such as existing and future development conditions), abstractions should be computed separately for lands likely to be affected by different land use scenarios.

A more detailed discussion of the flood computation is presented in Chapter E. However, the purpose of Chapter E is not to list all available models for streamflow simulation, but to illustrate the models used by most modellers, and those which were reviewed and found to be acceptable by the Ministry of Natural Resources.

4. HYDRAULIC ANALYSIS

Once the magnitude of a flood to be mapped has been determined by one of the methods previously described, 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 a flood plain.

Computation of water surfaces profiles from a given downstream starting point usually involves the use of backwater analysis. Such analysis is very tedious for manual calculation and therefore is generally achieved by using a computer program. 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. As a guide, the following points should be taken into consideration when selecting a program to compute water surface profiles.

4.1 Type of Flow

Most programs available are for steady, gradually varying flow only, using the standard step method of computation. As well, many of these programs were developed for subcritical flow although some can handle supercritical conditions.

4.2 Cross-Sections

A computer program 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 flood plain. It may also be advantageous to have the ability to interpolate between specified cross-sections where velocity changes are rapid.

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

4.3 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.

4.4 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.

4.5 Roughness

It is very important in a backwater analysis that the friction losses be computed as accurately as possible. 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.

4.6 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.

4.7 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. For analysis purposes, routines using a high speed printer are more flexible and provide faster turnaround than those using mechanical plotting devices. However, for documentation purposes, a good quality plotter should be used.

4.8 Bridge Losses

Any program 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.

4.9 Culvert Losses

If the backwater 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.

4.10 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 elevations.

4.11 Other Factors

There are several other factors that should be considered in the selection of a program 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 program 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 package.

(a) 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 risk maps. Under the option of two-zone concept, as described in Chapter B, 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 floodway is smaller in area than the flood risk zone, 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 flood plain. However, in cases where the floodway is designated, some development will be permitted within the designated flood fringe area. Where warranted, this can lead to improved use of the land within the flood plain and may provide an incentive for the development of floodproofing techniques.

Should it be necessary to delineate a floodway, a program 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.

(b) Ice Jams

The second problem that is encountered in some parts of Ontario is an increase in water levels caused by ice or log 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 to be mapped be the result of an ice jam, the backwater computations are started at 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.

Estimating the effects of such jams on a flood of a given magnitude is not a simple problem. The research carried out in this field is considerable and new techniques are being developed. Estimates should be based on the past 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.

4.12 Dykes

Area behind a dyke is regarded as fringe area if dykes are high enough to provide protection against the flood standard for the area and development in this area is subject to flood proofing requirements to flood standard, unless designated as a Special Policy Area. 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, the dykes would normally delineate the extent of inundation. This does not apply in cases where the dykes are not structurally inadequate and would fail under large floods.

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

4.13 Spills

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

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, less than 10% of the peak flow, the flood plain 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 route of the spill should be examined to determine the potential harmful effects of overland flow. No reduction should be made for the spill in the downstream flood plain computations.

D. FLOW COMPUTATION DATA REQUIREMENTS

1. RAINFALL ANALYSIS

1.1 Introduction

The following discussion is a collection and presentation of design storms and distributions applicable to hydrologic studies within Canada, particularly 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 Hazel, Timmins and 100-year storms), information is also provided on Probable Maximum and frequently occurring storms and distributions.

The storms and distributions have been grouped into three 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. The last group, **Probable Maximum**, considers the largest theoretical rainfall possible.

Until its recent reorganization, Atmospheric Environmental Services (AES) of Environment Canada provided the atmospheric data and analyses across the country. In this Guide all data and analyses prepared in the past by AES is referred to under the old name, as AES. For any new information the user should contact Environment Canada Ontario Region office.

Under the three categories mentioned above, the following storms and distributions are discussed:

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.

If the failure of a structure could result in the loss of life (see Table B-4), then the use of a Probable Maximum Rainfall is recommended.

To compute the extent of flood plain, one may use either of the two Historical storms described under the flood plain standards or a return period storm depending on the zone the watershed is located in. In the case of the Hazel or Timmins storms, the particular storm to be used is specified according to the zone. In other cases, the user is left with the decision to select the storm and the distribution. Figure D-1 presents a decision diagram to assist the user in the selection process.

A summary of storms used in the past in Ontario are presented in Table D-1. 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 AES data, as indicated in the table. In such cases, with the exception of 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.

Historical	·	Hurricane Hazel
	·	Timmins
Return Period	·	A.E.S - 1 Hour
		- 12 Hour Snowmelt
Probable Maximum	·	Small Dams.

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

- total precipitation
- duration
- temporal distribution
- time step
- areal reduction.

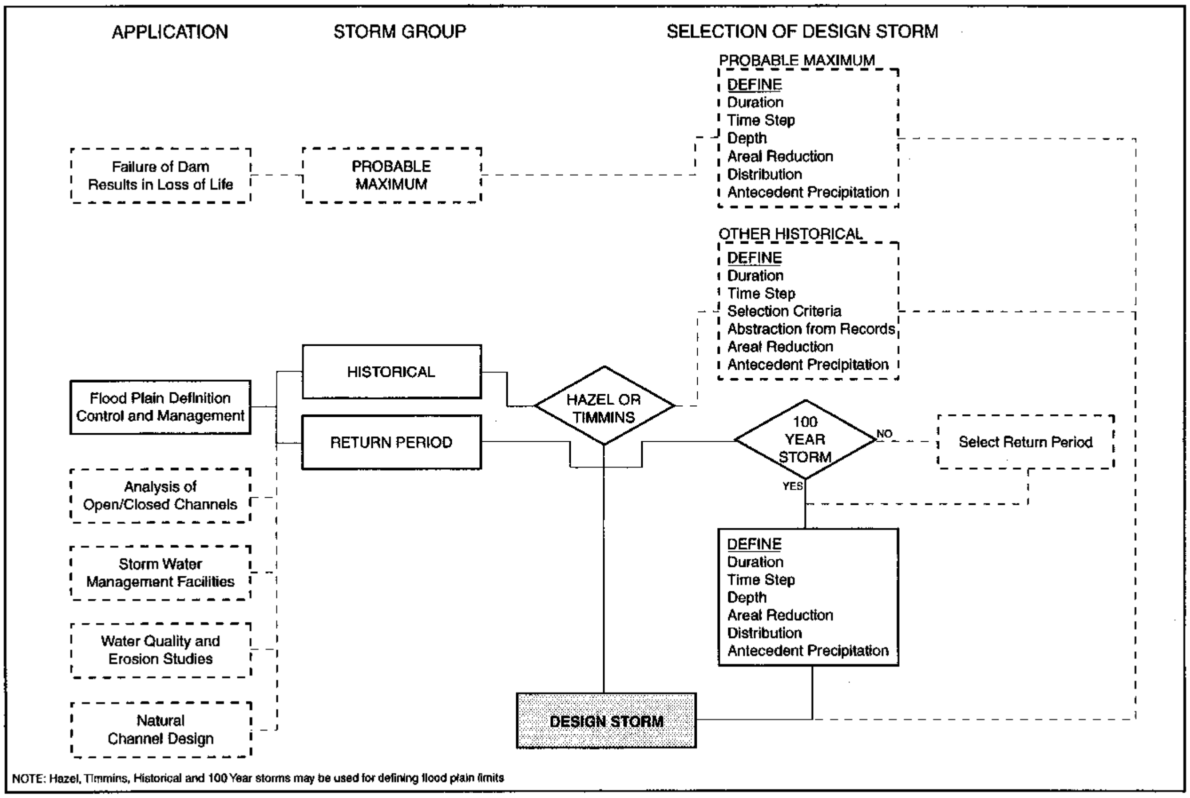


Figure D-1: Design Storm Selection

TABLE D-1
COMPARISON OF STORMS USED FOR
FLOOD PLAIN AND PROBABLE MAXIMUM PRECIPITATION COMPUTATION

	Duration	Time Interval	Compariso n with IDF Curves	Degree of Difficulty		Antecedent Precipitatio n	Areal Reductio n
				To Determine	To Apply		
1. Historical							
Hurricane Hazel	12 hour	1 hour	Poor	Low	Low	Recorded	Recorded
Timmins	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
2. Return Period							
Small Dams	6 hour	1 hour	Poor	Low	Low	User Assigned	User Assigned
Huff	Variable	Variable	Poor	Low	High	User Assigned	User Assigned
Keifer & Chu	Variable	Variable	Good	Medium	Low	User Assigned	User Assigned
S.C.S. 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	Fair	Low	Low	User Assigned	User Assigned
A.E.S. - 1 Hour	1 hour	5 min	Poor	Low	Low	User Assigned	User Assigned
A.E.S. - 12 Hour	12 hour	1 hour	Poor	Low	Low	User Assigned	User Assigned
Snowmelt	1-30 days	1 day	Good	Medium	Low	User Assigned	User Assigned
3. Probable Maximum							
Small Dams	6 hour	1 hour	Poor	Low	Low	User Assigned	User Assigned
HEC	48 hour	6 hour	Poor	Low	Low	User Assigned	User Assigned
Emergency Spillway	36 hour	1 hour	Poor	Low	Low	User Assigned	User Assigned
Bruce	6-48 hours	6 or 12 hour	Poor	Low	Low	User Assigned	User Assigned

Table D-1

1.2 Historical Storms

Two of the floods suitable for flood plain delineation are based on historical storms called "Hurricane Hazel" and "Timmins", both of which are described below.

Hurricane Hazel

Hurricane Hazel was adopted by the Ministry of Natural Resources as the storm for watersheds located within Zone 1 in the area delineated in Figure B-1. 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 D-2) 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.

For larger basins, the rainfall amounts listed in Table D-2 are to be modified by the reduction factor percentages shown for different drainage areas in Table D-3. These factors should be based on an equivalent circular area. In the case of an elongated watershed, the isohyetal technique should be applied to determine the rainfall input for the runoff simulation.

A hyetograph and a dimensionless storm distribution curve of the 12-hour design storm (36th and 48th hours) are shown in Figure D-2.

From the records, curves were produced for the 6, 12, 24, 36, and 48 hour portion of Hurricane Hazel for a range of drainage areas, as illustrated on Figure D-3, which incorporate the areal reduction effect.

Timmins

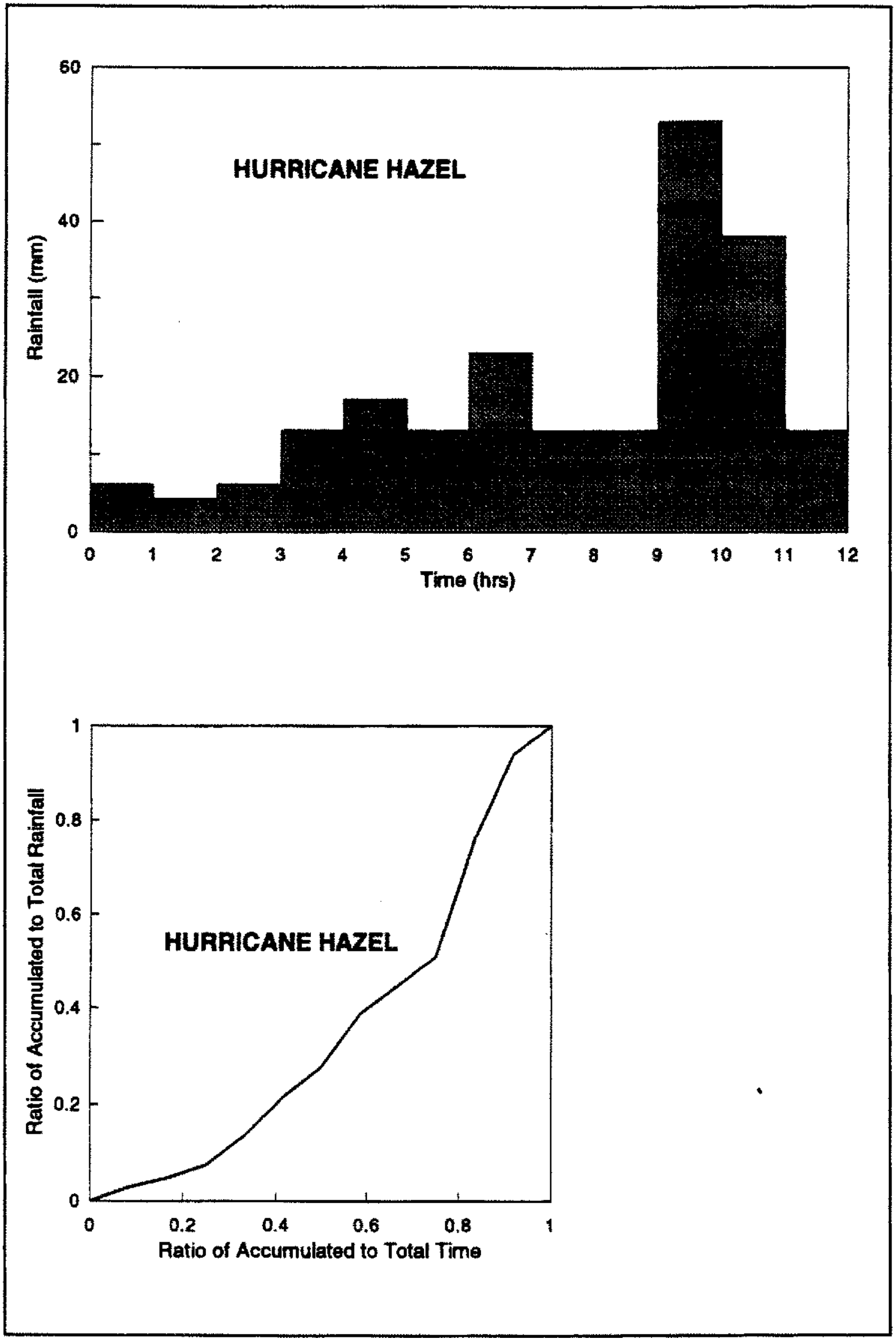
"Timmins" is the name applied to the summer storm which occurred over Timmins, Ontario on September 1, 1961. The event is described by McMullen in Circular 37456 published by the Meteorological Branch of the Department of Transport. The storm created severe property damage and resulted in loss of life on the banks of Town Creek.

The 12 hour design storm (Table D-4 and Figure D-4) was developed from information assembled from several gauges located in Timmins. It was adopted by the Ministry of Natural Resources, as the Storm to produce the flood standard for watersheds less than 25 km^2 in size within Zone 3 delineated in Figure B-1. Similar to the Hazel storm, an areal reduction is applied to the Timmins point rainfall, for drainage areas larger than 25 km^2 based on the 24 hour isohyets as shown in Table D-5.

1.3 Return Period Storms

The second group of storms used for flood plain 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.

Figure D-2: Hurricane Hazel Storm Hyetograph and Dimensionless Distribution



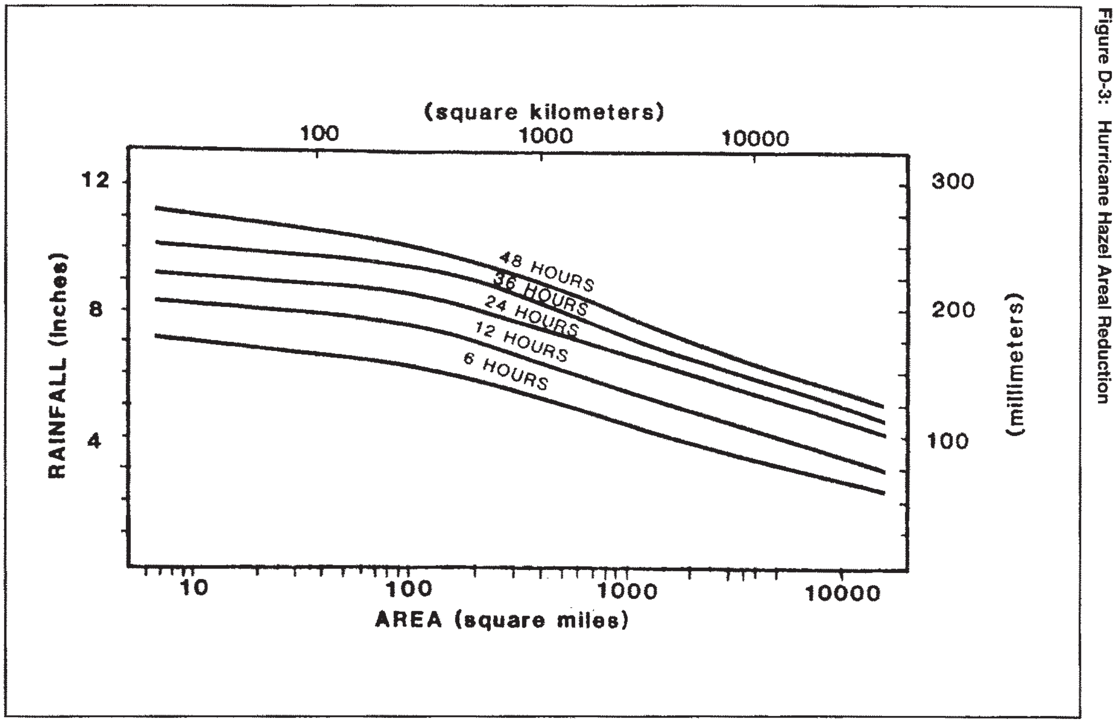


Figure D-3: Hurricane Hazel Area Reduction

TABLE D-2 HURRICANE HAZEL RAINFALL DEPTHS			
	Depth		Percent of Last 12 hours
	mm	inches	
First 36 hours	73	2.90	-
37th hour	6	.25	3
38th hour	4	.17	2
39th hour	6	.25	3
40th hour	13	.50	6
41st hour	17	.66	8
42nd hour	13	.50	6
43rd hour	23	.91	11
44th hour	13	.50	6
45th hour	13	.50	6
46th hour	53	2.08	25
47th hour	38	1.49	18
48th hour	13	.50	6
TOTAL	285	11.21	100

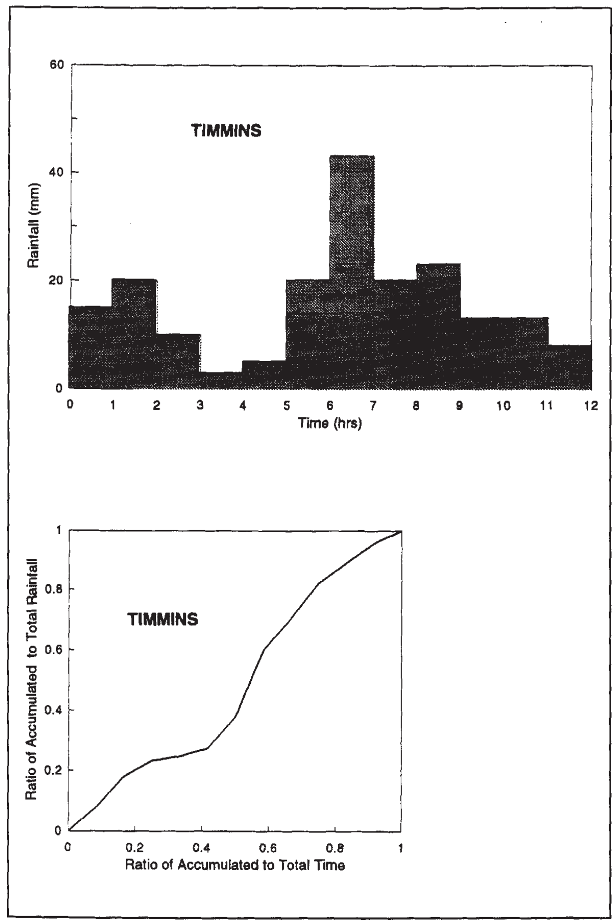


Figure D-4 Timmins Storm Hyetograph and Dimensionless distribution

TABLE D-3 HURRICANE HAZEL - AREAL REDUCTION	
Area (km^2)	**Reduction Factor Percentage**
0 to 25	100.0 (no reduction)
26 to 45	99.2
46 to 65	98.2
66 to 90	97.1
91 to 115	96.3
116 to 140	95.4
141 to 165	94.8
166 to 195	94.2
196 to 220	93.5
221 to 245	92.7
246 to 270	92.0
271 to 450	89.4
451 to 575	86.7
576 to 700	84.0
701 to 850	82.4
851 to 1000	80.8
1001 to 1200	79.3
1201 to 1500	76.6
1501 to 1700	74.4
1701 to 2000	73.3
2001 to 2200	71.7
2201 to 2500	70.2
2501 to 2700	69.0
2701 to 4500	64.4
4501 to 6000	61.4
6001 to 7000	58.9
7001 to 8000	57.4

Note: Reduction factor to be multiplied by the rainfall depth for watersheds larger than 25 km^2.

TABLE D-4 TIMMINS - RAINFALL DEPTHS			
	Depth		Percent of Last 12 hours
	mm	inches	
1st	15	0.6	8
2nd	20	0.8	10
3rd	10	0.4	6
4th	3	0.1	1
5th	5	0.2	3
6th	20	0.8	10
7th	43	1.7	23
8th	20	0.8	10
9th	23	0.9	12
10th	13	0.5	6
11th	13	0.5	7
12th	8	0.3	4
TOTAL	193	7.6	

TABLE D-5 TIMMINS - AREAL REDUCTION	
Area (km^2)	**Reduction Factor Percentage**
0 to 25	100.0 (no reduction)
26 to 50	97
51 to 75	94
76 to 100	90
101 to 150	87
151 to 200	84
201 to 250	82
251 to 375	79
376 to 500	76
501 to 750	74
751 to 1000	70
1001 to 1250	68
1251 to 1500	66
1501 to 1800	65
1801 to 2100	64
2101 to 2300	63
2301 to 2600	62
2601 to 3900	58
3901 to 5200	56
5201 to 6500	53
6501 to 8000	50

Note: Reduction factor to be multiplied by the rainfall.

Figure D-5A: 24 Hour Duration Mean Annual Extreme Rainfall

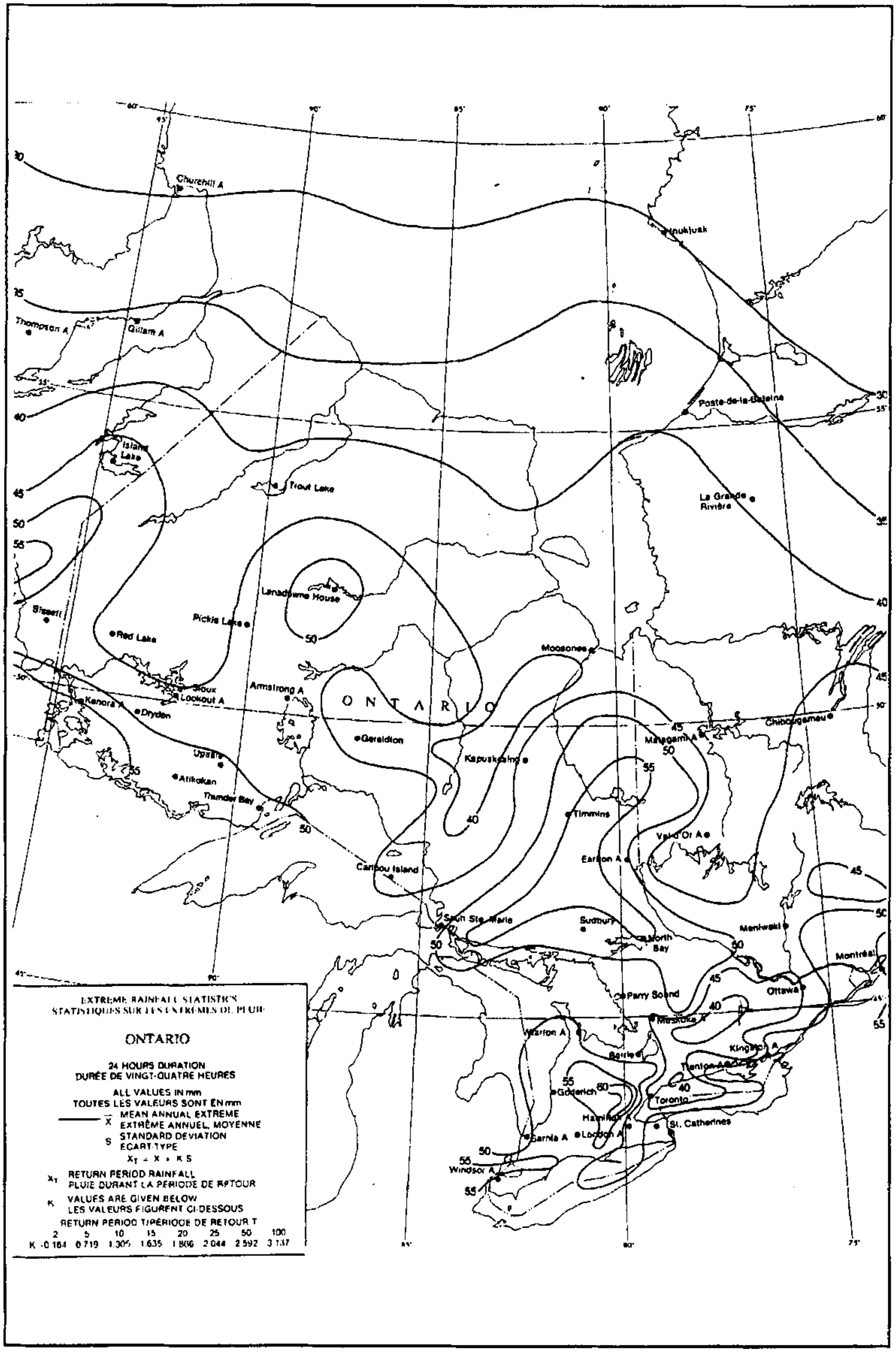
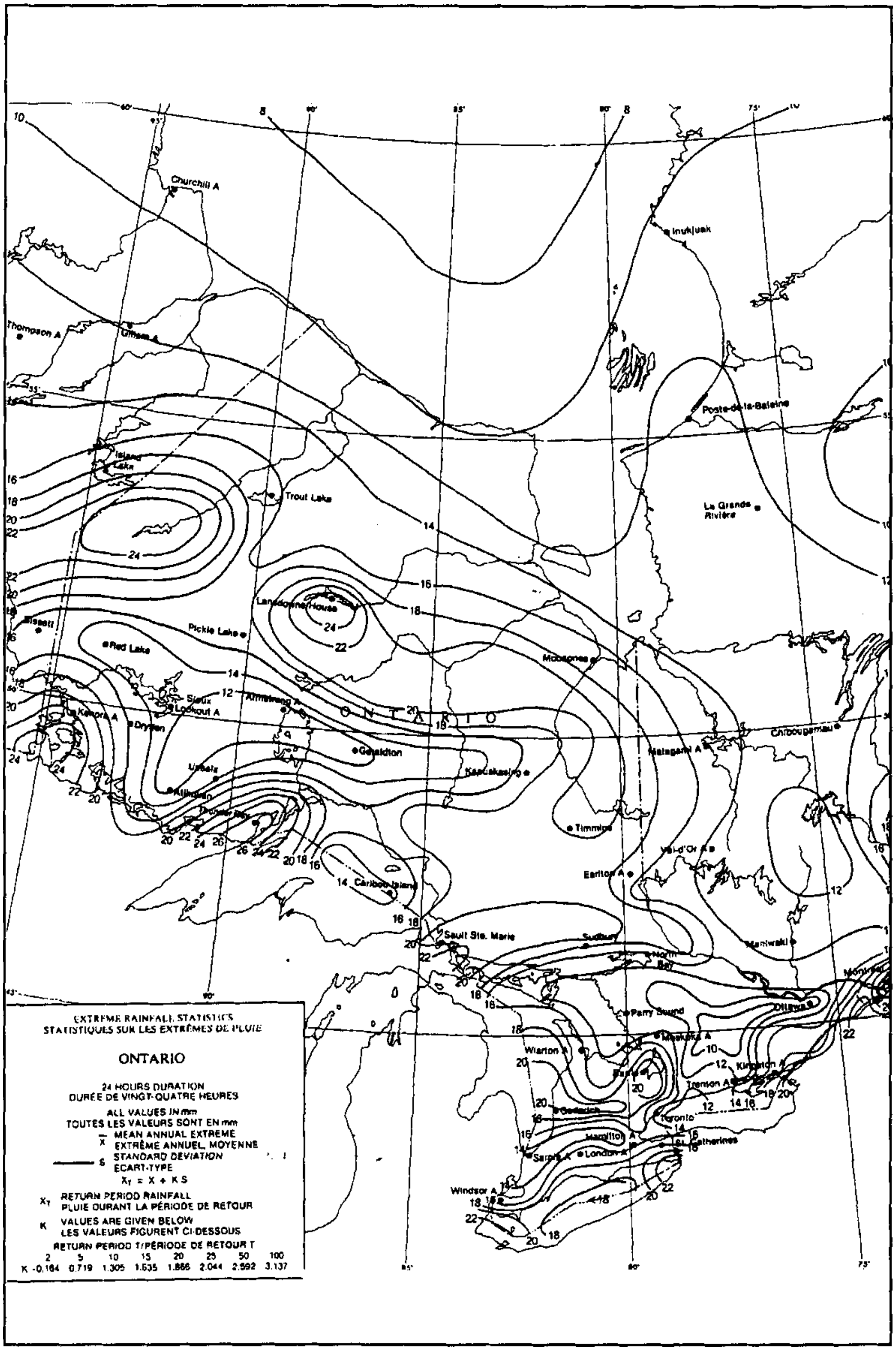


Figure D-5B: 24 Hour Duration Rainfall Standard Deviation



Intensity-duration-frequency curves prepared by AES are available from Environment Canada Ontario Region office for recording rain gauges with more than ten years of record. These gauges provide information on point rainfall. The point extreme rainfall statistics for individual recording precipitation gauges are periodically updated and are also available from Environment Canada Ontario Region office.

When computing flows from a given return period rainfall, the commonly made assumption is that rainfall of a given frequency produces streamflows of the same frequency. However, a rainfall event is made up of many components, each with a unique return period, therefore in practice it is not possible to have the same return period of each of the components. A second reason why this assumption may not be true in many cases is due to the variability of the antecedent moisture conditions.

There are two types of design storms: single event and continuous simulation. The single event can be based on an historical storm or a synthetic storm based on a statistical analysis of recorded data.

The second approach to the rainfall/runoff analysis is the use of continuous simulation. Instead of the assumption of a return period for a single event, the continuous simulation approach is based on the input of long-term precipitation data. 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.

The two most important selections the user has to make to define a return period storm are: rainfall duration and depth.

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 watershed or small watershed with storage facilities may require the use of long duration storms such as 12 or 24 hours.

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

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 hourly time steps.

Selection of Rainfall Depth

For Location Near a Gauge

In Canada, the source of most precipitation data is Environment Canada. More than 60 Environment Canada recording rainfall gauges are in operation in Ontario and 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).

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 by Environment Canada to prepare intensity-duration-frequency curves.

It should be noted that extrapolation of frequency curves for return periods greater than twice the length of record will result in increased inaccuracies.

Remote from a Gauge

For areas where no such recording gauges are available, information on rainfall for selected duration and for different return period events presented in Rainfall Frequency Atlas for Canada (Hogg and Carr, 1985) should be used.. The publication contains maps describing extreme rainfall events. The regional maps of both the mean and standard deviation of annual extremes of rainfall are 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 are presented on Figures D-5A and D-5B as examples from the Atlas.

Areal Reduction

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

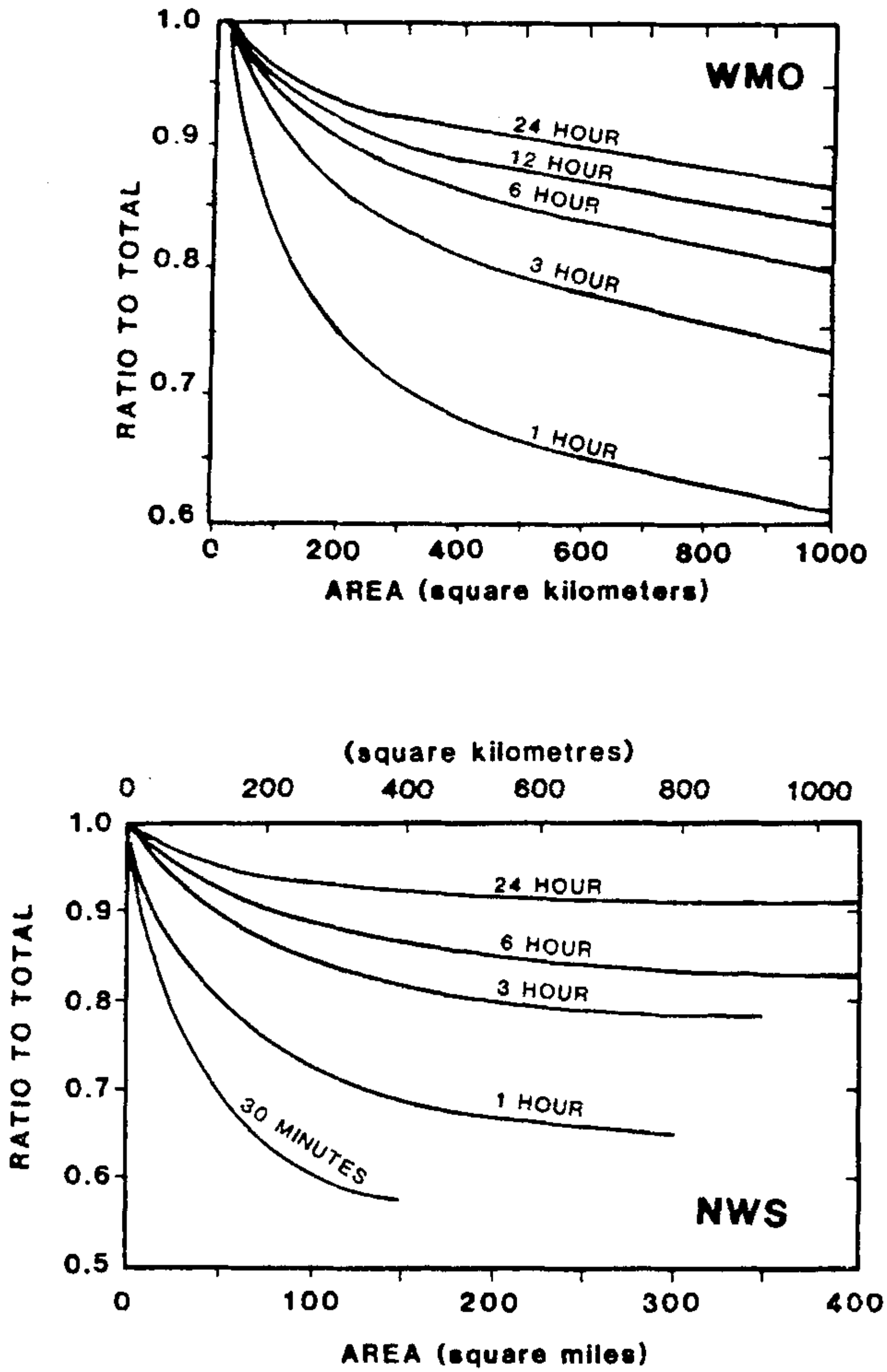
Intensity-duration-frequency curves describe the variation of point rainfall with time for a given frequency. The curves do not include an adjustment for the variation with area. Examples of two common methods of adjustment are shown in Figure D-6. The first method is used by the World Meteorological Organization and the second by the U.S. National Weather Service for non-historical storms.

The adjustment in rainfall depth for a particular point of interest can be calculated by either of the two methods. The two approaches can be used for the upstream drainage area or with the assumption of a circular area, using the longest length of the watershed as a diameter. A few examples of the circular area-watershed length relationship are shown in the following table.

Watershed Length (km)	**Circular Area (km^2)**
1	0.8
5	19.6
5.6	25.0
10.0	78.5

Generally, for studies undertaken in Ontario, no reduction to point rainfall is used for areas under 25 km^2.

Figure D-6: Areal Reduction Curves



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 km^2, which is more realistic than the NWS curves which will converge at 0 area for a ratio of 1.0.

When analyzing thunderstorm type rainfall in urban areas, storm distribution becomes an important factor. Variation in rainfall intensity over small areas, such as 5 km^2, can be significant. Unfortunately, due to lack of rainfall data, no guidelines are available on storm distribution within small urban areas.

Storm Distributions

A number of historical, return period and Probable Maximum Rainfall distributions are available for the designer. Table D-6 summarizes the distributions recommended for the various storms by different agencies.

For the Hazel and Timmins storms, the distribution is provided by the Ministry of Natural Resources. For Return Period storms, the temporal distribution of the rainfall will determine the peak intensity and time to peak. The most commonly used temporal distributions are: a) those based on Ontario and Canadian data, such as the AES distribution and HYDROTEK design storms, or b) those developed in the U.S., such as the Chicago or the SCS distribution.

a) Canadian Storm Distributions

Atmospheric Environment Services 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.

Examples of the 12 hour rain distribution curves produced by A.E.S. for southern and northern Ontario are shown on Figure D-7.

The designer has to exercise caution when selecting rainfall distribution from the enclosed graphs. For example, the selection of a 10% curve would result in an advanced pattern of excess rainfall, as the early portion of the rain contains a large percent 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. 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.

The 1 hour urban design storm (HYDROTEK, 1985) is also based on A.E.S. rainfall data. The design storm distribution is described by two parameters which can be applied in conjunction with the total rainfall values published with the intensity-duration-frequency data. The model assumes a linear rise and an exponential decay for the rainfall intensity during the storm. Parameter "a" defines the time of peak intensity, and "K" is a dimensionless exponential decay coefficient, as shown in Figure D-8.

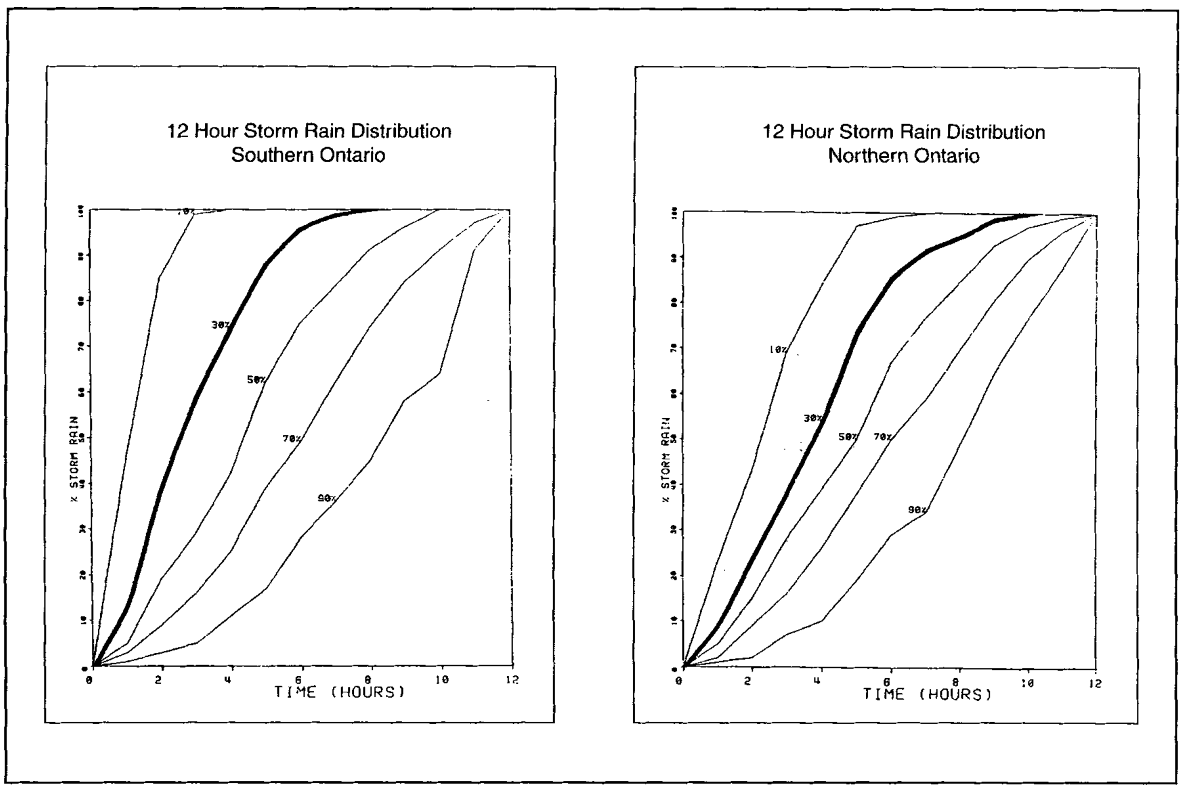


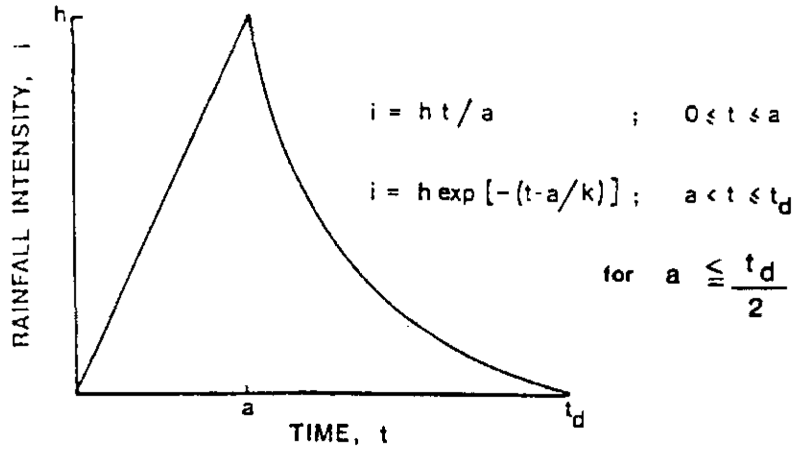
Figure D-7: 12 Hour Storm Distribution

TABLE D-6
RAINFALL DISTRIBUTIONS (PERCENT)

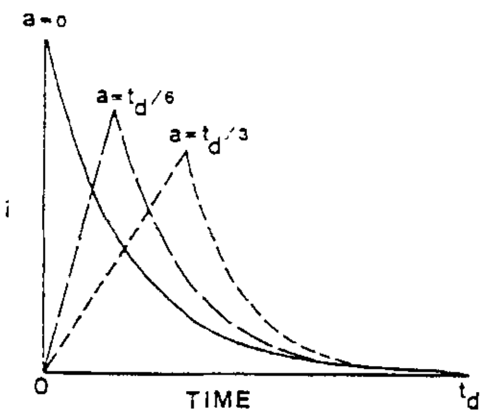
Type	Reference	Storm Duration	HOURS											
			1	2	3	4	5	6	7	8	9	10	11	12
Probable Maximum Storm	Small Dams	6 Hour	8	9	11	49	15	8	-	-	-	-	-	-
HAZEL	Ministry of Natural Resources	12 Hour	3	2	3	6	8	6	11	6	6	25	18	6
TIMMINS	Ministry of Natural Resources	12 Hour	8	10	6	1	3	10	23	10	12	6	7	4
Return Period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

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

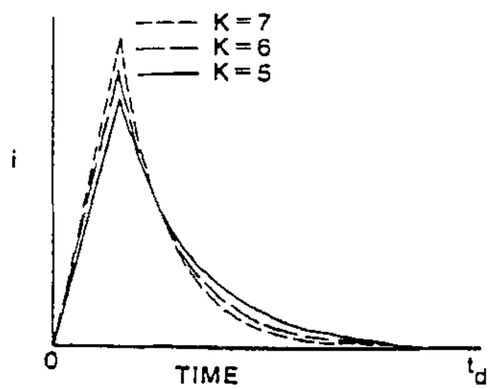
Figure D-8: 1 Hour Urban Design Storm



Definition sketch of urban design storm



Variation of design storm shape with a



Variation of design storm shape with K

Results of the storm distribution analysis for Ontario are presented in Table D-7. The regional K value for the Province is 7, and the individual "a" values for the 12 stations analyzed range from 21 to 28 minutes. Similar analysis can be carried out for other stations data by the user. Alternatively, the two parameters K=7 and a=25 minutes can be used with rainfall values obtained from intensity-duration-frequency data.

The above model has not yet been tested for longer duration storms.

TABLE D-7
ONTARIO VALUES FOR PARAMETER "a" (minutes)

Ear Falls	28
Central Patricia	23
Thunder Bay	25
Rayner	27
White River	27
Timmins	24
Sudbury	21
Windsor	27
St. Thomas	24
Toronto	21
Kingston	27
Ottawa	26

At present no appropriate storm distributions are available for any other duration, except the 1 and 12 hour storms. When the designer is faced with the task of establishing a design storm distribution for any other duration, historical rainfall data for the relevant gauge(s) should be analyzed to determine the appropriate distribution.

A simplified approach used in the HYDROTEK method involves a dimensionless duration and percent of storm plot, for the 1 hour and 12 hour A.E.S. storm distribution curves, for any of the 10%, 30%, 50% and 70% criteria.

Where the dimensionless plots for the two different duration storms are similar, the designer can estimate any duration storm distribution between 1 and 12 hours. However, this is an approximate method and should not be used without verifying the distribution with historical data.

b) Keifer and Chu (Chicago) Distribution

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

The 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.

c) SCS Type II Distribution

The original distribution derived by its author was based on a 24-hour storm. Subsequently, the 6 and 12 hour distributions were obtained by selecting the increments within the central 6 and 12 hour periods of the original 24 hour storm.

Prior to the development of distributions for Canada and Ontario by AES and HYDROTEK, the Chicago and SCS distributions were widely used in Ontario. More recent comparisons of the Chicago and SCS distributions with Ontario data found that for Ontario applications, the AES and the HYDROTEK distributions give more realistic results.

1.4 Probable Maximum Rainfall

The Probable Maximum Rainfall is the largest precipitation event that can be reasonably expected to occur over a selected basin. It is based on a rational consideration of the chance of simultaneous occurrence of the maximum of the various elements which contribute to the event.

The storm is used to calculate flow rates for the design of spillway structures and other hydraulic structures, where failure could result in the loss of life.

The estimate of the Probable Maximum Rainfall as presented in the Design of Small Dams (US Department of Interior Bureau of Reclamation, (1987) is based on analyses which consists of:

- determining the areal and time distribution of large storms;
- maximizing the observed storms by increasing their values to their physical upper limit and;
- considering the transposition of these storms.

Although the work presented specified the mainland United States, it did however overlap into areas of Southern Ontario and the Maritimes. J.P. Bruce (1957) found after a preliminary investigation, that the overlap was a result of a geographical extrapolation, but the meteorological conditions peculiar to Southern Ontario were not considered.

Figure D-9 presents generalized Probable Maximum Precipitation (PMP) estimates for Ontario, based on a series of PMP studies undertaken by Dillon (1987) and can be used to provide preliminary estimates of PMP's ranging from 6 to 96 hour duration.

1.5 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 as a result of snowmelt and rainfall. Past 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. For other methods, the reader is referred to the Ministry of Natural Resources, Snow Hydrology Studies, Phases I, II and III. (McLaren Plansearch 1984)

Five sets of snowmelt plus rainfall frequency values were developed by AES for Canada using the degree day method. Two of these prepared for Ontario are presented in Table D-8. A more detailed description can be found in Extreme Value Estimates of Snowmelt (Louise and Hogg, 1980) and in the Hydrology of Floods in Canada (National Research Council of Canada, 1989).

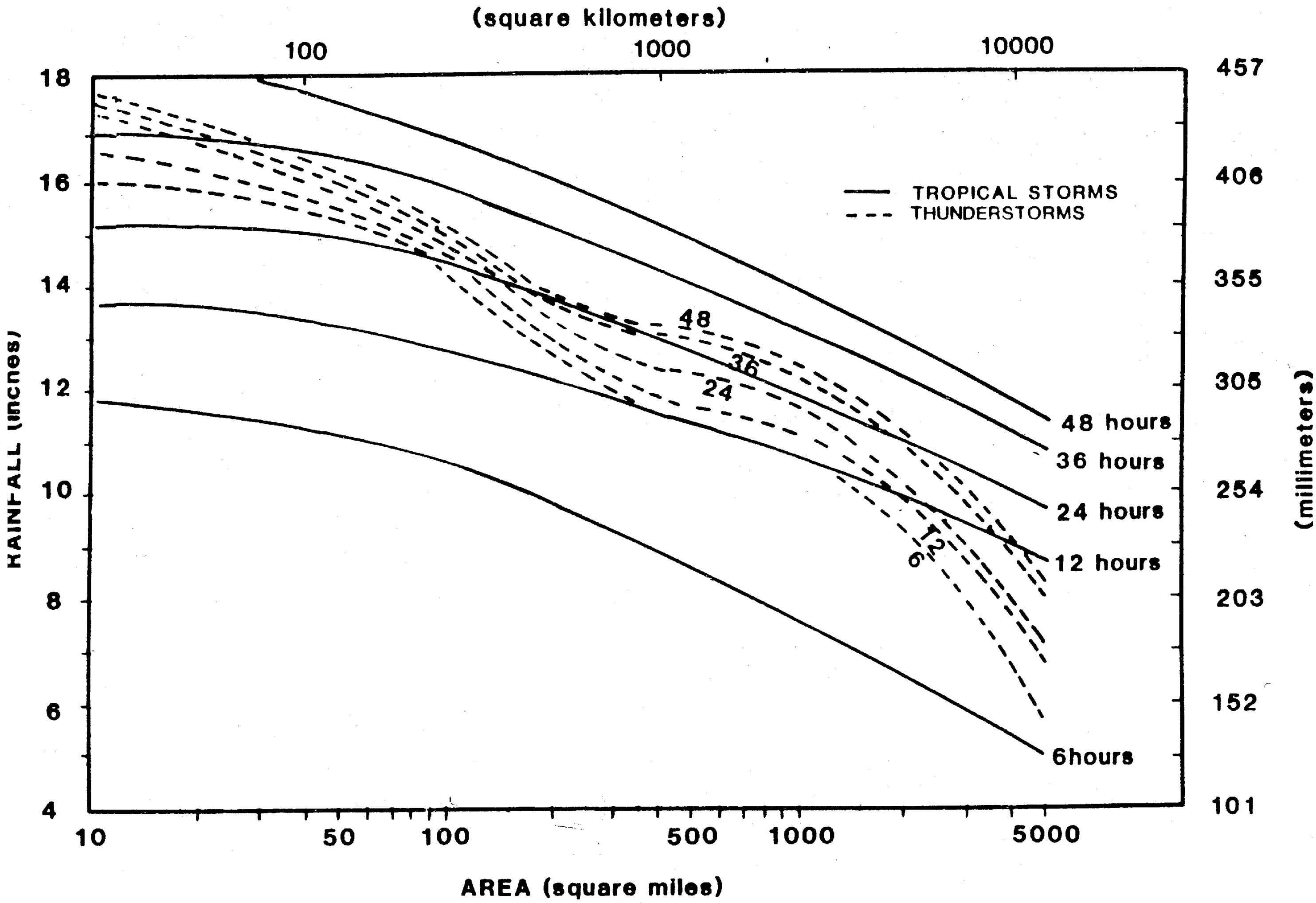


FIGURE D-9
PROBABLE MAXIMUM
SOUTHERN ONTARIO RAINFALL VALUES
BY J.P. BRUCE

TABLE D-8 AES SNOWMELT EQUATIONS		
Model 4 - Southern Ontario		
SM 4	=	$0.91\ T_{max}$
SM 4	=	mm/day
T_{max}	=	maximum daily air temperatures (C)
Model 5 - Southern Ontario		
SM 5	=	$3.66\ T_m$
SM 5	=	mm/day
T_m	=	mean daily air temperature, (C)

Environment Canada can provide snowmelt plus rainfall frequency values for AES 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 precipitation, and used as input in the precipitation/runoff simulation process, especially in cases where it exceeds the summer rainfall values.

The input data used in the 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 was 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. The Algorithm ceases to operate when the synthetic snowpack is reduced to zero. 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 Table D-8 using single station or regional frequency estimates should be carried out wherever feasible.

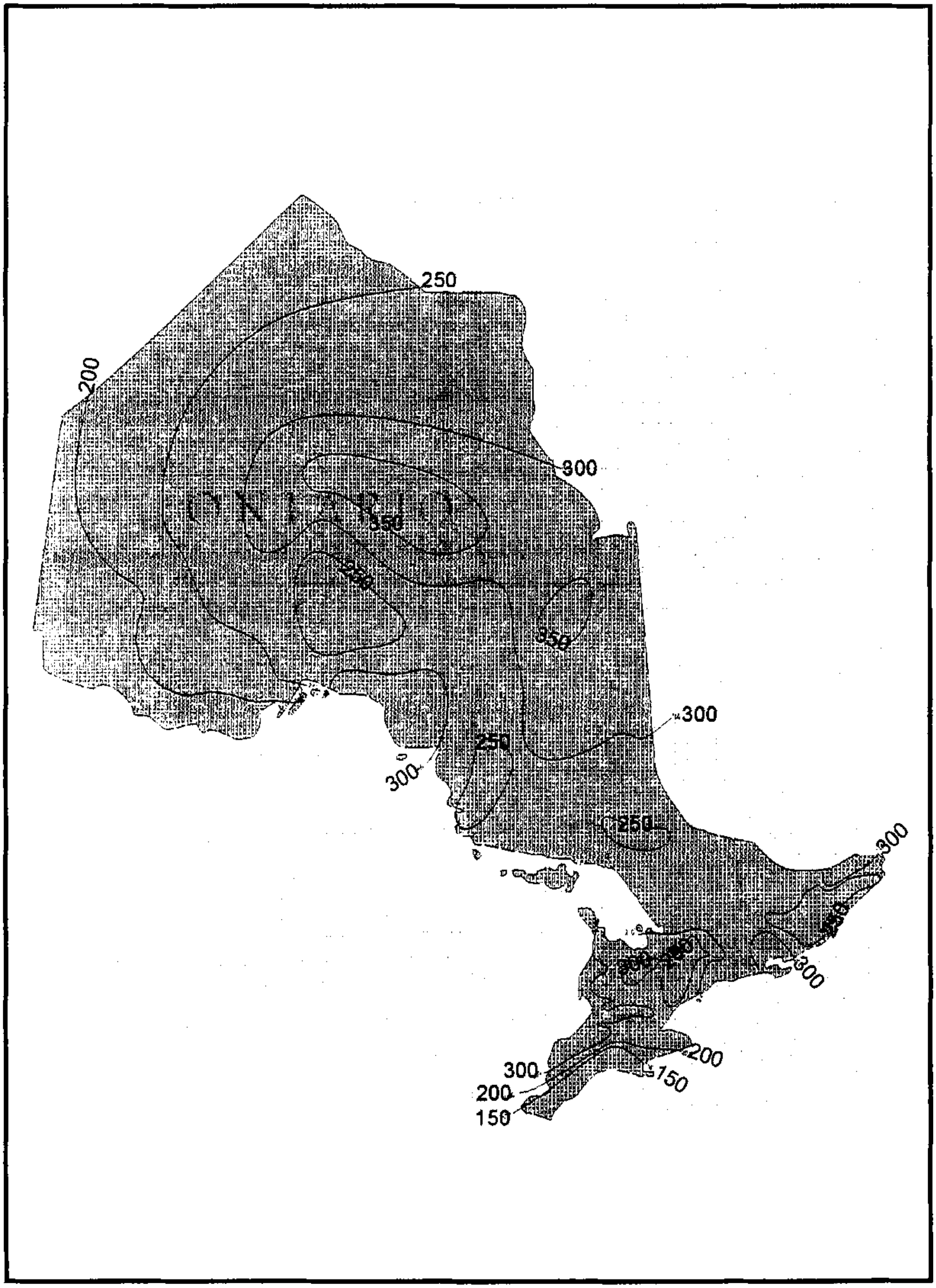
The extreme precipitation data, upon which the Environment Canada intensity-duration-frequency curves are based, is derived from storms that occur mainly in the summer or fall, which represents rainfall that falls on pervious surfaces that permit infiltration and thereby reduce runoff. In the winter months, rainfall is less intense, but may fall on frozen ground that prevents infiltration, and may be supplemented by snowmelt runoff. At present no separate winter rainfall intensity-frequency-duration curves are available. It is hoped Environment Canada will provide such data in the future.

As part of the series of snow hydrology studies undertaken by MNR, Dillon (1994) prepared a comprehensive study of 195 Ontario snow gauges. The study provides estimates of maximum snow water equivalent for a range of 2 to 100 year return periods. (See example on Figure D-10).

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 is 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.

Figure D-10: 100 Year Snowpack Water Equivalent Mapping



Note: Measurements are in Millimeters

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D 2. SOIL 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. The list presented in Table D-11 summarizes reports and maps released by the Ontario Centre for Soil Resource Evaluation on soils for Southern and Northern Ontario.

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 Drainage Manual) 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, 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 MTO 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 Drainage Manual, Chapter H.

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.

3. LAND USE

The land use types within a watershed may be determined from maps and/or aerial photo 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. Tables extracted from the Census of Canada data are available in the MTO Drainage Manual, Chapter H. Land uses may be grouped according to similar hydrologic characteristics.

Crop	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.

Large areas of urbanization should be measured from maps or air photos, but roads, scattered houses and other relatively small non-agricultural areas can be included as part of the agricultural area in all but the smallest basins. Future extent of urbanization should be extracted from Official Plans or other Municipal land use planning documents and the planning horizon should preferably extend 20 years into the future.

Variations in types of woodland should normally be ignored. Small clearings and cut-over land can be included in the wooded areas.

Where stormwater management facilities, existing or future, can affect the magnitude and/or timing of the flows, the cumulative effects of these structures should be incorporated in the flood plain studies.

Table D-9: Ontario Soil Surveys

SUMMARY OF SOIL SURVEY COVERAGE

County Area NTS Reference	Report No.	Date	Scale	No. of Maps	Available From	Information Slope	Information Data
Brant	55	In prep.	1:25 000	4	IC	S	D
Blind River–Sault Ste. Marie (41J, 41K)[1]	50	1983 1983	1:250 000 1:50 000	1 12	IC IC	N S	NA NA
Bruce	16	1954	1:63 360[2]	2	IC	G1	L
Chapleau–Foleyet (41-O, 42B)	61	1986	1:250 000	2	IC	G2	N
Dufferin	38	1963	1:63 360	1	IC	S	L
Dundas[3]	14	1952	1:63 000	1	IC	G1	L
Durham	9	1946	1:126 720	1	IC	G1	L
Elgin	63	1993	1:50 000	3	IC	S	D
Essex	11	1949	1:63 360	1	IC	G2	L
Fort Frances-Rainy River (52C, 52D)[1]	51	1986 1986	1:250 000 1:50 000	1 5	IC IC	N S	NA NA
Frontenac	39	1963	1:63 360	2	IC	G2	L
Glengarry	24	1957	1:63 360	1	IC	G2	L
Gogama (41P)[1]	59	1987 1987	1:250 000 1:50 000	1 2	IC IC	N S	NA NA
Grenville[3]	12	1949	1:63 360	1	IC	G2	L
Grey	17	1954	1:63 360	2	IC	G2	L
Haldimand–Norfolk	57	1984 1985	1:25 000 1:100 000	13 1	IC IC	S N	D NA
Halton	43	1971	1:63 360	1	IC	S	L
Hastings	27	1962	1:63 360	3	IC	G2	L
Huron	13	1952	1:63 360	2	IC	G2	L
Kenora–Dryden–Pointe du Bois (52E, 52F, 52L)[1]	52	1988	1:50 000	4	IC	S	NA
Kent	64	In prep.	1:50 000	3	OCSRE	S	D
Lambton	22	1957	1:63 360	1	IC	G2	L
Lanark	40	1967	1:63 360	2	IC	S	L
Leeds	41	1968	1:63 360	2	IC	S	L
Lennox–Addington	36	1961	1:63 360	2	IC	G2	L
Manitoulin Island[3]	26	1959	1:63 360	2	IC	G2	L
Middlesex	56	1992	1:50 000	3	IC	S	D
Niagara	60	1989 1989	1:25 000 1:100 000	7 1	IC IC	S N	D NA
North Bay (31L)[1]	54	1987 1987	1:250 000 1:50 000	1 7	IC IC	N S	NA NA
Northumberland	42	1976	1:63 360	1	IC	S	L
Ontario	23	1956	1:63 360	2	IC	G2	N
Ottawa–Carleton	58	1987	1:50 000	3	IC	S	D
Ottawa–Urban Fringe	47	1977	1:25 000	3	IC	S	D
Oxford	28	1961	1:63 360	1	IC	G2	L
Parry Sound[1]	31	1962	1:126 720	1	IC	G2	L
Peel	18	1953	1:63 360	1	IC	G2	L
Perth	15	1952	1:63 360	1	IC	G2	L
Peterborough	45	1981	1:63 360	2	IC	S	L

(Continued)

SUMMARY — Continued

County Area NTS Reference	Report No.	Date	Scale	No. of Maps	Available From	Information Slope	Information Data
Prince Edward	10	1948	1:63 360	1	IC	G2	L
Pukaskwa National Park[1]	53	1985	1:100 000	1	IC	S	NA
Renfrew	37	1963	1:63 360	4	IC	G2	L
Russell–Prescott	33	1962	1:63 360	2	IC	G2	L
Simcoe	29	1962	1:63 360	2	IC	G2	L
Soil Landscapes of Canada, Ontario–South		1989	1:1 000 000	1	IC NSDB	S	L
Soil Landscapes of Canada, Ontario–North		In prep.	1:1 000 000	1	NSDB	S	L
Stormont[3]	20	1954	1:63 360	1	IC	G2	L
Sudbury (41-I)[1]	49	1983 1983	1:250 000 1:50 000	1 10	IC IC	N S	NA NA
Thunder Bay (52A)[1]	48	1981 1981	1:250 000 1:50 000	1 8	IC IC	N S	NA NA
Timmins–Noranda–Rouyn (42A, 32D)[1]	46	1979 1979	1:250 000 1:50 000	1 6	IC IC	N S	NA NA
Victoria[3]	25	1957	1:100 000[4]	1	IC	G2	L
Ville-Marie (Ont. section)	OIP 90–2	1990 1990	1:250 000 1:50 000	1 8	OCSRE OCSRE	N S	NA L
Waterloo	44	1971 1971	1:20 000 1:100 000	45 1	IC IC	S N	D NA
Wellington[3]	35	1962	1:63 360	2	IC	S	L
Wentworth	32	1965	1:63 360	1	IC	S	L
York	19	1955	1:63 360	1	IC	G2	N

[1]No report prepared;
[2]Originally printed at a scale of 1:95 040 but reprinted at a scale of 1:63 360;
[3]Report available, maps out of print;
[4]Originally printed at a scale of 1:63 360 but reprinted at a scale of 1:100 000

Symbols used in the brochure

- NA - Not Applicable; no report prepared
- N - Not included; general soil descriptions only
- S - Specific slope information for map units; shown in map symbol
- G1 - General slope information; slope values given in report
- G2 - General slope information; no slope values given
- D - Detailed analytical data for most soils in the report
- L - Limited analytical data available in the report

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The National Soil DataBase contains computerized versions of many of the soil maps indicated in this brochure. These coverages are designed for use in a Geographic Information System. For further information regarding database contents, format, and availability, please contact your local CLBRR Land Resource Unit, or the

Canadian Soil Information System (CanSIS)
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Ottawa, Ontario K1A 0C6
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E. METHODS OF COMPUTING FLOOD FLOWS

1. INTRODUCTION

Hazel, Timmins, or the 100 year criteria require multi-stage computations to determine the water levels and the extent of flood plain, as shown below. Where the criteria is 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 flood plain. The second stage involves the computation of the corresponding flood level, and the final stage in the process of defining the flood plain in a watershed involves the delineation of floodlines on maps.

Table E-1 summarizes the alternative methods available to compute floods and water levels.

The 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 the 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.

The engineer or modeller may have to compute flood and water levels with return periods other than the 100-year flood for the design and economic appraisal of flood, stormwater management and erosion control works. The seven different alternative methods listed under the 100-year flood on Table E-1 can be used for computing various return period floods.

Numerous flow computation procedures were reviewed to identify their appropriateness for Ontario conditions. Three criteria were used for the selection of flow estimating procedures: accuracy, consistency and cost-effectiveness.

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.

Consistency identified the ease of reproducing similar results by different users at the site based on the same procedures.

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

Generally, floods and corresponding floodlines can be calculated from peak flows or from flood hydrographs. Each of these two approaches has a number of alternative methods available to compute the flood. Table E-2 lists the methods described in this document.

Before the appropriate method of computation is selected, the designer should establish the need to compute peak flow or a complete hydrograph. The following guide will assist in the selection process.

TABLE E-1
ALTERNATIVE METHODS OF FLOOD AND WATER LEVEL CALCULATIONS

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

TABLE E-2 COMPUTATION OF DIFFERENT CRITERIA FLOODS			
PEAK FLOWS		HYDROGRAPH	
Recommended Method for Design	Alternative Methods for Planning or Checking	Single Event Storm	Series of Storms
. Single station flood frequency analysis	. Regional regression equations	. 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	. Index Flood method		
. Combined Individual and Regional Analysis	. Watershed Classification (MTO)		

<u>Notes</u>:

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;
- watershed is regulated;
- Hazel or Timmins flood computation is required;
- remedial measures or watershed requirements warrant it.

Hydrograph simulation by computer analysis 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, (50 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 regulation effects due to manipulation of storage or flow within the drainage area;

iv) Watersheds for which the criteria remain the Hazel or Timmins flood as currently defined; and

v) Where different watershed 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, (for a description of the Ontario Hydrometric Network see report by Dillon, (1996). 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 cover less than a 10 year period, the records should not be used in the frequency analysis.

Where no gauging station data are available for analysis, a regional frequency study should be undertaken to determine the peak flow. For planning studies or for checking previously computed flows, three peak flow computation methods developed for Ontario conditions are presented in Table E-2; (i) Regional Frequency analysis, (ii) Index Flood method, and (iii) Watershed classification method, developed by the Ministry of Transportation, Ontario.

The Regional Frequency analysis for Ontario, originally undertaken in 1986 by Cumming Cockburn Ltd for the first Flood Plain Management Technical Guideline document, was based on pre-1986 streamflow data and therefore will require updating to incorporate the more recent streamflow data. Similarly, the Index Flood method report prepared by Moin and Shaw (1985), reproduced in the first addition of the Technical Guideline document, will require an update to reflect the present data base. The third method, based on Watershed Classification, is being currently updated by MTO and should be available in early 1997.

The above methods should not be used in urban watersheds, or for Hazel, Timmins and Historical flood computations.

Table E-3 summarizes the recommendations on the use of single station data and regional frequency analyses for (primary) instantaneous and (secondary) mean daily peak flows.

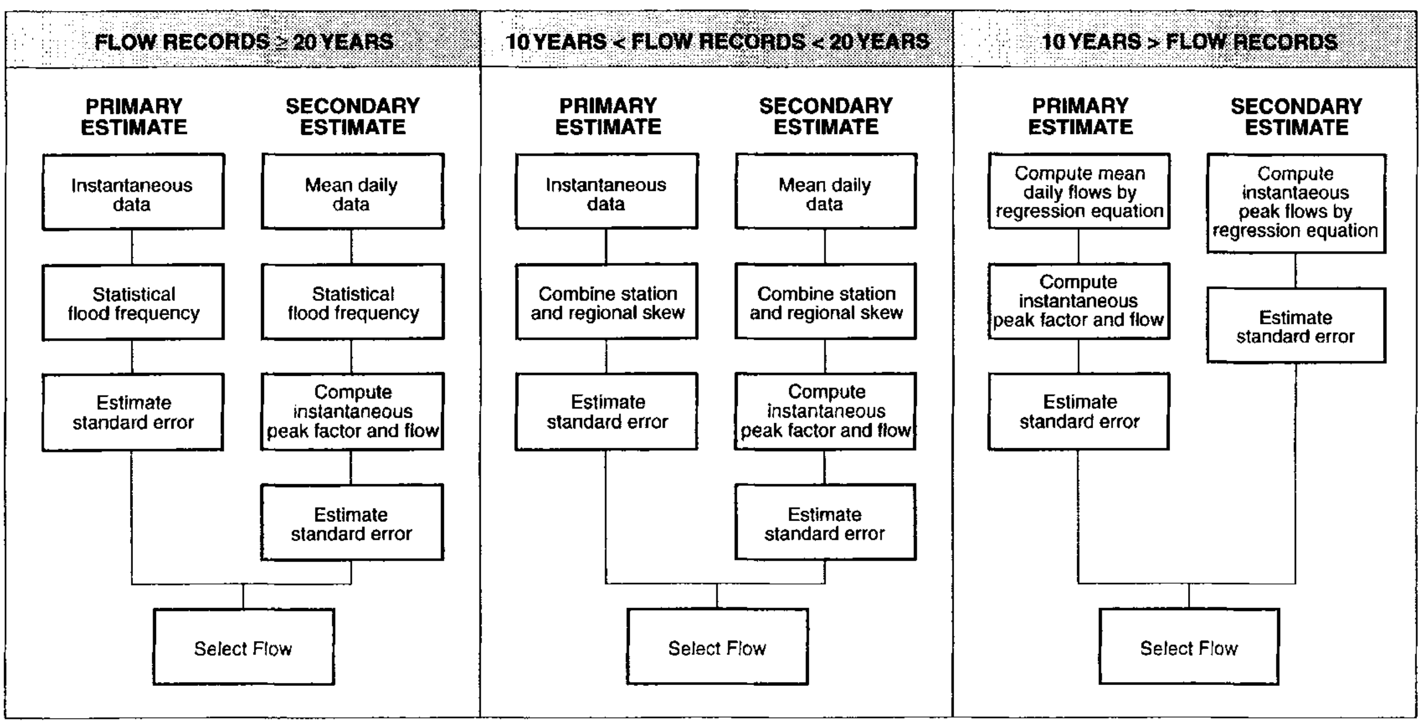
Where the user has the choice of computing the flows by hydrograph and the peak flow methods, the computer model results should be checked by comparing the results with flows obtained from Regression, Index Flood or Watershed Classification methods. Normally, the results should be within one standard error. However, as indicated, the Regression and Index Flood methods presented in the first edition of the Technical Guideline document are out of date and unless these methods are updated, the comparison would not be valid.

When computing the peak flow with the three methods listed for checking results, the outcome could show a wide range of flows. Particular attention should also be given to those situations where parameters of the regional frequency relationship fall near the limits of range and applicability.

Hydrograph simulation by computer analysis should be carried out to compare with the results obtained from Regional Frequency Analysis. The designer should use the standard error computation and considerable amount of judgement before selecting the return period peak design flow.

Results of a single station flood frequency analysis may be used to calibrate the computer model.

Table E-3: Summary of 100 Year Primary and Secondary Flood Peak Computation



NOTE: Compare standard error of estimate for each technique and use engineering judgement to select peak flow. Intantaneous peak flow data are preferred where available and are used for primary peak flow estimates. Secondary estimates are based on mean daily data used in conjunction with ratios of instantaneous and mean daily peaks.

2. HYDROLOGIC MODELS

2.1 Types of Hydrologic Models

Hydrologic modelling is an important tool for estimating flow hydrographs, including peak discharges, for flood plain management 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.

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

There are two main approaches to mathematical modelling. The first is to design a deterministic (or parameter) model 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; or one that uses a series of processes of the system. The second major approach to simulation is to determine the statistical parameters describing the response of the system, and to use these statistics to generate a record which is statistically indistinguishable from the measured record. This is called stochastic simulation.

For the most part, the models used in relation to flood plain 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; and
- rural versus urban.

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 storages, ground water storage, channel storage, and snow pack 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 streamflows 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.
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. In recent years, 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. One such model, the APIC model (API Continuous model) was developed by Walter T. Sittner of the U.S. Weather Bureau in 1969. MacLaren Plansearch calibrated it for some watersheds in Ontario for the Streamflow Forecast Centre (MacLaren Plansearch, 1982). Also a continuous version of GAWSER (Ghate and Whiteley, 1982) was developed by Harold Schroeter for the MNR Engineering Branch. Both of these models use carryover files for continuous mode. The GAWSER model is much more complex than the APIC model and as a result, also requires seasonalization of the parameters in order to run in continuous mode. Many of the parameters used in complex deterministic models vary throughout the year. Therefore, files have been developed to assist with startup of these models at any time during the year. These files are also required for running GAWSER in continuous mode.

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 file 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.

In practice, the state-of-the-art of modelling requires that caution be exercised in taking this approach.

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, etc., 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.

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 of 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 and has been tested on watercourses in the Metropolitan Toronto area.

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, for example STORM (U.S. Corps of Engineers, 1976, 1977), using average values of parameters such as maximum infiltration rate across the entire watershed. Other models, for example the HYMO-based model (Williams and Hann, 1973), are distributed in the sense that they permit the discretization of a watershed into a number of subareas. 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. The most truly distributed models are those which allow discretization of an individual subarea with simulation of the rate of precipitation falling within each discrete block. Such models have been used in research studies, but have not to date been applied in practice.

Over the last decade, super computers have been used for parallel programming where many identical computations can be performed simultaneously. Using this computing power on 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. Streamflow calculations from a very large number of segments of a watershed would be routed overland and through the ground into the streams. The anticipation is that using smaller and smaller pieces of the watershed to perform the calculations would produce a highly distributed model and allow for increased accuracy in both the timing and quantity of water moving through the watershed. The accuracy of the input data for these models continues to be a major stumbling block.

Currently there is a focus on applying these techniques to microcomputers and using them where there is insufficient streamflow data to justify other models.

Urban and Rural Models

The earliest computerized hydrologic models, for example the Stanford Watershed Model IV (Crawford and Linsley, 1966), 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, such as the EPA Storm Water Management Model (SWMM) (Huber et al., 1982), 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 plain 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 SWMM, 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 within recent years 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 state-of-the-art. As an example, the INTEROTTHYMO-89 (Wisner and Choon-Eng, 1984) and MIDUSS (Smith and Falcone, 1984) models contain two distinct unit hydrographs for use in urban and rural areas. The urban component uses a single linear reservoir with a storage delay constant defined in terms of watershed characteristics. Model use indicates that 200 ha may represent the upper limit of urban subwatershed application, due to the increasing significance of runoff lag effects. The rural component uses an n-linear reservoirs conceptualization first suggested by Nash in 1957 in his paper on instantaneous unit hydrograph. GAWSER (Whiteley and Gate, 1977) model has also added an urban algorithm to simulate urban land use.

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 such as DRAINMOD. These field-scale effects 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 such as GAWSER which 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 current 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, such as HSP-F, parameters related to overland flow length, slope roughness, infiltration and soil moisture storage can be modified to reflect altered practices. In the case of a simpler model such as INTERHYMO-89, based on the S.C.S. runoff method, the average Curve Number (CN) can be recomputed to reflect a different make-up of land use.

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 rainfalls, 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 occurs. If combined with rainfall, forest cover can be a liability rather than a flood control asset. Maintaining dissimilar land uses over a watershed now 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 would appear to be those such as HSP-F and the National Weather Service Model which 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 storages and infiltration rates between forested and open lands must also be correctly modelled.

2.2 Computational Procedures Used in Models

A description of the computational techniques in general use is given below. It is intended that this will assist the user in better understanding the individual models listed. In addition, it should also assist in assessing new models which will inevitably be developed in the future.

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 subsurface 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 of the currently utilized hydrologic models employ this "classic" concept in one form or another.

Snowmelt

A study of snowmelt modelling by MacLaren Plansearch (1984) sponsored by the Conservation Authorities and Water Management Branch indicated that the following three models are the most commonly used in current hydrologic practice:

i) 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. This approach is adopted in several operational models such as HSP-F, (Johanson et al., 1980) and (Anderson and Nichols, 1984), SSARR (U.S. Corps of Engineers, 1972) , and HEC-1 (U.S. Corps of Engineers, 1981). The equations used are generally based on the U.S. Army Corps of Engineers' 1956 Snow Hydrology Study.

Additional empirical equations are 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.

ii) Index models which identify snowmelt processes explicitly, but without the use of extensive meteorological input data. An example of this type of model is NWSRFS (Watt and Associates, 1979).

iii) Temperature index models in which air temperature is used as an indicator of snowmelt.

In some cases, additional factors such as variation in snow cover area are included in the equation. Models which incorporate such an approach are QUALHYMO (Rowney and Wisner, 1984a), USDAHL-74 (Holtan et al., 1975), and the UBC model (Quick and Pipes, 1977).

iv) GAWSER contains 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 in the edges.

Infiltration

The following infiltration methods are considered to be acceptable in hydrologic modelling:

i) Horton's equation;
ii) S.C.S. Curve Number Procedure;
iii) Holtan Infiltration Equation; and
iv) Green-Ampt Infiltration Equation.

Soil Moisture Accounting

Soil moisture is generally modelled by the use of 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 S.C.S. (1972) Curve Number procedure 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.

Groundwater Storage/Baseflow

Baseflow or groundwater response is generally not considered to be an important factor in determining the peak flows, which are of interest in flood plain mapping studies. There is, however, considerable evidence that rapid subsurface response can generate a large proportion of flows for certain watersheds. However, the current generation of models, particularly single-event models, would have limited abilities in reproducing such responses. In general, groundwater response can be modelled as a linear storage reservoir.

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;
- the Time-Area Curve method.

2.3 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. A general framework for model selection consists of the following steps:

1. Define the problem and specify the hydrologic information required to make flood plain management decisions.

2. Identify the available models and determine:
   - if watershed characteristics represented by model parameters govern watershed response in the intended application; and
   - if watershed and hydrometeorologic data required by models are available.

3. Specify the required performance of the hydrologic model:
   - the accuracy of the flow estimates; and
   - time frame available to obtain required hydrologic information.

4. Estimate the data preparation and computation costs for each model and compare to budget limits.

5. Rate candidate models and select.

Elaborate model evaluation is often not practical or possible due to a lack of published information on the accuracy of computational procedures. A preliminary screening of available hydrologic models was therefore carried out during the preparation of this Guide. The list of models selected which are conceptually sound and have demonstrated an ability to reliably estimate peak rates of streamflow in Ontario watercourses, are shown in Table E-4. The hydrologic features of the models are presented in Table E-5. These models are non-proprietary and may be obtained from the sources identified in the documentation for distribution fees ranging up to approximately $1,500.

Many of the models listed in the two tables are also described in the Provincial Urban Drainage Design Guidelines. These models, HYMO, INTERHYMO-89, VUH HYMO, SWMM, OTTSWMM, STORM, are also recommended to calculate major and minor system flows required for urban drainage studies.

TABLE E-4
LIST OF HYDROLOGIC MODELS

Class Model	Name of Model	Comment
Continuous Simulation	HSP-F HWSRFS (Stanford Version) NWSRFS (Sacramento Version) SSARR USDAHL-74 GAWSER QUALHYMO TUNIS	- complex continuous models - simplified continuous model
Single Event, Rural Areas	GAWSER HYMO VUH-HYMO HEC-1 FLOOD2 INTERHYMO-89 MIDUSS	- generally for use in rural watersheds
Single Event, Urban Areas	DDSWMM (formerly OTTSWMM) SWMM STORM INTERHYMO-89 MIDUSS	- generally for use in urban subareas

<u>New Models</u>

It is expected that new models or recoded existing models will continue to appear on a regular basis for many years. The new models, based on acceptable or new procedures, will require approval by the Ministry of Natural Resources before application.

When a modeller proposes a new model or an improved procedure for use with an existing model, the procedure should first be tested on well documented demonstration watersheds. These special test watersheds and data sheets should be maintained by the Ministry of Natural Resources. In addition, the proposed procedures should be submitted for publication in a technical journal by the author so that his or her contribution to the state-of-the-art can be scrutinized by peers. These technical journals include CSCE, ASCE, Water Resources Research, Canadian Water Resources Journal, Journal of Hydrology, etc. A thesis or a paper presented at a conference is not considered to be a published paper. Only when the superiority of the new procedure is clearly established, will the Ministry consider its use on flood plain mapping projects. The final decision should be made by the Ministry's staff, or by an appointed outside specialist.

Consultation with hydrology and hydraulic specialists of other Ministries is encouraged during the review procedures.

The Ministry will consider acceptance for general use only those models which have been demonstrated to be superior to some of the currently accepted models. The advantages and disadvantages of the candidate models will be evaluated from both theoretical and practical points of view. In general, only non-proprietary models will be considered for acceptance by the Ministry.

2.4 Model Calibration

Calibration of hydrologic and hydraulic models is an important part of the flood plain investigation. Frequently, lack of adequate data to calibrate and verify the model or lack of funding and expertise is used as an excuse to avoid calibration and validation of the models.

Calibration of the 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/verification, the uncertainty of parameter values can be reduced if the model is calibrated on a hydrologically similar watershed. The verification of a model is equally important, as this procedure will assess how the calibrated model predicts a set of events not used for calibration. Generally, the more complex a simulation model, the more data and expertise that is required; this will invariably result in a more expensive study, with results that are not necessarily more accurate.

In general, it is not feasible to directly estimate all model parameters from the physically measurable characteristics of the watershed. It is, therefore, necessary to calibrate and verify 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.

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.

TABLE E-5
HYDROLOGIC MODEL SELECTION MATRIX

APPLICATIONS	MODELS												
	Flood2	HEC-1	HYMO	HSPF	NWSRFS Stanford	NWSRFS Sacramento	INTERHY MO-89	SWMM/DD SWMM	STORM	SSARR	USDAHL-74	VUH-HYMO	QUALHY MO
Processes to be Simulated													
Precipitation	●	●	●	●	●	●	●	●	●	●	●	●	●
Snowmelt		●		●	●	●		●	●	●	●	●	●
Infiltration	●	●	●	●	●	●		●	●	●	●	●	●
Evapotranspiration				●	●	●		°	●	●	●		●
Surface Runoff	●	●	●	●	●	●	●	●	●	●	●	●	●
Subsurface Flow		●		●	●	●				●	●		●
Data Availability													
Precipitation	●	●	●	●	●	●	●	●	●	●	●	●	●
Temperature	●	●		●	●	●		°	●	●	●	●	●
Other Met. Variables		°		●						°	E	°	
Flow Records	●	●	●	●	●	●	●	°	°	●	●	●	●
Design Criteria													
Frequency Based				●	●	●		●	●	●	●		●
Historical Event	●	●	●		●		●	●				●	●
Design Rainfall	●	●	●				●	●				●	●
Land Use													
Rural	●	●	●	●	●	●	●		●	●	●	●	●
Urban				●			●	●	●				●
Agricultural													
Drainage Courses													
Significant Reservoir Storage	●	●	●	●			●	●		●		●	●
Watercourse Routing	●	●	●	●	●	●	●	●		●		●	●
Complex Models				●	●	●		●		●			

Note: ° = Optional
E = Pan Evaporation

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 verification data would be appropriate. For a single event model, five events of significant magnitude is recommended, this will prove invaluable for testing the accuracy of the calibration and should not be over looked.

Model parameters can be obtained by either measurement from physical data, computation from available records of flow or selection based upon previous experience. The accuracy of flow records is largely dependent on rating curves at gauge sites; every effort should, therefore, be made to investigate 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 elevation during spring breakup, it is important to consider ice conditions that can cause false high flow readings in the rating tables.

Certain parameters such as drainage area, channel slopes and impervious fraction of the watershed can be measured directly from topographic mapping or other sources, such as a GIS data bank.

Other type 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 which must be estimated based upon general experience, or experience from using the particular 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, etc. 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 users confidence in the suitability of the model for the intended application.

F. WATER LEVEL COMPUTATIONS - OPEN WATER

1. GENERAL

This chapter outlines and compares hydraulic analysis and modelling techniques that are suitable for application to a wide variety of Ontario rivers to estimate water levels. These techniques generally find their application in flood plain management, flood control, and preliminary engineering and design projects. 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.

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 the most commonly used method in computing water surface profiles under the above conditions. The calculations are made by successive approximation as follows. For subcritical flow conditions, the calculations begin with a known water surface elevation at the start of a reach (or downstream boundary). A water surface elevation is then estimated for the end of the reach, and energy losses are calculated. 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. If there is a significant difference between the estimated and computed elevations, a new estimate is made and the computation repeated. 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.

In all cases, field measurements of cross sections, discharges, and water surface profiles of actual floods are necessary 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 must 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.

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'.

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:

$$I - O = dS/dt$$

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. 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

$$S = K\,[XI + (1-X)O] \quad \text{or} \quad O = (1/K)S - x(ds/dt)$$

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. 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.

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

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 over-simplification 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.

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 one or two dimensional. 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.

4.CHOOSING A HYDRAULIC MODELLING TECHNIQUE

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.

The results from a flood routing study will, of course, be dictated by the nature of the overall project. For example, if a building is being constructed on the flood plain that does not significantly affect the flood characteristics of the river, the engineer will be concerned with the peak level of a flood hydrograph at the construction site. He may also be interested in how long the flood will be above a certain level, in which case he will need to know the shape of the stage hydrograph.

Similar information with respect to flood discharge hydrographs will be required when designing a spillway for a reservoir. Here the engineer may be concerned principally with the rising part of the discharge hydrograph and with its shape near the peak. If, however, alterations are being proposed, such as a flood damage reduction scheme, then a knowledge of peak levels and discharges from floods, and possibly the associated hydrographs, will be necessary at the improvement sites and downstream.

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 may be 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 the 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. Fortunately, it is usually possible to obtain general information on a river from topographic maps. 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.

Fortunately, many rivers 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 has to be built or discharge hydrographs for previous floods have to be derived at the upstream section of the reach from some hydrologic model.

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 plain 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.

There are numerous computer models available for backwater and flood routing computations. These models have been developed over the years based upon the previously described methods, and all have been tested and calibrated on real streams. A brief overview of several different hydraulic models with different capabilities and levels of complexity are presented in Table F-1.

Having regard to the foregoing discussion, the choice of model should be based upon the available data and information on the river, and the desired amount and quality of estimates. Additional factors that might be considered include:

- level of complexity (data input and interpretation of results)
- flexibility
- experience of user
- ease of calibration
- raining requirements.

Obviously, no model satisfies all requirements; a general rule should be to select the simplest model that provides the necessary information.

Table F.1: Hydraulic Model Selection Matrix

	TYPE OF FLOW						HYDRAULIC EQUATIONS				RESULTS								
	Gradually varied flow	Steady	Unsteady	One-dimensional	Sub-critical flow	Super-critical flow	Energy Equation	Manning's equation	St. Venant's equation	Continuity	Water surface profile	Velocity profile	Bridge structure	Flow hydrograph	Water elevation hydrograph	Sediment Transport	Flow distributions	Spill	Ice
Standard Step Method	●	●		●	●	●	●	●			●	●							
Direct Step Method	●	●		●	●	●	●	●			●	●							
Transform Curve Method	●	●		●	●		●				●	●							
WSP2	●	●		●	●		●	●			●	●	●				●		
HEC-2/HEC-RAS	●	●		●	●	●	●	●			●	●	●				●		●
USGS	●	●			●		●	●			●	●	●						
SHP	●	●	●	●	●		●	●	●	●	●	●							
FLOW ID	●	●	●	●	●	●		●	●	●	●	●		●	●				●
EXTRAN	●	●	●	●	●	●		●	●	●	●	●		●	●				
FLDWAV[1]	●	●	●	●	●	●		●	●	●	●	●	●	●	●				
HEC-GVF	●	●	●	●	●	●		●	●	●	●	●		●	●				
MOBED	●	●	●	●	●	●		●	●	●	●	●		●	●	●		●	
FERNS	●	●	●	●	●	●	●		●	●	●								
FLOW I-D[+]	●	●	●	●	●	●	●		●	●	●	●							
WSPRO	●	●		●	●	●	●	●			●	●	●				●		
SMS[2]	●	●	●	●	●	●	●		●	●	●	●	●			●	●		
UNET	●	●	●	●	●	●		●	●	●	●	●	●				●		

	INPUT DATA										MISC.			SENSITIVITY				COST			USAGE							
	Cross-section geometry	Bridge geometry	Weir and diversions	Flow and flow regime	Flow hydrograph	Water elevation hydrograph	Initial water elevation	Manning's "n"	Loss coefficients	Lateral Inflows	Tributary profile	Multiple profile	Automatic calibration	Manning's "n"	Loss coefficients	t	n	Low Medium	Medium	High	Applied in Ontario	Widely Applied	Documentation	Program support	Required training	Low Ease of Use	Medium Ease of Use	High Ease of Use
Standard Step Method	●			●			●	●	●					●	●			●				●	●			●		
Direct Step Method	●			●			●	●	●					●	●			●				●	●			●		
Transform Curve Method	●			●			●	●						●				●			●		●	●	●		●	
WSP2	●	●		●			●	●	●			●		●	●				●				●	●	●		●	
HEC-2/HEC-RAS	●	●		●			●	●	●		●	●	●	●	●				●		●	●	●	●	●			●
USGS	●	●	●	●			●	●	●	●		●		●	●				●				●	●	●		●	
SHP	●			●	●		●	●		●				●		●	●		●				●	●	●		●	
FLOW ID	●		●		●	●	●	●		●	●			●		●	●			●	●		●	●	●			●
EXTRAN	●		●		●	●	●	●		●				●		●	●		●		●		●	●	●			●
FLDWAV[1]	●		●	●	●	●	●	●	●	●	●		●	●	●	●	●		●		●	●	●	●	●			●
HEC-GVF	●				●	●	●	●		●				●		●	●			●			●	●	●		●	
MOBED	●		●	●	●	●	●	●		●				●		●	●		●		●		●	●	●			●
FERNS	●	●	●				●				●	●									●						●	
FLOW I-D[+]							●				●	●																●
WSPRO	●	●						●	●		●	●	●	●	●				●		●		●	●	●		●	
SMS[2]	●	●	●		●	●	●				●	●								●			●	●	●			●
UNET	●	●	●			●	●	●			●	●		●	●				●				●	●	●		●	

1 formerly DAMBREAK and DWOPER

2 includes RAMZ, FESWMS and SEDZD interfaces

\+ to be confirmed

Computer models continue to be refined and developed. In addition, a simplified manual approach based, say, on the Standard Step method, may be preferable in some cases to the use of a complex computer model. A simplified approach allows a much closer scrutiny of input data, closer involvement of the engineer in the computations, and more direct application of experience and judgement. Table F-1 represents a decision matrix designed to summarize the applicability of typical models. Ultimately, the selection of the appropriate model must be left to the judgement of the individual professional. Any model based upon the principles discussed in this chapter, and which can be calibrated and verified by the user, is an acceptable choice.

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 streamflows, 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 floodline around a lake. In such cases, a process of reservoir routing may be applied, in the absence of direct water level recordings.

The most commonly 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 is also used by some hydrologic models such as INTERHYMO 89, GAWSER and QUALHYMO in their respective reservoir routing routines.

In floodline mapping studies, the analysis of flow-frequency relationships require the conversion of regulated flows to de-regulated or natural flows. If the streamflows 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 streamflows to natural conditions prior to undertaking a flood frequency analysis. When the degree of flow regulation is not negligible, results from single-station flood frequency analyses can not 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 under-estimate 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 reconversion 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, etc.

Given adequate data, such as diversion flows, reservoir stages, outflows and stage-storage curves, it is a simple if tedious 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 de-regulated, 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.

6. EFFECT 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 must 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 floodlines around lake or reservoir perimeters; and
- 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 may 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. The HYDSTAT computer program, available from the Ministry of Natural Resources, estimates the joint probability of two related events from observed data.

If a significant amount of regulation exists in a lake, 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 (1 hour or so) will be needed in order to define the maximum levels reached under various floods. Most hydrologic models provide the means to perform these computations.

Water levels for the Great Lakes were published by the MNR Conservation Authorities and Water Management Branch in 1989. The report lists flood levels 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.

7. 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.

In relatively simple cases, such as where a hand calculator Standard Step method is being applied, these losses can be independently estimated and entered into the computations. The coefficient for expansion losses is a function of the change in velocity head between cross sections and is adequately discussed in most textbooks. 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, a computer model such as the HEC-2 or the more recent 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.

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. Some computer models include special routines for this purpose, but the most common approach is to modify the channel cross section to reflect the encroached section, and re-compute the flood profile. 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.

It is a current policy of the Ministry of Natural Resources 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.

8. MODEL CALIBRATION

Having selected an appropriate computer 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 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.

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.

Flows and water levels may be obtained from stream gauging stations operating in the study area. Environment Canada, Ontario, the Ministries of Natural Resources and Environment and Energy, Conservation Authorities, and various private 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.

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, but it is usually more practical to resort to a nearby stream gauge on the same river.

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

9. 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 are usually found to be the most sensitive parameters. However, in particular 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 floodlines.

G. WATER LEVEL COMPUTATIONS - ICE JAMS

1. INTRODUCTION

The potential for ice jam flooding is present in Southern Ontario during the three month period between freeze-up in late December and break-up in late March. In Northern Ontario, this period usually extends over a five month period beginning in late November. During these periods, flooding may result from accumulations of frazil slush, or fragmented sheet ice or both.

Many ice jam prone areas 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 (5 years) floods. The majority of problems 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 resulted in flooding.

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 underturning 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.

The shear force of the water flowing beneath the accumulation and the cover weight acting in the direction of the slope of the surface increase as the accumulation grows upstream. These forces are transferred through the accumulation to the river banks 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”. 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. This process continues as the cover advances upstream, with the 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 river banks after the release or melt of an ice jam event.

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.

During the 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 G-1. During this period of time the pack remains unconsolidated.

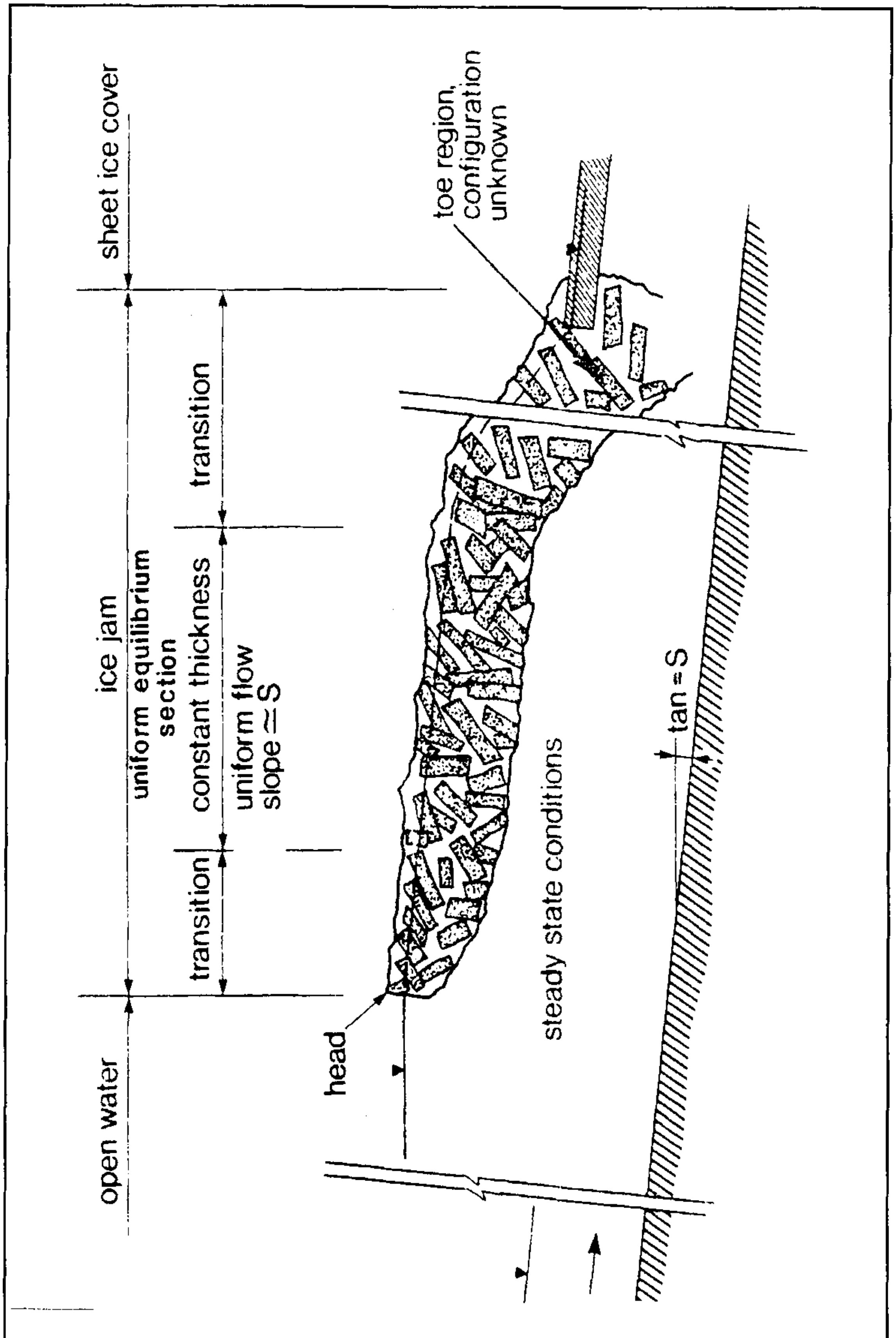
Some contact with the bed (grounding) is common for a short section near the toe of the jam. In a grounded or “dry” ice jam, this blockage extends to the bottom across the full width of the channel. The flow obstruction is almost complete and exceptionally high water levels can result with the majority 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.

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 equilibrium section of the jam.

The upstream edge of a stationary ice sheet (shown in Figure G-1) 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 the 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.

Figure G-1: Schematic Illustration of an Equilibrium Ice Jam



Reproduced from *"River Ice Jams: Theory, Case Studies, Applications"* S. Beltaos, 1983

2. ICE JAM COMPUTATIONS

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 the depth is the maximum depth that can be caused by a floating ice jam.

The forces acting on ice jams have been evaluated by a number of investigators to determine the equilibrium thickness. Based on this evaluation, two types of accumulation, termed "narrow" and "wide", have been identified in the literature.

In the "narrow" case, the internal stresses acting within the jam decrease in the downstream direction. 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.

In the "wide" case, 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.

The accumulation thickness of the floating ice jam at this location prevails throughout the equilibrium reach of the jam. As mentioned above, the depth is a maximum in this reach and the accumulation thickness is given by;

$$\mu\rho_i[1-s_i]gt^2 + 2Ct = B\,[\tau_i + P_i g t S_W]$$

where:

μ = internal frictional resistance coefficient

ρ = density of ice

s_i = Pi/P = specific gravity of ice 0.917 gm/cm^3

g = gravitational acceleration (9.8 m/sec^2)

t = ice-jam equilibrium thickness (m)

C = cohesion of the ice cover = 0 at breakup

τ_i = shear stress of water acting on the jam
= $\rho g R_i S_W$

S_W = slope of the water surface gradient (m/m)

R_i = hydraulic radius beneath the ice cover (m)

B = width of the equilibrium accumulation
(channel width at bottom accumulation)

A number of methods have been advanced to calculate the ice jam stage within the equilibrium reach of these "wide" channel jams. In general, it is assumed that:

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 following analytical models are all based on the same "wide" channel assumption and governed equations. The only variation in the models is in the representation of channel roughness.

Beltaos - "Detailed" Approach

Beltaos describes both a detailed analysis and a simplified one which may be sufficient for many engineering calculations. To make clear the nature of the simplifications made in the latter approach, the detailed method will be described first.

In the detailed approach, the channel section is initially taken to be divided into upper and lower subsections, with flow in the upper section controlled by the average shear stress on the underside of the jam, and in the bottom sections by shear stress on the river bed.

The computation begins with an assumed value of break-up ice jam thickness and proceeds to evaluate the hydraulic radius for the ice-controlled section.

The roughness of the bottom of the jam is then computed using equations derived from the author's study of observed thickness and roughness measurements (employing caution in situations of large relative roughness). This roughness is combined with that of the bed to determine the hydraulic radius of the bed-controlled section (Rb) and the composite Darcy resistance coefficient (fo). The depth (h) beneath the jam is:

$$h = Ri + Rb$$

and the ice-jam stage (hj) is:

$$hj = h + 0.917t$$

The discharge for this condition is based on the bottom width of the ice accumulation (W) using:

$$Q = Wh([4/fo]\ ghS)^{1/2}$$

The procedure is repeated for several other values sufficient to define a curve of thickness vs. discharge, as shown in the example in Figure G-3.

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.

Beltaos "Simplified" Approach

The calculation of break-up ice jam stage may be conducted quickly by using this approach. In the simplified method, it is further assumed that the internal frictional resistance (() and ice jam resistance factors (Fo and Fi) do not vary from site to site.

The maximum possible depth for the stage of a floating ice jam at break-up is estimated from the dimensionless functions shown in Figure G-2. These functions are based on data from ice observations at a number of sites across the nation, including the Thames and Grand Rivers in Ontario. The upper graph plots the data and the bottom graph plots the monotonic function derived from the data.

The dimensionless depth and discharge are each based on the average bottom width of the ice accumulation, slope and unit discharge (B, So and q). The peak ice jam stage for a given discharge is computed by (a) calculating (, (b) finding a corresponding from Figure G-2, and (c) calculating the ice-effected depth from:

$$Hj = SoB$$

Through this analytical approach, an upper envelope of peak break-up stages (such as shown in Figure G-3) can be computed for a range of flows at a site which is assumed to be in the equilibrium portion of an ice jam. The upper envelope shown in Figure G-3 generally exceeds the observed stages as it should. Also shown on Figure G-3 are the results of a "detailed" modelling approach.

The detailed approach allows for more account to be taken of the variations in the parameters involved than does the simplified approach (e.g. channel width variations), but more assumptions are made to obtain this detail.

Overall, the "detailed" approach to determine the upper envelope of ice jam stage appears to provide a more reasonable approximation of observed stages than the "simplified" approach in the few example cases in which the two have been applied. In the simplified approach, however, the author notes that special constraints such as low flood plains and bypass channels must be determined from careful field inspections and used to modify the calculations along the lines used by Gerard and Calkins (see below).

Calkins

The ice jam equilibrium thickness equation has also been used by this author in an approach which is generally the same as Beltaos. However, a limiting discharge concept is included in his formulation for the main channel of rivers having significant overbank spillage into the flood plain. 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. 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 field reconnaissance evidence.

Figure G-2: Dimensionless Depth Versus Dimensionless Discharge: River Ice Jams in Equilibrium

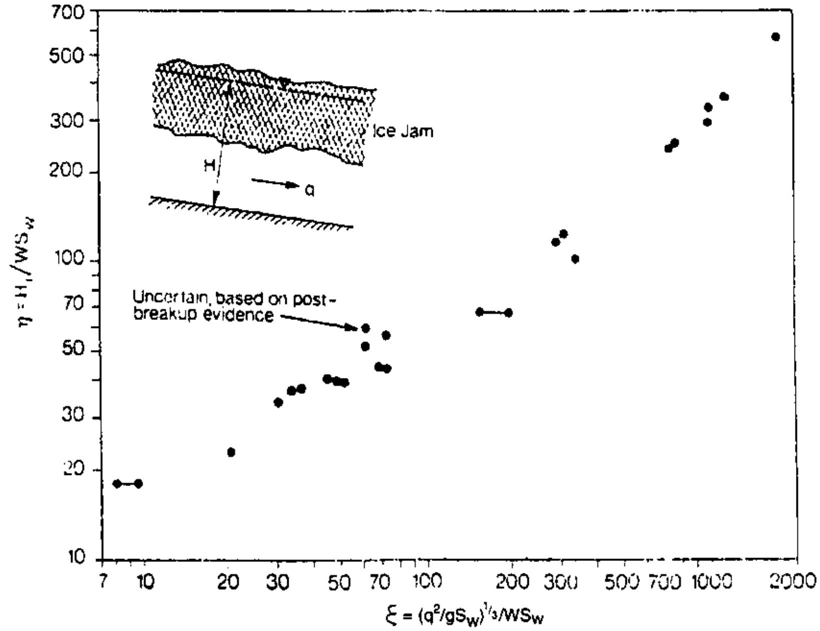


FIG.(a) DIMENSIONLESS DEPTH vs. DIMENSIONLESS DISCHARGE FOR RIVER ICE JAMS IN EQUILIBRIUM (Data)

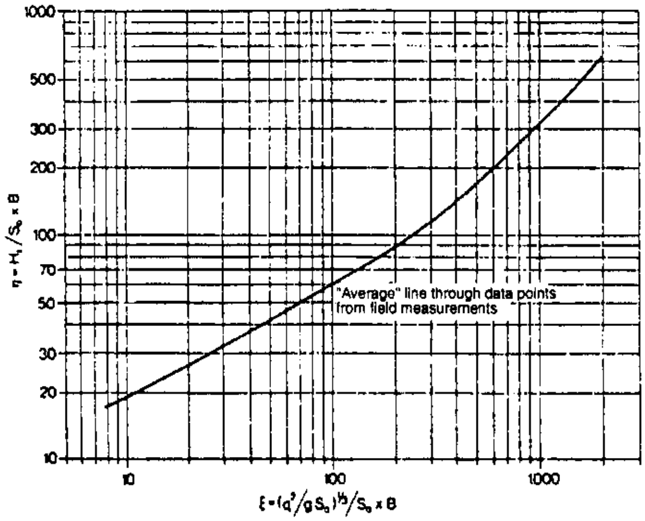
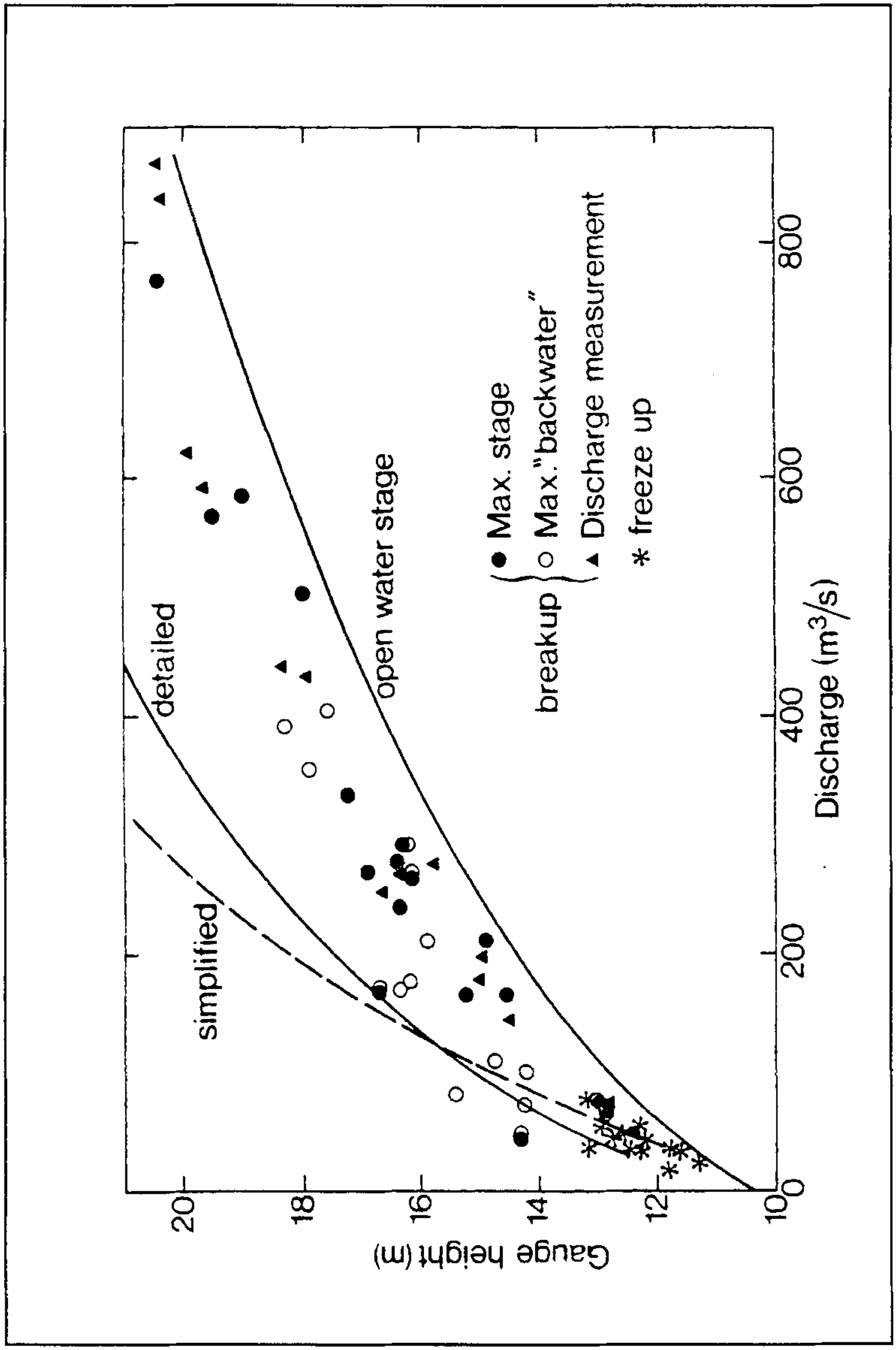


FIG.(b) DIMENSIONLESS WATER DEPTH DUE TO AN EQUILIBRIUM, FLOATING, WIDE CHANNEL JAM vs. DIMENSIONLESS DISCHARGE

Reproduced from *"Lecture Notes on Ice Jams"* S. Beltaos, 1984

Figure G-3: Freeze Up and Break Up Stages Versus Discharges; Thames River at Thamesville Ontario



Reproduced from *"River Ice Jams: Theory, Case Studies and Applications"* S. Beltaos, 1983

Gerard

The approach advanced by Gerard is similar to Beltaos “detailed” approach, except in the roughness formulation. In his approach, the absolute hydraulic roughness is employed in developing the depth of flow beneath the ice jam instead of friction factors or Manning’s ‘n’.

The approach is based on reach averaged geometry and culminates in a stage-discharge relationship. Trends for possible leakage spill to the flood plain must be evaluated outside of the model, and the author cautions that deep channels are required to obtain reasonable results when using this method for very rough ice covers.

HEC-2 (Ice Option)

The Hydrologic Engineering Centre has modified the HEC-2 backwater model to account for the presence of floating ice covers. Program modifications take into account the reduction in flow area, increased wetted perimeter and roughness caused by an ice cover. The program assumes steady state conditions, a stationary cover, and a floating cohesionless jam at break-up for computation of ice cover stability.

For determining ice jam water levels, the user must provide:

ï ice thickness;
ï Manning’s roughness coefficient for the ice cover;
ï specific gravity of the ice;
ï the discharge; and
ï information on the downstream control.

The ice thickness is assumed constant between each cross section and is estimated by the user. For equilibrium ice jam situations, it is appropriate to calculate the thickness for representative sections from the equation presented earlier in this chapter.

The ice cover roughness must also be stipulated by the user. Manning’s ‘n’ values are reported in the literature to range from 0.01 to 0.04 for single layer ice sheets and from 0.05 to over 0.1 for ice jam situations. The range is quite broad but again can be specified for a given condition using other analytical means (e.g. Beltaos “Detailed”).

Information on the downstream control or starting water level is also a critical input parameter for HEC-2. In an ice-jam situation, the downstream control is the blockage itself about which little is usually known. In cases where the control elevation is well defined (e.g. the crest of a dam) the process is relatively straightforward. In most cases, however, the starting water level must be generated by other analytical means.

Overall, the model requires the user to derive and input all of the ice parameters in an ice jam situation. Where downstream conditions, stage observations and ice thickness measurements are known or accurately estimated, the model appears to provide reasonable simulations of ice-effect profiles.

The HEC-2 ice option becomes extremely flexible in determining the stage when the flow 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 to HEC-2 to evaluate the conditions when flood plain flow occurs.

RIVJAM MODEL

A numerical model RIVJAM (Beltaos 1996) was developed at the National Water Research Institute as a tool for studying the configuration of ice jams and the water levels caused by river ice jams under a variety of conditions. The model computes longitudinal profiles of the ice jams in natural streams and the resultant water levels. Steady state conditions and one-dimensionality are assumed, which means that the flow and the jam configuration do not change with time; the jam thickness is assumed to be uniform across the river, though it may vary longitudinally.

RIVJAM solves two differential equations with two unknowns, jam thickness and water depth. The output of the model consists of the longitudinal variations of the jam thickness, flow depth and water level. The concept of flow through the voids of the jam is incorporated in the model. Past application of the model included determination of the safe height of a proposed bridge, comparison of different dyking schemes to alleviate ice-jam damages, assessment of the channelization, dredging and erection of ice retention structures, and changes to the regime of a river caused by climate changes and river regulation.

3. SUMMARY OF ICE JAM COMPUTATION METHODS

The following methods are suggested for consideration:

- Beltaos - “Simplified”
- Beltaos - “Detailed”
- Gerard
- Calkins
- HEC-2 Ice Option
- RIVJAM

The Beltaos “Simplified” approach provides the simplest method for quickly evaluating the range of possible ice-effect levels. One of the other detailed approaches, however, would appear more appropriate if studies continue beyond a simple, reconnaissance stage.

There are also a variety of limits which can be evaluated to estimate an upper bound on the maximum ice-effect stage. For example, in instances where wide flood plains provide considerable relief through “leakage”, the approach of Calkins can be used for establishing maximum ice jam thickness and flow in the main channel.

The HEC-2 ice option is a useful tool when flood plain flows are significant for showing reach-wide ice effects but it requires important input parameters to be derived from other analytical models or surveys (i.e., ice jam thickness and a good reconnaissance survey to determine relief channels, etc.). Similarly, the RIVJAM can assist in the study of water levels during ice jam conditions, however, at the time of the writing this Guide the model was still under testing and the user’s manual was only available in a draft form.

Overall, however, it must be noted that the above-described methods for determining ice jam flood stages have not been refined to the same extent as have models for open-water flood situations. There is still considerable ongoing discussion in the literature as to the merits of each modelling approach and, at present, no approach is clearly ‘better’ than another. There are still uncertainties and limitations to be resolved, and the assessment should always be based on careful analysis of historical information and detailed site inspection.

It is also worthy of note that the methods for estimating ice jam water levels have not been tested in a wide range of applications in Ontario, although evidence from other jurisdictions indicates that the simulation results can be reasonable. From the Ontario results, it can be shown that they can certainly be applied to establish upper bounds on the ice jam phenomenon, with correspondence with historical evidence resulting from site observations. Such observations may indicate the potential for some overbank spillage, and hence that the equilibrium depth cannot be reached at those sites.

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 levels may have arisen from;

a) flood peaks from the break-up of an upstream jam (attenuated effects or surge effects);

b) the backwater from a distant downstream ice jam;

c) evolving ice jams which release before building to their maximum flood depth at equilibrium;

d) ice jams which cannot be sustained beyond a given flow rate (i.e. an ice clearing discharge discussed by Beltaos);

e) ice jams which result in spillage over dykes or banks into the flood plain; or

f) ice jams which have their development limited by a limited supply of ice from upstream.

These conditions and their effect on water levels can be taken into account by careful field inspection, data review and analysis, 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. The 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 completely different at another site even on the same river, and are not amenable to such transposition.

4. STAGE-FREQUENCY OF ICE JAM FLOOD LEVELS

The frequency of ice jam flood levels can currently be estimated on the basis of historical information or by using analytical modelling procedures. The former may prove suitable where there is a long record of reliable ice stage estimates and the latter where there are few. Wherever possible, historical data should be used to check the results of analytically based approaches and vice versa for the historical approaches.

4.1 Historically Based Estimates

A number of 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 five to ten 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 jam flooding has been developed by Gerard and Karpuk. The methodology is as follows:

1. Draw together a complete history of ice jam flood stages from all possible sources (i.e., surveys, environmental evidence, interviews, etc.) and relate them to a reference stage, such as the zero discharge stage.

2. Examine topographic mapping, field surveys, air photography, etc. 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. 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 instruments).

4. Organize the information to display the stages identified from each source and their perception stages.

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. 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.

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 more likely than not, it won't. 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).

An example of this method is shown on Figure G-4, which indicates that this approach leads directly to a probability distribution of ice jam flood stages without a requirement 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. It requires that a reasonable number of years of observed levels be available at a floodprone 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. Given that these constraints are met, it provides a probability distribution which is consistent with other approaches for estimating ice stage frequency.

This approach requires the modeller 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 "desk-top" 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 perception stage information. Overall, such records may reduce the data acquisition phase, but, as mentioned above, should not replace the essential requirement for the modeller to evaluate the physical characteristics of the site and their implications on ice-effect stages.

4.2 Deterministic Estimates

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.

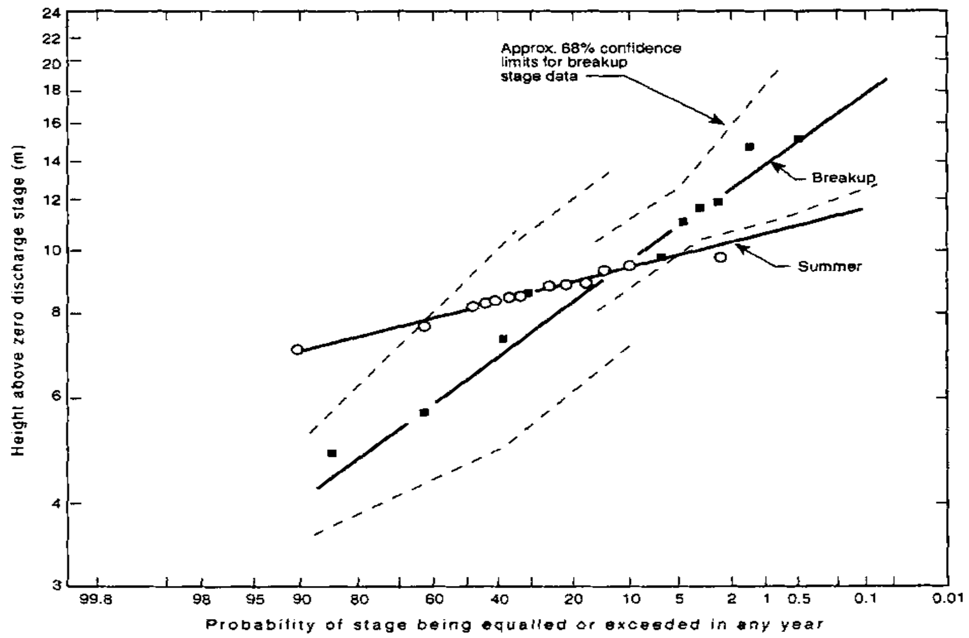
The analytical models described in the previous pages provide ice jam stage estimates as a function of streamflow. This flow data (unlike ice stage data) can be transposed to ice jam sites from non-jam areas and can be used to develop a probability distribution of ice jam stage.

The methodology for conducting this work has been outlined by several authors, most notable Gerard and Calkins. The procedure can be used with one or more of the analytical models outlined earlier and is as follows:

1. Develop stage discharge curves for ice-effected conditions. This includes the stage effect of a solid ice cover (a lower bound on ice stage) and open water conditions, as well as the equilibrium ice jam stage (an upper bound on ice stage) calculated by using the analytical approaches described previously.

2. Evaluate stage and discharge limits associated with each ice cover condition and incorporate the results in the stage-discharge curves taking into account the following notes:

Figure G-4: Probability Distribution of Annual Maximum Ice - Effect Stage and Open Water Stage



Reproduced from *"Probability Analysis of Historical Flood Data"* R. Gerard and E.W. Karpuk

- in river reaches with wide flood plains, environmental evidence (i.e., ice abrasion scars), as well as analytical estimates (Calkins), will indicate an approximate upper limit defined by leakage or overflow;

- 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; and

- the volume of ice required to develop an ice jam of sufficient length and thickness may also limit the stage (Calkins).

3. Derive an estimate of the maximum streamflow (observed, transposed or simulated) at the site during ice-effected periods for an extended number of years.

4. For the limited number of years for which historical ice stage data is available, 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 the maximum solid ice cover stage (from the stage-discharge curve for solid ice conditions) using the maximum ice-effect flow estimate. Similarly, determine 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.

   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, and use this to deduce a final compromise probability distribution between the upper and lower bounds described above using the relation:

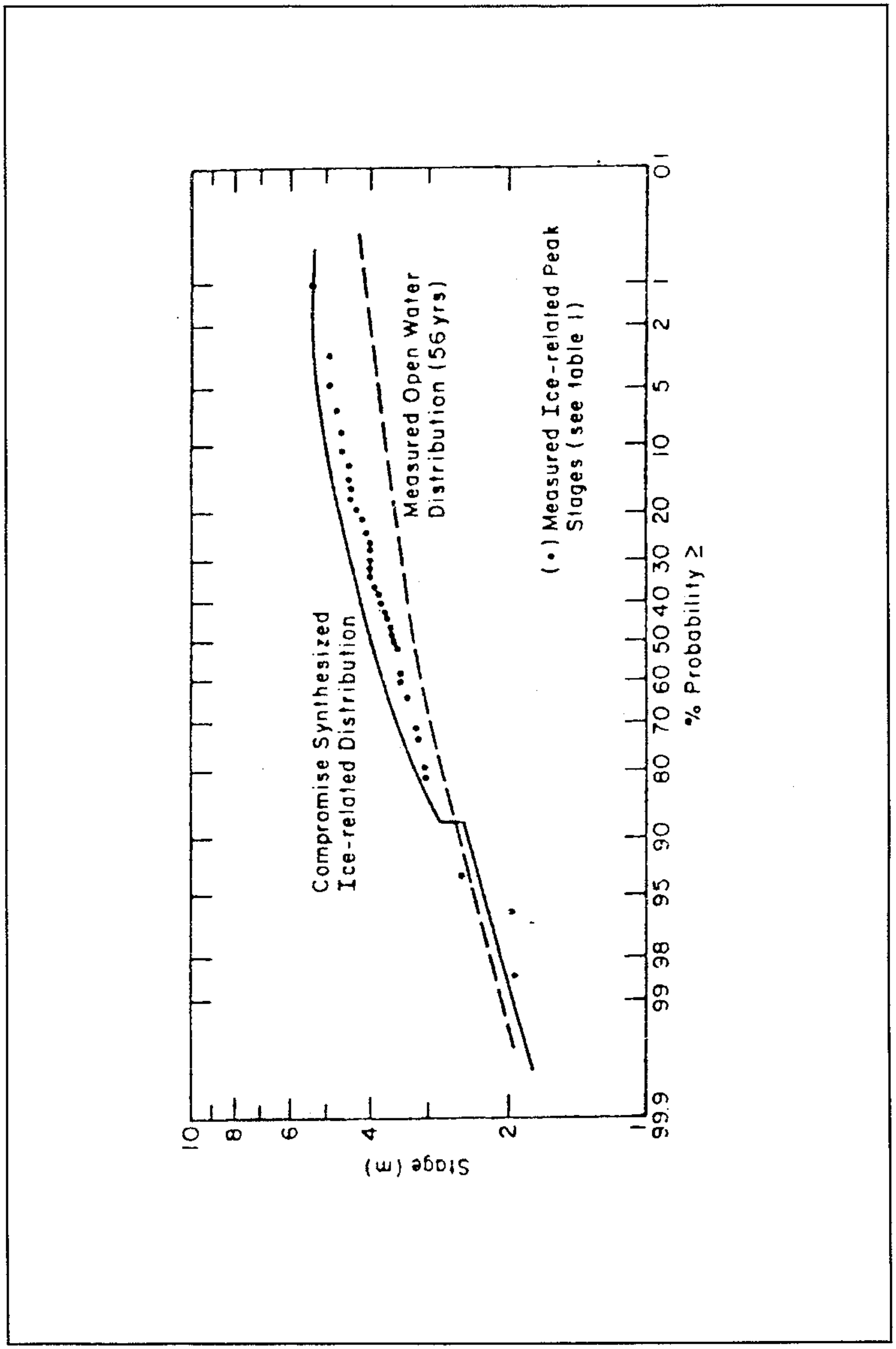
$$P = Pj + Pc$$

where 'P' is the probability of a stage being equalled or exceeded by ice effects, 'Pj' is the probability of a stage being equalled or exceeded by an ice jam, and 'Pc' is the probability of the stage being exceeded by a solid ice cover stage.

The final product of this work will be a stage probability distribution such as shown by the solid curve in Figure G-5. Also shown in the Figure, are actual ice-related stages measured at the site during 37 ice jam events in the 58 years of record. The agreement is quite good and demonstrates that this analytically-based deterministic approach can be used with some confidence.

As in the historical approach, this technique requires the modeller 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.

Figure G-5: Comparison of Synthesized and Measured Ice - Related Stage Probability Distributions



Reproduced from *"Ice Related Flood Frequency Analysis"* R. Gerard and D.J. Calkins, 1984

4.3 Annual Flood Stage Probability Distribution

The overall annual stage probability distribution is required for delineating the extent of flood plains in the Province, for estimating average annual flood damages or for determining the location/elevation of structures to be located outside the frequent effects of high water levels.

If the interest is simply in design stages such as the above, the open-water and ice-related stage-frequency distributions can be directly combined as with other mixed populations. As the two distributions are reasonably independent:

$$P_A = P_I + P_O - P_I P_O$$

Where:

P_A = the probability that a flood stage will be equalled or exceeded by either ice-related flooding or open-water flooding

P_I = the probability of the ice-related flood exceeding the given stage

P_O = the probability of open-water floods exceeding that stage

An alternative to combining the separate distributions to derive an annual curve is to choose 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.

If the evaluation also involves assessment of flood damages which differ significantly between open-water seasons and ice seasons, it is important that damage probability curves be developed from each stage distribution and then combined as above to determine the annual damage probability curve.

5. PROCEDURE FOR ESTIMATING THE FREQUENCY OF ICE-EFFECT STAGES

Figure G-6 presents a summary flow chart of the general procedure for estimating the frequency of break-up ice jam stages.

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 a result of ice jams, then ice-related levels need not be considered. If ice jamming has caused high levels, the following steps are required.

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-probability relationships.

Detailed river geometry is as important in the 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.

Once the field data is assembled, reviewed and prepared in a reference report, the frequency of significant ice jams 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 the frequency of ice jam levels.

If there is a long history of ice-jam 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), it is possible to go directly to the Historically Based approach for estimating the frequency of ice-jam stages (Gerard and Karpuk). If these qualifications are not met, an analytical procedure leading to a Deterministic Approach is required. This latter approach may also be used for stationary regimes with a long history of data (as shown on the flow chart by an optional arrow) to derive the best compromise estimates for upper and lower bounds on ice stage.

The study may only require a preliminary estimate of the ice jam stage-probability relationship. If there is some reliable data and the ice/river regime is statistically stationary, then the Historically Based approach may be used for this estimate.

The Deterministic Approach is used where there is limited ice-stage data or the river regime is statistically non-stationary (or will 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. 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 Beltaos "Simplified" method is appropriate.

If a detailed evaluation is deemed necessary (to account for channel width variations, for example) the Beltaos "Detailed" or similar detailed approaches would be appropriate.

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).

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.

Overall, these procedures give a deterministic estimate of the ice jam stage-probability distribution paralleling that of the 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 Historically Based approach is the most rapid because the analytical modelling procedures of the Deterministic approach are not required. Similarly, the 'Simplified" analytical method by Beltaos requires less effort than the "Detailed" approaches to develop a basic stage-discharge relationship for the site.

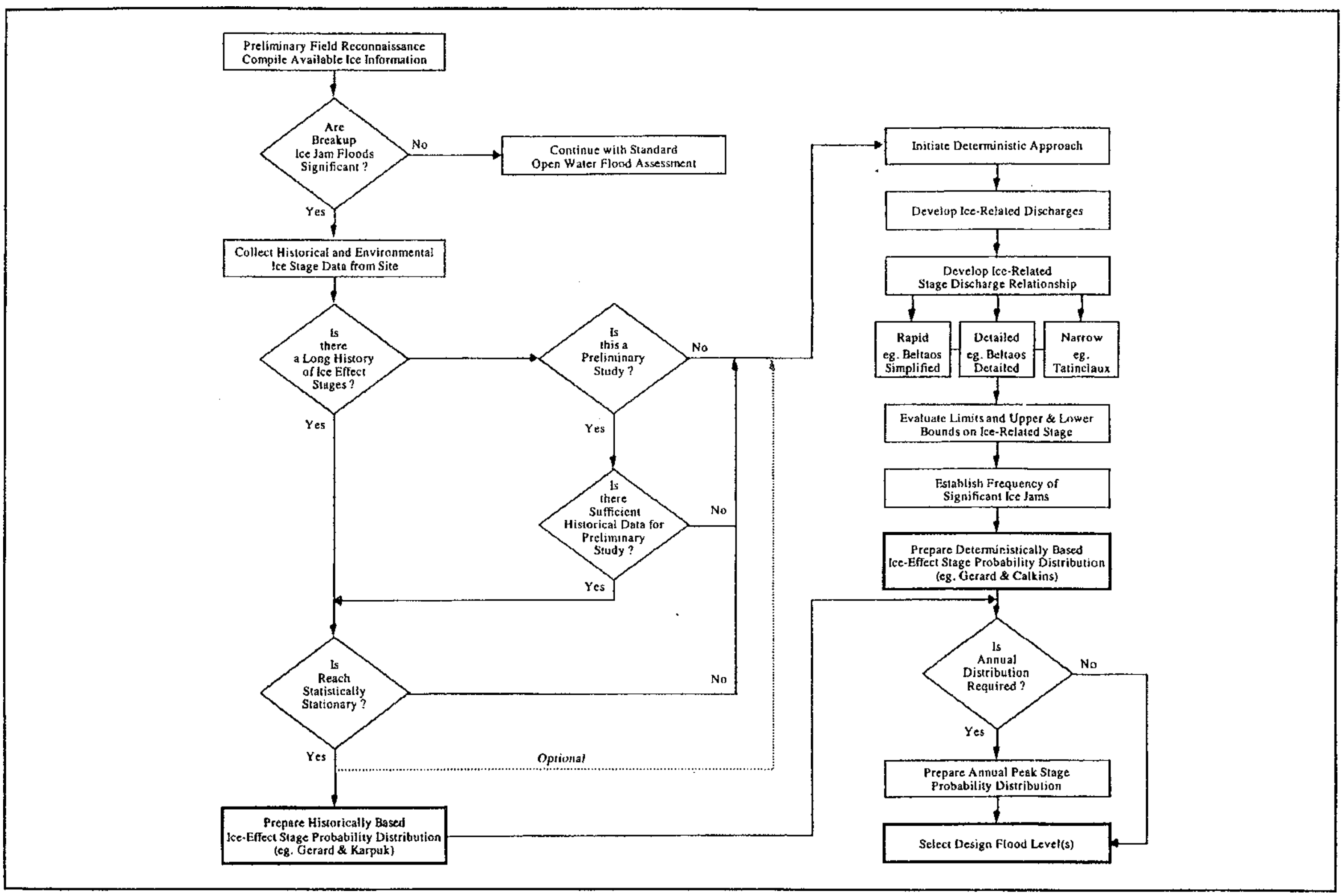


Figure G-6: Procedure for Estimating the Frequency of Breakup Ice Jam Stages

6. DATA COLLECTION FOR ICE-EFFECT LEVELS

Field observations and historical information on ice-related floods are required for forecasting 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 needs for ice studies are additional to current requirements.

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 - all tied to a reference datum. A reach length in the range of 20 times the width may be sufficient to adequately define the geometry;
- bed and water level profiles extending upstream and downstream of the site - particularly if ice jams in these areas are the cause of problems at the site; and
- identification of overbank spill areas or diversions, or obstructions to such diversions in the likely ice jam reach.

Detailed GIS topographic mapping is a definite asset, particularly if there is a possibility of spills or diversions around the ice jam site. With the extensive use of GIS, 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.

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;
- aerial and LANDSAT imagery; and
- stream gauge records (staff, crest or flow) and Water Survey of Canada ice condition notes.

Valuable data also resides with streamside residents, public works officials and government agencies. From 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 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-effect flood levels is given by:

- ice scars or abrasion marks (on trees, structures, drainage outlets), 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 analytical projections.

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, 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;
- freeze-up and break-up dates;
- descriptive observations at break-up (sequences, jam locations, thickness and strength);
- photographic record 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; and
- additional photographic surveys.

Also, it may be advantageous at some locations to:

- install new level gauges; and
- establish bench marks to assist in level monitoring in ice jam years.

If ice jamming does occur, as much quantitative work as can be conducted with the manpower resources at hand will be invaluable for later study. Particular focal points are;

- 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; and
- reconstruction of break-up sequence that led to the jam.

Post-jam observations and data collection can also provide highly valuable information:

- associated meteorological conditions;
- floe thickness along river banks, ice strength (qualitative);
- shear wall thickness, location, elevation; and
- high water mark elevations along the jam.

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 observers mind.

7. CONCLUSIONS

It is evident from the above discussions that a substantial commitment is 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; and
- careful evaluation of ice data.

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, it will be necessary to:

- generate peak winter streamflow data; and
- prepare ice-related stage-discharge curves.

Given that only limited experience has been gained in such assessments in Ontario, and that the ice-related site information and modelling approaches are not yet as refined as for open-water cases, it will likely be several years before this process is as streamlined as today's open water assessments. However, since techniques for assessing ice jam flood levels are available and are well documented, and ice-related flooding is significant in many rivers in the Province, it should not be long before these techniques are put to good use by consultants and regulatory agencies alike.

1. TECHNICAL REPORT

An important step towards meeting the goals of flood plain management is the completion of flood plain studies, which will provide communities and approval agencies with the necessary data base of clearly presented technical information. The following description summarizes the technical information required by approving 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 will be submitted to the Flood Plain Study Technical Review Committee. The report should present the studies in sufficient detail that specialists in this field can determine the adequacy of the work and its conformance to the procedures outlined in this document. 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 risk maps will be compiled. The technical report is not intended for wide distribution; a brochure summarizing the significant points will be prepared and made available to the public.

The draft submission should include the following material:

1.0 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.

2.0 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 control or prevention measures.

Describe the area to be mapped, the watercourse and its tributaries, watershed characteristics, climatic conditions and flood-generating mechanism.

3.0 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.

Flood Computations - Flood Frequency Analysis

Describe streamflow and rainfall groups and records. Information relevant to the conversion of regulated streamflows 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 an 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.

4.0 Hydraulic Analyses

Describe topography of the flood plain and the methods used for taking cross-sections. 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. Plots of all cross-sections must be shown, 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.

For lakes wind setup and wave estimates should be presented.

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.

Spill areas should be identified and the effect of spill should be commented on.

5.0 Flood Line Delineation

Describe mapping used, date of aerial photos, scale, contour intervals, accuracy and checking. 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.

6.0 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.

7.0 References

8.0 Exhibits

- location maps to identify study area;
- land use and soil maps;
- 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
- computer plot of flood profile.

9.0 Appendices

Input data and output summary should be presented in Appendices.

The different agencies and their roles and mandates are summarized below.

The position of the Province is that all public agencies exercising their planning authority shall have regard for the Provincial Policy Statement issued under Section 3 of the Planning Act as amended by Bill 20. Those policy statements address matters of Provincial interests including flooding and erosion hazards.

To assist public agencies in fulfilling this agreement, the Province identified the Ministry of Natural Resources to develop a technical guide for the calculation and mapping of flood and erosion limits of hazard lands.

As lead ministry responsible for administration of the provincial flood plain management program, the Ministry of Natural Resources is responsible for providing technical support for the implementation of the policy. This includes the responsibility for the development of a Technical Guide to provide direction on the definition of flood and erosion standards.

The Conservation Authorities or the Ministry of Natural Resources, where Conservation Authorities do not exist, will refer to the applicable flood plain mapping (approved under the Canada/Ontario Flood Damage reduction Program or successors thereto), for assistance and direction related to the implementation of the flood plain policies in the Provincial Policy Statement.

The Conservation Authorities will administer the provisions of the Conservation Authorities Act R.S.O 1990, and Fill, Construction and Alteration to Waterways regulations passed pursuant to Section 28(1) of the Act, or successors thereto, to assist in the implementation of this policy statement.

The Ministry of Natural Resources will administer the provision of the Lakes and Rivers Improvement Act, R.S.O, 1990, the Public Lands Act, R.S.O. 1990 and the Federal Fisheries Act or successors thereto, to assist in the implementation of this policy statement.

The Ministry of Natural Resources, in conjunction with Environment Canada will continue to administer the remainder of the Canada-Ontario Flood Damage Reduction Program through the Conservation Authorities and the municipalities. This includes the pursuance of flood and erosion hazard mapping and studies, and the preparation of information maps generated to the general public depicting flood and erosion susceptible lands. Once the program is completed, the Ministry of Natural Resources and Conservation Authorities will provide administrative and advisory roles to municipalities undertaking flood plain mapping studies.

Flood plain mapping

The flood standards to be used for delineating flood plains are described in the Provincial Policy Statement, (see Chapter B).

Methodologies for carrying out the hydrologic and hydraulic calculations and the preparing of base mapping for a flood plain analysis are outlined in this Technical Guide.

The flood plain mapping document will require approval from the local municipality and from the Conservation Authority, or where no Conservation Authorities exist, from the Ministry of Natural Resources.

Municipal Plan input and review

The Ministry of Natural Resources through the Ministry of Municipal Affairs and Housing, in cooperation with the Conservation Authorities, will administer the Natural Hazards Policies in the Provincial Policy Statement, as well advise and explain its content and application to municipalities.

The Ministry of Municipal Affairs and Housing and municipalities with delegated approval from the Minister will ensure that municipal planning documents subject to their review will have regard to the Natural Hazards Policies.

Municipalities, with input from Conservation Authorities, or the Ministry of Natural Resources where no Conservation Authorities exist, will put in place planning controls necessary to implement the Natural Hazards Policies in Official Plans.

The Conservation Authorities, or the Ministry of Natural Resources where no Conservation Authorities exist, are responsible for plan review related to natural hazard management. In this regard they will:

- Make available any existing mapping, flood and erosion data and studies and provide technical assistance to any government body or planning authority, in particular municipalities and planning boards, and assist municipalities and planning boards to incorporate the intent of the Policy Statement for the management of hazard lands susceptible to flood, erosion and other water related hazards into land use planning process and appropriate planning documents.

- Provide comments to approval agencies on proposed planning actions that may have implications for the management of hazards lands susceptible to flood or erosion and other water related hazards.

- Make representations or provide technical expertise to the Ontario Municipal Board or other appeal bodies, where a matter related to the Policy Statement may be an issue.

- Consult with ministries, public agencies, boards, authorities and municipalities on matters pertaining to the management of lands susceptible to flood, erosion or other water related hazards as may be appropriate.

- Inform and educate the general public on the principles and practices of flood and erosion hazard lands and provide information on the characteristics and consequences of flood, erosion and other water related hazards in flood plains.

The Ministry of Natural Resources in co-operation with the Ministry of Municipal Affairs and Housing will undertake periodic research programs to investigate and update planning implementation and natural hazard management techniques. The same Ministries, in cooperation with Conservation Authorities, will administer the flooding and erosion hazard policies, as well as advise and explain its contents and application to municipalities, planning boards, and other agencies.

The intent of the Natural Hazards Policies should be reflected in a municipality's Official Plan. Flooding and erosion are naturally occurring processes influenced by local watershed conditions. When addressing these physical processes from an ecosystem perspective, the local physical and ecological processes may need to be retained in an undisturbed state to the greatest extent possible, and possibly enhanced to sustain the overall health of the watershed ecosystem.

The order of preference in addressing the natural hazard concerns are prevention, protection and emergency measures. Frequently, a combination of the options will provide the most effective solutions to address the public safety and health issues. Effective hazard management can only occur on a comprehensive watershed basis. Therefore, site specific development activities should be evaluated on an overall watershed or jurisdictional base.

There is continuing research work being undertaken on the effect of developments on our ecological systems. Once a new approach has been field tested and proven to be effective in reducing or eliminating adverse effects on our environment and it is shown to provide a cost effective solution, the information will be disseminated by MNR and MOEE to all involved in land use planning, design and approval of developments and watershed management.

Benefits of flood plain and natural hazard policies

The favourable benefit-cost ratio produced by the ongoing flood plain mapping and zoning program was demonstrated by the very successful Canada-Ontario Flood Damage Reduction Program (FDRP). The program attempted to avoid the use of structural methods of flood control which subsidize those who develop in the flood plain. Instead, the beneficial effect of flood plain zoning and restriction of development in the flood plain resulted in significant reduction in the costs associated with flooding. This was done by drawing and publicising detailed flood plain maps. Potential and current landowners were expected to use this information to avoid or modify flood plain developments. This information was also useful to local authorities in planning and zoning decisions.

Although the FDRP program has been phased out, the continuation of flood plain and hazard land management (mapping and strict control of development along flood risk areas) will ensure that the benefits of the program will continue.

Past experience in flood plain management in Ontario identified three categories of benefits closely related to flood plain mapping:
- reduced flood damages;
- administrative and social benefits; and
- environmental, ecological and sustainable development benefits.

Keeping development out of flood risk areas will obviously reduce the risk of lives and property damages.
Flood plain management results in a number of administrative and social benefits as well, that include: the production of detailed flood line mapping which yield strengthened official plans and/or zoning by-laws that improve the precision of regulation, expediting the processing of permit applications, facilitating decision-making and enhancement of the defensibility of planning decisions. Finally, it provides a precise delineation of flood lands to identify and market the development potential of lands outside the risk areas.

The environmental benefits of flood plain designation include: 1) the maintenance of wetland areas for hydrologic and flood stage reduction purposes, 2) the preservation of environmentally sensitive areas, including provincially and regionally significant plant and animal species.

SUMMARY OF IMPLEMENTATION OF FLOOD PLAIN POLICIES

ACT	PURPOSE	IMPLEMENTATION
Planning Act	Enabling legislation	Land use planning decisions by Municipalities, local boards, planning boards, government agencies, committees
Official Plans	Provide background information Delineate flood and erosion hazard areas with supporting policies Identify development requirements Outline implementation policies to address flooding	Implementation by municipalities
Zoning By-Laws (in conformity with Official Plans)	Prohibiting erection of building or structure on lands subject to flooding or erosion	Implementation by municipalities through development control
Plan of Subdivision and Land severance	Lot/Block creation to delineate areas subject to flooding and erosion	Implementation by municipalities
Site plan control	Identification of landscape and buffer requirements	Implementation by municipalities
The Conservation Authorities Act	Administrative: to study watershed, to control the flow of surface water in order to prevent floods Regulatory: Make regulations prohibiting or regulating or requiring the permission of the Conservation Authority for the changing, diverting or interfering with existing water bodies Prohibiting or regulating or requiring the permission of the Conservation Authority for the construction of any building or structure, or placing or dumping of fill of any kind in flood hazard land . This mandate does not include the control of land use	Responsibilities for implementation with CA's, where they exist, and MNR where they do not.
Lakes and Rivers Improvement Act	Mandate: Provides MNR powers for the use of waters of Ontario and to regulate improvements to them	Application to MNR offices, and other agencies to be contacted MOE, Conservation Authority, MTO if affected Approval of project location, plans and specifications
Public Lands Act	Sale and disposal of public lands and forests, including lands covered by water or lands seasonally inundated by water	Administered by MNR
Building Code	No special provision for flood susceptibility, only indication where dynamic loading conditions apply. Also, for buildings three storeys or less and with area less than 557 square meters describes waterproofing, surface and subsurface drainage.	Ontario Building Code administered by Ministry of Housing. Under the Municipal Act municipalities can pass building by-laws which are not more restrictive nor lenient than the Building Code.
Building permits	Issuance of Building Permits	Issued by Chief Building Official, may require approval from Conservation Authority and MNR
Federal legislation		
Canada Water Act		
National Flood Damage Reduction	Canada-Ontario flood plain mapping	Federal-Provincial (MNR) program – local management of designated flood risk areas.

J. SURVEYS AND MAPPING

UNDER REVIEW

APPENDIX 1 : BIBLIOGRAPHY

1. RAINFALL FLOOD ANALYSIS

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APPENDIX - 2 - GLOSSARY

Adverse Effects

As defined in the *Environmental Protection Act*, means one or more of:

- impairment of the quality of the natural environment for any use that can be made of it;
- injury or damage to property or plant and animal life;
- harm or material discomfort to any person;
- an adverse effect on the health of any person;
- impairment of the safety of any person;
- rendering any property or plant or animal life unfit for use by humans;
- loss of enjoyment of normal use of property; and
- interference with normal conduct of business.

Areas of Natural and Scientific Interest (ANSI)

Means areas of land and water containing natural landscapes or features that have been identified as having life science or earth science values related to protection, scientific study, or education.

Cultural Heritage Landscape

Means a defined geographical area of heritage significance which has been modified by human activities. Such an area is valued by a community; and is of significance to the understanding of the history of a people or place.

Design Flood

The flood which controls the design of a specific flood related project, i.e. dam, diversion, bank protection, etc.

Development

Means the creation of a new lot, a change in land use, or the construction of buildings and structures, requiring approval under the *Planning Act*; but does not include activities that create or maintain infrastructure authorized under an environmental assessment process; or works subject to the *Drainage Act*.

Ecological Functions

Means the natural processes, products or services that living and non-living environments provide or perform within or between species, ecosystems and landscapes. These may include biological, physical and socio-economic interactions.

Erosion Hazards

Means the loss of land, due to human or natural processes, that poses a threat to life and property. The erosion hazard limit is determined using the 100 year erosion rate (the average annual rate of recession extended over a hundred year time span), an allowance for slope stability, and an erosion allowance.

Essential Emergency Services

Means services such as those provided by fire, police and ambulance stations and electrical substations, which would be impaired during an emergency as a result of flooding, the failure of floodproofing measures and/or protection works, and/or erosion.

Established Standards and Procedures

Means the following:

Floodproofing standard, which means the combination of measures incorporated into the basic design and/or construction of buildings, structures, or properties to reduce or eliminate flooding, wave uprush and other water related hazards along the shorelines of the Great Lakes - St. Lawrence River System and large inland lakes, and flooding along river and stream systems.

Fetch

Length of water surface exposed to wind.

Fish Habitat

Means the spawning grounds and nursery, rearing, food supply, and migration areas on which fish depend directly or indirectly in order to carry out their life processes.

Flood Fringe

Means the outer portion of the flood plain between the floodway and the flooding hazard limit. Depth and velocities of flooding are generally less severe in the flood fringe than those experienced in the floodway. The flood fringe is the area where development and site alteration may be permitted, subject to appropriate floodproofing to the flooding hazard elevation or another flooding hazard standard approved by the Ministry of Natural Resources.

Flood Plain

Flood plain of a river system means the area, usually low lands adjoining a watercourse, which has been or may be subject to flooding hazards.

Flood Standard

Flood used to define flooding hazard along rivers and stream systems.

Floodway (for river and stream systems)

Means the portion of the flood plain where development (other than uses which by their nature must be located within the floodway, flood and/or erosion control works, or where appropriate, minor additions or passive, non-structural uses which do not affect flood flows) and site alteration would cause a danger to public health and safety or property damage.

Where the one zone concept is applied, the floodway is the entire flood plain.

Where the two zone concept is applied, the floodway is the 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 thread to life and/or property damage. Where the two zone concept applies, the outer portion of the flood plain is called the flood fringe.

Great Lakes - St. Lawrence River System:

Means the major water system consisting of Lakes Superior, Huron, St. Clair, Erie and Ontario and their connecting channels, and the St. Lawrence River within the boundaries of the Province of Ontario.

Hazardous Lands

Means property or lands that could be unsafe for development due to naturally occurring processes. Along the shorelines of the Great Lakes - St. Lawrence River System, this means the land, including that covered by water, between the international boundary, where applicable, and the furthest landward limit of the flooding, erosion or dynamic beach hazard limits. Along the shorelines of large inland lakes, this means the land, including that covered by water, between a defined offshore distance or depth and the furthest landward limit of the flooding, erosion or dynamic beach hazard limits. Along river and stream systems, this means the land, including that covered by water, to the furthest landward limit of the flooding or erosion hazard limits.

Hazardous Sites

Means property or lands that could be unsafe for development and site alteration due to naturally occurring hazards. These may include unstable soils (sensitive marine clays [lead], organic soils) or unstable bedrock (karst topography).

Hazardous Substances

Means substances which, individually, or in combination with other substances, are normally considered to pose a danger to public health, safety and the environment. These substances generally include a wide array of materials that are toxic, ignitable, corrosive, reactive, radioactive or pathological.

Large Inland Lakes

Means those waterbodies having a surface area of equal to or greater than 100 square kilometres where there is not a measurable or predictable response to a single runoff event.

Maximum Probable Flood

The largest flood that can reasonably be expected to occur at a selected point. It is based on rational consideration of the chance of simultaneous occurrence of the maximum of the various elements which contribute to the flood.

Natural Heritage Features and Areas

Means features and areas, such as significant wetlands, fish habitat, significant woodlands south and east of the Canadian Shield, significant valleylands south and east of the Canadian Shield, significant portions of the habitat of endangered and threatened species, significant wildlife habitat and significant areas of natural and scientific interest, which are important for their environmental and social values as a legacy of the natural landscapes of an area.

Negative Impacts

Means:

a) in regard to fish habitat, the harmful alteration, disruption or destruction of fish habitat, except where it has been authorized under the *Fisheries Act*, using the guiding principle of no net loss of productive capacity;

b) in regard to other natural heritage features and areas, the loss of the natural features or ecological functions for which an area is identified.

One Hundred Year Flood (for river and stream systems)

Means that flood, based on an analysis of precipitation, snow melt, or a combination thereof, having a return period of 100 years on average, or having a 1% chance of occurring or being exceeded in any given year.

One Hundred Year Flood Level

Means:

- For the shorelines of the Great Lakes, the peak instantaneous stillwater level, resulting from combinations of mean monthly lake levels and wind setups, which has a 1% chance of being equalled or exceeded in any given year.

- In the connecting channels (St. Mary's, St. Clair, Detroit, Niagara and St. Lawrence Rivers), the peak instantaneous stillwater level which has a 1% chance of being equalled or exceeded in any given year.

- For large inland lakes, lake levels and wind setups that have a 1% chance of being equalled or exceeded in any given year, except that, where sufficient water level records do not exist, the one hundred year flood level is based on the highest known water level and wind setups.

- For rivers and streams, water leves caused by the 100 year return period flood, resulting in a 1% chance of occurring or being exceeded in any year.

One Hundred Year Precipitation

Precipitation, which on the average is expected to be equalled or exceeded once every 100 years. The chance of occurring in any one year is 1%.

Normally, the data is based on extrapolation of recorded events, as none of the Ontario rainfall stations have 100 years of record.

Other Related Hazards

Means water-associated phenomena other than flooding and wave uprush which act on shorelines. This includes, but is not limited to ice, ice piling and ice jamming.

Prime Agricultural Area

Means an area where prime agricultural land predominates. Prime agricultural areas may also be identified through an alternative agricultural land evaluation system approved by the province.

Prime Agricultural Land

Means lands that includes specialty crop lands and/or Canada Land inventory Classes 1, 2 and 3 soils, in this order of priority for protection.

Public Service Facilities

Means land, buildings and structures for the provision of public services, but does not include infrastructure.

Public Services

Means programs and services provided or subsidized by a government or other public body. Examples include social assistance, recreation, police and fire protection, health and educational programs, and cultural services.

Quality and Quantity (of water)

Is measured by indicators such as minimum base flow, oxygen levels, suspended solids, temperature, bacteria, nutrients, hazardous contaminants, and hydrologic regime.

Residential Infilling

Means the creation of a residential lot between two existing non-farm residences which are on separated lots of a similar size and which are situated on the same side of a road and are not more than 100 metres apart.

River and Stream Systems

Means all watercourses, rivers, streams, and small inland lakes or waterbodies that have a measurable or predictable response to a single runoff event.

Rural Areas

Means lands in the rural area which are not prime agricultural areas.

Sensitive Land Uses

Means buildings, amenity areas, or outdoor spaces where routine or normal activities occurring at reasonably expected times would experience one or more adverse effects from contaminant discharges generated by a nearby major facility. Sensitive land uses may be a part of the natural or built environment. Examples include: residences, day care centres, and educational and health facilities.

Significant

Means:

- In regard to wetlands and areas of natural and scientific interest, an area identified as provincially significant by the Ministry of Natural Resources using evaluation procedures established by the province, as amended from time to time.

- In regard to other features and areas, ecologically important in terms of features, functions, representation or amount, and contributing to the quality and diversity of an identifiable geographic area or natural heritage system. Criteria for determining significance may be recommended by the Province, but municipal approaches that achieve the same objective may also be used.

- In regard to other matters, important in terms of amount, content, representation or effect.

Site Alteration

Means activities, such as fill, grading and excavation, that would change the landform and natural vegetative characteristics of a site.

Special Policy Area

Means an area within a community that has historically existed in the flood plain and where site specific policies, approved by the Ministers of Natural Resources and Municipal Affairs and Housing, are intended to address the significant social and economic hardships to the community that would result from strict adherence to provincial policies concerning development.

Thalweg

Line connecting the deepest points along a stream channel.

Valleylands

Means a natural area that occurs in a valley or other landform depression that has water flowing through or standing for some period of the year.

APPENDIX 3 - APPLICATION TO CHANGE THE FLOOD STANDARD WITHIN A WATERSHED

For those watersheds with a flood standard greater than the provincial minimum acceptable standard of the 100 year flood, the option exists for municipalities and planning boards to apply to the Minister of Natural Resources, in accordance with the following procedures, to change the standard, subject to the following overriding conditions:[1]

- Changes to the existing flood standard will only be considered with the support of a significant majority of municipalities and/or planning boards within the watershed, in consultation with the local Conservation Authority or Ministry of Natural Resources, where Conservation Authorities do not exist; and

- the lowering of the existing flood standard where the past history of flooding reveals a higher level is more appropriate will not be considered.

It is recognized that complete information may not be available when a municipality comes forward with a request to change the flood standard. It is suggested, however, that reasonable information be provided to assess the impacts of the change.

Conservation Authorities and Ministry of Natural Resources Regional Offices, where Conservation Authorities do not exist, will be responsible for co-ordinating requests for changes.

To facilitate requests for changes in the flood standard, the following sequence will be followed:

(1) Responsibilities of the Initiating Municipality

- When considering requesting a change in the flood standard, the initiating municipality will first provide a suitable forum for public input for which prior notice is given to the general public.

A request made to the Minister of Natural Resources to consider a change will be in the form of a council endorsed resolution. The resolution, accompanied by documentation on the prior notification and opportunities for public input, will be forwarded to the Minister of Natural Resources together with an explanation as to the reasons for the request. The Minister may wish to seek clarification or additional information from the initiating municipality.

(2) Ministry's Action

- Main office of the Ministry of Natural Resources will inform the appropriate Conservation Authority or Ministry of Natural Resources Regional Office, where no Conservation Authorities exist, and the Ministry of Municipal Affairs and Housing, of the request for a change.

(3) Co-ordination/Technical Input by the Conservation Authority or Ministry of Natural Resources

- The Conservation Authority or Ministry of Natural Resources Regional office will provide written notification to each municipality within the watershed. Municipalities will include upper tier (i.e. regions, counties, etc.) and lower tier (i.e. cities, townships, etc.), whether situated totally or partially within the watershed. The transmittal letter will:

. identify that the Conservation Authority or Ministry of Natural Resources Regional office is acting in a co-ordinating role on behalf of the Minister of Natural Resources;

. identify that a request to change the flood standard has been made and include the actual resolution passed by the initiating municipality;

. request that all municipalities make their views known (whether for or against) in the form of a council endorsed resolution;

. identify that the resolutions are to be directed to the Minister of Natural Resources but forwarded to the local Conservation Authority or Ministry of Natural Resources Regional office for compilation;

. request that documentation of the public notification and the opportunities afforded for the public representation also be provided and forwarded in conjunction with the council endorsed resolution;

. identify that the comments of the Conservation Authority or Ministry of Natural Resources Regional office will be forwarded to each municipality in the near future;

. identify that a change in the flood standard may result in changes to municipal planning documents (i.e. official plan, zoning by-laws, etc.) and that the municipality may wish to contact the Ministry of Municipal Affairs and Housing in this regard;

- Under separate cover, the Conservation Authority or Ministry of Natural Resources Regional office will provide comments to the municipalities regarding the water management implications, if any, of a change in the flood standard. Comments may include specific information on:

. past history of flooding;
. time required to pass flood flows;
. extent and rate of urbanization;
. changes required to give effect to a change in the flood standard;
. funding implications;
. etc..

(4) Responsibilities of the Responding Municipalities

- In considering the request of the initiating municipality to change the flood standard for the watershed, each municipality will provide opportunities for public input for which prior notice is given to the general public.

(5) Additional Opportunity for Public Input

- If after a municipality has finalized its position through a council endorsed resolution, members of the public still wish to make their views known, they may do so directly to the Minister of Natural Resources.

(6) Minister's Decision

- Once obtained, the Conservation Authority or Ministry of Natural Resources Regional office will forward all municipal resolutions, including documentation of public notices and opportunities provided for public input, and its own comments to the Minister of Natural Resources and copies of the submission will be sent to each watershed municipality for information purposes.

- In considering a request to change the flood standard, the main office of the Ministry of Natural Resources will consult the Ministry of Municipal Affairs and Housing and may request the comments and opinions of any other groups and/or individuals, as deemed appropriate.

- Upon making a decision, the Minister of Natural Resources will directly inform, in writing, all watershed municipalities, the local Conservation Authority or Ministry of Natural Resources Regional Office, and the Ministry of Municipal Affairs and Housing.

(7) Communication Plan

- Upon formal approval of a change in the flood standard, the watershed municipalities, in conjunction with the local Conservation Authority or Ministry of Natural Resources Regional office, will jointly prepare a communication plan to inform:

  - the general public
  - interest groups/associations
  - other appropriate public agencies
  - consultants (i.e. planning, engineering, architectural, etc.)
  - developers/builders
  - others

The communication plan will also highlight the major water management changes to municipal planning documents that may be required as a result of the change in the flood standard.

1 For territories without municipal organization, planning boards, where they exist, will assume the responsibilities identified for "municipalities" within the context of this process. For territories without municipal organizations that do not have planning boards, requests to change the regulatory flood standard will be made directly to the Minister of Natural Resources by interested parties.

APPENDIX –4 APPLICATION OF THE TWO-ZONE CONCEPT FACTORS TO BE CONSIDERED

Evaluation of the following factors will assist in assessing the suitability of applying the two-zone concept.

(1) Frequency of flooding

Caution should be exercised in applying the two-zone concept for chronic problem areas. While development in such areas could adequately be floodproofed, maintenance and upkeep would continuously be required to ensure floodproofing measures and local services remain effective.

(2) Physical Characteristics of the Valley

Steepness of valley slopes, instability of banks and poor soil conditions in flood fringe areas can physically render the flood fringe unsuitable for development. Adopting the two-zone concept would show more promise for areas with a flat overbank and shallow flow. Topography varies, so evaluation is necessary on a local basis in determining suitability.

(3) Local Need

Suitability of flood fringe areas for development can be influenced by municipal planning considerations including availability of developable land elsewhere in the municipality. In urban areas where land values are high and pressure for development is usually the greatest, the concept shows promise. Lot sizes are usually larger in rural areas, and it is generally possible to locate development outside the flood plain. Therefore, proposed application of the two-zone concept in rural/agricultural areas will require detailed rationale/justification.

(4) Impacts of Proposed Development

Encroachment within the flood fringe area usually results in an increase in flood levels. The extent of potential increases will be dependent on a number of factors in watershed characteristics and the degree to which the two-zone concept is to be applied. As a result, it may be necessary to recalculate for the flood standard the flood levels for floodproofing purposes and identify and assess the upstream and downstream impacts where the two-zone concept is being considered. This is particularly true where the two-zone concept is to be applied over extensive areas.

(a) Flood Levels at the Site and Upstream

Filling and construction within the flood fringe area reduces the cross-sectional area of the waterway, so the corresponding flood level increases at the site and immediately upstream. This increase in the flood level can be estimated with reasonable accuracy and normally does not require major engineering studies.

(b) Flood Levels Downstream

General encroachment within the flood fringe area reduces the storage capacity of the flood plain and results in an increase in flood flows and the flood levels along the downstream reaches of the river. If undertaken during the initial flood plain mapping process, the revised levels can be computed without major additional expense. Where flood plain mapping was undertaken several years earlier and the data base utilized in preparing the maps is not readily available, the calculation of the revised flood levels may require major engineering studies at substantial cost.

(5) Feasibility of Floodproofing

One of the major factors in determining if a flood fringe area is suitable for development is the feasibility and cost of floodproofing.

(6) Constraints to the Provision of Services

Flood fringe areas are low-lying and it is often difficult and expensive to provide necessary services (watermains, sewers, drainage works, etc.) to serve the developments. Drainage systems should provide protection against the flood standard and it may be difficult to provide outlets above the level of flood standard. In these situations, it may be necessary to provide pumping facilities which would result in some additional expense in new developments.

(7) Ingress/Egress

Major accessways to development potentially located in the flood fringe must be examined. It is not acceptable to have development isolated during the flood conditions because roads and escape routes are not passable.

(8) Changes in Land Use

Land use is a key factor considered in flood plain studies and the calculation of flood lines. Proposed development, not anticipated in these calculations, could create increased flood risks and thus reduce the effectiveness of flood plain management programs.

It is therefore imperative that municipalities discuss proposed changes in land use with the local Conservation Authority or Ministry of Natural Resources, where one does not exist.

(9) Administrative Capability

The feasibility of the two-zone concept requires the examination of a number of factors and implementation requires assurance that various conditions are complied with. Therefore, staff availability and expertise must also be considered.

As well, certain planning tools (e.g. zoning, site plan control, subdivision control) are required to effectively implement the necessary land use controls. Where such tools are not available, e.g. areas without municipal organization, application of the two-zone concept is not a viable option unless supported by detailed methods of implementation.

It is not mandatory that a municipal official plan contain floodway - flood fringe policies prior to utilizing the two-zone concept. It is certainly intended that the municipal documents ultimately outline the basis for utilizing the two-zone concept and the areas of the municipality where it would apply. However, some municipalities in conjunction with the Conservation Authority

(Fill, Construction and Alteration to Waterways Regulation) or the Ministry of Natural Resources, may have already been utilizing the two-zone concept. In this regard, it is not the intent of the Provincial Flood Plain Policy that the water management options be applied retroactively, to municipal planning documents.

During the preparation of an official plan update or a major official plan amendment affecting flood plain areas, the municipality in conjunction with the Conservation Authority or Ministry of Natural Resources, should include policies addressing:

. existing areas of the municipality utilizing the two-zone concept and/or;

. a framework for analyzing potential areas of two-zone application, including both land use considerations and technical flood plain information and,

. the inter-relationship between the official plan, zoning by-law and the Conservation Authority's Fill, Construction and Alteration to Waterways Regulation.

The Regional Engineer of the Ministry of Natural Resources shall be involved in decision making regarding potential application of a two-zone concept.

APPENDIX 5 : SPECIAL POLICY AREAS -

A – REQUIRED CONSIDERATIONS

Special Policy Area application ONLY applies to River and Stream Systems and inland lakes that are less than 100 square kilometres in size and hydrologically respond to a single meteorological event. They do not apply to hazardous lands adjacent to Great Lakes – St. Lawrence River System and large inland lakes.

Only the Minister of Natural Resources and the Minister of Municipal Affairs and Housing have the authority to approve Special Policy Areas in Ontario. Since Special Policy Areas place the province at a greater degree of risk, this function CANNOT be delegated to municipalities or other planning bodies.

In determining whether or not an area potentially qualifies for special policy area status, the factors to be considered can be grouped into two major categories — community related and technical.

(1) Community Related

The characteristics of the community itself are important considerations in identifying eligibility for special policy area status. To potentially qualify, an area should:

. have a municipal commitment to area maintenance;
. be an area designated in the official plan for continued growth;
. have significant investment in infrastructure, i.e. services;
. limited opportunities for development elsewhere.

(a) Municipal Commitment

To qualify for special policy area status a municipality must have a commitment, reflected in its official plan policies, to the continued upkeep of the area, such as an active program to revitalize.

(b) Designated Growth Centre

To qualify for special policy area status, a community must be recognized as a centre for urban growth and development. This would be reflected in planning documents - regional municipality, county, joint planning area or local municipality plan. This criterion attempts to ensure there is a desire and commitment to further development on the community's part.

(c) Infrastructure Investment

A further measure of commitment to continued growth is the extent of investment in community infrastructure. Practical indicators include the extent of servicing that exists, i.e. water and sewage.

(d) Limited Alternatives

A community with feasible alternatives for expansion or redevelopment outside the flood plain area would not necessarily qualify for special policy area status.

(2) Technical Criteria

To determine if a community qualifies for special policy area status, various technical criteria relating to the flood hazard must also be considered:

- Appropriateness of other flood plain management measures, i.e. remedial works, two-zone approach.
- Depth of flooding and velocity of flow.
- Frequency of flooding.
- Feasibility of floodproofing measures.
- Upstream and downstream effects.
- Frequency of ice jams and other obstructions.
- Berms and flood walls.
- Reduced flood standard.

(a) Appropriateness of Other Measures

In order to contemplate eligibility for special policy status, other measures such as remedial works and the two-one approach must be proven to be unworkable.

In situations where remedial measures to permanently reduce flood levels may not be implemented in the immediate future, a special policy area might be considered as a water management option, until the remedial measures have been completed.

(b) Flow Characteristics

The depth of flooding and velocity of flow within a flood plain will have a bearing on the extent and location of a special policy area. No matter how strong the arguments relating to other criteria, an area susceptible to severe flooding may not be appropriate for special consideration.

(c) Frequency of Flooding

The frequency of flooding relative to the depth and velocity criteria also determines if special policy area status is appropriate. Potential special policy areas will be individually evaluated relative to flood frequency, both past and future.

(d) Floodproofing Measures

The feasibility of floodproofing new development, in general, within the special policy area must be examined. Based on flood characteristics, local conditions and type of land use proposed, alternative floodproofing measures can be examined as to their individual feasibility and desirability. Key in examining alternative floodproofing measures is the level of flood protection that can be afforded.

(e) Upstream and Downstream Effects

The effects on upstream and downstream areas caused by increased development in the flood plain must be taken into account. Normally, this is determined through the watershed planning process of the Conservation Authority or other special water management studies. These effects may be only minor in some instances. In others, because of the land use patterns and topography, the effects, though significant, may be acceptable. To determine upstream and downstream effects, all special policy area proposals will be evaluated on a case-by-case basis.

(f) Frequency of Ice Jams

Ice jams are a natural phenomena caused by topographic, hydraulic and meteorological factors. Resultant flooding has long been a problem and must be considered in the decision-making process regarding development in a flood plain.

It is almost impossible to predict in advance whether an ice jam will form or if any resultant flooding will occur and to what extent it will occur.

With ice generated floods, river flows are generally much below a flood standard (winter conditions), but due to ice constriction, levels may rise above the flood standard. Due to the unpredictable nature of ice jams, a conservative approach to development is needed where ice jams are known to have caused problems.

(g) Berms and Flood Walls

Where a berm or flood wall has been properly designed to the flood standard and constructed, and a suitable maintenance program is in place, the floodway would be considered to be contained within the berm or flood wall area. The area behind the berm or flood wall can be considered flood fringe. As such, a new development would be required to be floodproofed to the flood standard. If new development cannot be floodproofed to the flood standard, then special policy area status may be requested.

If there are any openings in the berm or flood wall (e.g. road crossings, watercourse confluence, etc.) which would require human intervention to complete the dyke during an impending flood through sand bagging, placing of stop logs etc., the berm or flood wall shall not be considered to contain the floodway.

The establishment of no or limited development zones behind a berm or flood wall will be dependent on local conditions (e.g. flood depth and velocity) and local approaches to flood plain management. As a precaution, certain areas immediately behind a berm or flood wall may be considered too hazardous for any or certain types of uses, if through ice jams, debris jams etc., failure of the berm or flood wall was ever to occur.

(h) Reduced Flood Standards

For watersheds where reduced flood standards levels have received approval by the Minister of Natural Resources in accordance with the provisions of Appendix "A", the options still exists for a municipality to apply for preliminary approval in principle for Special Policy Area status. However, if for example, the new flood standard for a watershed is the 100 year event and a floodway has been defined using the product of depth and velocity, it may be difficult to provide justification for the two-zone concept being too stringent.

Existing Special Policy Area policies may require revision once a reduced flood standard has been approved by the Minister or alternatively, the need for a special policy area may or may no longer exist.

(i) Evaluation

All criteria must be balanced against one another and a decision reached as to whether the community will qualify for special policy area status. The weighting of these factors will depend on the complexity and relative nature of the criteria. Before such weighing is undertaken, as much factual data as possible should be assembled.

(3) Types of Special Policy Areas

Each special policy area is unique, but there are two identifiable types:

(a) special policy areas where floodproofing to the flood standard is not provided; and

(b) special policy areas where development is proposed in the floodway.

Both types of special policy areas run counter to basic concepts within the provincial policy statement, namely; new development within the flood plain should be protected from flooding to the level of the flood standard and new development within the floodway, the more hazardous portion of the flood plain, should be prohibited or restricted to non-structural uses such as open space.

Therefore, great care must be exercised in proposing and approving special policy areas as susceptibility to flooding and damage are much greater in such areas.

B - PROCEDURES FOR APPROVAL

Procedures for seeking approval of a special policy area designation will generally consist of three phases:

Phase

Phase I	identification of need and preliminary approval in principle;
Phase II(a)	data collection and preparation of draft official plan policies;
Phase II(b)	review and formal approval of official plan policies;
Phase III	implementation and review/update.

(1) Phase I - Identification of Need and Preliminary Approval as Special Policy Area

(a) Request for Special Policy Area Status

Phase I (preliminary approval in principle) is designed to establish special policy area status in principle, and to lay the framework for further technical evaluation. It will prevent unnecessary expenditures prior to the approval agencies' acceptance of the request as being consistent with the principles of flood plain management. This phase will also identify the nature and extent of further studies necessary to accurately evaluate the limits and/or scope of the special policy area.

A special policy area is a flood plain planning option based on water management principles. It is necessary to consider policies for all land uses within the special policy area as policy decisions regarding one land use may adversely affect or limit the alternatives for other land use policies. Separate special policy area proposals for each land use within an overall special policy area is not appropriate.

It must be noted, approval in principle does not signify final approval of the proposed designated, nor is it an assurance the special policy area will be approved. The latter will depend largely on the conclusions and results under Phase II(a).

The initial request for special policy area status, having regard to the criteria outlined, must come from the municipality. The municipality should be satisfied it meets the criteria and that it has suitable expertise and financial capability to deal with the establishment of a special policy area. The request for special policy area status should be accompanied by a brief report addressing the criteria for special policy area eligibility. In this regard, the municipality should contact the local Conservation Authority where one exists, as an initial step to determine the type of flood related information that may exist. The Conservation Authority or Ministry of Natural Resources shall also provide detailed flood related information in report form indicating the rationale and justification why the provisions of the two-zone concept are too onerous.

The municipality should then approach the Ministry of Municipal Affairs and Housing, the local Conservation Authority and the Ministry of Natural Resources to obtain approval in principle of its request for special policy area status. Until it has been notified of approval in principle, the municipality should not proceed with any additional studies.

(b) Provincial Review of Municipal Request for Approval in Principle

The Ministry of Municipal Affairs and Housing will co-ordinate the review of material prepared by the municipality to decide whether it meets the criteria and if acceptance in principle can be given. Where the municipality seeking approval in principle for special policy area status is within a 'delegated' regional municipality, the region will be involved in the review. Regional municipal representatives may co-ordinate the review of all materials relating to a special policy area designation once the municipality has been given approval in principle and direction has been provided as to the additional studies required to support an approval of a specific special policy area.

If more information is required, the municipality will be advised what is required in support of its request for approval in principle and the agencies will reconsider the application when the additional material is available.

(c) Approval in Principle to Consider Special Policy Area Status

The municipality will receive written approval in principle jointly issued by representatives of the Ministries of Municipal Affairs and Housing, Natural Resources and, where one exists, the local Conservation Authority.

Upon acceptance in principle, the municipality will be advised regarding detailed studies required to support development of official plan policies for the special policy area.

(d) Refusal of Approval in Principle for Special Policy Area Status

If a municipality is ineligible under the criteria outlined, it will be notified and given reasons.

(2) Phase II(a) - Data Collection and Preparation of Draft Official Plan Policies

A municipality granted approval in principle for consideration of special policy area status, will be expected to collect appropriate data according to the approval in principle letter, and to produce policies meeting the guideline requirements. While data collection is the responsibility of the municipality requesting special policy area status, provincial agencies will assist as much as possible in providing information and in endeavouring to provide the municipality with guidance, technical advice, etc. A working group may be established to liaise with the municipality and monitor the study progress.

The municipality should carry out the following steps under the guidance of the working group composed of representatives of the Ministry of Natural Resources, the local Conservation Authority, the Ministry of Municipal Affairs and Housing, and possibly other Ministries such as the Ministry of the Environment and Energy and the Ministry of Northern Development and Mines. Municipal representation should include both planning and engineering staff. If there is a regional municipality, its representative should also be included.

(a) Municipal Data Collection

Before policies are developed, the municipality should collect data and demonstrate adequate consideration of alternatives. This stage should be monitored by the working group who will provide technical assistance to the municipality as needed and wherever possible.

(b) Evaluation of Alternatives

The municipality should consider alternative approaches to handling the problems of the floodprone area including upstream and downstream effects of the alternatives.

(c) Policy Formation

Once the data collection is completed, the municipality can prepare proposed policies. The policies should include and be supported by, but not necessarily limited to, the following information which shall be subject to the approval of the Conservation Authority, where one exists and the Ministries of Municipal Affairs and Housing and Natural Resources:

- An introductory statement containing an explanation of provincial policy, a brief description of the area proposed for a special policy area, and a justification for the proposal (including an evaluation of risk factors involved in permitting development in the flood plain).

- The boundaries of the special policy area shall extend to the appropriate floodlines on each side of the watercourse (if appropriate) and be closed at both the upstream and downstream limits. The policies will then address all land uses, additions, renovations and replacements within these boundaries.

- The flood levels must be defined by floodplain mapping studies for the area(a) under consideration. Such studies should consider both pre and post-development situations.

- The minimum acceptable level of protection (floodproofing) for development within the special policy area.[1]

- The land use policies and designations for the proposed special policy area.

- Detailed implementation policies identifying the mechanisms (i.e. zoning, site plan control) and means to be applied to ensure flood susceptibility and floodproofing are addressed by new development.

- Policies for new buildings, additions, renovations, infilling and replacements within the proposed special policy area.

- The roles of council, Conservation Authority(ies) and the Ministries of Municipal Affairs and Housing, Natural Resources and any other appropriate agency with respect to the circulation and review of development proposals including subdivision plans, consents, minor variances, and building permits.

- The delineation of the boundaries of the special policy area, as an overlay, on the land use schedule.

- An appendix which includes background reports and studies supporting the policies proposed.

The municipality in preparing an appendix to its official plan document would include background papers addressing the special policy area guidelines and how the draft official plan policies reflect them. It would outline various alternatives considered and studies carried out to support the proposed policies. Exceptions to the provincial policy statement on flood plain planning are considered on their own merits, and the Province will want to ensure there is a clear outline of the basis of these exceptions for the benefit of the public and others.

Agreement to the general policy proposals should be given in writing by representatives of Ministries of Municipal Affairs and Housing, Natural Resources and the local Conservation Authority, preferably before any public meeting, so as to avoid raising false expectations.

(3) Phase II(b) - Review and Formal Approval of Official Plan Policies

Processing the documentation for review and approval would comply with the requirements of the Planning Act, and the standard procedures established by the Ministry of Municipal Affairs and Housing for the review of all official plans and amendments would be followed.

(a) Public Involvement

The municipality has a responsibility to involve the public in considering proposed policies as they will form a component of the official plan. The municipality may request technical backup and support from the provincial ministries for presentation purposes at any public meetings.

(b) Review of Draft Official Plan Policies

Following public meetings, the official plan policies, including appendix material, should be finalized in draft form by the municipality and copies forwarded to the Ministry of Municipal Affairs and Housing, whose responsibility is to co-ordinate a response from the provincial agencies and the local Conservation Authority on all draft documents. This response should be received by the municipality within 30 days or other agreed to period. If necessary, meetings may be held to discuss the response.

(c) Municipal Adoption

Based on comments received, the municipality would make appropriate modifications and formally adopt the official plan policies.

(d) Conservation Authority Adoption

The special policy area provisions will determine the basis by which a Conservation Authority will administer applications pursuant to their Fill, Construction and Alteration to Waterways Regulation. Therefore, the agreed upon policies require a resolution of acceptance by either the Executive Committee or the Full Authority, whichever has been vested with the decision-making authority.

(e) Formal Submission for Approval

Once finalized at the local level, the municipality would then submit the policies to the Ministry of Municipal Affairs and Housing for approval, unless a regional municipality has the authority to receive them.

As the draft official plan policies request approval for special policy area status, they will also be forwarded to the appropriate Regional Office of the Ministry of Natural Resources to initiate the approval process by the Minister of Natural Resources. The Minister would then inform the Minister of Municipal Affairs and Housing of his/her support or objection to the official plan policies.

In all other respects, normal official plan policy circulation and approval procedures would be followed, as specified under the Planning Act, or by the Ministry of Municipal Affairs and Housing.

(4) Phase III - Implementation and Review/Update

(a) Implementation

The official plan/official plan amendment policies for a special policy area are implemented by a municipality and the Ministry of Natural Resources outside the area of Conservation Authority jurisdiction through the zoning by-law process.

The policies developed for the special policy area will have no legislative basis for enforcement under the Planning Act unless they are addressed in the zoning by-law. It is therefore important that close ongoing liaison among the agencies that developed the policies, be maintained after the approval of the official plan/official plan amendment to ensure that the proposed zoning by-law provisions adequately address all of the special policy area policies.

Ideally, the alternative implementation mechanisms will have been previously discussed or outlined in the implementation section of the official plan/official plan amendment. However, if additional information is required or unforeseen problems arise at the time the implementing zoning by-law is being prepared, it may be necessary to reconvene the special policy area technical committee. As a minimum, the implementing zoning by-law should be circulated in draft form to the agencies represented on the technical committee prior to public meetings and/or prior to the bylaw receiving three readings by council.

Special policy area policies are also implemented by Conservation Authorities where they exist, through the issuance of permits under Section 28(1) of the Conservation Authorities Act where such regulations have been adopted. It is therefore important to establish and maintain a close working relationship between the Conservation Authority and the municipality to ensure that any necessary approvals under the Planning Act and the Conservation Authorities Act are coordinated and mutually supportive.

(b) Review and Update

As flood plain information/works or reduced flood standards are approved and/or completed the Special Policy Area policies should be reviewed by the respective participants and the municipal documents amended as necessary.

Where no changes to the Special Policy Area policies, land use designations or boundaries are necessary and the policies/schedules are being transferred to another municipal document, further approvals of the Minister of Natural Resources are not required.

1 A minimum acceptable level of protection for Special Policy areas has not been included in the Guide due to the extent of variation in flood plain characteristics which exists province-wide. However, the 100 year flood has been used almost exclusively as the minimum acceptable flood standard and the CMHC lending policy is also based on this level. (See Section 5.4, Federal Legislation.) SPAs which include a minimum acceptable level of protection which is less than the 100 year flood will require substantial justification. In this regard it should not be interpreted as either the flood standard or the 100 year flood elevation. In all situations as much of floodproofing as possible should be incorporated in the policies.

APPENDIX 6 : FLOODPROOFING

INTRODUCTION

Floodproofing is defined as a combination of structural changes and/or adjustments incorporated into the basic design and/or construction or alteration of individual buildings, structures or properties subject to flooding so as to reduce or eliminate flood damages. It is acknowledged that this term is somewhat misleading, since total protection from flood damage cannot always be assured. However, if applied effectively, floodproofing can play a significant role in comprehensive flood plain management.

Floodproofing is generally most appropriate in situations where moderate flooding with low velocity and short duration is experienced and where traditional structural flood protection, such as dams and channels are not considered to be feasible. Although measures can be applied to both existing and new developments, it is usually impractical, expensive and extremely difficult to flood proof existing buildings.

Since floodproofing is best incorporated into the initial planning and design stages, new development has the greatest potential for permanent structural adjustment. In general, floodproofing can be applied most economically and effectively in the design of new buildings in developing areas. It can also be applied to infilling situations and proposed additions in developed areas. However, as well as providing adequate flood protection, new development within developed areas will have to take into account special considerations such as the aesthetic blend with neighbouring properties.

Floodproofing, whether wet or dry should be no lower than the 1:100 year flood level. The only exceptions are in cases where an addition is proposed to an existing structure or there is one remaining infilling lot in a neighbourhood. In these instances, the floodproofing level should be no lower than the first floor levels of the existing structure or the adjacent structures.

TYPES OF FLOODPROOFING

All floodproofing measures can be described as active or passive and providing wet or dry protection.

Active vs Passive

Active floodproofing requires some action, i.e. closing watertight doors or sandbagging for the measure to be effective. Advance flood warning is almost always required in order to make the flood protection operational.

Passive floodproofing measures are defined as those that are in place and do not require flood warning or any other action to put the flood protection into effect. These include construction of development at or above the flood standard, or the use of continuous berms or floodwalls.

Dry vs Wet Protection

The object of dry floodproofing is to keep a development and its contents completely dry. Such can be carried out by elevating the development above the level of the flood standard or by designing walls to be watertight and installing watertight doors and seals to withstand the forces of flood waters. The benefit of elevated floodproofing is that it is passive and advance warning of an impending flood is not required. Temporary watertight closures, on the other hand, are considered to be active floodproofing usually requiring advance warning for operation.

Wet floodproofing is undertaken in expectation of possible flooding. Its use is generally limited to certain specific non-residential/non-habitable structures (e.g. arena, stadium, parking garage), but many of the techniques of wet floodproofing can be used with certain dry floodproofing approaches. The intent of wet floodproofing is to maintain structural integrity by avoiding external unbalanced forces from acting on buildings during and after a flood, to reduce flood damage to contents, and to reduce the cost of post flood clean up. As such, wet floodproofing requires that the interior space below the level of the flood standard remain unfinished, be non-habitable, and be free of service units and panels, thereby ensuring minimal damage. Also, this space must not be used for storage of immovable or hazardous materials that are buoyant, flammable, explosive or toxic. Furthermore, access ways into and from a wet floodproofed building must allow for safe pedestrian movement.

For new development, dry floodproofing above the level of the flood standard can generally be economically and easily achieved in the design and early construction phase. However, dry floodproofing of structures which will have portions below the level of the flood standard will require additional special design attention so that the structure will resist all loads including hydrostatic pressures.

TECHNICAL CONSIDERATIONS

Once flood waters enter a development, the risk of loss of life and flood damage will be determined by the location of the habitable portion of the buildings. The habitable portion of a structure is defined as living space intended for use by the occupant with the key concern being overnight occupancy. This includes buildings used for residential, commercial, recreational, and institutional purposes. In considering appropriate floodproofing measures, the habitable portion of the building should be designed to eliminate or minimize the risk of flood damage and loss of life.

As a rule, damages increase rapidly with the depth of flooding. Major structural damage occurs when a structure is weakened, totally collapses or is displaced. Damage to contents, such as finishes, trimwork, furniture, appliances, equipment and storage materials, also represents a substantial portion of the total loss. In addition, it is difficult to assign a dollar value to compensate for human suffering caused by a flood.

Thus, protection to at least the level of the flood standard is significant in reducing human suffering and property damage. In selecting between wet or dry flood protection, consideration must be given to the type of development, need for floodproofing and cost effectiveness. Further, selection of active or passive measures will depend on location of the habitable portion of the development below or above the level of the flood standard, local flood warning, and accessways.

As well, all mechanical and electrical systems should be designed and installed so that the heating, lighting, ventilation, air conditioning and other systems are not vulnerable to flood damage during the flood standard. Where flooding could interrupt key power supplies, it may be necessary to provide stand-by or backup systems, with power and controls located above the level of the flood standard.

In order to determine the most appropriate floodproofing measure, the full extent of the flood hazard must be evaluated. This section outlines technical considerations which can assist in determining the most suitable floodproofing measure.

(1) Flooding as a Threat to Life

Hazard to life is linked to the frequency of flooding, and to depth of flood waters and the velocity of flow in the floodplain. Depth increases buoyancy and velocity increases instability, so that each of depth and velocity should be studied independently or as a combined function.

(a) Depth

Any person in the midst of a flooded area will be acted upon by a buoyant force equal to the weight of water displaced by that person. The volume of displaced water and this force increases with depth until neutral equilibrium is reached and the person begins to float.

Average adults and teenage children remain stable when standing in flood depths up to about 1.37 m (4.5 ft.). The average school child 6 - 10 years old would float at about 1.1 m (3.5 ft.), although smaller, younger children in this range would float at a depth of about 0.98 m (3.2 ft.).

Hence, in terms of depth and individuals who could be present in the floodplain during a flood:

- depths in excess of about 0.98 m (3.2 ft.) would be sufficient to float young school children;

- a depth of about 1.37 m (4.5 ft.) is the threshold of stability for teenage children and most adults.

(b) Velocity

Moving water in the floodplain exerts a lateral force resulting from momentum thrust of the flood flow. This force acts to displace objects in a downstream direction. The shear force of friction of a person on the wet surface of the floodplain resists this force. However, even relatively low velocities of flow in the floodplain can pose possible flood hazards.

The force exerted by various flow velocities can be developed for different age and size groups, but because its effect is tied to depth, a better appreciation of velocity effects can be gained by looking at both depth and velocity in combination.

(c) Combination of Depth and Velocity

As a guide for personnel involved in stream flow/depth monitoring, the simple “3 x 3 rule” was developed in the U.S. based on 3 ft depth and 3 ft/s velocity values. The rule suggests that people would be at risk if the product (multiple) of the velocity and the depth exceeded 0.8 m^2/s (9 $ft.^2/s$).

The Water Survey of Canada has the same rule of thumb and its Hydrometric Field Manual (1981) states, “a general rule of thumb which has been used in the past is arrived at through the product of the depth and velocity. Generally speaking, if the bed is firm and provides good footing, the product of these two factors should be slightly less than 1 m^2/s, or roughly 9 $ft.^2/s$”.

It should be noted that this rule of thumb applies to trained professionals whose regular work accustoms them to the dynamic forces of river flows, buoyant forces from partial submergence and recognition of potential hazards, e.g. rocks, depressions, etc. They also enter the stream with equipment which will assist them in maintaining stability, e.g. tag line, wading rod, strap-on cleats for greater stability.

It is considered highly unlikely that such equipment would be available to most occupants of floodproofed buildings in the flood plain. It seems equally unlikely that these occupants would have the same level of experience as water survey staff in dealing with high depths, current speeds, unsteady footing, or cold weather/water conditions.

As a result, it is likely that the simple rule of 3 x 3 product (1 m^2/s or 9 $ft.^2/s$) represents an upper limit for adult male occupants in the flood plain and that it would be reasonable to consider something lower as being more representative of a safe upper limit for most flood plain occupants.

As noted earlier, any person on foot during a flood may be subject to a number of forces in the floodplain. Excluding impact by ice and/or other debris, these forces include:

- an upward buoyant force, equal to the weight of the fluid displaced;

- a lateral force exerted by the moving water (linear momentum);

- unbalanced hydrostatic forces.

Resisting these forces are:

- the shear force of friction acting through the weight of the person standing on a wet surface in the floodplain.

Figure 6-1 provides a graphical representation of depth and velocity hazards in the flood plain to show the limits of stability. Unit weights of 976, 1464 and 1952 kg/m^2 (200,300 and 400 $lb/ft.^2$) are used. Adults of average size would fall into the range between 976 -1952 kg/m^2 (200 - 400 $lb/ft.^2$) but young children would more appropriately fall into a range of 732 - 1464 kg/m^2 (150 - 300 $lb/ft.^2$). Only 7% of Ontario’s population is within the 6 - 10 year age range, i.e. young children (Statistics Canada, 1981).

The coefficient of friction between foot apparel and wet grass, gravel, bare soils, pavements or other wet surfaces under flood conditions is not well known. A standard table of friction coefficients suggests that friction factors in the order of 0.3 to 0.6 could be characteristics of the ratio of the force to body weight required to initiate movement over unlubricated, dry surfaces. It is assumed that a lower friction factor range would be representative of the same state for a person standing on wet grass or pavement under flood conditions. The sensitivity of the stability calculation to friction factors of 0.15 and 0.3 is shown on Figure 6-1.

Any flood plain situation giving velocity and depth conditions lower than the appropriate curve for that individual is one where that person would be in a stable condition in the flood plain. Conditions of velocity or depth exceeding the appropriate stability curve would be unstable conditions for the same individual.

It is also appropriate to note that this analysis is based on a person standing still in the flood plain. Once a person begins to move to install floodproofing measures or leave the flood-prone area, stability is reduced further.

Figure 6-2 presents the same chart with overlays of different "product rules" (products of depth x velocity in m and m/s (ft. and ft./s). The 3 x 3, 3 x 2 and 2 x 2 rules are shown on the figure with the use of 0.3 friction factor to represent wet conditions. The cross hatched area defines a region of depth and velocity combinations which are stable, low risk combinations for most individuals likely to be present in the floodplain during a flood.

The 3 x 3 line encloses a large area of depth and velocity conditions which would lead to instability for most individuals. The 3 x 2 line represents a general average, but it too encompasses areas of instability for many individuals.

The 2 x 2 line excludes most of the unstable conditions for most individuals and would appear adequate at first glance. However, the 2 x 2 rule also has limitations as shown on the graph. At low velocity but depths greater than 0.9 - 1.2 m (3 - 4 ft.), most individuals would become buoyant. Similarly, in areas where flood plain depths may be less than 0.3 m (1 ft.) but where velocities exceed 1.5 - 1.8 m/s (5 - 6 ft./s) encountered on roadways or bridge crossings, for example, stability conditions would be exceeded and some individuals would be swept off their feet.

Although no product rule exactly defines this region, a reasonable approximation of the low risk area can be made with a product rule that includes some constraints on the domain of depth and velocity. For example, a product depth and velocity less than or equal to 0.4 m^2/s (4 $ft.^2/s$) defines the low risk area providing that depth does not exceed 0.8 m (2.6 ft.) and that the velocity does not exceed 1.7 m/s (5.5 ft/s). By contrast, in a situation where the depth and velocity are 1.1 m (3.5 ft.) and 0.3 m/s (1 ft./s) respectively, the product is less than 0.4 m^2/s (4 $ft.^2/s$) but the depth limit is exceeded. Hence, these conditions define a high risk area for some individuals.

It is evident that this approximate classification is somewhat conservative; but until further research is undertaken, it provides a reasonable factor of safety for all individuals - young and old - who may be present in the floodplain.

(2) Duration of Flood

The duration of a flood or the length of time a river overflows its banks, reaches its crest and recedes to within its banks depends on the efficiency of the river to transport the flood waters. Since the size of the watershed, time of concentration and duration of a flood affects the type of impact and pressure on the development, floodproofing measures must be designed to withstand these forces for the required period of time.

(3) Rate of Rise and Fall

The rate of rise and fall of a flood to and from its crest can affect the type and extent of floodproofing. For example, where the rise and fall are very sudden, there may not be time to implement active floodproofing measures, such as watertight seals and doors and thus these approaches would be deemed unacceptable. The rate should also be considered in investigations of slope stability for certain types of soils where a quick drawdown of flood waters may pose problems.

(4) Flood Warning System

The availability of advance warning can play an important role in determining the most appropriate measure. Where active floodproofing procedures are contemplated, lead time for implementation of appropriate protective measures and devices must be related to the amount of advance warning.

(5) Structural Integrity

When buildings and structures are surrounded by flood waters, they cause unbalanced pressures and loadings on all wetted surfaces, which increase rapidly with depth. Unbalanced pressures can cause structural and sub-structural damages which can completely collapse or displace the development. In order to design the most appropriate floodproofing measures, it is important to determine the effect of stresses on the proposed building.

The stresses imposed on a building are due to hydrostatic, hydrodynamic and impact loadings, depending on its location. Hydrostatic loads are developed by water that is either still or moving at a low velocity. These loads may be defined as acting vertically downward (i.e., on floors), or vertically upward (i.e., uplift), or laterally when acting horizontally on walls. Hydrodynamic loads results from the flow of water against or around a structure at moderate or higher velocities. These loads are directly dependent on the velocity of flow, and can also adversely affect the floodproofing measures by causing erosion and scour. Impact loads are caused by water-borne objectives, debris and ice. Their effects become greater and more crucial as the velocity and weight of objects increase. Impact loads are difficult to predict and define accurately. However, a reasonable allowance can be made with the knowledge of the conditions of the site.

(a) Superstructures (Above Ground)

Hydrostatic Loading Effects

Until the mid-1970s, it was assumed that standard design and construction practices - without modification - would be adequate to ensure that floodproofing by closures and seals could be conducted to moderate depth/ hydrostatic loading without threatening the structural integrity of the above ground/superstructure of most buildings. However, various research by the U.S. Corps of Engineers over the years, has suggested otherwise.

Studies on structures of conventional design have determined that:

· brick veneer, frame structures (such as a typical home) would resist hydrostatic loading up to about 0.8 m (2.5 ft.) without damage;

· concrete block structures with limited or no reinforcement (such as the small warehouse building) displayed similar resistance characteristics and would not be damaged by hydrostatic loading up to 0.8 m (2.5 ft.). Above this at 0.9 and 1.2 (3 and 4 ft.) depths deflection and cracking became significant;

· solid brick structures responded in a similar manner. Tests with these also included end and side walls and walls with and without door openings. Walls with ceiling joists (with and without door openings) were found adequate to resist loadings to about 0.8 m (2.5 ft.). Walls with ceiling joists provide much stronger, but failed explosively when 2 x 4 supports were snapped;

· poured concrete walls were not tested, but from experience with other structural designs it was presumed that conventional design techniques would prove adequate against hydrostatic loads to at least 0.9 (3 ft.).

Therefore, 0.8 m (2.5 ft.) would appear to be the upper limit of effective flood depth (static plus equivalent hydrodynamic head) which can be resisted by conventionally designed structures without affecting structural integrity.

Studies on structural integrity during flow conditions have also given an appreciation of the permeability of conventional structures, in that:

· brick structures of conventional design begin to leak almost immediately and badly, when in contact with flood waters;

· concrete block structures of conventional design also leak badly at a rate that exceeds that of brick structures.

Tests also conducted to determine if materials or surface coatings would enhance water tightness found:

· no clear sealants (e.g. epoxy) were completely effective;

· no asphaltic material was completely effective;

· embedded roofing felts with polyethylene sheeting laid between a second brick course were found effective - but exceptionally stringent quality control of workmanship was required (particularly at joints);

· flood shields/bulkheads also presented difficulties and were for the most part ineffective unless designed especially with gaskets, smooth surfaces and locking bolts;

· certain thick, non-tear materials can be used as external "wrappings" to effectively seal buildings against infiltration. These are very special materials and fall into the category of "active" measures vs "passive", permanent measures.

In summary then:

· conventional designs are not water resistant/waterproof for even low depths of flooding;

· new structures should be designed from scratch for complete water tightness (or if not completely watertight must incorporate an internal system to collect and remove water seepage);

· new structures using conventional designs can be made watertight (without re-design) but the only proven approach so far uses external "wrapping".

Erosion

Flow velocities which will cause erosion of grass covered slopes or erosion around foundations are difficult to determine. Factors such as type of cover, slope and soil conditions must be taken into account. For most common situations, the range lies between 0.8 m/s and 1.2 m/s (2.5 ft/s and 4 ft/s) for easily eroded soils and 1.1 m/s to 1.5 m/s (3.5 ft./s to 5 ft./s) for more erosion resistant soils.

Impact Loading and Debris Accumulation

This aspect of structural integrity has not been studied in the field because it is practically impossible to establish velocity/depth limits associated with loadings caused by debris accumulation and the impact of floating objects on the flood plain. The nature of debris accumulations and size and shape of floatables simply varies too significantly.

Ice, debris and other floating materials can result in significant impact loading on buildings within the flood plain or increase the loads on buildings as a result of blockage. Although these loads are difficult to estimate a reasonable allowance must be made in design. Sites where the potential for such loading is high should simply be avoided or buildings should be designed/landscaped to intercept/deflect materials before the building is affected.

In cases where floodproofing is achieved by elevation on columns or piles, the clearing space between the columns or piles should measure perpendicular to the general direction of flood flow and should be adequately designed to minimize possible debris blockage. The open space created below the level of the flood standard should remain essentially free of more buoyant or hazardous materials.

(b) <u>Substructures/Basements (Below Ground)</u>

Based on normal (conventional) construction methods, any hydrostatic head in excess of 0.2 m (0.7 ft.) may result in damage to basement floors (i.e. the upward force of groundwater on the basement floor).

Even where the basement of a single storey brick or masonry structure has been structurally reinforced and/or made watertight, structural integrity or buoyancy may pose problems when groundwater (saturated soil) levels are 1.2 - 1.5 m (4 - 5 ft.) above the level of the basement floor. Much depends on the duration of the flooding, type of soil and the presence/effectiveness of the drainage system.

(6) <u>Vehicular Access</u>

Little or no information exists in the literature regarding ingress/egress criteria for vehicles.

The question of safety for the passage of vehicles can be subdivided into:

· flood depth and velocity considerations affecting egress of private vehicles from floodproofed areas;

· flood depth and velocity affecting access of private and emergency vehicles to floodproofed areas.

(a) <u>Private Vehicles</u>

In general, water contact is one critical issue in terms of its effect on the ignition/electrical system and the exhaust system. In the former, the distributor and/or spark plugs are the main items of concerns and those which are typical problem areas for most motorists.

Private vehicles come in all shapes and sizes and it is practically impossible to identify "typical" vehicles for assessing the elevation of key electrical components from the road surface. It appears likely that a depth of about 0.4 m - 0.6 m (1.5 - 2 ft.) would be sufficient to reach the distributor or plugs of most private vehicles. They would fail to start at this depth and hence vehicular egress will be halted. Cars may start at lower depths but then "splash" from driving on wet pavement or from the radiator fan would become a concern.

The issue of the exhaust system and the effect that flooding can play on engine back pressures/expulsion of exhaust gases appears to be the controlling factor. Difficulty would probably be experienced in starting most vehicles if the vehicle is standing in water at a depth that covers the muffler. The vehicle may start and continue to run if it is quickly removed from the water but if remains at that depth, there is a strong possibility that it will fail soon after.

Again, it is practically impossible to generalize this depth but for most family automobiles something in the range of about 0.3 m - 0.4 m (1 - 1.5 ft.) would be the maximum depth of flooding before potential egress problems would result.

A hazard diagram such as Figure 2 may also be derived to evaluate the significance of flood velocity (and depth) on vehicles. Such a diagram would indicate that a "typical" North American car would not be significantly affected by velocities up to about 4.5 m/s (15 ft./s) or more at flood depths at less than 0.3 m (1 ft.). At running board depth or slightly above 0.3 m (1 ft.) the maximum velocity for stability drops to about 3 m/s (10 ft./s) and at about 0.4 m (1.5 ft.) depth an average vehicle may be displaced by velocities as low as 0.3 - 0.6 m/s (1 - 2 ft./s), with smaller vehicles becoming buoyant.

(b) Emergency Vehicles

Emergency vehicles operate under the same constraints relating to the electrical/exhaust system. Most police vehicles and ambulances would be limited by exhaust considerations, although emergency vans are better equipped to avoid splash problems since the key electrical components are higher above the road surface.

Diesel fire vehicles with top exhausts appear best suited for flood conditions. Their road clearance is high and it is suggested that 0.9 m - 1.2 m (3 - 4 ft.) of flood depth would not present a problem. These vehicles are about 10 times heavier than most automobiles and hence are resistant to displacement by higher velocity flood flows. Operations at velocities in excess of 4.5 m (15 ft./s) would probably not pose a problem when these vehicles are moving over a good/non-eroding base.

(7) Portable or Mobile Buildings and Structures

A portable or mobile building is one that is not permanently tied or anchored to a foundation and can be transported by means of a hauler. Portable or mobile buildings can be located on individual sites or in a park or subdivision. They can be used for temporary purposes, such as for construction crews or as full-time residences/seasonal homes with overnight occupancy.

When located in flood plains, portable or mobile buildings are highly susceptible to flood damage. Since they are not affixed to a permanent foundation, flood waters may easily sweep such buildings off their sites. Without advance warning, residents can be entrapped in the building. In addition, portable or mobile buildings can increase the flood hazard as they collide with other structures or block bridge openings or culverts. Despite this, portable or mobile buildings often are located in flood plains because:

· flood plain land acquisition costs may be lower;

· swamp conditions and higher water table which prevail in flood plain areas may preclude construction of permanent homes with basements; and

· potential recreational access by locating close to the water's edge.

Ideally, portable or mobile buildings should not be located in the flood plain. However, when located in the flood fringe, they should be properly floodproofed to the flood standard, in order to prevent flotation, collapse and lateral movement. Due to the inherent hazard of remaining in a mobile building during a flood, contingency plans indicating escape routes and alternative vehicular accessways should be prepared.

Where the portable or mobile building is on site temporarily, it may not be feasible to meet all the requirements for floodproofing. In such cases, temporary location of portable and mobile buildings in the flood fringe may be considered where the time frame is very short and sufficient flood warning would allow the structure to be hauled away in advance of the flood.

(8) Floodproofing Complexity

The complexity of floodproofing techniques (and to a degree the cost) is best related to depth and type of floodproofing considered.

(a) Closures and Seals

It appears that external walls can be floodproofed by closures and seals to a flood depth of about 0.8 m (2.5 ft.). Beyond this depth, structural integrity is threatened and special reinforcing or revised designs (with poured concrete walls for example) are required.

Dry floodproofing to this depth can be completed with the use of impervious external "wrappings". These contingency wrappings are anchored beneath the ground surface along the foundation and rolled upward and hung into place along the walls of building prior to flooding. Equivalent dry floodproofing using internal sealants, doubled walls, etc. with flood shields at openings is more complex, expensive and uncertain as to effectiveness.

Basements can be closed and sealed to levels of about 1.2 - 1.5 m (4 - 5 ft.) above the floor slab with poured concrete designs employing additional reinforcement and special attention to monolithic construction. Beyond this level, the procedure becomes complicated as buoyancy/uplift must be addressed through anchors and/or added wall and slab thickness.

Overall, closures and seals is fraught with possible problems and is considerably more complicated than other floodproofing approaches.

(b) Elevated structures

Structures on Fill

Floodproofing on fill is generally considered for slab on grade construction. It is not a complex procedure and conventional building techniques are employed once the pad is down. The principal concern is fill compaction which must usually be done in 0.2 - 0.3 m (0.5 - 1 ft.) lifts. Beyond 0.6 - 0.9 m (2 - 3 ft.). however, pad sizes increase, compaction requirements become more important and an engineer or soils consultant should be employed for design review and inspection. Increased elevation may also lead to requirements for pad sizes in excess of lot size and, hence, additional requirements for erosion protection, etc.

Houses with conventional basements can also be placed in fill to elevate the first floor to a level about 2.1 - 2.4 m (7 - 8 ft.) above grade (i.e. the basement is founded on grade and the basement walls are surrounded by fill). At 1.2 - 1.5 m (4 - 5 ft.) above grade, the procedure is complicated by the need for wall and slab reinforcement, and anchors to prevent buoyancy.

Elevation on Columns, Piles, Piers and Extended Foundation Walls

Elevated structures using these techniques must be designed with consideration for debris loading, orientation of supports, effective submergence on foundation soil conditions and anchorage, bracing and connection details, availability of mechanical equipment, etc. In most instances, an engineer should be consulted to ensure that the possible effects of flooding are considered in the design. There are more factors to consider than conventional house construction on fill and, hence, these approaches could be considered more complex.

The majority of elevated buildings use posts for support (steel or timber). Installation becomes more complex at lengths in the range of 3.6 - 4.8 m (12 - 16 ft.) since machinery is needed for installation. A range of 3 - 3.6 m (10 - 12 Ft.) seems typical for most homes which use extended posts.

Mechanically-driven piles are reported to be the best solution if severe erosion is anticipated. Pile driving equipment and skilled operators are at a premium and, because of the initial expense, this technique may be too complex/unnecessary for flood depths less than 1.5 - 1.8 m (5 - 6 ft.)

Piers/columns are generally constructed with brick, concrete block or poured concrete. The common elevation range for each of these approaches is as follows, beyond which increasing complexity is assumed:

- 0.4 - 1.8 m (1.5 - 6 ft.) for brick piers;
- 0.4 - 2.4 m 91.5 - 8 ft.) for reinforced concrete masonry piers;
- 0.4 - 3.6 m (1.5 - 12 ft.) (or more) for poured in place, reinforced concrete piers.

Extended foundation walls make a relatively simple and effective foundation for elevated structures but again must be designed with consideration for loads and pressures anticipated in the flood plain.

Berms and Floodwalls

Berms (or levees) and floodwalls used for floodproofing are low structures built around single homes or individual industrial complexes. Property design is more complex since material and construction practices must be closely monitored, they must be regularly maintained (in the case of berms), and they usually require adequate pumping facilities to handle interior drainage and seepage. Both berms and floodwalls usually have some opening for access and consideration must be given to closure.

In many instances, berms and floodwalls should be designed by qualified professional engineers.

Intentionally Flooding a Building (Wet Floodproofing)

Intentionally flooding a building for the purpose of balancing internal and external pressures so as to maintain structural integrity is in itself not complex. To ensure minimal damage and quick clean up, a number of conditions have been placed on the use of wet floodproofing by agencies such as Canada Mortgage and Housing Corporation. Requirements include:

· at least two openable windows located on opposite sides of the building;

· tops of window sills to be not less than 150 mm below grade (to allow flood water into the basement);

· basements to remain unfurnished and contain non-habitable space only;

· mechanical and electrical equipment, heating units and duct work to be located above the flood standard;

· sump pump required;

While wet floodproofing may be designed and provided for in a building, there is no guarantee over time that the requirements will be maintained. In particular, it is difficult to control the “finishing off” of basements which would then result in damages when wet floodproofing measures were put into effect. Therefore, while wet floodproofing may appear desirable initially, the ability to ensure the principles and requirements of wet floodproofing are maintained in the future must also be considered.