PART 3

FLOODING HAZARD

FLOODING HAZARD

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3.1 INTRODUCTION

Lake level records dating back to 1860 demonstrate the variable nature of the Great Lakes. Of the two key factors influencing long-term and short-term changes in lake levels, natural phenomena (i.e., rainfall, evaporation, wind, storms, etc.) by far, cause the greater magnitudes of changes, measured in terms of metres of change, than does human intervention (i.e., diversions, water control structures, etc.) which can be measured in terms of centimetres of change.

The most familiar changes in lake levels are seasonal fluctuations as evidenced by average differences of about 0.6 to 1.1 metres (2.0 - 3.5 feet) in lake levels between the summer and winter months. Superimposed on these seasonal fluctuations are some extremely short periods of significantly larger magnitudes of lake level changes. The most temporary of these are caused by storm winds which blow over the lake surfaces pushing the water to the opposite side or end of the lake. These "wind setups", or "storm surges" have frequently caused total differences of more than 4 metres (13 feet) and occasionally as high as 5 metres (16+ feet) in lake levels at opposite ends of Lake Erie.

Over 130 years of lake level records demonstrate that large, long-term lake level changes, thought by many shoreline residents to occur in "cycles", actually vary between 7 to 30 years. As such, these periods of changes in lake levels are neither regular or readily predictable and are instead the direct influence of changes in climate and hydrological patterns across the Great Lakes Basin.

Historical lake level records further demonstrate that long-term changes in lake levels are neither immediate or short in duration. Contrary to the duration and magnitudes of riverine flooding threats, which can be measured in terms of hours or days, high lake levels impose long-term threats to shoreline residents which are normally measured in terms of months or years. These threats are often dramatically heightened when combined with wind setup impacts.

The intent of Part 3 of the Technical Guide is to provide an indepth analysis of the *flooding hazard* as defined in the Provincial Policy 3.1, Public Health and Safety: Natural Hazards for the *Great Lakes - St. Lawrence River System*. For the purposes of clarification, Part 3 will examine the factors influencing the definition, magnitude, duration and potential impacts associated with flood levels, wave uprush and other water related hazards in the following manner:

- **Section 3.2** provides an introduction to *Great Lakes - St. Lawrence River System flooding hazard*s, identifying the three main components used to define shoreline flooding: 1) **flood level**; 2) **wave uprush**; and 3) **other water related hazards**;

- **Section 3.3** presents the *flooding hazard* as contained in the Provincial Policy Statement (1996), and the supporting Natural Hazards Training Manual (1996);

- **Section 3.4** introduces and defines the **100 year flood level**, the first of three main components of the *flooding hazard*. The calculated 100 year flood levels are provided for all sectors of the *Great Lakes - St. Lawrence River System* except for those along the Niagara River. In addition, for each of these sectors, the **calculated flood levels, mean monthly levels and wind setups for various return periods** are provided in Appendix A3.1 which also outlines the relationship between the three datums in use (e.g., GSC for topographic mapping; IGLD 1955 for calculated flood levels; IGLD 1985 for present water level data). An explanation of the methods used to calculate the flood levels for the Great Lakes and the connecting channels is presented in Appendix A3.1. The **water levels information** service which is available is provided in Appendix A3.2.

  Section 3.4 includes a **review of studies required to evaluate conveyance of critical flow in connecting channels**;

- **Section 3.5** provides an overview of **wave uprush**, the second primary component of the *flooding hazard*. The discussion of wave uprush also includes **wave overtopping**, **ponding** and **wave spray**. Supporting this section is a separate report, ***Wave Uprush and Overtopping: Methodologies and Applications*** (Atria 1997) which contains a detailed **description on wave uprush and wave overtopping** including an outline of the theory and hydraulic processes involved, recommended procedures for selecting the wave height, nearshore slope and for estimating wave uprush and wave overtopping, the limitations of these procedures, technical guidance for determining acceptable rates of overtopping, including considerations of public health and safety, structural stability and property damage, measures for providing protection against overtopping waves and for addressing drainage concerns and typical worked examples.

  The provincial standards for wave uprush, in the absence of detailed engineering studies, are described as standard horizontal allowances. Two separate horizontal allowances to address wave uprush are identified first, along the shores of the Great Lakes and secondly, along the shores of the connecting channels. The description of these horizontal allowances is further supported by information presented in Appendix A3.7;

- **Section 3.6** provides a description of **other water related hazards**, the third primary component of the *flooding hazard*. Other water related hazards includes such factors as **ice jamming and piling** and **ship generated waves**.

  The discussion of **ice hazards**, as they relate to flooding in the *Great Lakes - St. Lawrence River System*, includes a description of how **ice jams** and **ice piling** occur, and the present scientific state of predicting ice jams and ice piling. This discussion is further supported by more detailed information in Appendix A3.6.

  Finally, this section provides a discussion on **ship waves** or **ship wakes** and of how they relate to the calculation of the *flooding hazard*. The discussion is supported by Appendix A3.6 which includes: a brief overview of studies that have been carried out in Canada and the United States; a discussion of the "energy method" which is the basis for the most recent Canadian research on ship waves; and a description of work currently being carried out by the U.S. Army Corps of Engineers to develop a numerical model to determine ship wave heights, based on a different approach;

- **Section 3.7** provides a description of how to combine the *flooding hazard* limits at junctions of the *Great Lakes - St. Lawrence River System* and *river and stream systems*;

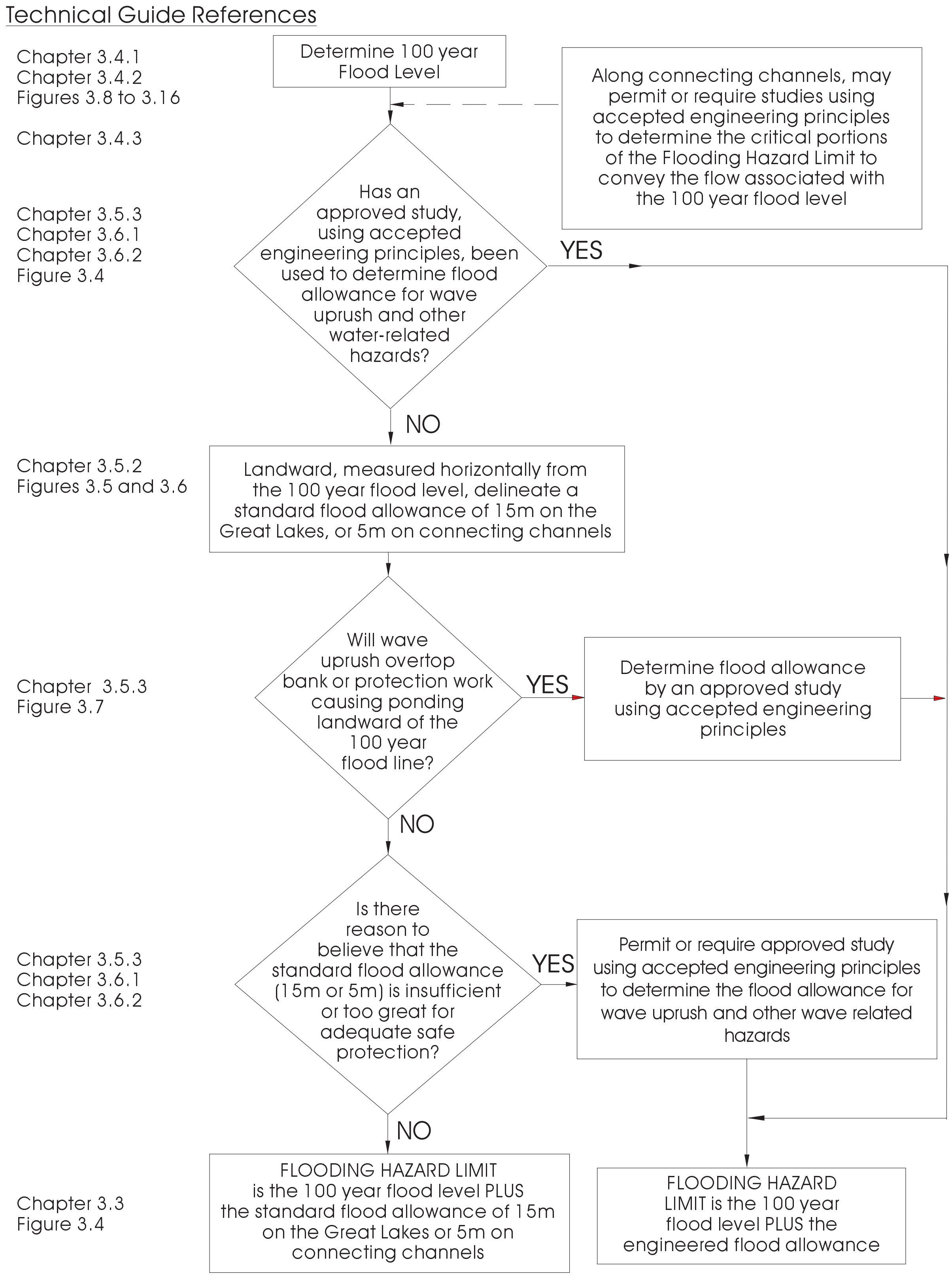
- **Appendix A3.1** describes the *Great Lakes - St. Lawrence River System* **flood levels, monthly mean levels, wind setups** for various return periods, including an outline of difference between Geodetic Survey of Canada Datum and International Great Lakes Datums.

  Appendix A3.1 details the **methods for calculating the 100 year flood levels for the Great Lakes** (i.e., using a combined probability of the monthly mean lake level and wind setup) **and the connecting channels**; provides a **description of the water level data set** which was used including an outline of the **basis of comparison for the water level data set**; and presents a discussion on the **potential influence of serial correlation and climate change**. The models used in undertaking the flood level calculations, **SURGE and HYDSTAT**, are briefly outlined in Appendices A3.3 and A3.4 respectively;

- **Appendix A3.2** provides an overview of the **water levels information** service
- **Appendix A3.3** provides summary outline on the **SURGE Model**
- **Appendix A3.4** provides outline on the **HYDSTAT Model**
- **Appendix A3.5** discusses the standard horizontal **flood allowance for wave uprush** (i.e., 15 metres for the Great Lakes and 5 metres for connecting channels)
- **Appendix A3.6** provides a description of how **ice jams and piling** occurs and present positions on the ability to predict ice impacts and provides an overview of **ship waves or wakes** and of the present studies used to determine the impacts associated with ship waves or wakes

A flowchart summary of the procedure for delineating the *flooding hazard* is presented in Figure 3.1. Figure 3.1 also serves as a guide to the various sections of Part 3.

Figure 3.1: Delineating the Flooding Hazard Limit



3.2 FLOODING WITHIN THE GREAT LAKES - ST. LAWRENCE RIVER SYSTEM

Flooding has historically and repeatedly caused considerable damage along the shorelines of the *Great Lakes - St. Lawrence River System*. As discussed in Part 1 and Part 2 of this Technical Guide, the entire shoreline of the *Great Lakes - St. Lawrence River System* can be considered to have some degree of flood susceptibility which must be calculated and delineated.

Various magnitudes and durations of shoreline flooding, described in terms of three general types of shoreline flooding (see Figure 3.2), are the result of a combination of:

- higher, lakewide, static water levels due to abnormally high levels of precipitation and runoff and the annual lake level fluctuations occurring with changes in the season (see Figure 3.2a);
- short-term, storm induced wind setups or surges (see Figure 3.2b); and
- wave action which rushes up, and possibly beyond, the shore and other water related hazards, including wave overtopping, ice jamming and piling, and ship generated waves (see Figure 3.2c).

The flood level is a combination of the static water level and the wind setup. A wide range of natural hydrological, physical and geological factors have an influence on the higher, lakewide, static water levels, annual water level fluctuations and/or wind setups. As introduced in Part 1 of this Technical Guide, these factors may include rainfall, evaporation, winds, shore configuration and shore processes.

As shown in Figure 3.3a, wave uprush, or wave runup, is essentially the vertical distance reached by the uprushing wave above the stillwater level or flood level. The stillwater level is the level the water would assume in absence of wave action. Wind setup is included in the stillwater level. When the height of the natural shoreline, or of the protection works, above the stillwater level is less than the limit of runup, wave overtopping occurs (see Figure 3.3b). The phenomena of wave overtopping can result in backshore flooding and can potentially threaten the structural stability of the overtopped protection works.

Ice can be damaging to shoreline causing erosion and flooding. Ice piling essentially occurs when wind generated currents and flows carry ice flows onto the shore causing erosion. Ice jams, usually occur during spring ice breakup, involve the clogging of rivers or connecting channels by the broken pieces of ice which in turn can cause shoreline flooding. Ship generated waves, frequently posing a significant impact along the shorelines of the connecting channels, can uprush onto the shoreline and beyond the defined 100 year flood level. The subsequent ship generated drawdown can also then cause scouring and damage to adjacent shorelines and protection works.

In summary, the shoreline *flooding hazard* consists of three components:

- **flood level, which is the sum of the static lake level (i.e., no wind action) and the wind setup;**
- **wave uprush and overtopping; and**
- **other water related hazards, such as ice jamming, ice piling and ship generated waves.**

Figure 3.2: Types of Shoreline Flooding

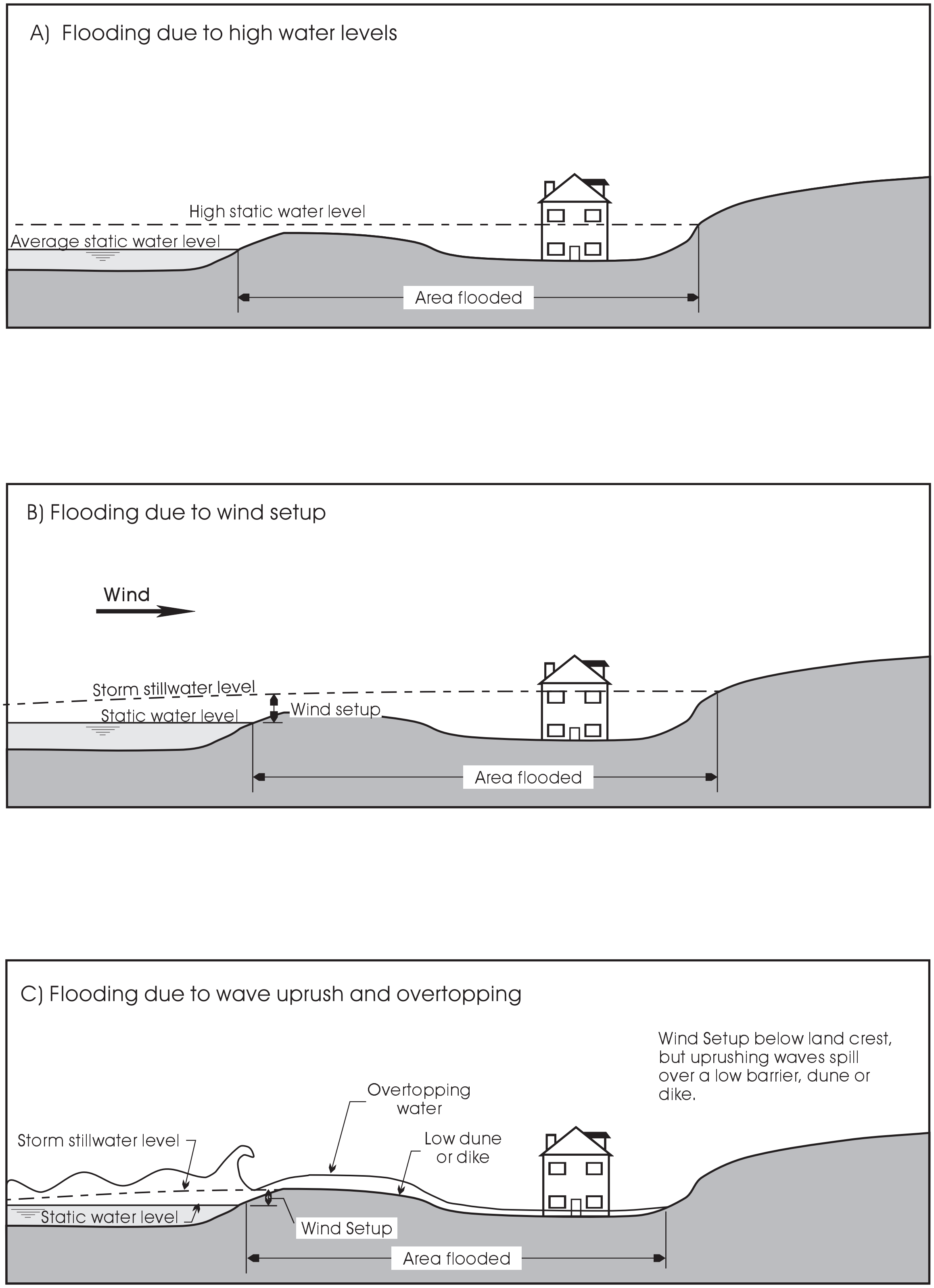
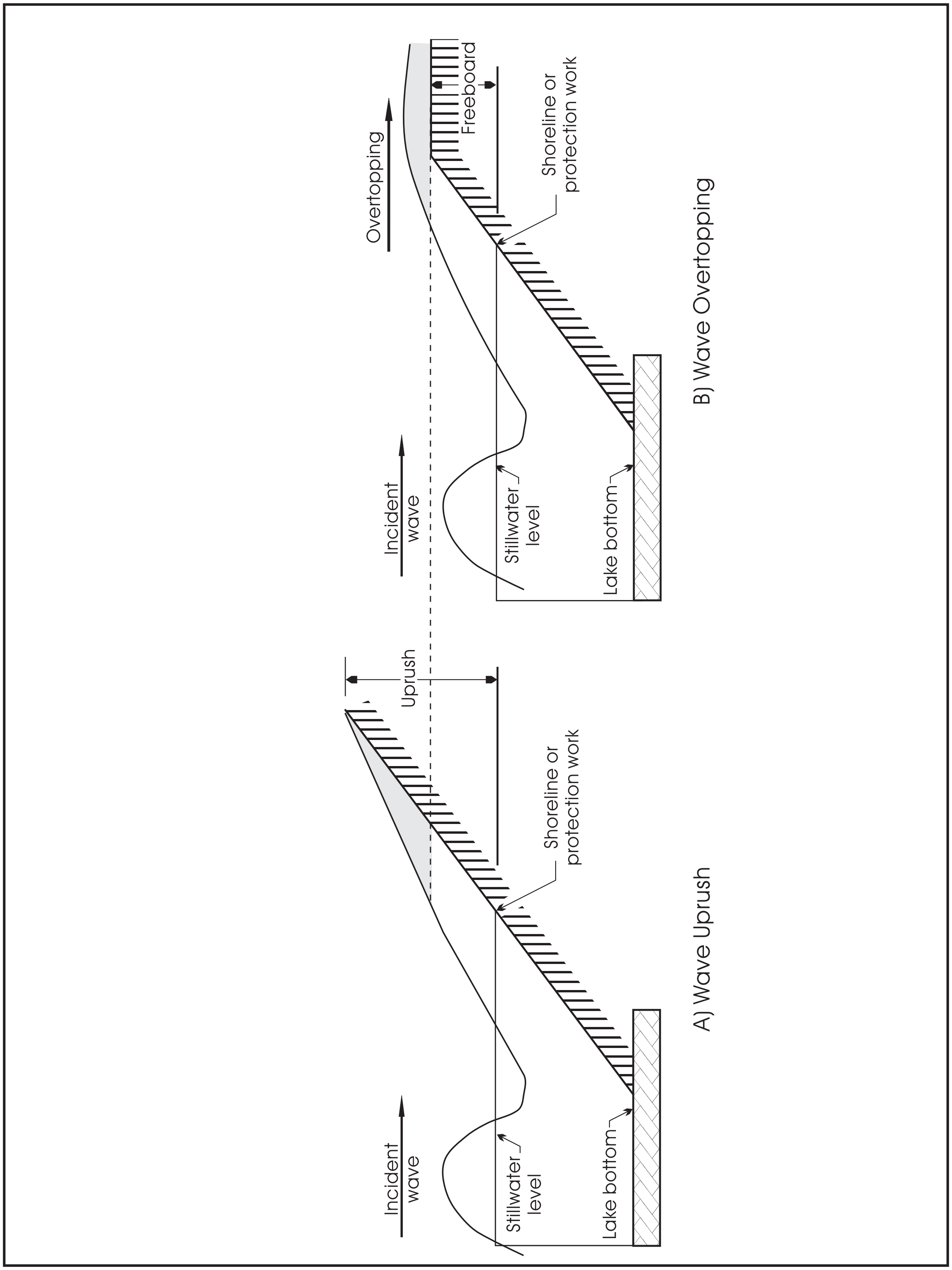


Figure 3.3: Wave Uprush and Wave Overtopping



3.3 PROVINCIAL POLICY: FLOODING HAZARD

The *flooding hazard* along the shorelines of the *Great Lakes - St. Lawrence River System*, as defined in the *Provincial Policy Statement (1996)*, involves the calculation of the cumulative impact of the 100 year flood level, wave uprush and other water related hazards. Specifically, the *flooding hazard* (Figure 3.4) combines the **100 year flood level** (i.e., static water level and wind setup), and a **flood allowance** for **wave uprush and other water related hazards**. The 100 year flood level is represented by a contour line or elevation on the shoreline mapping; the flood allowance for wave uprush, unless done on a site specific basis, is represented by a specified horizontal distance measured landward from the 100 year flood level; and the second component of the flood allowance associated with other water related hazards must be examined on a site specific basis. In the absence of studies to determine the allowance for wave uprush and other water related hazards two standards are identified namely:

- 15 metres for shorelines of the Great Lakes (Figure 3.5); and
- 5 metres for shorelines of the connecting channels (Figure 3.6).

For shorelines of the *Great Lakes - St. Lawrence River System* where flooding and/or wave action overtops a natural bank or protection works, causing ponding landward of the 100 year flood level, the flood allowance for wave uprush and other water related hazards is to be determined by a study using accepted engineering principles (Figure 3.7).

Figure 3.4: Flooding Hazard Limit

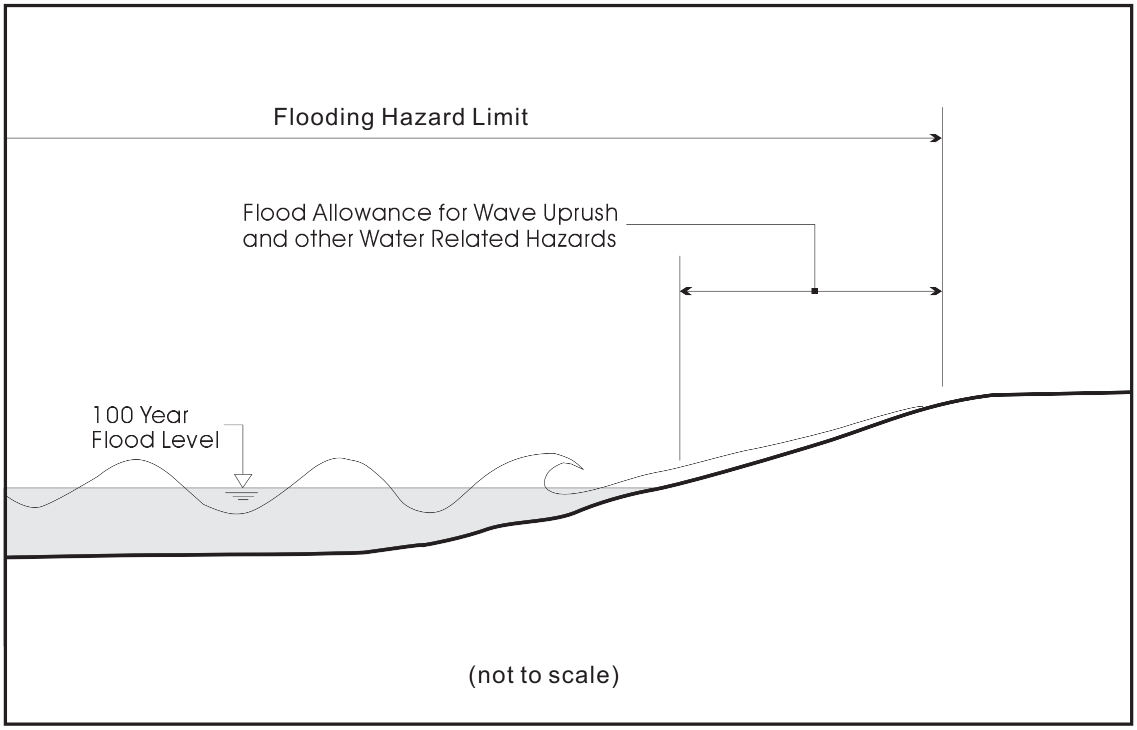


Figure 3.5: Flooding Hazard Limit - Standard 15m Flood Allowance for Great Lakes

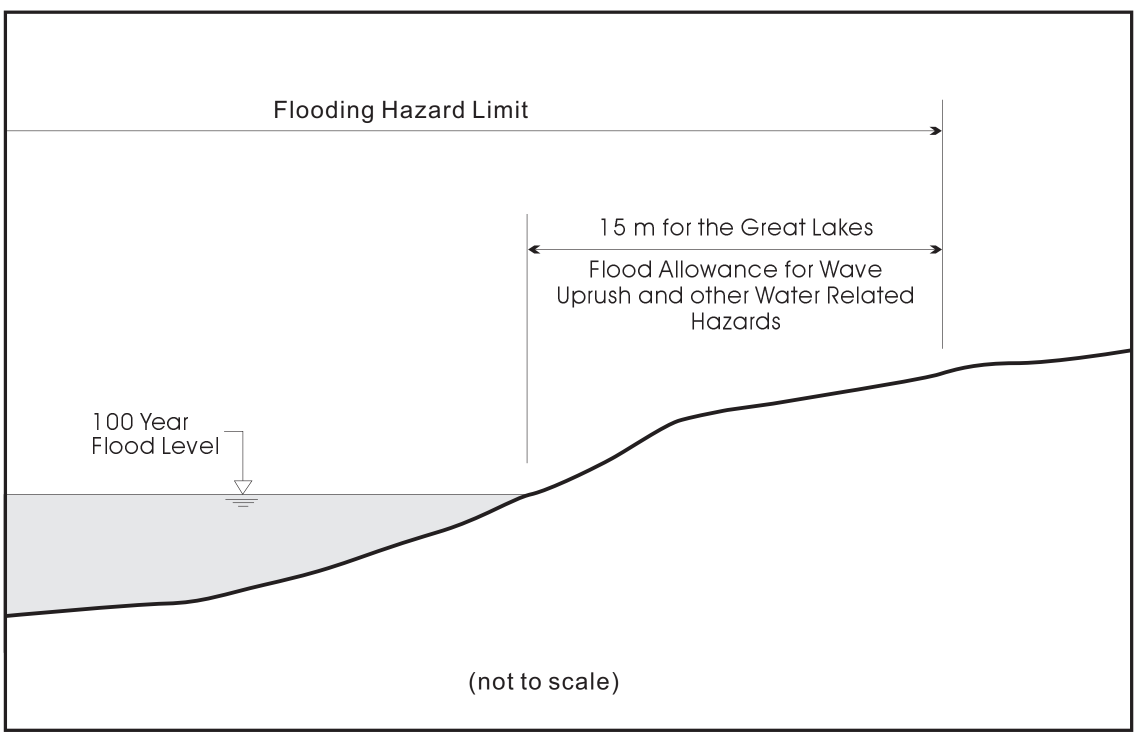


Figure 3.6: Flooding Hazard Limit - Standard 5m Flood Allowance for Connecting Channels

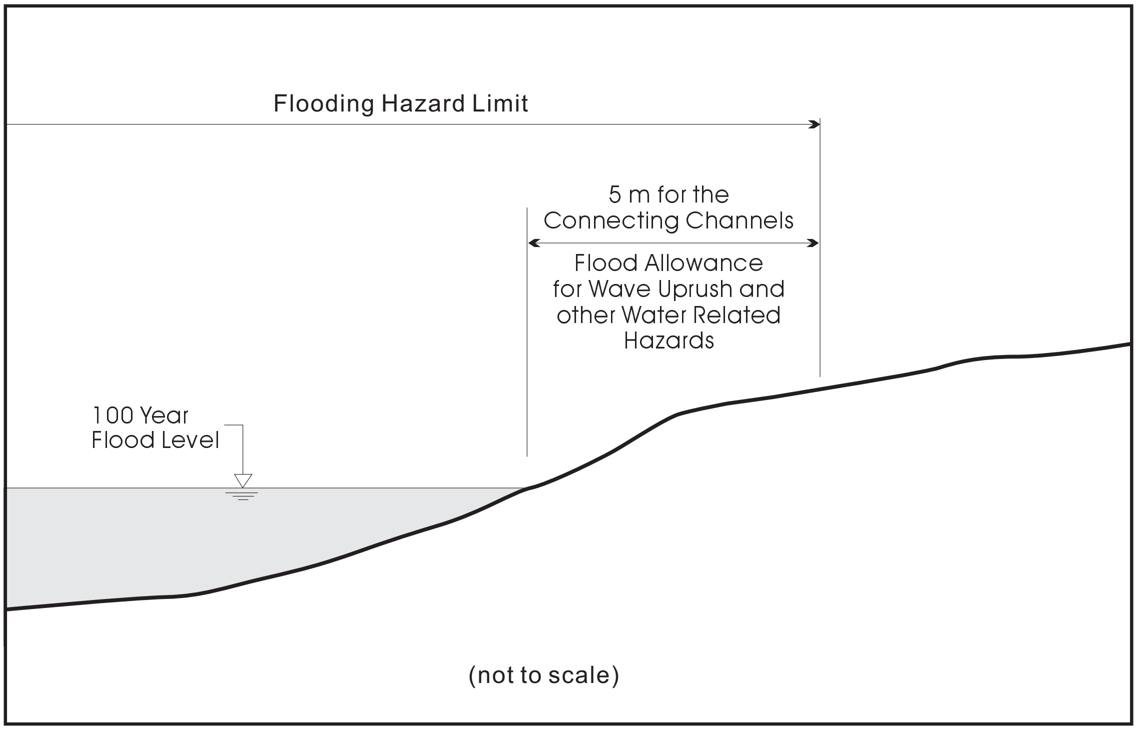
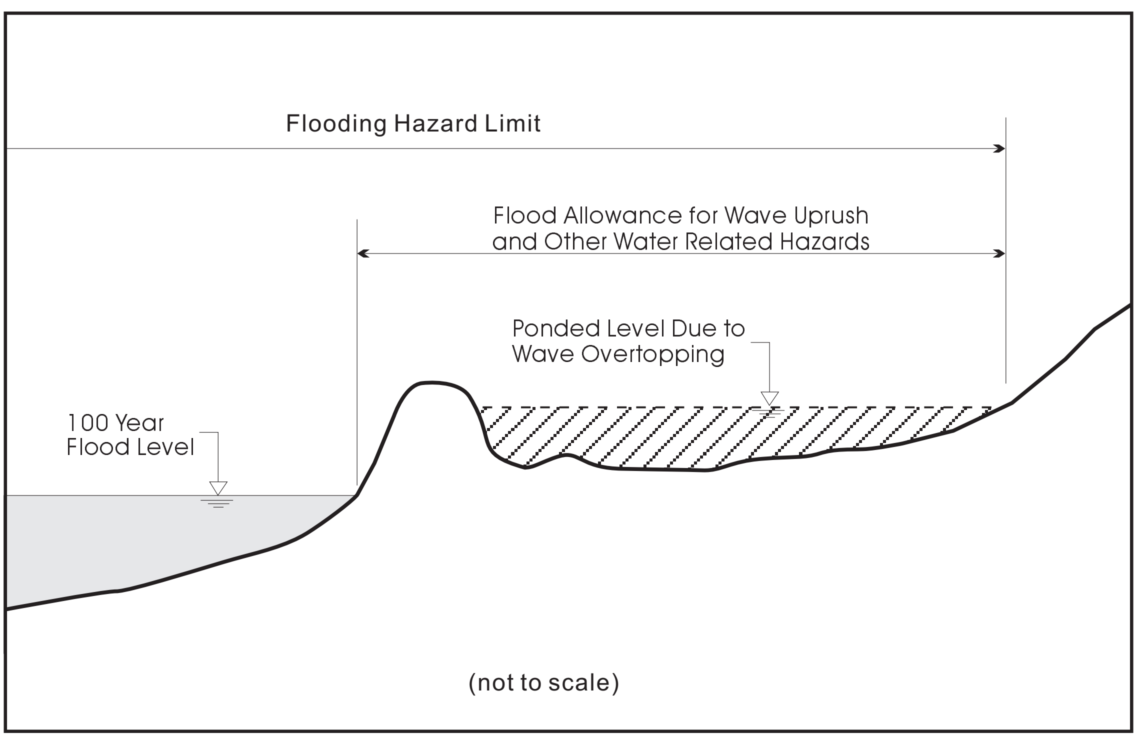


Figure 3.7: Flooding Hazard Limit with Overtopping



3.4 100 YEAR FLOOD LEVEL

3.4.1 Flood Level Definition

For the purpose of this Technical Guide, the 100 year flood level is defined as the peak instantaneous stillwater level (i.e., static water level plus wind setup) having a total probability of being equalled or exceeded during any year of 1% (i.e., probability, P=0.01). The term *return period* is used to denote the reciprocal of the annual probability (i.e., return period = 1/P). Therefore, a flood level with an annual probability of 1% is equivalent to a 100 year return period flood level. This means that on average, during a 100 year period, the 100 year flood level is expected to be equalled or exceeded once; during a 1,000 year period, the 100 year flood level would be equalled or exceeded 10 times on average. Further discussion is provided in Appendix A3.1.

The concept of the 100 year return period flood level must be clearly understood. The 100 year flood does not mean that the 100 year flood level will only occur once every 100 years, or that it will occur only 100 years from now. The 100 year flood level means that it will occur, on average, once every 100 years, and that during any one year, there is a 1% probability of occurrence.

In addition to understanding the probability, P, of a flood event happening in any year, it is necessary to understand the probability, R, called *risk*, that the same flood event will occur at least once in n successive years, where n is the number of years. This is of particular interest when considering the risk of flooding during the life of the project under consideration.

Risk is a function of the project life and the return period of the flood level. Specifically, the risk, R, that a flood level with a return period, T, will occur at least once in the project life of n successive years is defined by the equation:

$$R = 1 - \left(1 - \frac{1}{T} \right)^{n}$$

Table 3.1 shows return periods associated with various levels of risk for project lives of 50 and 100 years. From the table one can see that a flood level with a return period of 100 years would have a 39% risk of being equalled or exceeded at least once during a project life of 50 years, and a 64% risk for a project life of 100 years. Even a 10,000-year return period event has about a 1% risk of being equalled or exceeded over a project life of 100 years. In practice risk can not be eliminated, it can only be reduced to an acceptable level by undertaking the appropriate measures to address the hazard. Additional discussion of acceptable levels of risk is provided in Part 7 (Addressing the Hazards) of this Technical Guide.

Table 3.1 Return Periods in Years (*T*) for Various Levels of Risk (*R*) and Different Project Lives (*n*).

RISK, *R* (%)	RETURN PERIOD, *T* (years)	
	PROJECT LIFE, *n*=50 years	PROJECT LIFE, *n*=100 years
1	4977	9953
10	475	950
25	174	348
39	100	200
50	73	145
64	50	100

For each of the Great Lakes, there are many separate combinations of static water levels and storm surges, or wind setups which could result in the same local flood level. For illustrative purposes, Table 3.2 provides two combinations of different static water levels and different wind setups for Lake Ontario that result in the same flood level. There are many more different combinations than are illustrated in Table 3.2. The 100 year flood levels for the Great Lakes were determined, therefore, by calculating the probability of all possible combinations of the entire range of monthly mean lake levels and wind setups which could combine to result in a peak instantaneous stillwater level having a total probability of being equalled or exceeded 1% any year.

Table 3.2 Example of Different Combinations of Static Water Levels and Wind Setups for Lake Ontario

STATIC WATER LEVEL	WIND SETUP	FLOOD LEVEL
75.36 m (infrequent monthly mean level from April to June)	0.15 m (frequent event from April to June)	75.51 m
75.16 m (relatively frequent monthly mean level from April to June)	0.35 m (infrequent event from April to June)	75.51 m

For the connecting channels of the Great Lakes, the 100 year flood level, defined as the peak instantaneous stillwater level having a 1% probability of being equalled or exceeded in any year, is based on a frequency analysis of recorded data adjusted to Basis of Comparison conditions.

3.4.2 Calculated Flood Level Values

A 1978 study to calculate flood levels focused on Lake Ontario, Lake Erie and the southern portion of Lake Huron. Under the auspices of the Canada-Ontario Flood Damage Reduction Program, the original study was expanded and updated to assist in the delineation of the shore hazard zone for the purposes of shoreline land use planning.

With the return to record high water levels occurring on all but Lake Ontario between 1985 and 1987, the availability of ten years of additional recorded lake level data, the introduction of new methods of calculation and the need for flood level information for all shorelines throughout the *Great Lakes - St. Lawrence River System*, the Ministry of Natural Resources and Environment Canada, through the Hazard Land Technical Committee, undertook a new frequency analysis of Great Lake flood levels.

In November, 1988, the Hazard Land Technical Committee calculated and provided the 100 year return period peak instantaneous stillwater flood levels for 97 shoreline sectors on the Great Lakes and upper connecting channels (i.e., excluding the St. Lawrence and Niagara Rivers). In addition, the report included the complete exceedance frequency distributions produced for: peak instantaneous stillwater levels at 72 lakeshore sectors and gauge locations on channels; static water levels for the five lakes; and, wind setups at the 72 lakeshore sectors.

The 100 year flood levels outlined in the Hazard Land Technical Committee report were subsequently reviewed by technical/engineering staff of the then Conservation Authorities and Water Management Branch, Ministry of Natural Resources regional engineers and select Environment Canada staff. Based on the comments received, the flood hazard information was revised and released in February, 1989, by the Conservation Authorities and Water Management Branch of Ontario's Ministry of Natural Resources in a report entitled, *Great Lakes System Flood Levels and Water Related Hazards* (MNR 1989).

In July, 1993, Environment Canada's Water Planning and Management Branch released a report which was prepared in consultation with MNR and Conservation Authority staff. The report (Environment Canada 1993) contained the 100 year flood elevations along the St. Lawrence River from Kingston to the Ontario-Quebec border based on a frequency analysis of available water level data. In the report it was noted that "while the 100 year levels established for each reach of the River appear reasonable and consistent with previous studies, it should be noted that during the past 30 years, concentrated efforts have been taken to regulate levels of Lake Ontario and the St. Lawrence River. Past data and experience combined with modelling techniques have enabled the International Joint Commission's International St. Lawrence River Board of Control to regulate the levels of the Lake Ontario - St. Lawrence River System within a limited range of conditions. The impact of this control is reflected in the frequency analysis results upon which the 100 year levels are based. Many of the factors influencing lake supplies and levels such as precipitation and wind cannot be controlled. In addition, future changes in regulation procedures can also impact on levels experienced. One must not gain a false sense of security based on levels experienced over the past 30 years."

Appendix A3.1 details the methods for calculating the 100 year flood levels for the Great Lakes (i.e., using a combined probability of the monthly mean lake level and wind setup) and the upper connecting channels (i.e., frequency analysis of recorded peak instantaneous levels); and provides a description of the water level data set which was used including an outline of the basis of comparison (BOC) conditions. A brief discussion of the St. Lawrence River flood levels is presented in Appendix A3.1. For complete details regarding the calculation of the St. Lawrence River flood levels, the Environment Canada (1993) report should be consulted.

MNR (1989) concluded that Canadian and U.S. agencies currently analyze Great Lakes data assuming highest annual events are independent and that serial correlation does not significantly alter results. The effects of climate change were also considered in developing the 100 year flood levels. It was concluded that over the next 50 years water levels are unlikely to appreciably exceed modern records. Appendix A3.1 provides additional discussion on the potential influence of serial correlation and climate change.

The models used in undertaking the flood level calculations, SURGE and HYDSTAT, are briefly outlined in Appendices A3.3 and A3.4 respectively.

In summary, distributions of the combined probability of annual maximum monthly mean lake level and annual maximum surge levels were estimated for the lakeshores. For shorelines between gauge locations, a calibrated numerical model was used to estimate wind setups/surges. Flood level frequencies for the upper connecting channels were based on a frequency analysis of recorded peak instantaneous levels. The flood profiles for the connecting channels were established using sectors based on changes of 0.1 m using changes in slope based on known flow profiles. The results obtained for the Federal Emergency Management Agency (FEMA) by the U.S. Army Corps of Engineers (USACE 1988) were also taken into consideration. In developing the 100 year flood levels for the St. Lawrence, Environment Canada adopted the methodologies used for the MNR 1989 report. A numerical hydraulic model was used to estimate the flow profile where it is known that the St. Lawrence River does not fall uniformly. They also considered the results obtained in other previous studies and the operating procedures of the hydro-electric utilities.

The calculated **100 year flood levels,** in metres GSC, for the shoreline sectors of Great Lakes and the St. Mary's, St. Clair, Detroit Rivers and St. Lawrence Rivers are shown in Figures 3.8 to 3.16.

Additional information on flood levels throughout the *Great Lakes-St. Lawrence River System* is provided in Appendix A3.1 as follows:

- Figures A3.1.1 to A3.1.6 provide a series of *Great Lakes-St. Lawrence River System* maps indicating the 100 year flood levels (GSC) by sector;
- Table A3.1.1 lists 100 year flood levels (GSC) by lake/connecting channel and sector;
- Table A3.1.2 lists flood levels (IGLD 1955) for all sectors for recurrence intervals from 2 to 200 years;
- Table A3.1.3 lists the highest annual monthly mean lake levels (IGLD 1955) for recurrence intervals from 2 to 200 years; and
- Table A3.1.4 lists the wind setup values for lakes for recurrence intervals from 2 to 200 years.

Figure 3.8: 100 Year Flood Levels: Lake Superior

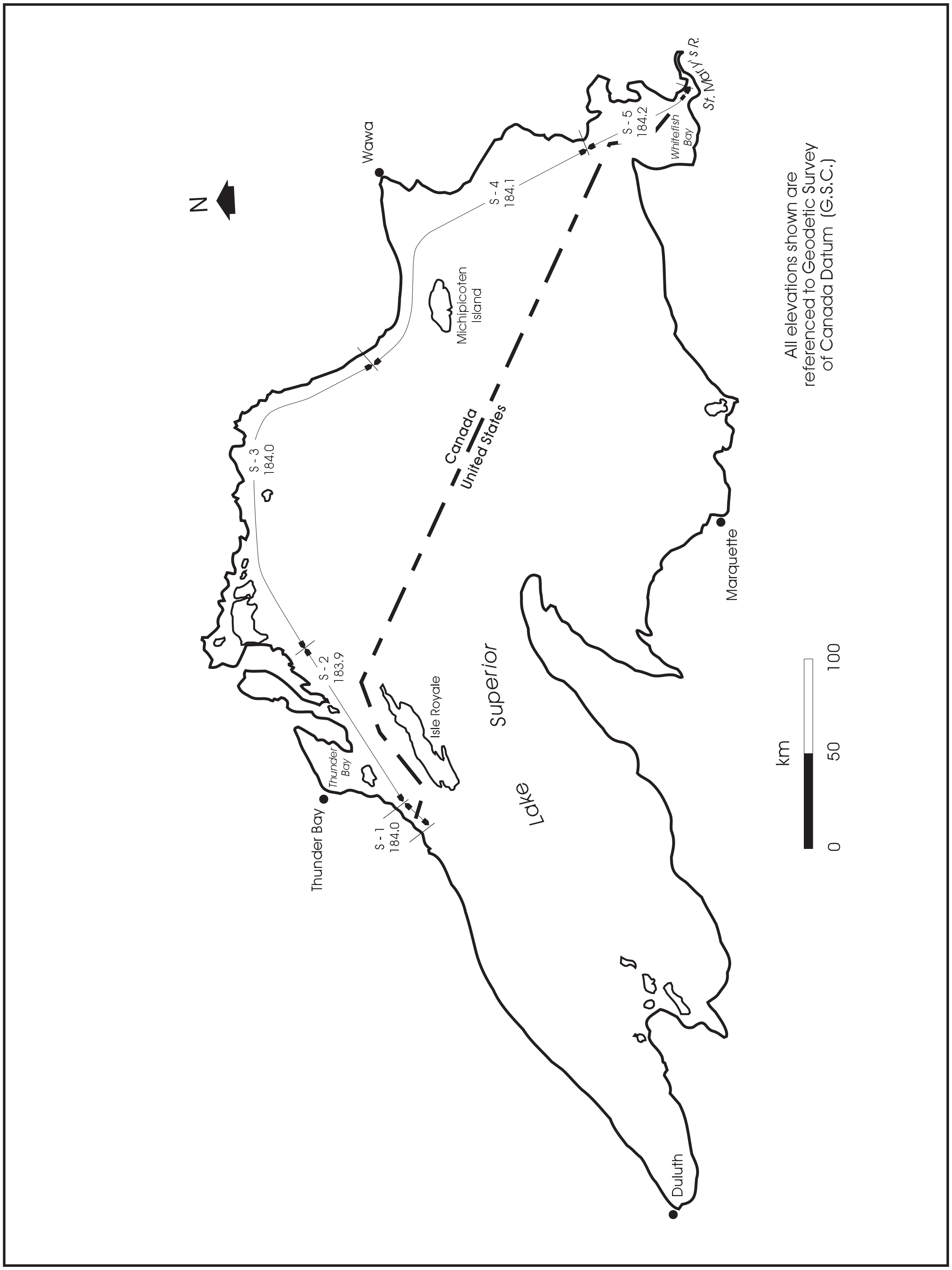


Figure 3.9: 100 Year Flood Levels: Lake Huron and Georgian Bay

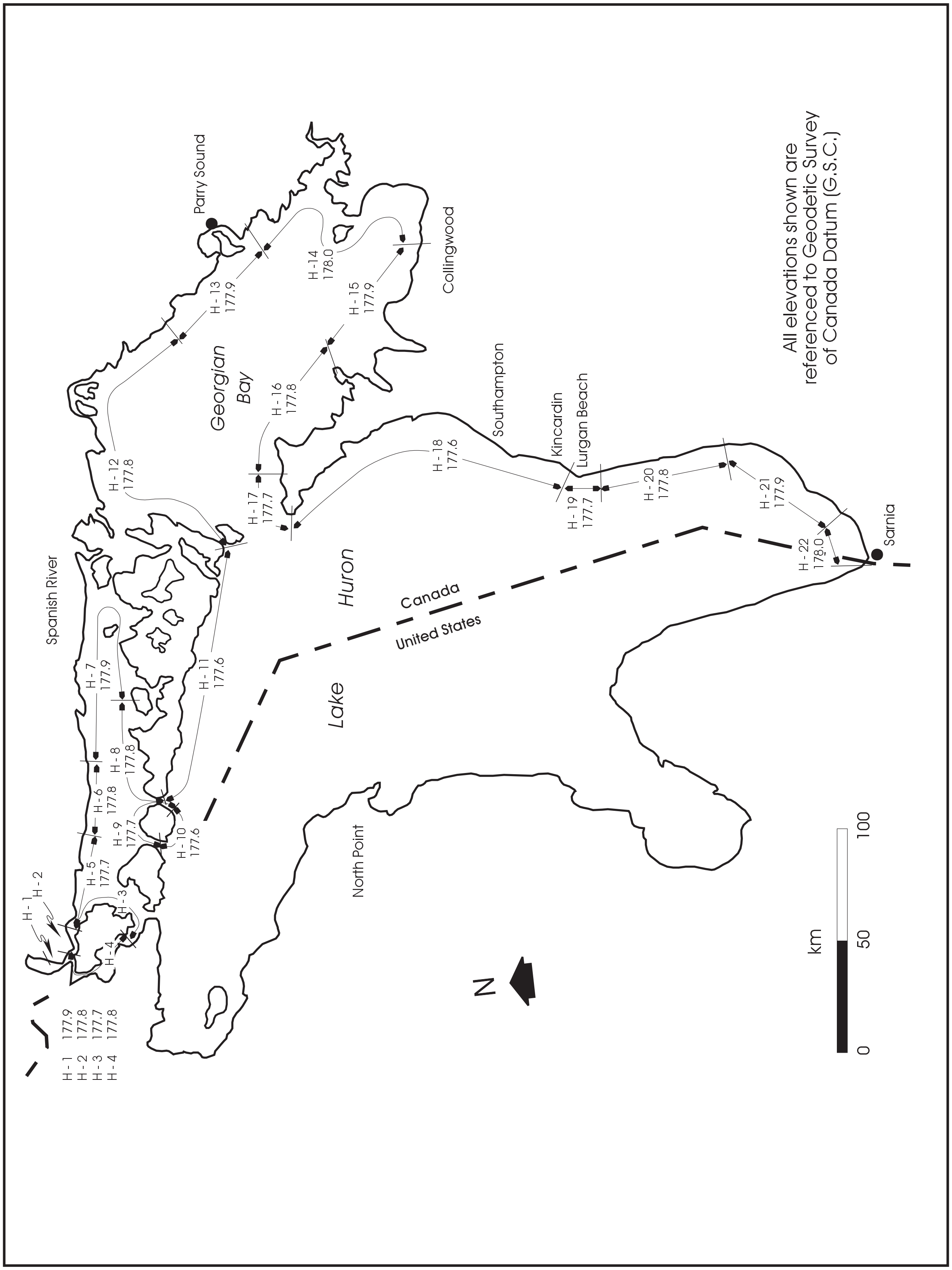


Figure 3.10: 100 Year Flood Levels: Lake Erie and Lake St. Clair

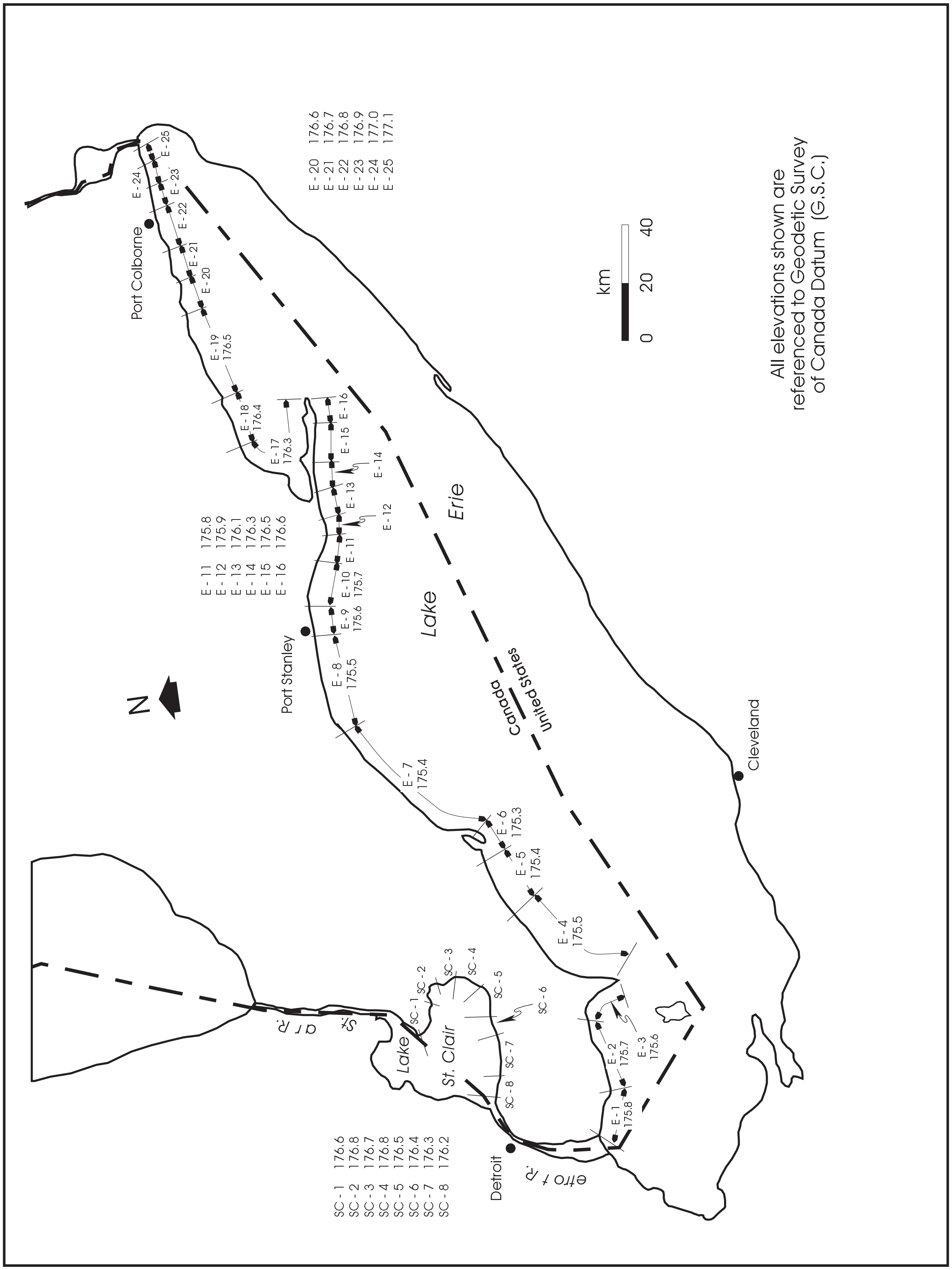


Figure 3.11: 100 Year Flood Levels: Lake Ontario

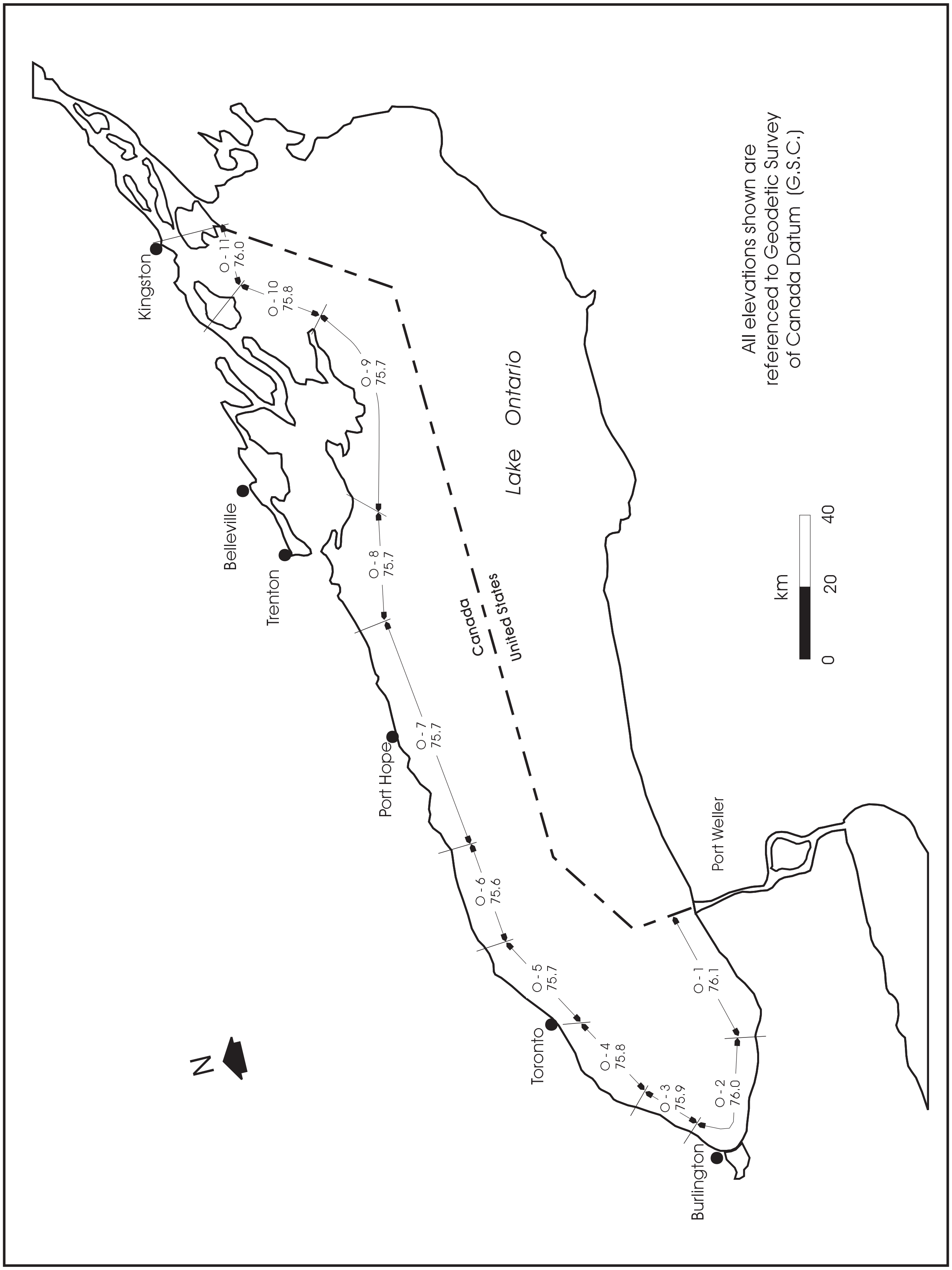
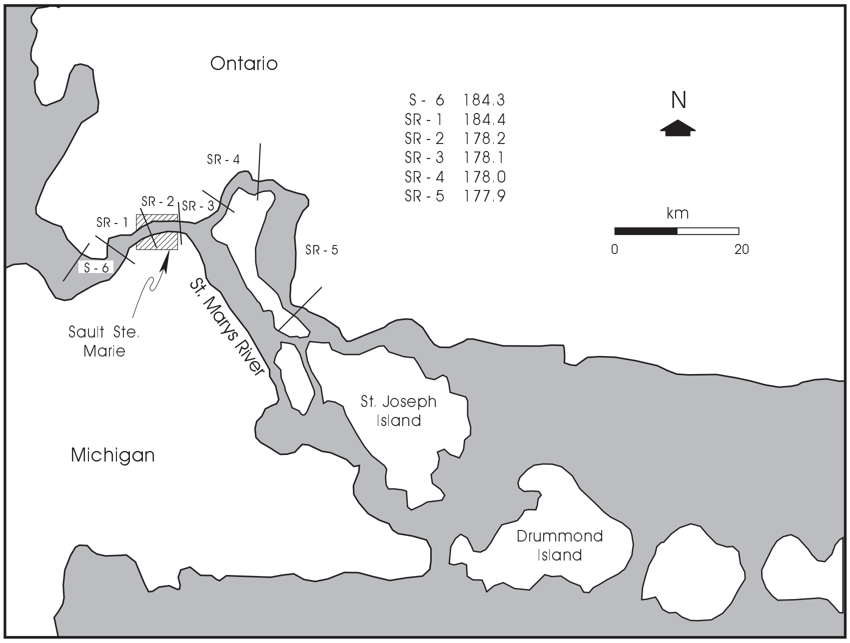
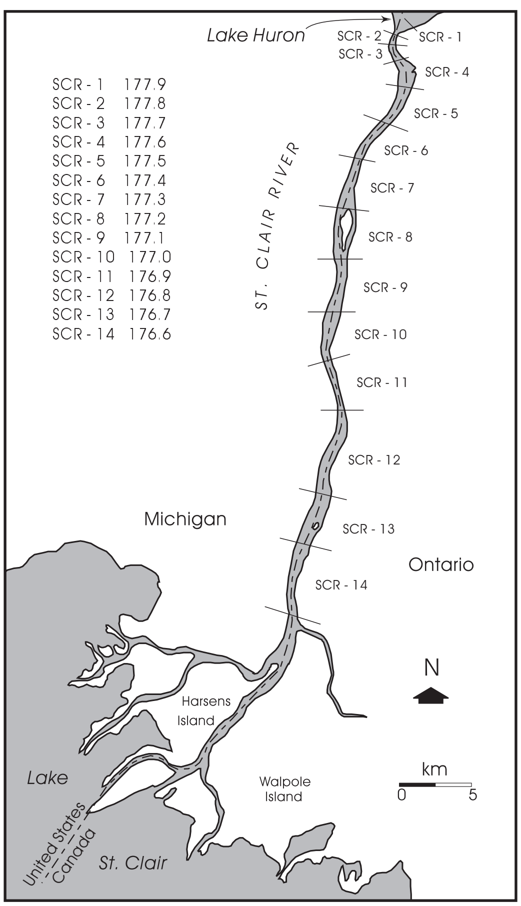
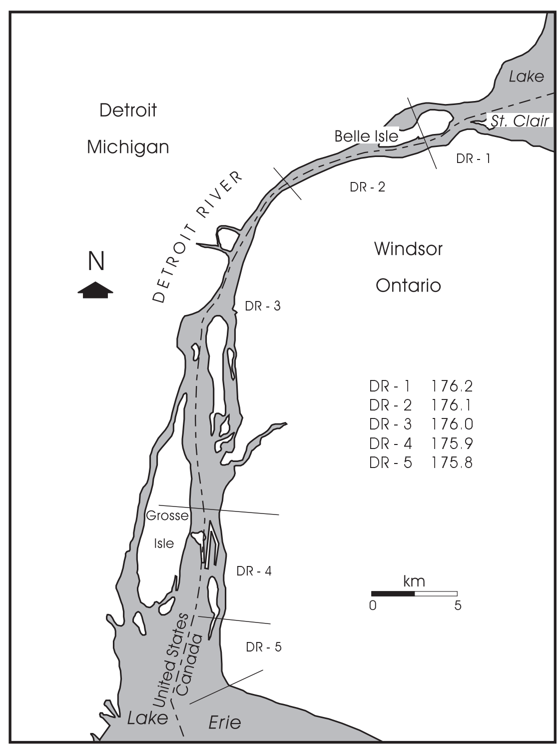


Figure 3.12: 100 Year Flood Levels: St. Marys River, St. Clair River and Detroit River







All elevations shown are referenced to Geodetic Survey of Canada Datum (G.S.C.).

Figure 3.13: 100 Year Flood Levels: St. Lawrence River (SLR-1 and SLR-2)

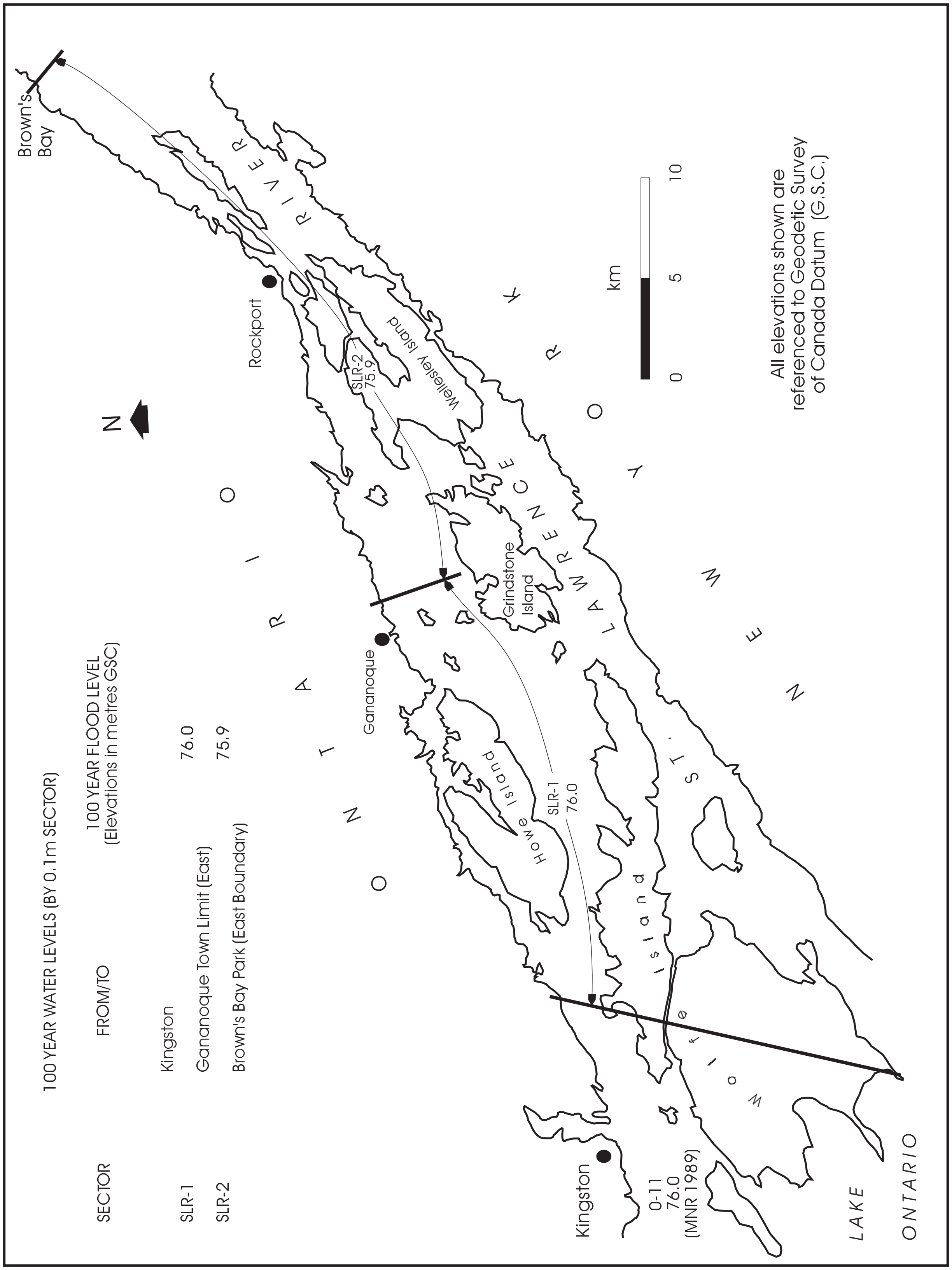


Figure 3.14: 100 Year Flood Levels: St. Lawrence River (SLR-3 to SLR-7)

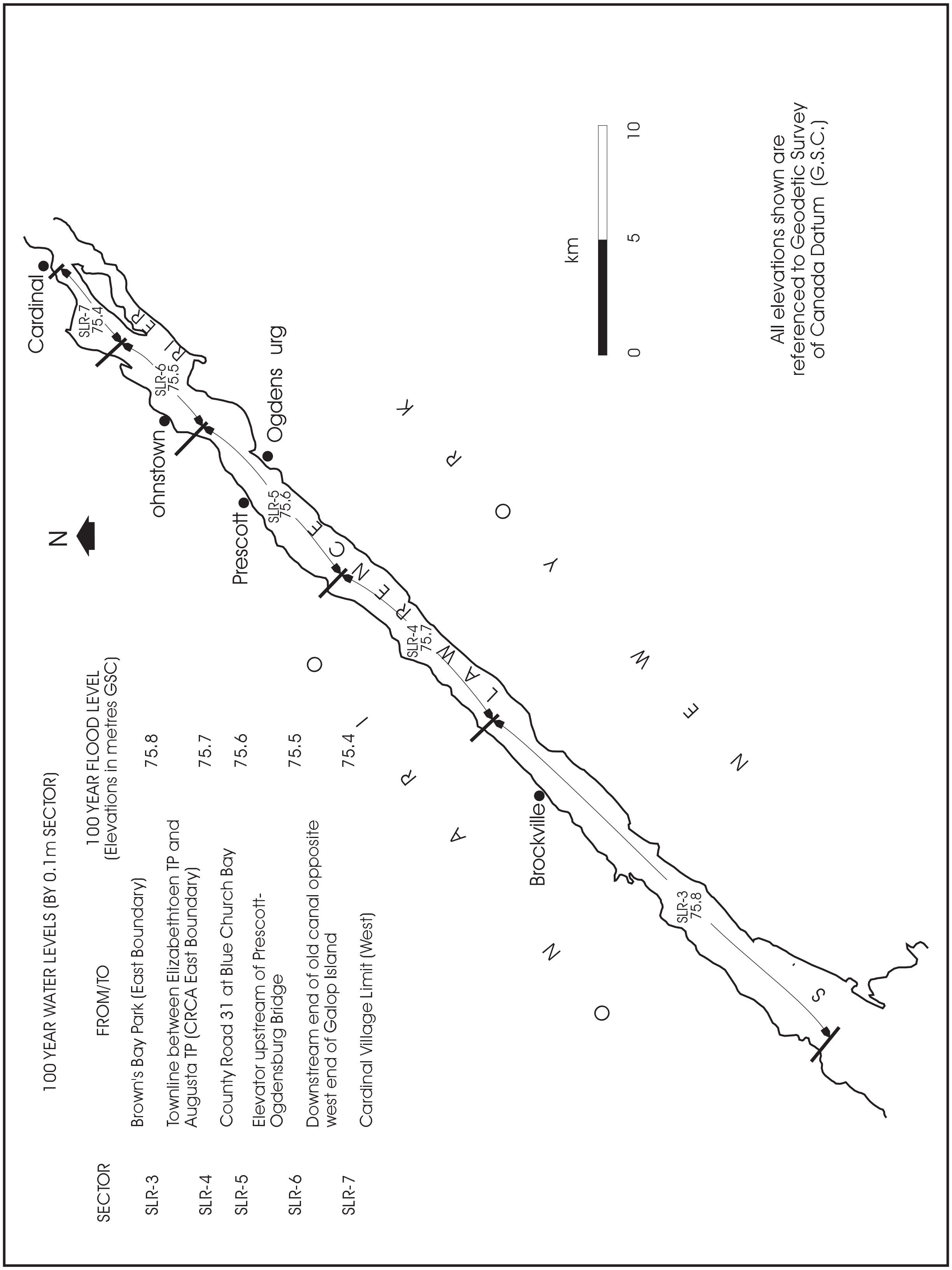


Figure 3.15: 100 Year Flood Levels: St. Lawrence River (SLR-8 to SLR-11)

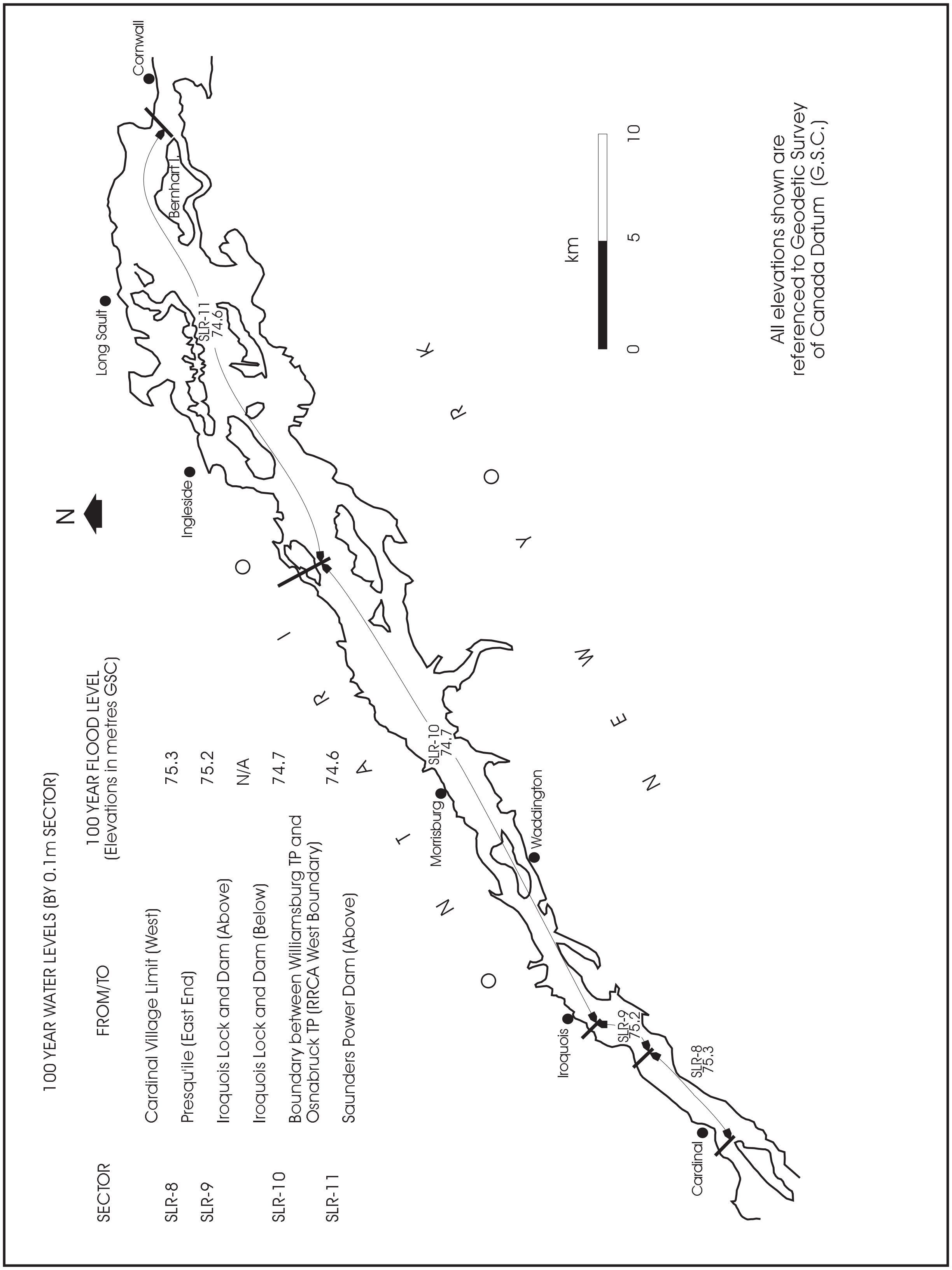
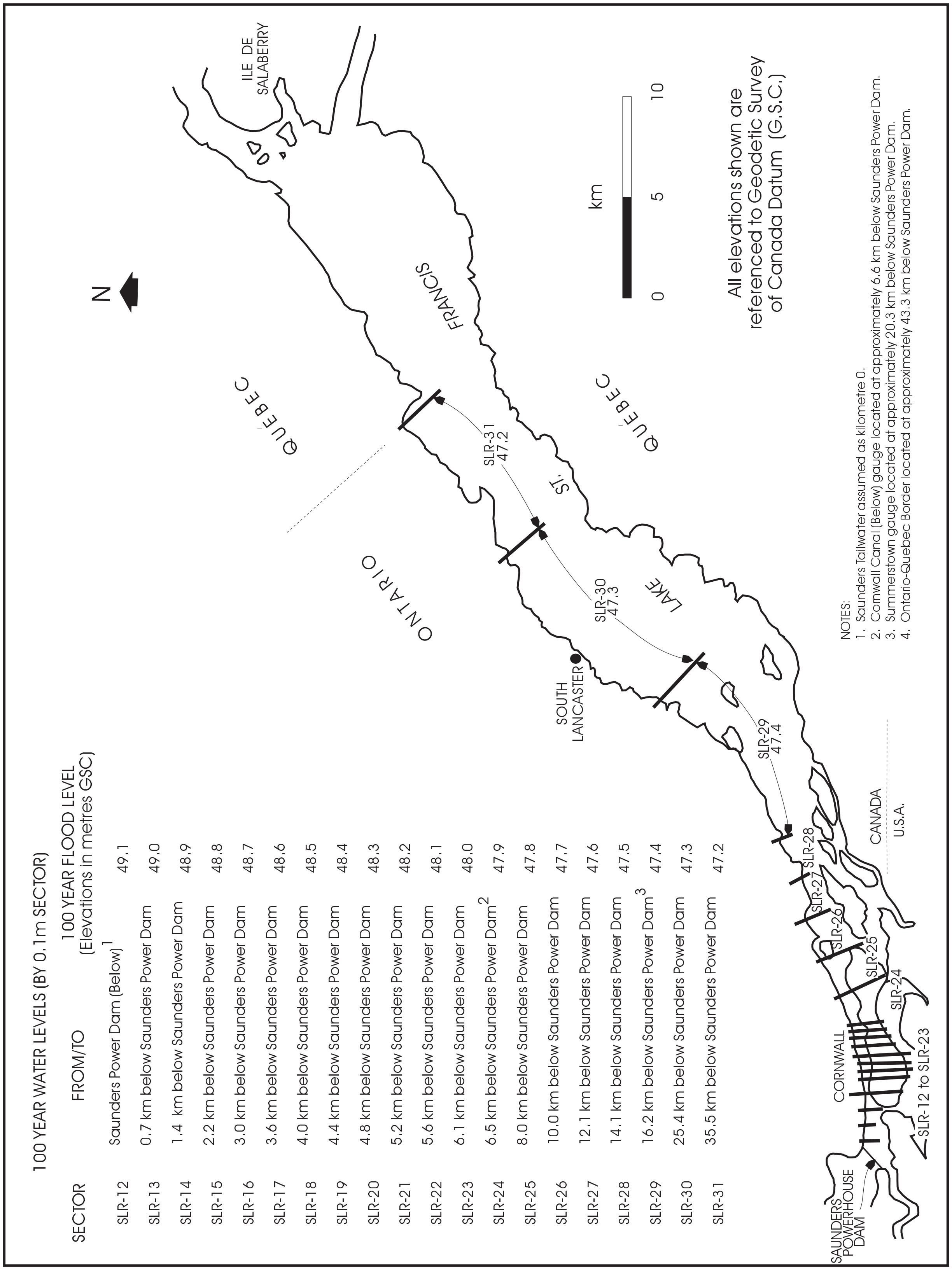


Figure 3.16: 100 Year Flood Levels: St. Lawrence River (SLR-12 to SLR-31)



100 YEAR WATER LEVELS (BY 0.1m SECTOR)

SECTOR	FROM/TO	100 YEAR FLOOD LEVEL (Elevations in metres GSC)
SLR-12	Saunders Power Dam (Below)[1]	49.1
SLR-13	0.7 km below Saunders Power Dam	49.0
SLR-14	1.4 km below Saunders Power Dam	48.9
SLR-15	2.2 km below Saunders Power Dam	48.8
SLR-16	3.0 km below Saunders Power Dam	48.7
SLR-17	3.6 km below Saunders Power Dam	48.6
SLR-18	4.0 km below Saunders Power Dam	48.5
SLR-19	4.4 km below Saunders Power Dam	48.4
SLR-20	4.8 km below Saunders Power Dam	48.3
SLR-21	5.2 km below Saunders Power Dam	48.2
SLR-22	5.6 km below Saunders Power Dam	48.1
SLR-23	6.1 km below Saunders Power Dam	48.0
SLR-24	6.5 km below Saunders Power Dam[2]	47.9
SLR-25	8.0 km below Saunders Power Dam	47.8
SLR-26	10.0 km below Saunders Power Dam	47.7
SLR-27	12.1 km below Saunders Power Dam	47.6
SLR-28	14.1 km below Saunders Power Dam	47.5
SLR-29	16.2 km below Saunders Power Dam[3]	47.4
SLR-30	25.4 km below Saunders Power Dam	47.3
SLR-31	35.5 km below Saunders Power Dam	47.2

NOTES:
1. Saunders Tailwater assumed as kilometre 0.
2. Cornwall Canal (Below) gauge located at approximately 6.6 km below Saunders Power Dam.
3. Summerstown gauge located at approximately 20.3 km below Saunders Power Dam.
4. Ontario-Quebec Border located at approximately 43.3 km below Saunders Power Dam.

For the purposes of clarification, known elevations are also generally described in accordance with different datums. For shoreline areas, elevations are typically reported in reference to one of three types of datum as follows:

- Elevations on land are usually referenced to Geodetic Datum as determined by the Geodetic Survey of Canada (GSC). For example, elevations on the 1:2,000 scale FDRP shoreline mapping are referenced to the Geodetic Datum.

- Lake levels, on the other hand, are referenced to the datum for each lake which is called the chart datum. The chart datums provide elevations above the International Great Lakes Datum (IGLD).

- Within certain municipalities or harbour areas, older plans and maps may be referenced to the municipal or local datum.

The differences between the land-based Geodetic Datum and the water-based International Great Lakes Datum (IGLD) are not constant, but vary slightly with latitude and elevation. In addition, the datums are adjusted occasionally to account for glacial uplift. For example, starting in 1992, the Great Lakes water levels are no longer referenced to IGLD (1955) and as such, are now referenced to IGLD (1985). Local datums are site specific. It is important to know which datum the elevations are referenced to and how to convert from one datum to another. Information on procedures for converting between GSC, IGLD (1955) and IGLD (1985) datums for various locations around the Great Lakes is provided in Appendix A3.1 of this Technical Guide.

Other water level information services are outlined in Appendix A3.2 of this Technical Guide.

The combined probability approach can be appropriately used to calculate 100 year peak instantaneous stillwater levels for the Great Lakes. As a result of the longer period of record used for the February 1989 report (MNR 1989), the changes in Basis of Comparison conditions for the lakes, different calculation techniques and the improvements made in estimating surge levels in ungauged sections, the 100 year return period flood levels calculated for the 1989 study are to be used in place of the 100 year levels calculated in 1978 for delineating *flooding hazard* areas along the Great Lakes shoreline.

The 1989 updating of the 100 year return period flood levels resulted in different flood level standards being established from those calculated in the 1978 study. For example, for the 100 year return period annual maximum monthly mean levels, the increased period of record and updated Basis of Comparison conditions resulted in no change for Lake Superior, an increase of 0.07 m for Lake Huron, an increase of 0.15 m for Lake St. Clair, and increase of 0.07 m for Lake Erie and a lowering of 0.04 m for Lake Ontario.

For purposes of clarification and discussion, Table 3.3 compares the 100 year return periods for the Great Lakes and the upper connecting channels resulting from the combined probability analyses of annual maximum monthly mean levels and surges at gauge sites ("1989 Study") with: a) the highest recorded lake levels from 1900 to 1987; and b) the 1978 study results.

3.4.3 Conveyance of Critical Flow in Connecting Channels

As a general rule, development within the 100 year flood level along connecting channels reduces the cross-sectional area of the waterway, so the corresponding flood level increases at the site and immediately upstream. General encroachment within the 100 year flood level also reduces the storage capacity of the channel and results in an increase in flood flows and the flood levels along the downstream reaches of the connecting channel. There may be specific locations along the connecting channels, at the outer limits of the 100 year flood level, that could potentially be safely developed with no adverse impacts. Due to the specific hydraulic conditions, including very shallow flood depth and minimal flood flow velocity, these outer portions of the *flooding hazard* limit within the 100 year flood level are generally not critical to the conveyance of flow associated with the 100 year flood level. The inner portions of the *flooding hazard* limit within the 100 year flood level along connecting channels which are critical to the conveyance of the flow associated with the 100 year flood level are those portions where development will not be permitted as it would result in a significant and unacceptable increase in the 100 year flood level.

Table 3.3 Comparison of 100 Year Flood Levels: Great Lakes and Upper Connecting Channels

Gauge Location	Highest Recorded Instantaneous Stillwater Level (1900-1987) (m G.S.C.)	100 year Flood Levels	
		1978 Study (m G.S.C.)	1989 Study (m G.S.C)
Thunder Bay	184.02 (1952)	-	183.84
Rossport	183.84 (1968)	-	184.03
Michipicoten	184.22 (1939)	-	184.18
Gros Cap	184.09 (1972)	-	184.18
Sault Ste. Marie	184.24 (1985)	-	184.40
Thessalon	177.61 (1986)	-	177.72
Little Current	177.89 (1985)	-	177.92
Parry Sound	177.79 (1986)	-	177.96
Collingwood	177.78 (1986)	-	178.01
Tobermory	177.53 (1986)	-	177.65
Goderich	177.79 (1986)	178.2	177.81
Point Edward	177.64 (1987)	178.0	177.65
Tecumseh	176.11 (1986)	176.4	176.20
Belle River	176.23 (1972)	176.3	176.35
Bar Point	175.90 (1974)	176.3	175.80
Kingsville	175.61 (1987)	175.9	175.68
Erieau	175.37 (1973)	175.8	175.32
Port Stanley	175.33 (1985)	175.7	175.52
Port Dover	175.79* (1985)	176.1	176.23
Port Colborne	176.59 (1985)	176.2	176.77
Port Weller	75.89 (1973)	76.0	76.11
Burlington	76.45 (1973)	76.2	76.01
Toronto	75.77 (1952)	76.2	75.69
Cobourg	75.78 (1952)	76.1	75.74
Kingston	75.90 (1952)	76.3	75.95

* Water level was above 175.79 m G.S.C. on December 2, 1985 but was not recorded due to a malfunction of the gauge.

Development and site alteration, as defined by the policy, will not be permitted within:

> *defined portions of the one hundred year flood level along the connecting channels* (the St. Mary's, St. Clair, Detroit, Niagara and St. Lawrence Rivers) (Policy 3.1.2(b)).

Defined portions of the one hundred year flood level along connecting channels means those areas which are critical to the conveyance of the flows associated with the *one hundred year flood level* along the St. Mary's, St. Clair, Detroit, Niagara and St. Lawrence Rivers, where *development and site alteration* will create *flooding hazards*, cause updrift and/or downdrift impacts and/or cause adverse environmental impacts. The *defined portions of the one hundred year flood level along connecting channels* are to be identified based on studies using accepted engineering principles.

To determine the critical or defined portions of the *flooding hazard* along connecting channels, various factors should be considered including but not limited to:

- physical characteristics of connecting channel;
- each individual component of *flooding hazard* (i.e., water level, wave uprush, other water related hazards);
- duration and frequency of flooding;
- pre-development and post-development flood conditions and impacts;
- date of flood information;
- reliability of the flood information;
- availability, accuracy, applicability of existing engineering studies; and
- long-term maintenance costs where flood mitigation measures are proposed.

The *Technical Guide for River and Stream Systems* (MNR 1996) outlines accepted engineering principles for determining flows and levels. It may be necessary to recalculate the *flooding hazard* (e.g., 100 year flood level plus an allowance for wave uprush and other water related hazards) for floodproofing purposes and to identify and assess the upstream and downstream impacts.

Any increase in the 100 year flood level must be estimated. If undertaken during the initial *flooding hazard* mapping process, the revised flood levels can be computed without major additional expense. Where flood hazard 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. Regulatory agencies in the United States, on the opposite side of the connecting channels, are also opposed to increases in the 100 year flood level and they must be consulted should any potential for increase be contemplated.

Caution should be exercised in designating chronic flood problem areas as non-critical. 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.

Flooding hazard areas along connecting channels can be low-lying and it is often difficult and expensive to provide necessary services (e.g., watermains, sewers, drainage works, etc.) to serve the developments. Drainage systems should provide protection against the 100 year flood level and it may be difficult to provide outlets above the 100 year flood level. In these situations, it may be necessary to provide pumping facilities which could result in some additional expense in new developments.

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

Land use is a key factor considered in *flooding hazard* studies and the calculation of 100 year flood lines. Proposed development, not anticipated in these calculations, could create increased flood risks and thus reduce the effectiveness of shoreline flood hazard management programs.

3.5 WAVE UPRUSH AND OVERTOPPING

Along shorelines susceptible to wave action, *flooding hazard* areas extend landward beyond the 100 year flood level to the limit of wave action. All shorelines in the *Great Lakes - St. Lawrence River System* should be considered to be susceptible to wave action unless site specific study demonstrates that wave action is not significant. Wave uprush and other water related hazards, including ship generated waves, ice piling and ice jams, are important factors that are critical to the proper definition and delineation of the shoreline *flooding hazard*.

Typically, wave action includes:

- wave uprush — also known as wave runup, is the time varying height above the stillwater level that the water runs up the shore face
- wave setup — is the mean increase in water level caused by the onshore transport of water due to waves breaking at the shoreline
- wave overtopping and/or wave spray — essentially occurs when the height of the natural shoreline, or of the protection work, above the stillwater level is less than the limit of uprush (see Figure 3.3b). As a result, waves overtopping the protection work can cause flooding of the onshore and can threaten the structural stability of protection works.

In general, methods used to estimate wave uprush, or wave runup, are measured from the stillwater level and as such, inherently include wave setup. The stillwater level is the level the water would assume in absence of wave action. Wind setup, also known as storm surge, is included in the stillwater level. Therefore, for the purposes of this Technical Guide, the term "wave uprush", will include wave setup. For straight, uniform reaches without protection works, the landward limit of wave action can be represented by the maximum limit of wave uprush. In areas where waves act on protection works, and in areas with irregular shorelines, the wave action may include overtopping and spray which are more difficult to determine and may require detailed site specific study.

3.5.1 Characteristics of Wave Uprush and Overtopping

Wave uprush, or wave runup, is typically defined as the vertical height above the stillwater level to which water, from an incident wave, will rush up to on a shoreline or shoreline protection work (see Figure 3.3a). As previously noted, the available engineering guidance for predicting wave uprush levels includes wave uprush and wave setup as an inseparable combination of factors. Since most uprush prediction methods are empirical and the wave uprush/runup data collected during the laboratory tests was measured from the stillwater level, the resultant wave uprush calculations automatically included wave setup.

The relationship between the vertical wave uprush/runup value, R, and the horizontal offset for wave runup is shown in Figure 3.17. The same geometrical consideration could be used to obtain the horizontal component of wave uprush/runup for other simple slope configurations.

If waves break on the beach, or protection work, due to instability caused by decreasing depths, the incident wave energy will be distributed in wave uprush/runup, wave reflection, wave breaking, slope roughness losses, and losses due to permeability. As such, the primary controlling parameters are summarized as follows (see Figure 3.18):

- stillwater level (*SWL*);
- the incident wave climate (wave height, H, and wave period, T);
- the beach or protection work slope ($\tan \theta$);
- the lake bottom slope (m);
- the water depth at toe of the protection work's slope or beach slope (d_s); and
- surface roughness and protection work permeability (Δ_r, and P).

Figure 3.17: Uprush Characteristics for Wave Breaking on a Slope

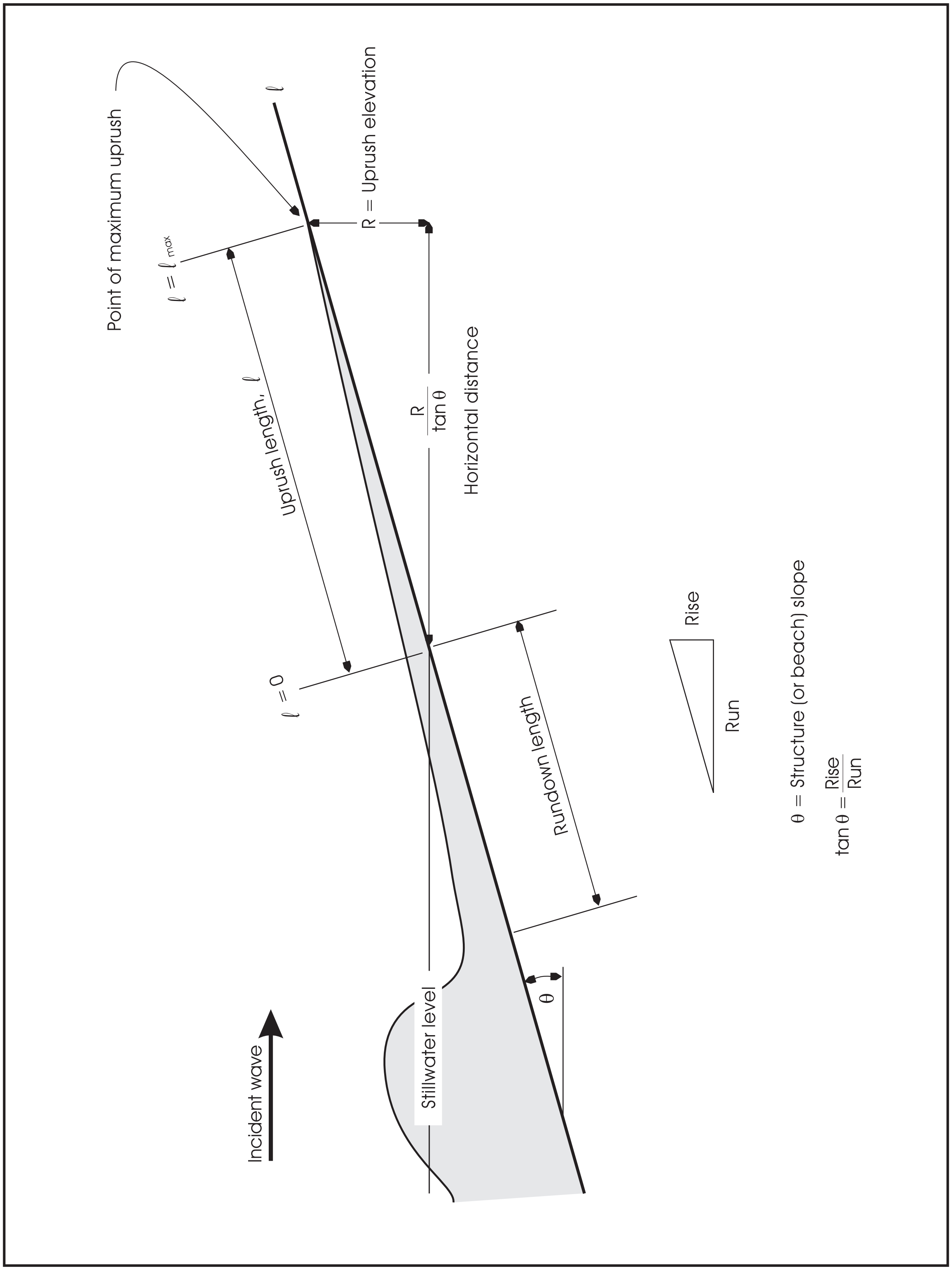
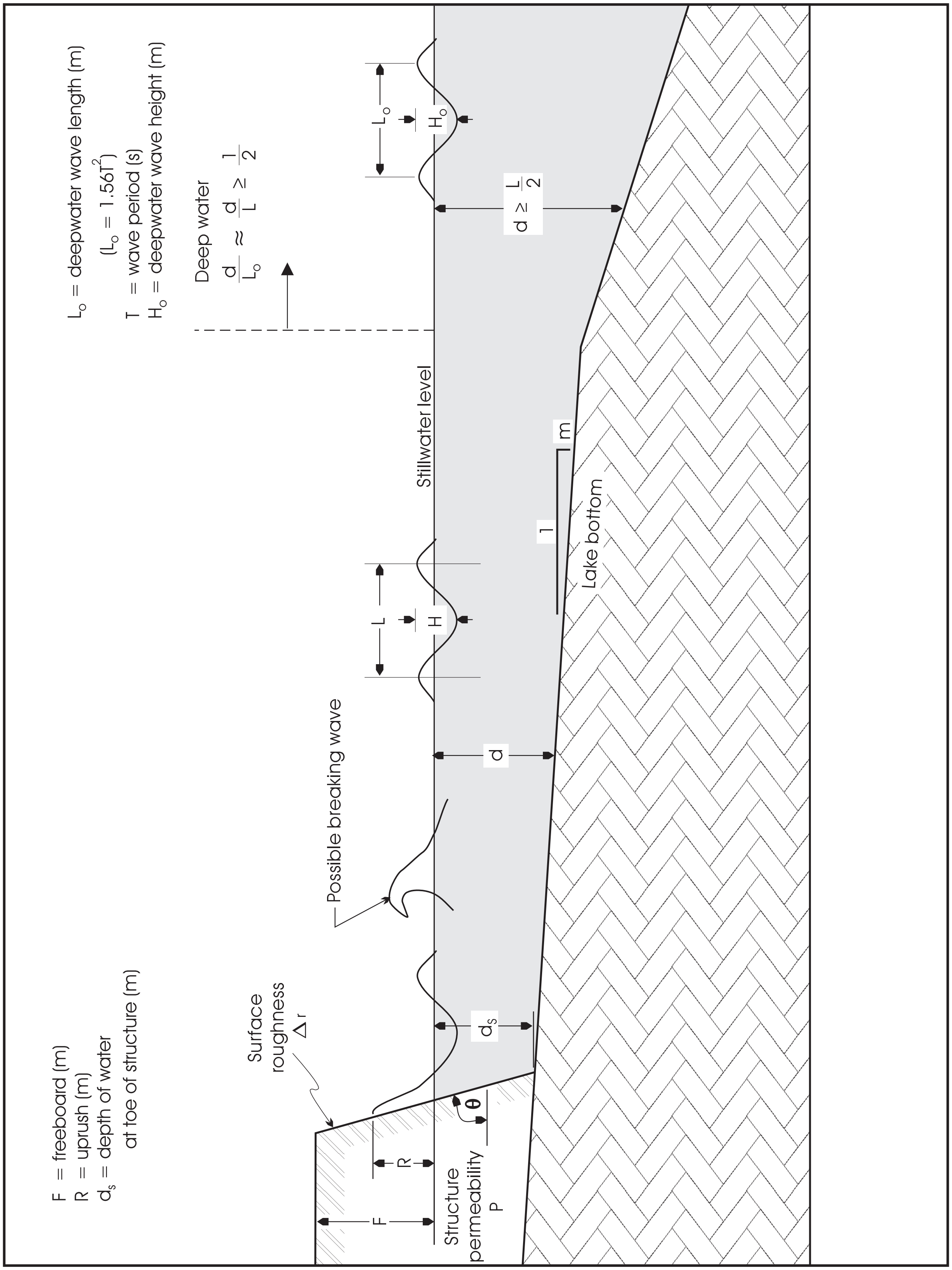


Figure 3.18: Definition Sketch for Some Variables Applicable to Wave Uprush



Under certain conditions, such as for protection works with steep slopes, long period and small amplitude waves, waves may reach the protection works, or beach, without breaking. In this case, the waves can be assumed to be reflected totally like a standing wave, and the wave uprush/runup is then directly related to the wave amplitude.

Other factors such as the local bathymetry (e.g., offshore bars and composite slopes), berms in front of protection works and oblique wave attack may also change the magnitude of the wave uprush/runup. Ice cover of the shore can also influence the wave uprush by making a rough permeable slope into smooth impermeable slope and/or limiting the depth of water, therefore, limiting the wave action by the presence of an ice foot.

Wave overtopping occurs when the limit of wave uprush or incident wave action passes over, or exceeds, the top of a shoreline bank or the top elevation of a shoreline protection work (see Figure 3.2). The basic parameters controlling wave overtopping are essentially those affecting wave uprush (see Figure 3.18).

The commonly accepted measure of overtopping is the mean discharge, *Q*. The mean discharge is the total volume of overtopping water over a given time period (e.g., one storm of several hours), divided by the length of that time period. The mean discharge, *Q*, is usually specified in terms of the mean discharge per unit length of shoreline or protection works (i.e., $m^3/s\bullet m$; note that 1000 litres(l) = 1 m^3).

The amount, magnitude, duration and landward extent of overtopping water and wave spray are of direct concern in assessing shoreline flood susceptibility and degree of risk. The degree of risk is measured in terms of:

- risk to the safety of persons and property behind the shoreline protection works (e.g., usage considerations);
- the threat to the stability of the shoreline protection work itself; and
- the magnitude and impact of flooding and ponding behind the protection work (e.g., drainage considerations).

Protection works that permit a safe amount of wave overtopping are not uncommon and their proper design, installation and use is considered to be acceptable practice. Initial costs of protection works that preclude overtopping may be prohibitive and, depending on the proposed land use to the lee of the protection work, a non-overtopping work may not be necessary. The controlling criteria in any decision-making process is whether the intensity and/or the amount of wave overtopping endangers people or property, threatens the structural stability of the protection works and/or permits excessive water to pond in the onshore area.

Protection works which are subject to overtopping must be carefully designed to withstand the forces of the overtopping water. Special attention must be given to the details of the crest and backside of the protection works to ensure that its stability is not jeopardized. Proper provisions for the drainage of the overtopping water must be specifically incorporated into the design of the shoreline protection work to prevent upland flooding and ponding. Additional supporting information, providing more details on wave uprush and overtopping, is outlined in a separate report, *Wave Uprush and Overtopping: Methodologies and Applications* (Atria 1997). It provides a review and summary of acceptable practice for determining wave uprush and overtopping, as well as typical examples of application.

3.5.2 Flood Allowances for Wave Uprush and Overtopping

By definition, the *flooding hazard* along shorelines of the *Great Lakes - St. Lawrence River System* involves the combination of the 100 year flood level and a flood allowance for wave uprush and other water related hazards. In the absence of studies to determine the allowance for wave uprush and other water related hazards two standards, measured horizontally landward from the 100 year flood level, are identified, namely:

- **15 metres for the lakes Superior, Huron, St. Clair, Erie and Ontario (Figure 3.6); or**

- **5 metres on connecting channels (Figure 3.7).**

Shoreline managers responsible for determining and applying these standards should note that the 15 m and 5 m horizontal allowances are not to be interpreted as, or used as, the required setback for development behind protection works. The required setback may be further inland due to *erosion hazards* or *dynamic beach hazards* or it may be closer to the water if appropriate floodproofing or erosion protection works are in place. Floodproofing requirements and flood and erosion allowances, as part of the Protection Works Standard, are presented in Part 7 (Addressing the Hazards) of this Technical Guide.

It should be recognized that the standard 15 m and 5 m flood allowances, in the absence of detailed, site-specific studies, are intended to provide a means of defining and delineating the landward extent of the wave uprush and other water related hazards component of the *flooding hazard* on natural unprotected shorelines. If field indicators provide reasonable grounds to believe that these standard allowances are insufficient or too great to provide safe adequate protection, the policy provides the flexibility to determine the appropriate allowances by undertaking studies using accepted engineering principles. Additional information regarding the use of these studies is presented in Section 3.5.3.

The standard 15 m and 5 m flood allowances are based on calculations of wave uprush for a number of sites along the shorelines of Lakes Huron, Erie and Ontario. The wave uprush level is the combined level resulting from wave setup and wave uprush. The procedure for calculating the standard 15 m and 5 m flood allowances and the results of the calculations at the various sites are presented in Appendix A3.5. It has been determined through this use of this procedure that the resultant 15 m and 5 m flood allowances, in the absence of detailed engineering studies, are appropriate and satisfy wide areas of applicability, are flexible enough to adapt to site-specific conditions, and provide a reasonable estimate of areas subject to wave action.

On the Lake Superior shoreline, almost 70% of the residential properties in 1987 had either highly or medium erodible soils. Only 2% of existing properties had a high wave susceptibility. It would appear that most existing development is located along shorelines that are similar to the other Great Lakes (i.e., most development is not located on exposed bedrock shoreline with steep nearshore depths that might allow fetch-limited wave conditions to govern wave uprush). As such, the 15 metre wave uprush offset was deemed to be suitable for application across the Great Lakes. In cases where developments are planned for irregular or bedrock shoreline, the policy provides the flexibility to undertake wave uprush calculations using **accepted engineering principles** and site-specific information.

A wave uprush offset allowance of 5 metres is generally suitable for application along the connecting channels and at other sites located away from the influence of lake waves. For the connecting channels, wave uprush levels are normally the result of ship-generated waves (see Section 3.6.2 for additional information on ship generated waves, including a discussion regarding the wave uprush allowance along the St. Lawrence River).

Where wave overtopping causes ponding landward of a natural shoreline feature (e.g., bank) or a protection works, the *flooding hazard* is to be defined by a study using accepted engineering principles. A summary of methodologies which fulfil the criteria of "accepted engineering principles" for estimating wave overtopping rates is presented in Section 3.5.4.

3.5.3 Using Studies to Determine Wave Uprush and Overtopping

Where the standard 15 m or 5 m flood allowances are considered to be insufficient or greater than necessary to safely address the wave uprush component of the *flooding hazard*s, the policy provides the flexibility to define the limit of wave uprush and other water related hazards through the use of a study using accepted engineering principles.

The flood allowance of 15 m, for Great Lakes, or 5 m, for connecting channels, may be insufficient if one or more of the following indicators are present landward of the 15 m or 5 m limit:

- materials washed up by waves (e.g., cobbles/shingles/gravel, driftwood, debris);
- the lakeward extent of mature, established vegetation;
- erosion of the backshore (e.g., beach dune, berm or bluff) or a high vertical bluff or cliff; or
- historical evidence (e.g., photographs, surveys).

Field, or on-site, indicators that the flood allowances may be insufficient or too great with respect to wave uprush generally would be based on sound field evidence and local knowledge of the limit of wave uprush experienced during high water periods (i.e., 1972-1973, 1985-1986).

The presence of materials washed up by waves or evidence of erosion of the backshore, lakeward of the 15 m or 5 m flood allowance, is not proof that the flood allowance is too great unless this is supported by information regarding the recent flood levels. Field evidence must be backed up by further analysis of the limit of wave uprush using accepted engineering principles.

To assist in the determining "accepted engineering principles", a review of wave uprush and overtopping and the various current methods for their calculation or prediction is provided in a separate report, *Wave Uprush and Overtopping: Methodologies and Applications* (Atria 1997). A summary of the review is presented in the following subsections, including acceptable methods for estimating wave uprush and overtopping, acceptable rates, data requirements, and alongshore considerations.

a) Summary of Accepted Methods for Estimating Wave Uprush

At present, the understanding of wave uprush/runup processes is limited, and there seems to be no generic methodology for the prediction of the limit of wave uprush/runup. Existing guidance is mainly based on empirical research work, carried out in laboratory facilities.

Extensive data are available for monochromatic waves and smooth impermeable slopes (i.e., smooth asphalt or concrete slopes; also, saturated sand beaches are often assumed to be smooth slopes). These smooth slope data, when coupled with a slope surface reduction factor, *r*, representing further analysis of the influence of several roughness elements (i.e., rip-rap, armour stone) and permeabilities (i.e., quarry stone core versus earth core with geotextile filter), are then used for practical applications.

Several uprush methodologies may be considered as "accepted engineering principles" provided they are used in the same context for which they are based. Some of the "accepted" methods include:

- Ahrens and McCartney (1975);
- Stoa (1978; 1979);
- Losada and Gimenez-Curto (1981);
- Ahrens (1981);
- Ahrens and Heimbaugh (1988a);
- Walton and Ahrens (1989);
- Mase (1989);
- Pilarczyk (1990);
- U.S. Army Corps of Engineers (1990);
- Van der Meer and Stam (1992); and
- physical model testing.

The procedures outlined for the various "accepted" wave uprush/runup methods must be followed closely and they should not be extrapolated much beyond the tested conditions.

For any one of the previously noted methodologies used to estimate wave uprush, the proponent should be required to provide a brief summary of how the methodology was derived and why it is applicable to the situation under study. This would help to demonstrate the proponents' understanding of the limits of the method used. In addition, since most of the methods are based upon physical model tests, the proponent should discuss the differences between the model layout and the prototype situation and evaluate whether or not any adjustments should be made to the predicted wave uprush/runup to account for the differences.

This should be valid for small <u>and</u> large project evaluations, where the former is done with existing guidance and the later with the aid of site-specific model tests.

b) Methods of Calculating Wave Overtopping

The prediction of wave overtopping through an entirely theoretical basis of analysis is not yet possible. The inability to predict wave overtopping is essentially due to the complex processes which govern wave interaction with shorelines and protection works. These processes cannot yet be fully explained and put into the form of mathematical equations.

Existing literature on wave overtopping deals mainly with empirical relationships derived from the results of physical model tests carried out in hydraulic laboratories. These tests are limited in number and, for the most part, are site specific. They involve various wave conditions (i.e., wave height, wave period, breaking and non-breaking waves), numerous protection work geometries (i.e., depth of water, uniform and composite profiles, freeboard) and many different materials (i.e., smooth or rough and impermeable or permeable). Until recently, these model tests were carried out with monochromatic waves. In addition, researchers use different techniques for measuring and reporting overtopping, making it difficult to directly compare the various results.

Due to the limited number of laboratory studies, the wide range of wave conditions, and the vast variety of protection work geometries and materials, a generic, empirical methodology for the prediction of the magnitude of wave overtopping does not yet exist. There is, however, some limited guidance in the literature for predicting overtopping rates of some protection works with simple profiles (i.e., uniform vertical seawalls and uniformly sloping armour stone revetments, see Figure 3.19).

For these simple protection works, the results of a number of model tests have been assembled and analyzed and some predictive empirical wave overtopping methodologies have been developed. Most wave overtopping prediction models include the following input parameters (see Figure 3.18), taken at the toe of the protection work:

- wave height, H;
- wave period, T;
- water depth, d_s; and
- freeboard, F (i.e., distance between the stillwater level and the top of the protection work)

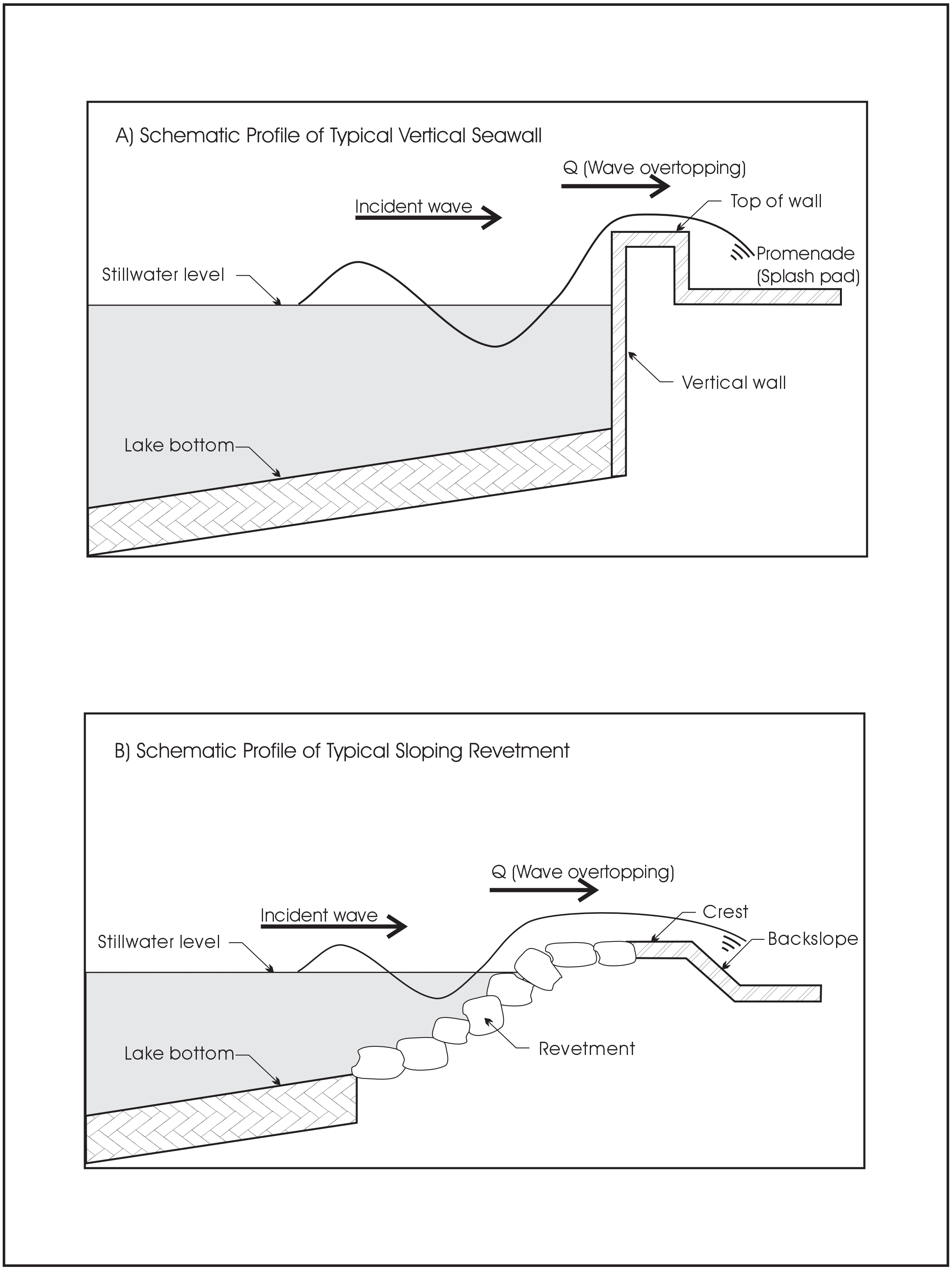
Other models require the equivalent unrefracted wave height, H_o'.

For simple vertical seawalls (Figure 3.19a) and sloping armour stone revetments (Figure 3.19b), typical of the Great Lakes and connecting channels, the following methods are applicable:

For Vertical Seawalls	For Sloping Armour Stone Revetments
Ahrens and Heimbaugh (1988b)	Ahrens and Heimbaugh (1988b)
Goda (1985)	Goda (1985)
	Owen (1982)

These wave overtopping methodologies are described in the technical support document *Wave Uprush and Overtopping* (Atria 1997) and when used appropriately, they may be considered as acceptable practice, for the time being. They are limited to estimating wave overtopping of simple seawall or revetment structures proposed for small scale development and where there is no risk of loss of life. The three methodologies have been commonly used within the coastal engineering industry and may be deemed as being the best estimates available and the most applicable to the types of shoreline protection works and environmental conditions of the Great Lakes - St. Lawrence River System. However, caution must be exercised when applying these methods due to the significant margin of error associated with the results. The technical guidance provided in this document should not be used for final design. Each final design must be site specific and should be carried out by a qualified coastal engineer.

Figure 3.19: Application of Wave Overtopping Models



The three methodologies should not be extrapolated much beyond the tested conditions from which they were derived. Other factors such as the local bathymetry (e.g., offshore bars and composite slopes), berms in front of protection works, wind, and oblique wave attack may change the magnitude of the wave overtopping. As such, these "other" factors must be considered in the estimation of wave overtopping.

For larger scale developments or where there is a risk of loss of life, it is recommended that physical model studies be conducted to measure site specific overtopping rates. Model study results provide a greater degree of certainty when estimating wave overtopping rates.

Existing methodologies for predicting wave overtopping have been reviewed, summarized and presented in Wave Uprush and Overtopping (Atria 1996). In addition, the report supports the description of these methodologies by providing guidance for assessing acceptable rates of overtopping and examples of typical applications.

c) Upper Limit of Wave Uprush

i) Upper-Bound Limit of Wave Uprush Procedures

For the definition and delineation of shoreline *flooding hazard*s in the *Great Lakes - St. Lawrence River System*, an appropriate **upper-bound** curve of the "accepted" uprush procedures for smooth slopes can be used and is given as follows:

$$\frac{R_s}{H_s} = 1.25\,\xi$$

together with and limited by the maximum value as a function of the protection work (or beach) slope

$$\frac{R_s}{H_s} = \sqrt{2\pi}\left(\frac{\pi}{2\theta_r}\right)^{1/4}$$

where:

R_s = the significant uprush (m)
H_s = the significant wave height (m) at d_s
ξ = the surf similarity parameter; and
θ_r = slope of protection works (in radians).

The surf similarity parameter is as follows:

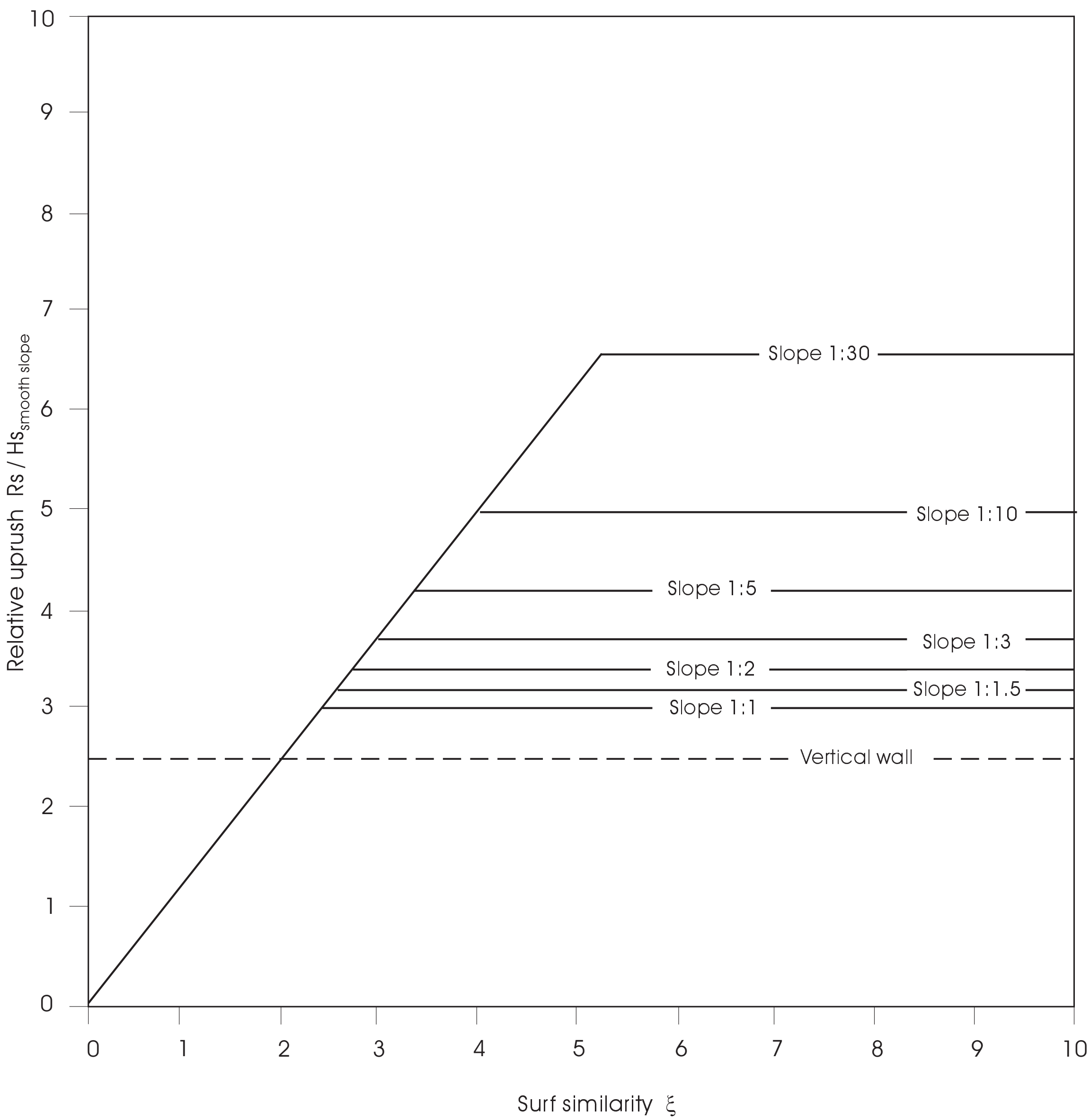
$$\xi = \frac{\tan\theta}{\left(\frac{H_s}{L_o}\right)^{1/2}}$$

where:

θ = slope of protection works (in degrees)
L_o = deep-water wave length (m); $L_o = 1.56T^2$; and T is wave period (s).

Figure 3.20 graphically shows this upper-bound, smooth slope, uprush method.

Figure 3.20: Relative Uprush: Upper-Bound Curves for Smooth Slopes



For rough slopes apply the slope surface reduction factor, r.

$$R/H_{rough\ slope} \approx R/H_{smooth\ slope} \ * \ r$$

For rough slopes, the smooth slope runup values can be modified by the appropriate slope surface reduction factor, *r*, as follows:

$$\left[\frac{R}{H}\right]_{rough\,slope} = \left[\frac{R}{H}\right]_{smooth\,slope} * r$$

Table 3.4 provides an upper bound of the slope surface reduction factors, *r*.

Table 3.4 Upper Bound of Slope Surface Reduction Factors

Slope Surface Characteristics	*r*
Smooth	1.0
Concrete or Gobi blocks	0.9
Grass	0.9
Quarrystone, rubble	0.8
Stepped surface	0.8

ii) Distribution of Irregular Wave Uprush

During a storm, the maximum limit of wave uprush will exceed the value of uprush determined by R_s/H_s. In deep water, approximately 13% of the waves will be larger than H_s, if Rayleigh distributed. This is because even though the wave conditions during a storm are characterized by the single value, H_s, the waves actually vary in height (i.e., they are "irregular"). H_s represents the "significant wave height" and is described as the average of the highest one-third of all the wave heights. Therefore, there are waves that are higher than the H_s value. The same assumption is often made for wave uprush. There will be values of wave uprush that exceed R_s. Thus, overtopping will occur if the top height of the structure above the stillwater level is less than the R_s value.

It is often assumed that the wave uprush is Rayleigh distributed resulting in

$$R_2 = 1.4\,R_s\,; \qquad R_2 = 2.23\,R_m$$

where: R_m = the mean uprush level
R_2 = the uprush level exceeded by only 2% of all the uprush levels.

The R_2 uprush value could be considered as a reasonable limit for minimal overtopping for Great Lakes' shoreline protection works. However, it should be noted that making allowances for safe overtopping is considered to be acceptable engineering design practice (see Section 3.6.1).

In shallow water, the assumption of Rayleigh distribution is only an assumption and is not strictly correct. Due to the limiting effect of depth, the higher deep-water waves break before they reach the shore and there will be a truncation in the wave height distribution. Some research (Ahrens 1981; Mase 1989) indicates that R_2/H_s is approximately equal to 1.3 to 1.4 times R_s/H_s.

To obtain an upper-bound estimate of the R_2/H_s level, the upper-bound curves (Figure 3.19), represented by the equations presented above, could be increased by a factor of 1.4. Alternatively, the curves presented could be used with the local H_2 or H_{max} value at the toe of the protection work. H_2 is the wave height exceeded by only 2% of all the waves. H_{max} is the maximum wave height. H_2 or H_{max} could be estimated using an appropriate methodology such as Goda (1985).

d) Acceptable Rates of Overtopping

A summary of acceptable mean overtopping rates is presented in Figure 3.21. Further guidance for assessing the acceptable rate of overtopping is provided in *Wave Uprush and Overtopping: Methodologies and Applications* (Atria 1997). The guidance provided by Figure 3.21 is grouped according to three categories:

1) usage considerations;
2) structural stability considerations; and
3) flooding/drainage considerations.

All three considerations must be addressed when assessing the allowable level of overtopping. The usage consideration is further subdivided into "vehicles", "pedestrians" and "buildings". The usage guidelines are generally applicable for situations in close proximity (i.e., less than 10 m) to the shoreline or protection works.

Usage considerations should be evaluated using severe wave and water level conditions that are likely to occur during periods of normal operation. Structural stability and flooding/drainage considerations should be evaluated using the 100 year flood level and the associated wave conditions.

It is important to note that during a storm, overtopping is characterized by sporadic, intense events when the larger waves overtop the shore. It is during these brief, intense moments when most of the overtopping volume takes place. For example, over a period of one hour, approximately one-half of the total overtopping volume may be the result of the single largest overtopping wave. The volume of water in this single overtopping wave, and the rapidness with which it occurs (i.e., of the order of a couple of seconds), will determine the hazard level. As a result, it is not so much the average rate that determines the level of inconvenience or danger, although average rates can be used as criteria for acceptable overtopping (Jenson and Juhl 1987).

Wind can have an important influence on the quantity and extent of wave overtopping. Wave spray can be carried much further inland. Building and structure designers should be made aware of the potential for significant icing during freezing weather.

e) Data Requirements

The data required for determining wave uprush and wave overtopping includes (see Figure 3.18):

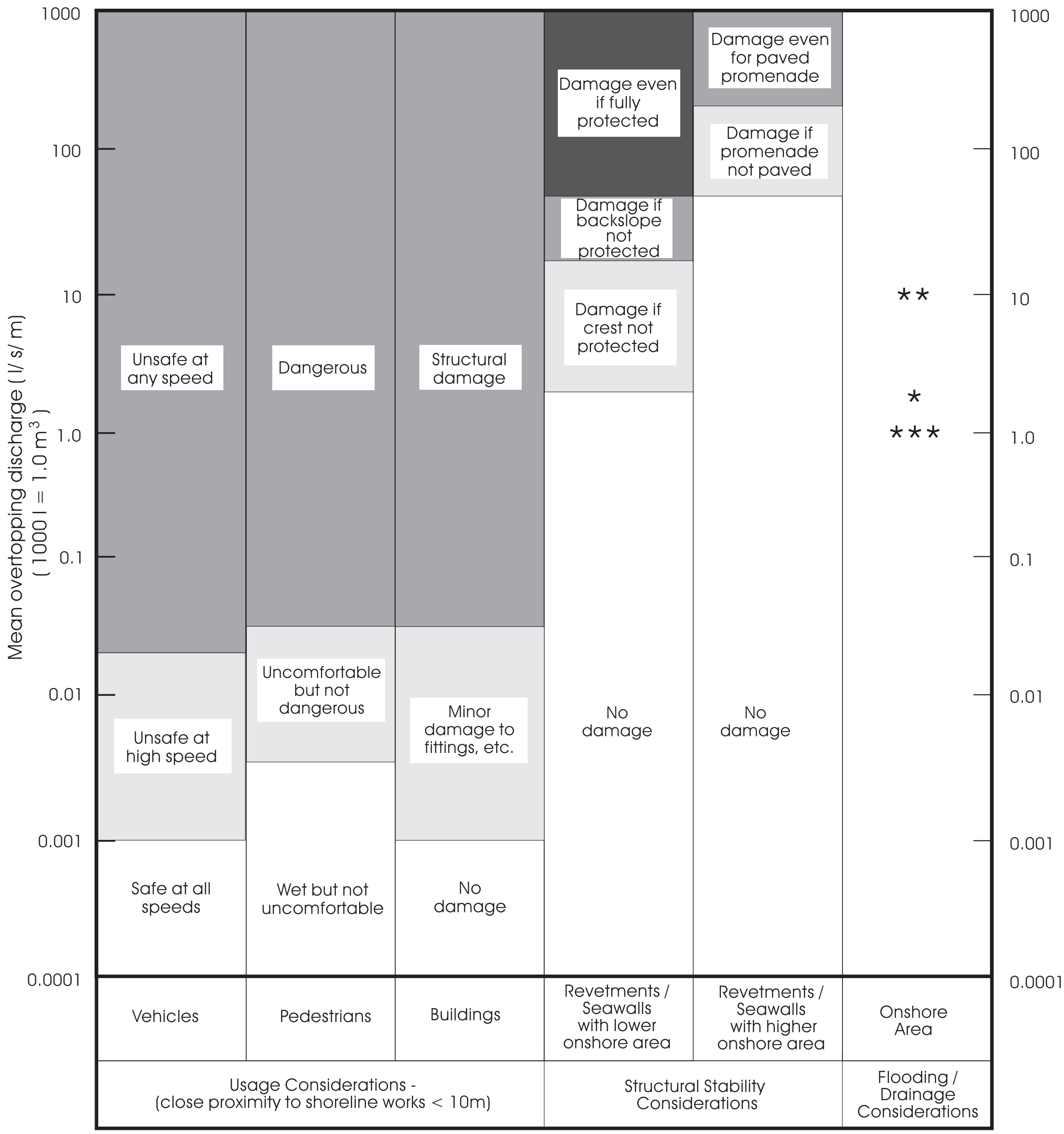
- the stillwater level (*SWL*);
- the incident wave climate (wave height, H, and wave period, T);
- the beach or protection work slope (tan θ);
- the lake bottom slope (m);
- the water depth at toe of the protection work's slope or beach slope (d_s); and
- surface roughness and protection work permeability (Δ_r, and P).

The stillwater level should be the 100 year flood level. The 100 year flood level includes both the mean lake level plus wind setup. More detailed information on water levels is provided in Section 3.4.

When defining wave conditions, for shoreline reaches with significant fetch areas, wind generated deep-water waves should be calculated, or depth-limited nearshore wave conditions assumed, to determine the wave uprush offset allowance. A fetch is considered to be significant where the maximum fetch distance measured over an arc extending 60° on either side of a line perpendicular to the shoreline is greater than 5 km.

If deep-water waves are used, the deep-water waves then have to be transformed to the nearshore to take into account the effects of refraction, shoaling, diffraction and other influences. Simple techniques for estimating refraction, shoaling and diffraction, based on linear wave theory, are available in the *Shore Protection Manual* (USACE 1984). In practice, most nearshore wave transformation studies carried out at the present involve more sophisticated computerized models. These models must be used and interpreted by qualified coastal engineers.

Figure 3.21: Summary of Acceptable Overtopping Rates (adapted from CIRIA / CUR 1991)



* 2 l / s / m considered acceptable for dykes in Holland (Pilarczyk 1990).

** Suggested limit by GODA (1985) for utilization considerations and requires drainage channel of sufficient width (few tens of metres wide) and strong paving to withstand impact of falling water.

*** Suggested limit by GODA (1985) if ground elevation behind structure is lower than flood level or if insufficient facilities for proper drainage. Keillor and Miller (1987) suggest this limit for residential properties.

i) Waves and the Flooding Hazard

Storm surge and wave action are not independent events as both are wind driven phenomena; when you have a storm surge on the Great Lakes, you are likely to have wave action as well. Further, the 100 year flood level is a combination of mean lake level and storm surge. It follows that the 100 year flood level is not independent of wave action. In order to determine the limit of wave uprush for the purpose of determining the limit of the *flooding hazard*, it is necessary to know the wave conditions which are reasonably likely to accompany the 100 year flood level. There are three approaches that can be used: the first, is based on the appropriate return period of the deep-water waves; the second, is derived from the wave conditions resulting from a maximum sustained wind speed; and the third, is the based on the depth limited wave condition. Both the first and second approaches require transformation of the deep-water waves to the site in the shallow nearshore (i.e. local wave conditions).

. Return Period of Deep-Water Waves

Very little research has been carried out on deep-water wave conditions which are likely to accompany the 100 year flood level. In *Guidelines for Great Lakes Wave Runup Computation and Mapping*, the U.S. Federal Emergency Management Agency (FEMA 1991) recommends using the 100 year deep-water wave condition. To determine the 100 year deep-water wave, it is necessary to complete a return period analysis of the deep-water wave hindcast data (e.g., hourly wave heights over a period of 10 to 20 years). Deep-water wave databases for the Great Lakes include: Ministry of Natural Resources (Philpott 1988; Sandwell 1988; and MacLaren Plansearch 1988); and U.S. Army Corps of Engineers, Waterways Experiment Station, Wave Information Study (WIS).

Preliminary work by Dewberry and Davis (1994), on wave action with extreme floods on the U.S. side of the Great Lakes, suggests that expected waves coincident with the 100 year flood on the four upper lakes (Superior, Huron, Michigan and Erie) may be described by the wave height with a recurrence interval of three years. On Lake Ontario, their preliminary work indicates that the expected wave height is summarized as having a recurrence interval of one-half year. Dewberry and Davis (1994) also reported that their guidance appears to be consistent with the approach of using wave heights derived from a maximum sustained wind speed of 65 km/h (40 mph).

It should be noted that the preceding findings of Dewberry and Davis are preliminary and were developed for the U.S. shores of the Great Lakes. The U.S. shores are exposed to prevailing and storm wind directions which are different from those at the Ontario shores. Hence, the Dewberry and Davis results may not be directly applicable to the Ontario shoreline. In developing this Technical Guide, wave heights were estimated for 13 of the MNR Great Lakes deep-water wave database sites: 2 on Lake Superior, 3 on Lake Huron, 2 on Lake St. Clair, 3 on Lake Erie and 3 on Lake Ontario. The wave heights were estimated by using a maximum sustained wind speed of 65 km/h (90 km/h for Lake St. Clair), as outlined in the following subsection. The results for the 13 sites were then compared to the return periods of the corresponding wave heights in the MNR wave database. It was estimated that wave heights calculated using the 65 km/h winds were generally less than the 10-year to 20-year wave heights from the MNR wave database for Lakes Superior, Huron, St. Clair (using the 90 km/h wind), and Ontario. For Lake Erie, the 5-year MNR database wave heights were greater than the waves generated by the 65 km/h wind.

It is recommended that until further study is carried out for the Ontario shore, that 10-year to 20-year return period wave heights, calculated from the MNR deep-water wave database, should be reasonably safe estimates of wave heights to be used in conjunction with the 100 year flood level to determine the limit of uprush for the purpose of delineating the *flooding hazard limit*.

. Maximum Sustained Wind Speed

If a deep-water wave database is not available, the deep-water wave conditions (H_s and T_p), to be used to determine the *flooding hazard limit*, can be estimated using wave prediction techniques presented in the 1977 edition of the *Shore Protection Manual* (USACE 1977) along with the maximum sustained wind speed for the Great Lakes and the appropriate straight fetch length. The deep-water wave is then transformed to the location at the structure toe. Bishop et al. (1989) recommend the wave prediction method presented in the 1977 edition of the Shore Protection Manual.

A wind speed of 65 km/h is considered to be a reasonable estimate of the maximum sustained wind speed on the Great Lakes (FEMA 1991). Wind measurements at various lake-centre sites demonstrate that winds of 65 km/h constitute a "moderately extreme condition, occurring about 5 to 20 hours per year" (Dewberry and Davis 1994). For Lake St. Clair, because it is relatively shallow and has shorter fetch distances, the appropriate sustained wind speed used should be greater than 65 km/h. A preliminary recommendation is to use a wind speed of 80 to 100 km/h for Lake St. Clair.

. Depth-Limited Waves

An alternative approach to determining the deep-water wave conditions and then transforming the waves to the nearshore is to assume depth-limited wave conditions in shallow nearshore waters. Depth-limited simply means that the wave height is physically limited by the depth of the water. That is to say, a given depth of water can only support a certain maximum wave height. There are a number of depth-limited wave height or breaking wave criteria as follows:

Linear wave theory (approach slope flatter than 1:100)

$$H_s = 0.78\, d_s$$

Hughes (1984) (engineering approximation for typical beach slopes)

$$H_{mo} = 0.6\, d_s$$

Kamphuis (1991)

$$H_{sb} = 0.095\, e^{4m} L_{pb} \tanh k_{pb} d_b$$

Two procedures which incorporate the depth-limited approach for estimating local wave conditions are Goda (1985) and FEMA (1991).

It is useful to know the deep-water wave height associated with the depth-limited wave height (i.e., what deep-water wave height is necessary to produce the depth-limited wave, including effects of wave refraction, shoaling, etc.). Comparison of the depth-limited associated deep-water wave height with actual deep-water wave statistics characteristic of the site will give some indication of how often the structure could be subjected to waves as high as the depth-limited wave. It is likely for many shallow nearshore sites, depth-limited conditions will prove to be very frequent events (possibly annually or more frequently). In many cases, this will reduce the importance of deciding the accuracy of the 10 year to 20 year deep-water wave height and the 50 year to 100 year deep-water wave height criteria.

ii) Waves and the Floodproofing Standard

The wave condition to be used for defining the *floodproofing standard* should be more extreme than the wave condition for the *flooding hazard*. The *floodproofing standard* for the Great Lakes is the sum of the 100 year monthly mean lake level plus the 100 year storm surge plus an allowance for wave action. Wave action that is likely to accompany the 100 year storm surge has not been defined by any studies but can be expected to be more extreme than wave action which accompanies the 100 year flood level.

It is recommended that until further study is carried out, a 50-year to 100-year return period wave height, determined from an appropriate deep-water wave database, be used in conjunction with the 100 year monthly mean lake level and the 100 year storm surge to determine the *floodproofing standard*.

iii) Sensitivity Analysis

A range of wave conditions (i.e., height and period) should be used to determine sensitivity of the wave uprush methodologies. Depending on the site conditions (i.e., slope, approach slope and water depth), the maximum wave height may not necessarily produce the maximum value of wave uprush.

iv) Incident Versus Transmitted Waves

In instances where a structure, such as a detached breakwater, may act to reduce the incoming, or incident, wave action at a site, it may be necessary to estimate the transmitted wave height (i.e., the wave height on the leeside or shoreward side of the structure) for the purpose of determining the *floodproofing standard*. Estimates of transmitted wave height can be made by qualified coastal engineers using guidance from the literature (i.e., CIRIA/CUR 1991; Allsop 1983; Bremner, Foster, Miller and Wallace 1980; Seelig 1979; van der Meer 1990).

v) Shoreline Slope, Approach Slope and Water Depth

Detailed data on the nearshore bathymetry (e.g., site specific sounding surveys and/or field sheets from the Canadian Hydrographic Service) and lake bottom profiles and onshore topography (e.g., information can be obtained from FDRP mapping or site survey) at the study site are needed for evaluations of wave uprush and overtopping. In undertaking these evaluations, it is important to obtain the nearshore bathymetry near the site in order to define the transition from the approach slope (i.e., lake bottom) to the shore or protection works slope and to determine if a composite slope procedure is applicable. A composite slope consists of various slopes as opposed to a single constant slope.

A further complicating factor arises when the extent of the calculated uprush extends past the top of the shore or structure slope such as might occur along a low bluff or bank shoreline. Guidance on the inland extent of the wave uprush is limited (FEMA 1989; Cox and Machemehl 1986).

Further information on beach slope, approach slope and water depth is provided in *Wave Uprush and Overtopping: Methodologies and Applications* (Atria 1997). Appendix A3.5 outlines the procedure of FEMA (1989) for estimating the inland extent of wave uprush, when the uprush extends beyond the top of the slope.

vi) Surface Roughness and Structure Permeability

The surface roughness and structure permeability is dependent on the structure type. The surface roughness of various materials is demonstrated in Table 3.4. Saturated sandy beaches are typically assumed to be smooth and impermeable. Permeability of the structure depends on the thickness and number of armour layers, the underlayers and filter. Single layer armour stone revetments with a minimal under layer of rip-rap and a geotextile filter may be considered as a rough impermeable structure for the purpose of wave uprush. This due to the fact that the velocity of the uprushing and downrushing water greatly exceeds the ability of the geotextile filter to pass water. Further information on roughness and permeability is provided in *Wave Uprush and Overtopping: Methodologies and Applications* (Atria 1997).

f) Alongshore Considerations

For isolated protection works (i.e., no protection at adjacent alongshore properties) or for protection works with low level protection at adjacent properties, the provision of adequate wave uprush protection, at the one site only, may be insufficient to protect the property against flooding. Water overtopping the shoreline at the adjacent properties may flow unimpeded onto the subject property. Consideration must be given to the provision of drainage of water from adjacent properties.

g) Shoreline Protection Works

The structural stability of protection works can be threatened when they are overtopped by waves. Overtopping can erode the area behind or above the protection work, resulting in the removal of material which supports the structure and may ultimately lead to the potential failure of the protection work.

The effects of overtopping can be reduced by increasing the crest elevation (i.e., increase the freeboard, *F*) or by protecting the surface area behind the protection work. The protection work can consist of armour stone, rip-rap, concrete or asphalt pavement, proprietary reinforced grass or soil products or other suitable materials. Of primary importance is the provision of a proper bedding and filter for the selected protection works and a continuous connection with the filter of the primary protection. A description of some of the methods for estimating the overtopping protection requirements for shoreline structures are outlined in Atria (1997).

h) Drainage Provisions

In the design of seawalls, the drainage system for the overtopped water should be well planned, ensuring that the volume of overtopping water due to storm waves, which can be considerable, is properly calculated and accommodated. The drainage system must ensure rapid drainage otherwise recovery operations may be hampered.

Factors to take into consideration, when designing a proper drainage system for any development, include the following:

- the temporal and spatial distribution of the overtopping water (i.e., most of the water volume is the result of relatively few waves resulting in short periods of high flow with the greatest intensity closest to the protection works);

- while overtopping intensity may decrease as you move away from the shoreline, the increased distance does not diminish the flooding risk due to the total volume of water which comes over the structure;

- the potential for blockage of drains by wave carried sediments and debris; and

- the potential freezing of the drains.

The predictors of wave overtopping can provide estimates of the average overtopping rate. Using established, standard civil engineering drainage design methods as a guide, a designer should then ensure that these factors are incorporated into a drainage system for the selected protection works. The resulting shore drainage should be much larger than typical land-based systems.

Where drainage provisions landward of a protection work are uncertain or are not sufficient, the elevation of the development should be greater than the level of the limit of wave action (i.e., the development must meet the conditions of the *floodproofing standard*).

3.6 OTHER WATER RELATED HAZARDS

By definition, the term "*other water 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, ice jamming and ship generated waves.

Ice piling occurs when wind driven currents and waves carry ice sheets or flows on top of one another and eventually onto the shore. Ice jams, usually occurring during spring ice breakup, involves the broken pieces of ice moving to join a stationary piece of ice to form a jam. The most significant impacts of ice jams, when they clog a river or a connecting channel, is that of shoreline flooding.

Depending on the shoreline configuration and slope characteristics, ship generated waves or surface waves from boats can rush up the shoreline past the 100 year flood level. The subsequent ship generated wave drawdown can scour and damage a shoreline or protection works.

3.6.1 Ice

Ice is often seen as a natural protector of shoreline environments by acting as a protective layer. Shore ice and ice piling along the shoreline or channel bank can reduce the influence and overall destructive impact of wave action. Ice can also be damaging to shorelines and cause erosion of beaches.

Ice can also cause severe flooding, particularly during spring breakup. During this spring period, damage occurs most often when ice jams, frazil dams or other ice initiated restrictions increase water levels and deflect or increase water flows onto shoreline areas.

The ice cover, itself, can also cause changes to the hydraulics of the *Great Lakes - St. Lawrence River System* posing local and regional flooding threats. This usually occurs where a portion of the system which is normally open channel flow is restricted or closed and becomes closed to conduit flow, resulting in flooding.

Hazards related to ice, including ice jams and ice piling are to be addressed and included in the calculation of the flood allowance for wave uprush and other water related hazards. Where local conditions suggest that the flood allowance standards (i.e., 15 metres for Great Lakes and 5 metres on connecting channels) are not sufficient to address recurring problems associated with ice piling and ice jams, a more detailed engineering study, using accepted engineering principles, should be considered.

a) Ice Jams

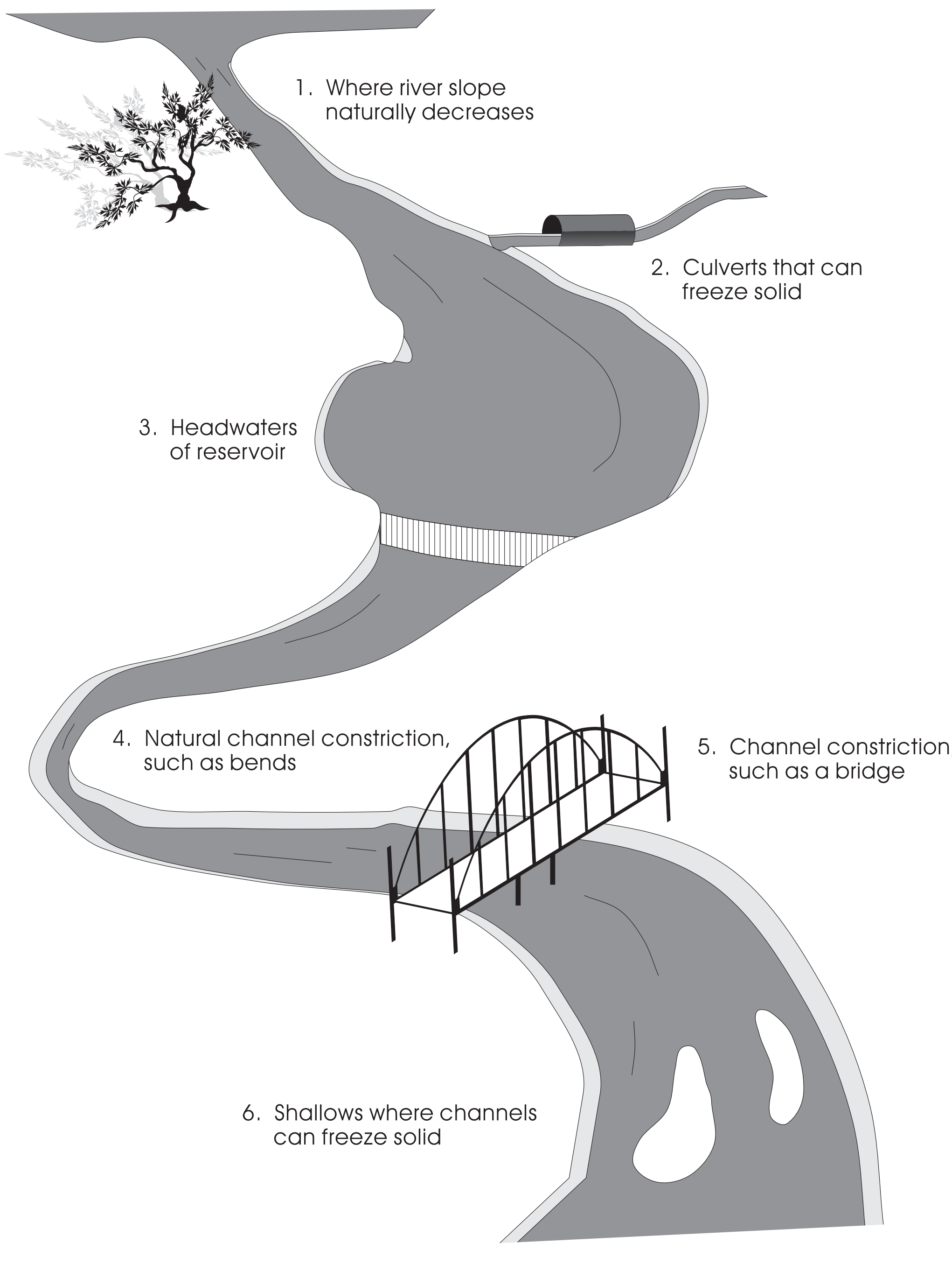
Ice jams generally occur during spring ice breakup, where the broken pieces of ice floes move downstream and/or downdrift and join a stationary piece of ice to form a jam. Ice jams can be found in restricted or shallow areas of the connecting channel, and where velocities upstream may be slower (e.g., connecting channel entrances, islands, sand bars and around protection works such as piers or bridges). Figure 3.22 indicates potential ice jam areas.

There are basically two types of ice jams:

- simple or floating jam; and
- grounded or "dry" jam.

The simple jam involves the accumulation of the fragmented or frazil ice that does not obstruct the flow of the water under the jam. The grounded or "dry" ice jam, by comparison, will completely restrict the flow of the water. The dry ice jam usually begins with a solid piece of ice which will not move or breakup. The broken ice on the updrift side of the dry jam then continues to build up behind the jam restricting the regular flow of water. This results in a rise in water levels and increased flooding and damage to adjacent shoreline and protection works.

Figure 3.22: Potential Ice Jam Areas



Ice jams are natural phenomena which tend to have a direct impact on flooding in lower reaches of Lake Huron and in the Niagara, Detroit, St. Clair and St. Mary's Rivers. For additional information on ice jamming controlling factors and potential impacts and for maps delineating the general locations throughout the *Great Lakes - St. Lawrence River System* which are susceptible to ice jamming problems, Appendix A3.6 of this Technical Guide should be consulted.

b) Ice Piling

Ice piling or rafting generally occurs when wind driven currents and waves carry ice sheets or floes on top of one another. This rafted ice can continue to build up and may eventually ride up on shore. Ice piling can occur at mouths of the connecting channels, when flood floes carrying ice within the connecting channel meet a frozen lake and spread onto adjacent lands, or along the shoreline of the lakes.

Ice piling or rafting can pose considerable threat to shoreline protection works as a direct result of scouring action and the extreme forces exerted by the ice piling action.

Extensive and consistent annual erosion damages as a result of ice piling generally tends to be an infrequent phenomena. Although damages resulting from ice piling may be experienced over a long stretch of shoreline, only a small portion of that shoreline is likely to sustain extensive erosion damages due to ice piling in any given year.

As such, to require that protection works be designed to withstand these ice piling forces which occur infrequently may prove unjustifiable and uneconomical. Conversely, in local shoreline areas where ice piling is a significant concern, inclusion of appropriate design criteria to address ice piling impacts should be reviewed in detail and appropriate measures taken to ensure that these hazards are overcome prior to development approval.

Ice piling or ridging events are known to have occurred on Lakes Erie, Ontario and St. Clair. Discussions with Raisin River Conservation Authority and Environment Canada staff located in Cornwall, Ontario indicate that ice piling has caused damage at various locations along the Lake St. Francis shoreline. Local experience with the impacts of ice piling is the best guide to help define the extent of hazard land (Environment Canada 1993). For additional information on ice piling and on the areas throughout the *Great Lakes - St. Lawrence River System* which are susceptible to ice piling, Appendix A3.6 of this Technical Guide should be consulted.

c) Methods of Calculation

At the present time there are no sufficiently accurate methods of forecasting ice jamming or ice piling. To predict the probability or exact time of an ice jam or ice piling event, many variables which cause such an event to occur (e.g., time of year, air temperature, stage of ice decay, changing wind conditions, ice thickness and strength, snow thickness, strength of the ice cohesion to the shoreline or shore ice) must be fully understood, measurable and incorporated into a theoretical model. To date, a complete theoretical form to predict ice jamming and/or ice piling events does not exist. As a result, these events are unpredictable and are not represented by generalized theories.

3.6.2 Ship Generated Waves

Ship generated waves are a familiar and common phenomenon to people living and interacting near harbours, embayments, navigation channels, or other bodies of water on which ships operate.

In areas susceptible to ship wave action, *flooding hazard* areas may extend landward beyond the 100 year flood level to the limit of the ship wave action. Not all shorelines in the *Great Lakes - St. Lawrence River System* are considered susceptible to ship wave action.

The St. Lawrence River and Seaway, St. Clair, Detroit and St. Mary's Rivers have encountered problems in the past and can be considered susceptible to ship waves. In these areas, local conditions such as bathymetry, water levels, soil conditions, ice conditions, protection works, currents and waves have a direct effect on the degree to which a site is susceptible to ship waves (Wuebben 1983). As such, site specific conditions may require that investigations using acceptable engineering principles be carried out in these and other locations affected by ship generated waves.

For additional information on ship generated waves and for maps delineating the general locations throughout the *Great Lakes - St. Lawrence River System* which are susceptible to ship or small craft wave problems, Appendix A3.6 of this Technical Guide should be consulted.

The following discussion regarding the wave uprush allowance along the St. Lawrence River is taken from Environment Canada (1993):

> "Up to now, a 0.5 metre vertical freeboard for wave uprush has been used along much of the St. Lawrence River and local staff have indicated that the 5 metre horizontal setback generally recommended for interconnecting channels may not be sufficient for much of the St. Lawrence River. Within the St. Lawrence River, a study by the St. Lawrence Seaway Authority Operations Branch (1971) completed for the St. Lawrence River below Cornwall shows that, for the recommended velocities, commercial vessels will produce waves on average about 0.3 m in height. The maximum observed wave measured was 0.67 m from trough to crest. The wave height above normal water level is one-half the above values. The existence of flood rights upstream of the Moses-Saunders power dam provides further support for the maintenance of the freeboard approach. It should be noted that even with a 0.5 m freeboard added, the flood zone elevations still fall below the 76.3 metres GSC [upstream of Iroquois Control Dam to Cardinal] (or 76.0 metres GSC [upstream of Long Sault dam]) flood entitlements. A wave uprush allowance based on the highest elevation resulting from either a 0.5 metre vertical freeboard or a 5 metre horizontal setback might represent the best alternative. If this combined approach or the horizontal setback approach alone is adopted, it is assumed that a 15 metre setback for wave uprush, as opposed to a 5 metre setback, should apply to Lake St. Francis based on fetch lengths, etc.. Other shoreline sections along the study reach where the 5 metre setback may not be adequate should be identified."

By definition, the calculation of the flood allowance for wave uprush and other water related hazards includes the influence of ship generated waves. Where local conditions suggest that the flood allowance standards (i.e., 15 metres for Great Lakes and 5 metres on connecting channels) are not sufficient to address recurring problems associated with ship generated waves, a more detailed engineering study, using accepted engineering principles, should be considered.

a) Methods of Calculation

At the present time the level of analysis and understanding of the processes related to ship waves in the *Great Lakes - St. Lawrence River System*, and particularly in the connecting channels, is limited. There is no predictive method available for determining the limit of wave uprush due to ship generated waves.

Although present techniques do not translate the ship generated wave information into a horizontal distance, they are useful in providing an understanding of some of the processes which are occurring as a result of ship generated wave activity. Appendix A3.6 provides a review of ship wave processes.

Despite the usefulness of this information, it is apparent that the existing information that is available for the assessment of ship generated waves on the *Great Lakes - St. Lawrence River System* is insufficient to assess the necessary flood allowance for new development. Additional study and analysis on the calculation and impacts of ship generated waves is required.

Based on existing information, a translation of ship wave characteristics is required for the calculation of wave uprush. The present uprush prediction methodologies (Section 3.4) are primarily empirical and are based on wave height and wave period data characteristic of wind waves. To ensure the effectiveness of these theories to address ship generated wave action, additional analysis on the conversion of ship wave characteristics, both long and short period waves, to wave height and wave periods appropriate for the use in the calculation of wave uprush must be undertaken.

3.7 FLOODING HAZARD LIMITS AT JUNCTIONS OF THE GREAT LAKES - ST. LAWRENCE RIVER SYSTEM AND RIVER AND STREAM SYSTEMS

Determining the relevant *flooding hazard* limit at the junction of a lake and river or stream is based on a comparison of which *flooding hazard* governs the site, namely the *Great Lakes - St. Lawrence River System* hazard limit or the river and streams hazard limit. In other words, the decision on which limit applies is based on which factors most influence the level of flood risk or hazard at a given location.

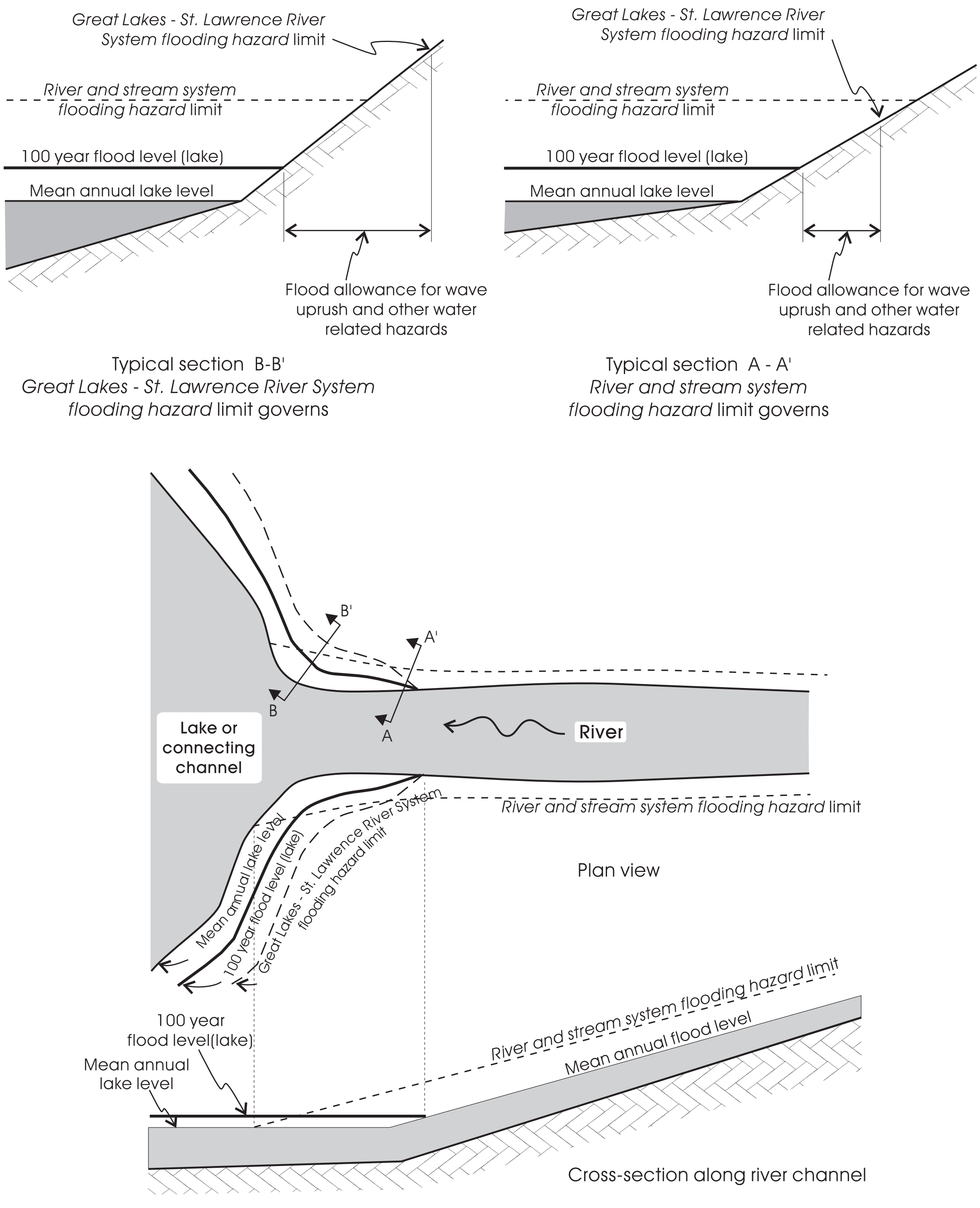
This determination may become particularly difficult at a wide river mouth due to the interaction of lake-generated waves with the channel currents and the complex bathymetry at the river mouth, as the waves propagate into the river or stream system flood plain. If the currents in the riverine flood plain are significantly high, the wave propagation pattern (i.e., refraction and diffraction) will be altered and the wave conditions along the connecting channel, which are essential for wave uprush calculation, may be affected.

Determining which *flooding hazard* limit applies is based on the same principles outlined in the *Technical Guide for River and Stream Systems* (MNR 1996) and are as follows:

> Rivers flowing into the Great Lakes require an analysis of the respective river and lake flood levels. Where the high water conditions at the junction are generated by two independent flood events, the *flooding hazard* limit should be based on the <u>higher</u> of:
>
> i) mean annual lake level and the *river and stream systems flooding hazard* limit as shown in Figure 3.23, Section A-A';
>
> <u>or</u>
>
> ii) *Great Lakes - St. Lawrence River System flooding hazard* limit and the mean annual flood level in the *river and stream sytem*, as shown in Figure 3.23, Section B-B'.
>
> Rivers flowing into connecting channels also require an analysis of the respective flood levels. Where the high water conditions at the junction of the connecting channels and a riverine flood plain are generated by two independent flood events, the *flooding hazard* limit should be based on the <u>higher</u> of:
>
> ii) mean annual flood level in the connecting channel (*Great Lakes - St. Lawrence River System*) and the *river and stream systems flooding hazard* limit;
>
> <u>or</u>
>
> ii) the *flooding hazard* limit in the connecting channel (*Great Lakes - St. Lawrence River System*) and the mean annual flood level in the *river and stream system*.

In certain areas, ice jamming or the closure of barrier beach outlets, at the junction of rivers flowing into the Great Lakes, may result in flooding upriver which is greater than either of the storm-centred events or the 100 year flood, for *river and stream systems* as outlined in the *MNR Technical Guide for River and Stream Systems* (1997). In these situations, local experience of actual floods should be used to establish the appropriate *flooding hazard* limit.

Figure 3.23: Flooding Hazard Limits at Junction of River and Lake



Note: *Erosion* and *dynamic beach hazard* limits for *Great Lakes - St. Lawrence River System* and e*rosion hazard* limits for *river and stream systems* must still be evaluated and applied if further landward.

3.8 REFERENCES

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INDEX - PART 3

FLOODING HAZARD

TECHNICAL GUIDE FOR
GREAT LAKES - ST. LAWRENCE RIVER SHORELINES

APPENDIX A3.1

GREAT LAKES - ST. LAWRENCE RIVER SYSTEM

FLOOD LEVELS

GREAT LAKES - ST. LAWRENCE RIVER SYSTEM FLOOD LEVELS

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A3.1 GREAT LAKES - ST. LAWRENCE RIVER SYSTEM FLOOD LEVELS

The purpose of Appendix A3.1 is to provide information and data sets specific to the *Great Lakes - St. Lawrence River System* to assist shoreline managers in the calculation and definition of shoreline *flooding hazards.*

A3.1.1 Provision of Data Sets

A series of *Great Lakes - St. Lawrence River System* maps indicating the 100 year flood levels (m, GSC) are provided in Part 3, Figures 3.8 to 3.16.

Tables A3.1.1 to A3.1.4 are provided to assist in the calculation and definition of the *flooding hazards* along the shorelines of the *Great Lakes - St. Lawrence River System*:

- Table A3.1.1 — list of 100 year flood levels (GSC) by lake/connecting channel and sector
- Table A3.1.2 — peak instantaneous stillwater (i.e., flood) levels (IGLD 1955) for all sectors for recurrence intervals from 2 to 200 years
- Table A3.1.3 — highest annual monthly mean lake levels (IGLD 1955) for recurrence intervals from 2 to 200 years
- Table A3.1.4 — wind setup values for lakes for recurrence intervals from 2 to 200 years

A3.1.2 Differences between Geodetic Datum and International Great Lakes Datum

To assist shoreline managers in determining the potential impact on the calculation of shoreline *flooding hazards* with the use of Geodetic Datum (i.e., Geodetic Survey Canada, GSC; also referred to as Canadian Geodetic Datum, CGD) in relation to the use of International Great Lakes Datum (i.e., IGLD 1955 and IGLD 1985), Table A3.1.5 provides information on the differences between the three datum at a number of specific sites throughout the *Great Lakes - St. Lawrence River System.*

Table A3.1.1
100 Year Flood Levels

Sector No.	Sector Description	100 Year Flood Level (m) GSC
	LAKE SUPERIOR	
S-1	Pine Point	184.0
S-2	Thunder Bay	183.9
S-3	Rossport	184.0
S-4	Michipicoten	184.1
S-5	Gros Cap	184.2
S-6	Pointe Louise	184.3
	St. Mary's RIVER	
SR-1		184.4
SR-2		178.2
SR-3		178.1
SR-4		178.0
SR-5		177.9
	LAKE HURON	
H-1	Neebish	177.9
H-2	Richards	177.8
H-3	Hilton	177.7
H-4	St. Joseph	177.8
H-5	Thessalon	177.7
H-6	Mississagi Bay	177.8
H-7	Little Current	177.9
H-8	Cape Robert	177.8
H-9	N. Cockburn Is.	177.7
H-10, -11	S. Shore	177.6
H-12	N. Georgian Bay	177.8
H-13	Parry Sound	177.9

Sector No.	Sector Description	100 Year Flood Level (m) GSC
	LAKE HURON (cont'd)	
H-14	Collingwood	178.0
H-15	Meaford	177.9
H-16	Dyer's Bay	177.8
H-17	Tobermory	177.7
H-18	Southampton	177.6
H-19	Point Clark	177.7
H-20	Goderich	177.8
H-21	Kettle Point	177.9
H-22	Bright's Grove	178.0
	ST. CLAIR RIVER	
SCR-1		177.9
SCR-2		177.8
SCR-3		177.7
SCR-4		177.6
SCR-5		177.5
SCR-6		177.4
SCR-7		177.3
SCR-8		177.2
SCR-9		177.1
SCR-10		177.0
SCR-11		176.9
SCR-12		176.8
SCR-13		176.7
SCR-14		176.6

Sector No.	Sector Description	100 Year Flood Level (m) GSC
	LAKE ST. CLAIR	
SC-1	Walpole	176.6
SC-2	Mitchell	176.8
SC-3	Dover	176.7
SC-4	Thames	176.8
SC-5	Tremblay	176.5
SC-6	Stoney Point	176.4
SC-7	Belle River	176.3
SC-8	Tecumseh	176.2
	DETROIT RIVER	
DR-1		176.2
DR-2		176.1
DR-3		176.0
DR-4		175.9
DR-5		175.8
	LAKE ERIE	
E-1	Bar Point	175.8
E-2	Kingsville	175.7
E-3	Pelee West	175.6
E-4	Wheatley	175.5
E-5	Port Crewe	175.4
E-6	Erieau	175.3
E-7	Port Glasgow	175.4
E-8	Port Stanley	175.5
E-9	Port Bruce	175.6
E-10	Port Burwell	175.7
E-11	Hemlock	175.8
E-12	Clear Creek	175.9
E-13	Erie View	176.1

Sector No.	Sector Description	100 Year Flood Level (m) GSC
	LAKE ERIE (cont'd)	
E-14	Long Point Park	176.3
E-15	Long Point Central	176.5
E-16	Long Point East	176.6
E-17	Long Point Bay	176.3
E-18	Nanticoke	176.4
E-19	Selkirk	176.5
E-20	Port Maitland	176.6
E-21	Mohawk Point	176.7
E-22	Port Colborne	176.8
E-23	Point Abino	176.9
E-24	Crystal Beach	177.0
E-25	Fort Erie	177.1
	LAKE ONTARIO	
O-1	Port Weller	76.1
O-2	Burlington	76.0
O-3	Oakville	75.9
O-4	Mississauga	75.8
O-5	Toronto	75.7
O-6	Oshawa	75.6
O-7	Cobourg	75.7
O-8	Wellington	75.7
O-9	Point Petre	75.7
O-10	Prince Edward	75.8
O-11	Kingston	76.0

Table A3.1.1 100 Year Flood Levels cont'd
(from Environment Canada, 1993)

SECTOR		100 Year FLOOD LEVEL (m GSC[1])
No.	DESCRIPTION (From / To)	
SLR-1	Kingston / Gananoque Town Limit (East)	76.0
SLR-2	Gananoque Town Limit (East) / Brown's Bay Park (East Boundary)	75.9
SLR-3	Brown's Bay Park (East Boundary) / Townline between Elizabethtown TP and Augusta TP (CRCA East Boundary)	75.8
SLR-4	Townline between Elizabethtown TP and Augusta TP (CRCA East Boundary) / County Road 31 at Blue Church Bay	75.7
SLR-5	County Road 31 at Blue Church Bay / Elevator upstream of Prescott-Ogdensburg Bridge	75.6
SLR-6	Elevator upstream of Prescott-Ogdensburg Bridge / Downstream end of old canal opposite west end of Galop Island	75.5
SLR-7	Downstream end of old canal opposite west end of Galop Island / Cardinal Village Limit (West)	75.4
SLR-8	Cardinal Village Limit (West) / Presqu'ile (East End)	75.3
SLR-9	Presqu'ile (East End) / Iroquois Lock and Dam (Above)	75.2
SLR-10	Iroquois Lock and Dam (Below) / Boundary between Williamsburg TP and Osnabruck TP (RRCA West Boundary)	74.7
SLR-11	Boundary between Williamsburg TP and Osnabruck TP (RRCA West Boundary) / Saunders Power Dam (Above)	74.6
SLR-12	Saunders Power Dam (Below)[2] / 0.7 km below Saunders Power Dam	49.1
SLR-13	0.7 km below Saunders Power Dam / 1.4 km below Saunders Power Dam	49.0
SLR-14	1.4 km below Saunders Power Dam / 2.2 km below Saunders Power Dam	48.9
SLR-15	2.2 km below Saunders Power Dam / 3.0 km below Saunders Power Dam	48.8
SLR-16	3.0 km below Saunders Power Dam / 3.6 km below Saunders Power Dam	48.7
SLR-17	3.6 km below Saunders Power Dam / 4.0 km below Saunders Power Dam	48.6
SLR-18	4.0 km below Saunders Power Dam / 4.4 km below Saunders Power Dam	48.5

SECTOR		100 Year FLOOD LEVEL (m GSC[1])
No.	DESCRIPTION (From / To)	
SLR-19	4.4 km below Saunders Power Dam / 4.8 km below Saunders Power Dam	48.4
SLR-20	4.8 km below Saunders Power Dam / 5.2 km below Saunders Power Dam	48.3
SLR-21	5.2 km below Saunders Power Dam / 5.6 km below Saunders Power Dam	48.2
SLR-22	5.6 km below Saunders Power Dam / 6.1 km below Saunders Power Dam	48.1
SLR-23	6.1 km below Saunders Power Dam / 6.5 km below Saunders Power Dam[3]	48.0
SLR-24	6.5 km below Saunders Power Dam[3] / 8.0 km below Saunders Power Dam	47.9
SLR-25	8.0 km below Saunders Power Dam / 10.0 km below Saunders Power Dam	47.8
SLR-26	10.0 km below Saunders Power Dam / 12.1 km below Saunders Power Dam	47.7
SLR-27	12.1 km below Saunders Power Dam / 14.1 km below Saunders Power Dam	47.6
SLR-28	14.1 km below Saunders Power Dam / 16.2 km below Saunders Power Dam[4]	47.5
SLR-29	16.2 km below Saunders Power Dam[4] / 25.4 km below Saunders Power Dam	47.4
SLR-30	25.4 km below Saunders Power Dam / 35.5 km below Saunders Power Dam	47.3
SLR-31	35.5 km below Saunders Power Dam / Ontario-Québec Border[5]	47.2

Notes:

1. Conversions from GSC to IGLD 1955 are given in Table A3.1.5.
2. Saunders Tailwater assumed as kilometre 0.
3. Cornwall Canal (Below) gauge located at approximately 6.6 km below Saunders Power Dam.
4. Summerstown gauge located at approximately 20.3 km below Saunders Power Dam.
5. Ontario-Québec Border located at approximately 43.3 km below Saunders Power Dam.

Table A3.1.2 Peak Instantaneous Stillwater Level Frequencies (m IGLD 1955) (from MNR, 1989)

Sector	Recurrence Interval (yrs)							HYDSTAT Parameters (Log Pearson III)		
	2	5	10	25	50	100	200	Location	Scale	Shape
LAKE SUPERIOR										
S-1 Pine Point	183.42	183.58	183.67	183.77	183.85	183.91	183.97	5.52075	0.00022036	19.734
S-2 Thunder Bay*	183.43	183.56	183.62	183.69	183.74	183.77	183.81	5.2280	-0.000042826	377.020
S-3 Rossport*	183.46	183.62	183.71	183.81	183.97	183.94	183.99	5.2067	0.000178480	29.803
S-4 Michipicoten*	183.60	183.77	183.87	183.97	184.04	184.10	184.16	5.2004	0.000096459	128.880
S-5 Gros Cap*	183.61	183.76	183.84	183.92	183.98	184.03	184.08	5.1714	0.000023506	1760.400
S-6 Pointe Louise	-	-	-	-	-	184.20	-	-	-	-
ST. MARY'S RIVER										
SR-1*	183.74	183.93	184.03	184.15	184.23	184.30	184.37	5.2063	0.000187230	38.819
SR-2*	177.12	177.50	177.69	-	178.01	178.12	178.22	A=0.1051	B=10.43	M=3.370
SR-3	-	-	-	-	-	178.0	-	-	-	-
SR-4	-	-	-	-	-	177.9	-	-	-	-
SR-5	-	-	-	-	-	177.8	-	-	-	-
LAKE HURON										
H-1 Neebish	-	-	-	-	-	177.7	-	-	-	-
H-2 Richards	-	-	-	-	-	177.6	-	-	-	-
H-3 Hilton	-	-	-	-	-	177.5	-	-	-	-
H-4 St. Joseph	-	-	-	-	-	177.6	-	-	-	-
H-5 Thesssalon*	176.76	177.04	177.19	177.34	177.44	177.52	177.60	5.2071	-0.000117500	275.670
H-6 Mississagi Bay	176.84	177.13	177.28	177.43	177.53	177.62	177.69	5.2104	-0.00011001	319.63
H-7 Little Current*	176.89	177.20	177.35	177.52	177.63	177.73	177.82	5.2796	-0.000041126	2531.500
H-8 Cape Robert	176.82	177.11	177.25	177.41	177.50	177.59	177.67	5.2087	-0.00011402	294.67
H-9 N Cockburn Is	176.78	177.06	177.21	177.36	177.46	177.54	177.62	5.2076	-0.00011631	281.82
H-10, H-11 S Shore	177.63	176.91	177.05	177.20	177.30	177.38	177.46	5.2045	-0.00012259	248.81
H-12 North Georgian Bay	176.81	177.10	177.25	177.41	177.50	177.60	177.67	5.21	-0.00011341	305.24

H-13 Parry Sound*	176.91	177.22	177.37	177.54	177.64	177.74	177.82	5.2446	-0.000060426	1142.000
H-14 Collingwood*	176.99	177.29	177.45	177.61	177.71	177.80	177.89	5.2267	-0.000081248	623.440
H-15 Meaford	176.87	177.17	177.32	177.48	177.58	177.67	177.75	5.22	-0.000097832	422.14
H-16 Dyer's Bay	176.80	177.09	177.24	177.40	177.50	177.58	177.66	5.21	-0.00010662	345.41
H-17 Tobermory*	176.74	177.03	177.18	177.33	177.43	177.52	177.60	5.2107	-0.000107530	335.330
H-18 Southampton	176.66	176.95	177.09	177.25	177.34	177.43	177.51	5.2097	-0.000106960	331.800
H-19 Point Clark	-	-	-	-	-	177.50	-	-	-	-
H-20 Goderich*	176.83	177.12	177.27	177.42	177.52	177.61	177.69	5.2100	-0.000110790	314.080
H-21 Kettle Point	176.93	177.22	177.37	177.53	177.63	177.72	177.80	5.2161	-0.000098794	408.560
H-22 Brights Grove	177.01	177.32	177.48	177.65	177.75	177.84	177.94	5.2097	-0.000106960	331.800
ST CLAIR RIVER										
SCR-1	-	-	-	-	-	177.7	-	-	-	-
SCR-2	-	-	-	-	-	177.6	-	-	-	-
SCR-3	-	-	-	-	-	177.5	-	-	-	-
SCR-4	-	-	-	-	-	177.4	-	-	-	-
SCR-5	-	-	-	-	-	177.3	-	-	-	-
SCR-6	-	-	-	-	-	177.2	-	-	-	-
SCR-7	-	-	-	-	-	177.1	-	-	-	-
SCR-8	-	-	-	-	-	177.0	-	-	-	-
SCR-9	-	-	-	-	-	176.9	-	-	-	-
SCR-10	-	-	-	-	-	176.8	-	-	-	-
SCR-11	-	-	-	-	-	176.7	-	-	-	-
SCR-12	-	-	-	-	-	176.6	-	-	-	-
SCR-13	-	-	-	-	-	176.5	-	-	-	-
SCR-14 Port Lambton*	175.64	175.93	176.07	NA	176.29	176.37	176.43	A=-0.01785	B=13.78	M=7.165
LAKE ST. CLAIR										
SC-1 Walpole	175.61	175.90	176.05	176.21	176.31	176.40	176.49	5.2916	-0.000030566	4033.4
SC-2 Mitchell	175.74	176.04	176.20	176.38	176.48	176.58	176.67	34.708	-1.4289E-07	206730000
SC-3 Dover	175.65	175.95	176.10	176.26	176.37	176.46	176.55	5.2932	-0.000032085	3887

SC-4 Thames	175.71	176.01	176.17	176.34	176.45	176.55	176.64	3.9675	-3.5236E-06	34093
SC-5 Tremblay	175.41	175.68	175.83	175.98	176.07	176.16	176.24	5.2506	-0.000040971	2038.3
SC-6 Stoney Point	175.38	175.65	175.78	175.93	176.02	176.10	176.14	5.2083	-0.000079299	521.77
SC-7 Belle River*	175.38	175.66	175.80	175.96	176.05	176.14	176.22	5.2061	-0.000038259	2434.7
SC-8 Tecumseh*	175.33	175.59	175.71	175.85	175.93	176.01	176.08	5.193	-0.00011537	228.68
DETRIOT RIVER										
DR-1	-	-	-	-	-	176.0	-	-	-	-
DR-2	-	-	-	-	-	175.9	-	-	-	-
DR-3	-	-	-	-	-	175.8	-	-	-	-
DR-4	-	-	-	-	-	175.7	-	-	-	-
DR-5	-	-	-	-	-	175.6	-	-	-	-
LAKE ERIE										
E-1 Bar Point*	174.79	175.08	175.23	175.39	175.50	175.59	175.67	5.2298	-0.000061375	1079.100
E-2 Kingsville*	174.77	175.03	175.17	175.32	175.41	175.49	175.57	5.2020	-0.000087765	439.640
E-3 Pelee West	174.63	174.90	175.04	175.20	175.30	175.38	175.47	4.7283	7.8498E-06	55332
E-4 Wheatley	174.58	174.84	174.97	175.12	175.21	175.29	175.36	5.2157	-0.000059296	899.47
E-5 Port Crewe	174.49	174.87	174.87	175.01	175.09	175.17	175.24	5.1964	-0.000086291	400.05
E-6 Erieau*	174.47	174.71	174.83	174.96	175.05	175.12	175.19	5.1901	-0.000099727	284.66
E-7 Port Glasgow	174.49	174.75	174.89	175.03	175.12	175.20	175.28	5.2474	-0.000036678	2332.3
E-8 Port Stanley*	174.60	174.87	175.01	175.17	175.26	175.35	175.44	5.1625	0.000010770	29641
E-9 Port Bruce	174.71	174.98	175.11	175.26	175.35	175.43	175.50	5.1805	-0.00014416	142.36
E-10 Port Burwell	174.78	175.04	175.18	175.32	175.41	175.49	175.56	5.1876	-0.00014413	167.14
E-11 Hemlock	174.88	175.17	175.31	175.46	175.56	175.64	175.72	5.1873	-0.00017496	133.18
E-12 Clear Creek	174.97	175.27	175.42	175.58	175.68	175.77	175.85	5.1875	-0.00019372	118.56
E-13 Erie View	175.08	175.40	175.56	175.73	175.83	175.93	176.01	5.1904	-0.00051904	127.63
E-14 Long Point Park	175.14	175.49	175.68	175.88	176.00	176.12	176.23	5.0173	-0.000038103	3891.1
E-15 Long Point Central	175.23	175.60	175.80	176.02	176.16	176.29	176.40	5.1073	-0.00010708	548.88
E-16 Long Point East	175.29	175.68	175.89	176.11	176.26	176.40	176.53	5.124	-0.00015867	267.61
E-17 Long Point Bay	175.33	175.62	175.77	175.93	176.03	176.13	176.21	5.2673	-0.000038677	2601.5

E-18 Nanticoke	175.42	175.72	175.87	176.03	176.14	176.23	176.32	5.2832	-0.00003476	3338.9
E-19 Selkirk	175.46	175.76	175.92	176.08	176.19	176.29	176.38	5.3279	-0.000026185	6130.3
E-20 Port Maitland	175.57	175.88	176.05	176.21	176.32	176.42	176.50	5.2358	-0.000066124	1024.7
E-21 Mohawk Point	175.62	175.94	176.10	176.28	176.39	176.49	176.58	5.2412	-0.000063822	1141.9
E-22 Port Colborne*	175.51	175.88	176.09	176.32	176.47	176.61	176.74	5.1453	0.00027407	81.874
E-23 Point Abino	175.78	176.12	176.30	176.49	176.62	176.73	176.83	5.0396	0.000040564	3196.2
E-24 Crystal Beach	175.87	176.21	176.39	176.58	176.70	176.81	176.91	5.2696	-0.000054835	1821.2
E-25 Fort Erie	175.97	176.33	176.52	176.71	176.84	176.95	177.05	5.2841	-0.000051535	2208
LAKE ONTARIO										
O-1 Port Weller*	75.09	75.33	75.50	75.72	75.88	76.04	76.20	4.3147	0.0026394	1.7856
O-2 Burlington*	75.28	75.49	75.61	75.75	75.84	75.93	76.02	4.3111	0.00088175	11.79
O-3 Oakville	-	-	-	-	-	75.83	-	-	-	-
O-4 Mississauga	-	-	-	-	-	75.73	-	-	-	-
O-5 Toronto*	75.10	75.27	75.36	75.47	75.54	75.61	75.67	4.3039	0.00043745	34.351
O-6 Oshawa	75.05	75.22	75.31	75.42	75.49	75.55	75.61	4.304	0.00044283	32.256
O-7 Cobourg*	75.15	75.32	75.42	75.53	75.60	75.67	75.73	4.3035	0.00043239	37.111
O-8 Wellington	75.07	75.24	75.33	75.44	75.51	75.58	75.64	4.3042	0.00045958	31.209
O-9 Point Petre	75.04	75.21	75.30	75.41	75.48	75.54	75.61	1.43041	0.00045106	31.301
O-10 Prince Edward	75.09	75.27	75.36	75.48	75.55	75.62	75.69	1.43042	0.00048204	30.474
O-11 Kingston*	75.25	75.44	75.54	75.66	75.73	75.81	75.87	4.3041	0.00046649	36.177

NOTES:

1. The frequencies of peak instantaneous LAKE levels are calculated by combining the individual frequency distributions of monthly mean lake levels and surge values (wind setup) using the MNR HYSTAT computer program.

2. The frequencies of peak instantaneous RIVER levels aare calculated with recorded peak instantaneous levels using the computer program CFA88; this program does not list a 25 year value.

3. Levels for sectors marked with and * are based on recorded surges; surges at other sections are calculated using the computer model SURGE from AES.

4. Conversions from I.G.L.D. to G.S.C. are given in Appendix 1.

Table A3.1.2
Peak Instantaneous Stillwater Level Frequencies (m IGLD 1955) cont'd
(from Environment Canada, 1993)

Station	Recurrence Interval (yrs)						
	2	5	10	20	50	100	200
Kingston	75.25	75.44	75.54	--	75.73	75.81	75.87
Ogdensburg	74.94	75.10	75.20	75.28	75.39	75.46	75.53
Iroquois Is. (Above)	74.39	74.56	74.68	74.80	74.95	75.07	75.19
Iroquois Is. (Below)	74.27	74.38	74.45	74.51	74.58	74.63	74.69
Morrisburg	74.14	74.25	74.29	74.31	74.33	74.34	74.35
Long Sault	74.00	74.17	74.26	74.33	74.41	74.46	74.51
Saunders HW	74.02	74.12	74.17	74.21	74.25	74.27	74.29
Saunders TW	48.46	48.69	48.81	48.89	48.99	49.04	49.09
Cornwall Canal (Below)	47.41	47.55	47.62	47.69	47.77	47.82	47.87
Summerstown	46.94	47.03	47.07	47.10	47.13	47.15	47.17
Lake St. Francis							
Summerstown End	46.97	47.07	47.13	--	47.25	47.30	47.34
Côteau-Landing End	46.69	46.78	46.83	--	46.93	46.97	47.01

Notes:

1. The frequencies of peak instantaneous LAKE levels are calculated by combining the individual frequency distributions of monthly mean lake levels and surge values (i.e., wind setup) using the OMNR HYDSTAT computer program; this program does not list a 20 year value.

2. The frequencies of peak instantaneous RIVER levels are calculated with recorded peak instantaneous levels using the Environment Canada CFA88 computer program.

3. The flood frequency regime estimated using the Log Pearson Type III distribution.

Table A3.1.3
Highest Annual Monthly Mean Lake Level Frequencies (m IGLD 1955)
(from MNR 1989)

Lake	Water Level (m IGLD 1955) Return Period (years)							HYDSTAT Parameters (Log Pearson Type III)		
	2	5	10	25	50	100	200	Location	Scale	Shape
Superior	183.17	183.29	183.35	183.41	183.45	183.48	183.51	5.2183	-0.0000615	96.86
Huron	176.47	176.76	176.91	177.06	177.16	177.24	177.32	5.1976	-0.0001639	149.42
St. Clair	175.05	175.31	175.44	175.57	175.65	175.73	175.8	5.1847	-0.0001628	120.71
Erie	174.18	174.42	174.55	174.67	174.76	174.83	174.89	5.1805	-0.0001442	142.36
Ontario	74.92	75.1	75.2	75.31	75.39	75.46	75.53	4.3057	0.0006083	18.03

Notes:

1. "Basis of Comparison" water levels for the period 1900 to 1987 were used in the analysis.
2. For all lakes, the "best fitting" distribution was the Log Pearson Type III according to the least squares criterion in HYDSTAT.

Table A3.1.4 Wind Setup/Surge Frequencies (m)
(from MNR 1989)

Sector	Wind Setup (m)							HYDSTAT Parameters			
	2	5	10	25	50	100	200	Type	Location	Scale	Shape
LAKE SUPERIOR											
S-1 Pine Point	0.24	0.33	0.40	0.52	0.62	0.75	0.91	LG	-1.529958	0.271399	-
S-2 Thunder Bay*	0.26	0.31	0.35	0.38	0.41	0.43	0.45	LP III	4.342	-0.008635	658.88
S-3 Rossport*	0.28	0.37	0.45	0.56	0.66	0.76	0.88	3 LN	-2.19325	0.72085	-
S-4 Michipicoten*	0.42	0.56	0.64	0.74	0.80	0.86	0.93	P III	0.061455	0.056367	6.7672
S-5 Gros Cap*	0.43	0.53	0.60	0.67	0.71	0.76	0.80	P III	0.061235	0.03567	10.598
S-6 Pointe Louise	-	-	-	-	-	0.96	-	-	-	-	-
LAKE HURON											
H-1 Neebish	-	-	-	-	-	0.48	-	-	-	-	-
H-2 Richards	-	-	-	-	-	0.48	-	-	-	-	-
H-3 Hilton	-	-	-	-	-	0.48	-	-	-	-	-
H-4 St. Joseph	-	-	-	-	-	0.48	-	-	-	-	-
H-5 Thesssalon*	0.28	0.33	0.37	0.41	0.45	0.48	0.51	LP III	-2.34430	-0.03746	28.971
H-6 Mississagi Bay	0.35	0.42	0.47	0.53	0.58	0.63	0.68	LP III	-1.51300	0.07458	6.656
H-7 Little Current*	0.40	0.51	0.59	0.69	0.78	0.87	0.96	LP III	-1.96870	-0.74045	14.373
H-8 Cape Robert	0.34	0.40	0.44	0.49	0.54	0.58	0.63	LP III	-1.51580	-0.72405	6.191
H-9 N Cockburn Is	0.29	0.35	0.39	0.44	0.48	0.53	0.58	LP III	-1.58060	0.09059	4.201
H-10, H-11 S Shore	0.15	0.18	0.20	0.22	0.24	0.25	0.27	LP III	-4.77250	0.01553	185.110
H-12 North Georgian Bay	0.34	0.41	0.46	0.50	0.53	0.56	0.59	LP III	0.15276	-0.05508	22.914
H-13 Parry Sound*	0.42	0.53	0.61	0.72	0.82	0.92	1.03	LP III	-1.35750	0.11729	4.498
H-14 Collingwood*	0.50	0.61	0.68	0.78	0.85	0.93	1.01	LP III	-1.35360	0.06756	10.116
H-15 Meaford	0.39	0.49	0.55	0.62	0.68	0.73	0.78	LP III	-5.08660	-0.01602	259.070
H-16 Dyer's Bay	0.32	0.39	0.45	0.51	0.56	0.61	0.67	LP III	-2.49740	0.04603	29.558
H-17 Tobermory*	0.25	0.32	0.36	0.43	0.48	0.54	0.60	LP III	-1.82390	0.11297	4.346
H-18 Southampton	0.19	0.23	0.25	0.27	0.28	0.30	0.31	LP III	-0.71061	-0.06452	15.412
H-19 Point Clark	-	-	-	-	-	0.49	-	-	-	-	
H-20 Goderich*	0.36	0.43	0.48	0.55	0.61	0.67	0.74	LP III	-1.45400	0.09080	5.005
H-21 Kettle Point	0.44	0.53	0.59	0.66	0.72	0.78	0.84	LP III	-1.60450	0.05311	15.038
H-22 Brights Grove	0.52	0.64	0.72	0.83	0.91	1.00	1.10	LP III	-1.39380	0.07053	10.736
LAKE ST. CLAIR											
SC-1 Walpole	0.55	0.70	0.79	0.91	0.99	1.07	1.14	LN	-0.59216	0.28193	-
SC-2 Mitchell	0.68	0.86	0.97	1.10	1.18	1.27	1.34	LP III	1.973	-0.0372	63.797
SC-3 Dover	0.59	0.76	0.86	0.97	1.04	1.11	1.17	LP III	1.1464	-0.0059	28.666
SC-4 Thames	0.65	0.83	0.94	1.07	1.16	1.24	1.32	P III	0.026512	0.06853	9.7852
SC-5 Tremblay	0.34	0.45	0.53	0.64	0.74	0.85	0.96	LP III	-1.18036	0.11994	6.3629
SC-6 Stoney Point	0.32	0.41	0.47	0.56	0.63	0.71	0.79	LP III	-1.22	0.076521	14.066
SC-7 Belle River*	0.31	0.44	0.53	0.64	0.72	0.81	0.90	LN	-1.1648	0.40826	-
SC-8 Tecumseh*	0.28	0.33	0.36	0.40	0.42	0.45	0.47	LN	-1.27839	0.20194	-

LAKE ERIE											
E-1 Bar Point*	0.61	0.80	0.89	0.98	1.03	1.07	1.11	LP III	0.23225	-0.18941	4.1496
E-2 Kingsville*	0.60	0.72	0.79	0.85	0.90	0.94	0.97	P III	12.7366	-0.001771	6855.88
E-3 Pelee West	0.43	0.56	0.66	0.78	0.88	0.99	1.10	LP III	-2.0266	0.073952	16.374
E-4 Wheatley	0.51	0.65	0.73	0.84	0.91	0.98	1.05	3 LN	-0.68838	0.28436	-
E-5 Port Crewe	0.31	0.39	0.44	0.51	0.55	0.60	0.64	3 LN	-1.23364	0.29466	-
E-6 Erieau*	0.28	0.34	0.37	0.42	0.46	0.49	0.53	LP III	-1.8176	0.06283	9.1582
E-7 Port Glasgow	0.30	0.40	0.47	0.58	0.67	0.77	0.87	3 LN	-1.9131	0.61814	-
E-8 Port Stanley*	0.40	0.53	0.63	0.75	0.85	0.96	1.07	LP III	-2.7365	0.059921	30.625
E-9 Port Bruce	0.56	0.67	0.72	0.77	0.79	0.81	0.83	LP III	0.97221	-0.053782	8.057
E-10 Port Burwell	0.63	0.74	0.78	0.81	0.81	0.82	0.83	LP III	-0.19712	-0.2959	1.121129
E-11 Hemlock	0.72	0.88	0.94	0.98	1.00	1.01	1.02	LP III	0.02158	-0.29084	1.14995
E-12 Clear Creek	0.81	0.99	1.07	1.15	1.19	1.23	1.26	P III	1.15583	-0.072874	10.558
E-13 Erie View	0.92	1.13	1.23	1.32	1.38	1.43	1.47	P III	2.0139	-0.065833	16.945
E-14 Long Point Park	0.96	1.23	1.37	1.54	1.64	1.74	1.84	3 LN	1.31776	0.0818	-
E-15 Long Point Central	1.05	1.34	1.51	1.70	1.82	1.94	2.05	P III	-0.91089	0.05821	33.954
E-16 Long Point East	1.11	1.42	1.60	1.81	1.94	2.07	2.19	P III	-0.63955	0.073059	24.221
E-17 Long Point Bay	1.15	1.32	1.42	1.52	1.59	1.66	1.72	P III	-0.027084	0.026833	53.406
E-18 Nanticoke	1.24	1.42	1.52	1.63	1.71	1.77	1.84	P III	-0.45576	0.043159	67.591
E-19 Selkirk	1.28	1.47	1.58	1.69	1.77	1.84	1.91	P III	-0.5325	0.026695	68.229
E-20 Port Maitland	1.40	1.60	1.71	1.82	1.90	1.96	2.02	N	1.4027	0.24019	-
E-21 Mohawk Point	1.45	1.66	1.77	1.89	1.97	2.04	2.10	N	1.45111	0.25151	-
E-22 Port Colborne*	1.32	1.61	1.80	2.01	2.17	2.32	2.46	LN	0.27407	0.24342	-
E-23 Point Abino	1.60	1.85	1.99	2.14	2.25	2.34	2.43	LP III	1.9755	-0.02167	69.884
E-24 Crystal Beach	1.70	1.95	2.08	2.22	2.31	2.39	2.47	N	1.6961	0.30026	-
E-25 Fort Erie	1.80	2.07	2.21	2.36	2.46	2.55	2.63	N	1.8028	0.3206	-
LAKE ONTARIO											
O-1 Port Weller*	0.16	0.27	0.39	0.59	0.79	1.06	1.41	LG	-2.05023	0.478792	-
O-2 Burlington*	0.33	0.44	0.53	0.67	0.79	0.94	1.12	LG	-1.19962	0.248271	-
O-3 Oakville	-	-	-	-	-	0.81	-	-	-	-	-
O-4 Mississauga	-	-	-	-	-	0.72	-	-	-	-	-
O-5 Toronto*	0.16	0.21	0.24	0.28	0.31	0.34	0.37	3 LN	-2.07162	0.38076	-
O-6 Oshawa	0.12	0.15	0.17	0.20	0.21	0.23	0.25	LN	0.125	0.03411	-
O-7 Cobourg*	0.21	0.27	0.31	0.36	0.40	0.44	0.47	LN	0.22	0.06826	-
O-8 Wellington	0.13	0.17	0.21	0.27	0.32	0.39	0.47	LP III	-1.24069	0.24073	1.7135
O-9 Point Petre	0.10	0.13	0.16	0.20	0.23	0.27	0.32	LG	-2.347132	0.227774	-
O-10 Prince Edward	0.14	0.21	0.26	0.34	0.40	0.47	0.55	3 LN	-2.45682	0.67569	-
O-11 Kingston*	0.31	0.40	0.46	0.54	0.60	0.66	0.72	LP III	-4.9361	0.025067	150.16

NOTES:

* based on recorded surgers: surges at other sectors are calculated using the computer model SURGE from AES

Where frequency relationships have not been calculated, the 100 year wind setup has been interpolated from adjacent sites.

Table A3.1.5 Differences Between Geodetic Datum and International Great Lakes Datum (IGLD 1955 and IGLD 1985) for Holding Bench Marks at Permanent Gauging Stations Operated by the Canadian Hydrographic Service (CHS)

The following differences between elevations referenced to Geodetic Datum and elevations referenced to International Great Lakes Datum (IGLD) 1955 and IGLD 1985 are from CHS (Sept. 1992).

GAUGING STATION	BENCH MARK	IGLD 1985 - IGLD 1955 (m)	IGLD 1985 - GSC (m)	GSC - IGLD 1955 (m)
Thunder Bay	346-E	0.38	0.31	0.07
Rossport	70-U-652	0.46	0.41	0.06
Michipicoten	698 GSC	0.44	0.40	0.04
Gros Gap	GROS 3-1963	0.38	0.28	0.10
Sault St. Marie	MIDDLE SOO	0.27	0.16	0.11
Thessalon	THES 2-1959	0.26	0.07	0.19
Little Current	LICU 9-1965	0.34	0.13	0.21
Parry Sound	420-A-3	0.33	0.12	0.22
Collingwood	DCLXVIII	0.24	0.04	0.20
Tobermory	101-R2	0.28	0.14	0.14
Goderich	72-U-108	0.19	0.00	0.19
Point Edward	PTED 1-1959	0.18	0.04	0.14
Port Lambton	POLA 1-1959	0.20	0.03	0.17
Belle River	BELL 1-1961	0.22	0.01	0.21
Tecumseh	TECU 2-1959	0.18	-0.01	0.19
La Salle	MMMDXLVIII	0.19	0.00	0.19
Amherstburg	71-U-117	0.20	0.00	0.20
Bar Point	3016	0.18	-0.02	0.21
Kingsville	3031	0.18	-0.02	0.20
Erieau	HS 1-1957	0.18	-0.01	0.18
Port Stanley	JW 1975	0.19	0.03	0.17
Port Dover	MMDCCXXX	0.17	0.02	0.14
Port Colborne	71-U-032	0.19	0.04	0.15

GAUGING STATION	BENCH MARK	IGLD 1985 - IGLD 1955 (m)	IGLD 1985 - GSC (m)	GSC - IGLD 1955 (m)
Port Weller	HS 3	0.13	0.03	0.10
Burlington	60-U-3327	0.08	0.00	0.08
Toronto	579-F	0.13	0.05	0.08
Cobourg	67-U-057	0.13	0.06	0.07
Kingston	75-U-502	0.18	0.04	0.14
Brockville	68-U-339	0.15	0.02	0.13
Iroquois Upper	HS2	0.12	0.00	0.12
Iroquois Lower	HS L2	0.12	0.00	0.12
Cornwall	2-1958	0.10	0.00	0.11
Summerstown	2616	0.09	0.00	0.10
Côteau-Landing				0.08

Notes:

GSC = Geodetic Survey Canada (also known as Canadian Geodetic Datum (CGD))
IGLD 1955 = International Great Lakes Datum 1955
IGLD 1985 = International Great Lakes Datum 1985

A3.1.3 Method of Calculation

a) Great Lakes and Upper Connecting Channels

The following description of the flood level calculations is taken from MNR (1989) and from *"Calculation of Flood Levels Along the Ontario Shoreline of the Great Lakes"* (Fay and Moulton 1990).

The steps taken to calculate the flood levels are discussed in detail in the following sections and are summarized as follows:

- **Step 1** A frequency distribution of highest annual monthly mean lake levels was derived for each lake based on recorded water level data adjusted to 1988 "Basis of Comparison" conditions (see Section A3.1.4).

- **Step 2** Wind setup or storm surge values were obtained from recorded surges at gauging stations and by modelled surges between gauges using the SURGE model obtained from Environment Canada (see Appendix A3.4).

  At gauging stations, the highest annual wind setups were calculated by subtracting static levels from recorded peak instantaneous levels. A frequency distribution of highest annual recorded wind setups was then obtained using the HYDSTAT model (see Appendix A3.5).

  For shoreline areas between gauges, wind setup values were modelled using SURGE and average wind speed data from Atmospheric Environment Service of Environment Canada. The model was calibrated by comparing the modelled wind setup values with the recorded values at nearby gauge sites.

- **Step 3** A combined probability analysis was then completed of the highest annual monthly mean water levels and the best-fitting frequency distribution of wind setup values at each grid point to obtain 100 year peak instantaneous stillwater levels. Those grid points with 100 year levels within 0.1 m and common physiographic features and shore alignments were then grouped together into a common sector. The resulting 100 year flood level is assumed to be applicable throughout the sector.

- **Step 4** For the connecting channels of the *Great Lakes - St. Lawrence River System*, 100 year peak instantaneous levels were calculated using recorded data adjusted to "Basis of Comparison" conditions. Sectors are based on changes of 0.1 m using changes in slope based on known flow profiles. Environment Canada's Consolidated Frequency Analysis computer program was used to calculate the stage frequency relationships at Canadian gauge locations. The results obtained for the U.S. Federal Emergency Management Agency (FEMA) by the U.S. Army Corps of Engineers (USACE 1988) for the U.S. gauges in the connecting channels were also taken into consideration in determining the 100 year flood levels.

Further details regarding the water level data set, flood level calculations, serial correlation and climate change are provided in the following sections.

i) Water Level Data Set

Records of water levels for the Great Lakes exist from the early 1800's. These early records mainly consisted of occasional readings of the levels at a limited number of gauge locations along the U.S. shoreline. In the later part of the 19th century, as the lakes grew in importance for shipping, there was an increase in the frequency and number of locations at which water level measurements were recorded. With the turn of the century, the frequency and accuracy of gauge recordings increased considerably and by the early 1900's continuous recordings of water levels were initiated at several Canadian locations.

By the late 1950's or early 1960's many of the Canadian gauges on the Great Lakes had been established and by 1987 a total of 22 water level gauges were operational along the Canadian shores of the Great Lakes, excluding the connecting channels.

As a result of the different time periods in which water level gauges were established, record lengths at individual gauge locations vary considerably across the Great Lakes. The Port Colborne gauge, for example, has continuous records dating from 1912, while the Erieau gauge has records from only 1959.

Over the time period during which water level records have been kept, a number of physical and operational changes have occurred within the Great Lakes system. These changes include:

- implementation of the Long Lac and Ogoki diversions into Lake Superior in the 1940's;
- updates to the operating procedures used at the control works at the outlet of Lake Superior;
- changes in the diversion out of Lake Michigan at Chicago;
- dredging of the St. Clair and Detroit Rivers on several occasions;
- variations in the diversion through the Welland Canal; and
- introduction of control works in the St. Lawrence River at Cornwall.

The water level data recorded since 1900 have been adjusted to account for these changes. The adjustments modify the recorded levels to the levels that would have occurred if the present regime had been in place throughout the century. The adjusted data are referred to as Basis of Comparison, (BOC) levels (see Section A3.1.4).

The water level data for each of the lakes for the period 1900–1987 were obtained from the Great Lakes–St. Lawrence Study Office of Environment Canada, which maintains a coordinated Great Lakes hydrologic database in cooperation with the U.S. Army Corps of Engineers.

Water level information also exists for several gauge locations along the connecting channels. There are six recording gauges along the Canadian side of the upper connecting channels, the St. Mary's, St. Clair and Detroit Rivers. Four of these gauges have periods of record exceeding 60 years. There are also several gauges located along the U.S. sides of the connecting channels.

The water levels within the connecting channels have also been affected by changes within the Great Lakes system. The gauge records for the connecting channels were adjusted to account for these changes by using adjustment factors prepared for the lakes by the U.S. Army Corps of Engineers (USACE 1988). For the record of each connecting channel gauge, the adjustment factors for the closest lake were applied.

ii) Combined Frequency Analysis Methodology

The most common approach used to obtain the frequency distribution of peak instantaneous stillwater levels is to extract and analyze the maximum instantaneous levels for each year from the gauge records. This method was used in the recent U.S. analysis (USACE 1988), however, it was deemed inappropriate for the Canada/Ontario analysis due to the generally shorter period of record at Canadian gauges.

The alternative method selected to analyze the frequencies of peak stillwater lake levels makes use of combined probability concepts. Using this method, if two variables are independent, then the combined probability distribution of the sum of the two variables is calculated by integration, or convolution, of their individual probability distributions. Peak stillwater level, as defined under this method, is the sum of static lake level and wind setup. Since static levels of the Great Lakes can be approximated by their monthly average levels, the annual exceedance probabilities of peak stillwater levels can then be determined from the combined probability of annual maximum monthly mean levels and annual maximum wind setups.

By using the combined probability approach, it was possible to:

- use 88 year series of annual maximum monthly average levels (1900–1987) which include a larger range of static levels than the typical (±) 30 year series for which peak instantaneous values are available; and

- separately analyze the frequencies of modelled annual maximum wind setup levels and frequencies of annual maximum monthly average lake levels to produce combined frequencies of peak stillwater levels in areas distant from gauge sites.

· Static Lake Level Frequencies

Monthly mean water level data for the period 1900–1987 were used to compute static water level frequencies for lakes Superior, Huron, St. Clair, Erie and Ontario. The monthly data were an average of Basis of Comparison level records from several gauges on each lake.

For the purposes of this analysis, it is assumed that the present Great Lakes hydraulic regime and climate will not change in the future (see Section A3.1.3(a)(iv)). While it is likely that changes will occur some time in the future, it is impossible to accurately predict what those changes will be. If changes to the hydraulic regime of the Lakes are implemented in the near future, or if the climate of the region is significantly altered, then the present frequency analysis should be reviewed and possibly repeated.

For each lake, the highest of the twelve monthly mean levels for each year were selected to form series of annual maximum monthly mean levels. The parameters for several common frequency distributions used in hydrology were calculated for each of the 88 point series through the use of the computer program HYDSTAT (MNR 1982). For an outline of the HYDSTAT computer program, Appendix A3.5 should be consulted.

The 88 point series used across the five lakes was consistently "best fit" by the log Pearson Type III (LPIII) distribution according to the least square error criterion of HYDSTAT and therefore was selected for use. The LPIII distribution has also been selected by U.S. agencies in the past to model frequencies of Great Lakes annual maximum monthly mean levels. A comparison between application of LPIII and Pearson III distributions to the lake levels data indicated that the differences between estimates in 100 year return period levels of the two distributions were less than 0.01 metres.

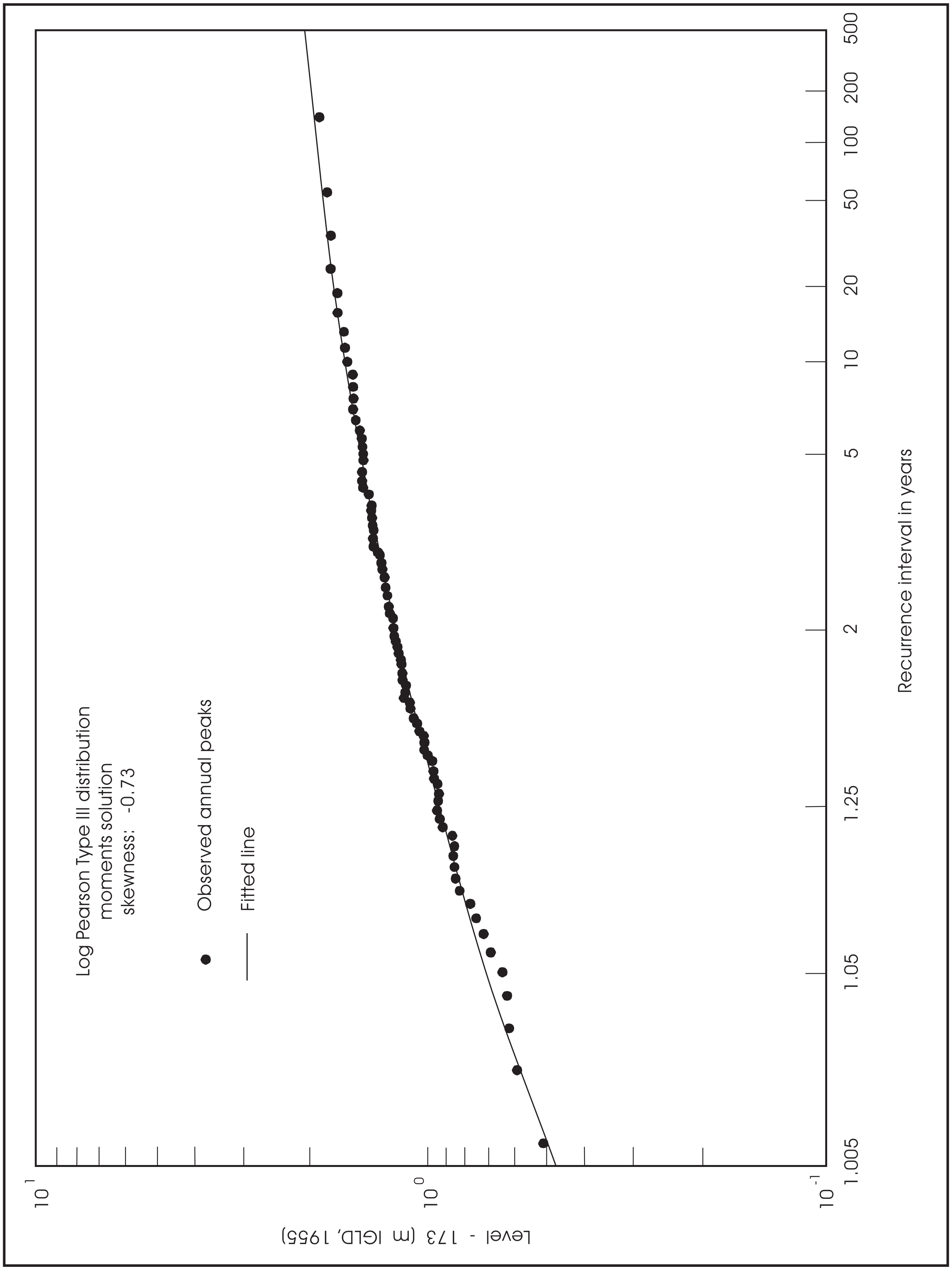
It should be noted that significant serial correlation is evident in each of the annual maximum monthly mean water levels series. This correlation between successive years of maximum levels violates the assumption of serial independence for frequency analysis and makes the exceedance frequencies calculated in the analyses not strictly correct. Nonetheless, the exceedance frequencies calculated for two separate sample series with reduced serial correlation, one series made up of even years and the other made up of odd years in the annual series, did not change significantly from that of the annual series (see Section A3.1.3 (a)(iii)). Canadian and U.S. agencies continue to use the frequency analyses of annual maximum lake levels since it appears that serial correlation does not significantly alter the results in this case.

The fitted LPIII frequency distribution and the adjusted observations of annual maximum monthly mean for Lake Erie have been plotted and are shown in Figure A3.1.1. For a better visual display of the data on the logarithmic plot, a constant of 173.0 metres was subtracted from all data points and the fitted distribution prior to plotting. The plots of the fitted distributions for the other four lakes had a similar appearance.

· Frequency Analysis at Gauge Locations

Water level records for many of the Canadian water level gauges extend over about 30 years, with a few gauges having records of up to 80 years. From these instantaneous stillwater level records, the maximum surge recorded at the gauge for each year was extracted for a minimum period of 30 years or the period of record, whichever is less.

Figure A3.1.1: Lake Erie Annual Maximum Monthly Mean Level



Wind setups were calculated by taking the difference between the instantaneous level at the time of the peak wind setup and the average level at the gauge during the preceding few days. For the purposes of the determination of the flood standards, a record period of 30 years was considered to be of a sufficient length to estimate the statistical distribution of surges for two reasons:

1) in virtually all cases these wind setups were the result of storms passing over the lake; and

2) these storms were random, independent events.

The program HYDSTAT was used to estimate the parameters for the frequency distributions for the annual maximum recorded surge series at each of the 22 Canadian gauges located on the Great Lakes. For each series, the "best fit" distribution was selected according to the least square error criterion of HYDSTAT. For the purpose of graphically describing the results of this process, Figure A3.1.2 displays the log-normal distribution which best fit the 1957–1986 observations of annual maximum wind setup for the Port Colborne gauge on Lake Erie. The 100 year return period surge for this gauge was calculated to be 2.32 metres (7.6 ft.).

The multivariate frequency analysis feature of the program HYDSTAT was used to estimate a combined probability distribution from the fitted distributions for annual maximum monthly mean lake levels and the wind setups for each gauge location. The HYDSTAT program uses discreditization and numerical integration techniques to complete the convolution of the individual cumulative probability distributions of each of the two components. The HYDSTAT program fits several common univariate probability distributions to the mathematically complicated combined probability distribution. In all cases the LPIII distribution fit the calculated combined probability distribution the best. These LPIII distributions of the combined probabilities represents the frequency of peak stillwater levels at each gauge location.

- **Frequency Analysis Between Gauge Locations**

To estimate frequencies of peak stillwater levels along the shoreline of the lakes between gauge locations, a combined probability analysis of annual maximum monthly mean levels and modelled annual maximum wind setup levels was completed. The numerical model SURGE was used to simulate wind setup elevations from available hourly wind velocities.

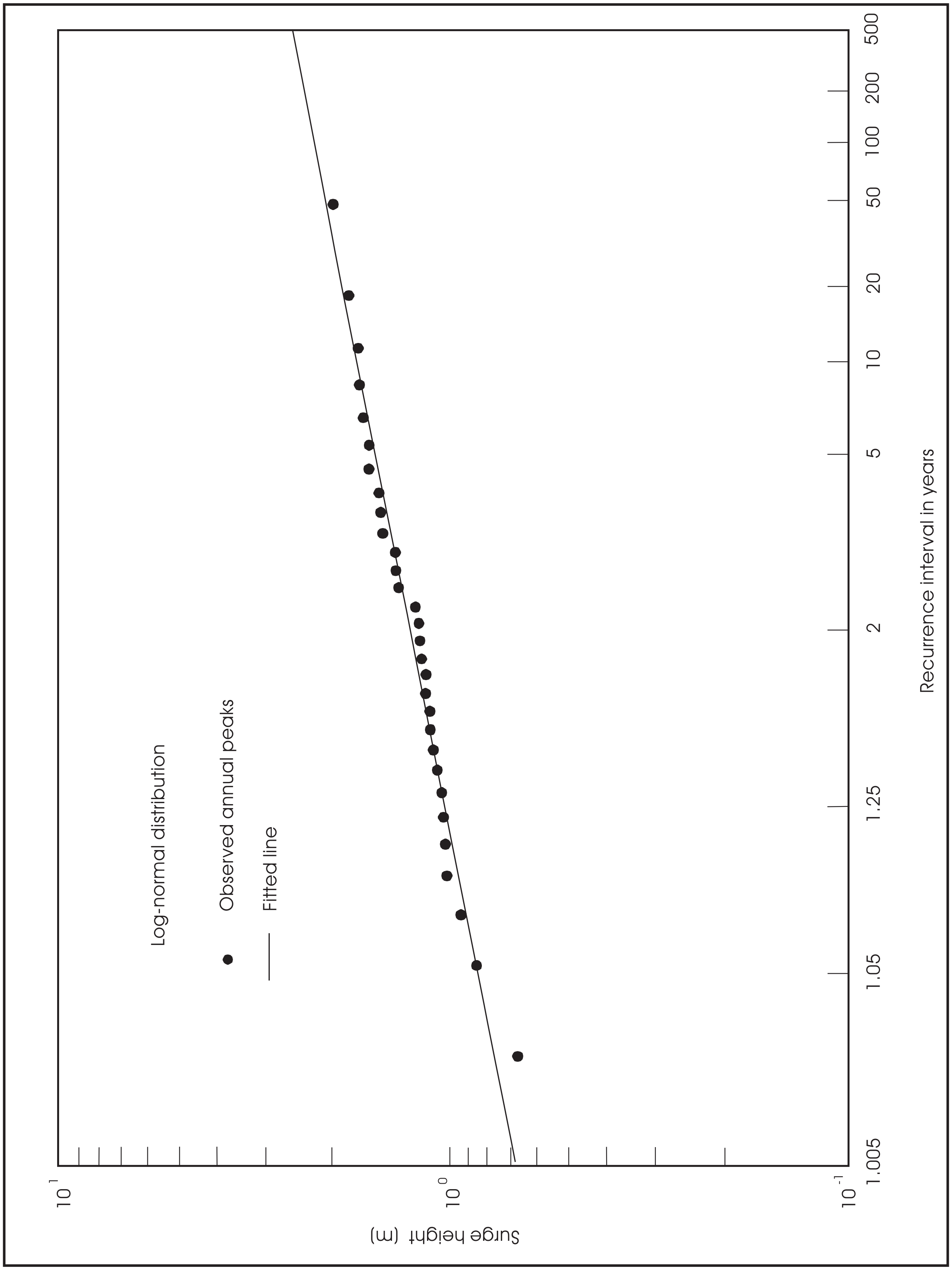
The SURGE numerical model was originally developed by Environment Canada staff at the National Water Research Institute in Burlington and was subsequently modified and calibrated for Great Lakes conditions by the U.S. Great Lakes Environmental Research Laboratory of NOAA. and by the Atmospheric Environment Service of Environment Canada. Further details on SURGE numerical model are provided in Appendix A3.4.

The SURGE model describes the lake as a single layer of grid squares of constant size for the entire lake. Each grid square has a corresponding depth taken from bathymetric chart information. The SURGE program calculates the average water level surface position in each grid square in each hour of the user defined analysis period. The water surface position is calculated using simplified basic hydrodynamic equations (Welander 1961) which include wind stress and bottom friction terms. The wind stress relation in the model assumes that the wind stress is equal to the square of the wind velocity times a drag coefficient. The program output is in the form of a digital map showing the water surface elevation for each grid on the lake on an hour by hour basis for the period defined by the user.

The following steps describe the process in determining the frequency distributions for ungauged portions of the lake:

- **Step 1** Wind speed data for use in the SURGE model were obtained from the Atmospheric Environment Service of Environment Canada. The data included maximum monthly wind speeds of 1, 2, 3, 6 and 12 hour duration for 8 wind directions (i.e., N, NE, E, SE, S, SW, W, NW) at selected stations in the Great Lakes Basin. The station selection was based on the period of record, completeness of the record within each year, and the proximity to the shore. The annual highest wind speed for each duration and direction was extracted from the data for further use.

Figure A3.1.2: Port Colborne Annual Maximum Wind Setup (1957-1986)



- **Step 2** An iterative process was then used to calibrate the SURGE model for the sector being analyzed to the recorded surges at the adjacent gauge location.

  First, the annual maximum wind series for the period of record which produced the highest surge at the gauge site of interest was determined. This series was made up of winds of various durations and directions.

  Once this governing annual maximum measured wind velocity series was determined, it was multiplied by a selected constant and input to the model to produce a series of modelled annual wind setup values at the gauge site. Various constants were selected until the maximum annual wind setup population produced by the model at a gauge site did not differ significantly from the recorded highest annual surge population at the site. This was then tested by comparing the fitted distribution of the recorded wind setup data at the gauge to the fitted distribution of the modelled wind setups using the program HYDSTAT. Differences between 100 year modelled wind setups and 100 year recorded wind setups at a gauge site were generally less than 0.1 m.

  The final selected wind speed calibration constants were assumed to apply to adjacent sectors. Wind setups were modelled for the highest annual 1, 2, 3, 6 and 12 hour duration winds for each applicable wind direction for the period of record of the wind data.

  From this data the highest annual wind setup population for the sector (i.e., SURGE model grid location) was selected. This represents, for a particular year, the highest wind setup that would occur from a 1, 2, 3, 6 or 12 hour duration wind speed from any wind direction relevant to a particular sector of shoreline.

- **Step 3** A combined probability analysis was then completed of the annual maximum monthly mean water levels and the modelled wind setup values at each grid square. The best fitting frequency distribution for the combined analysis at each grid square was then selected to represent the stillwater level frequency relation at that grid square. Those adjacent grid squares with 100 year levels within 0.1 metres and common physiographic features and shore alignments were then grouped together into a common sector. The 100 year stillwater flood level is assumed to be applicable throughout the sector.

- **Frequency Analysis for Connecting Channels**

Water levels in the connecting channels vary in a different manner than the levels of the Great Lakes as a direct result of the different hydraulic characteristics of the connecting channels. In the upper connecting channels (i.e., St. Mary's, St. Clair and Detroit Rivers), the levels are dependent on the upstream and downstream lake levels. Although a wind setup on one or both of the lakes at the ends of the connecting channel can influence the level within the channel, the impact of the wind setup is usually dampened as it moves up or down the channel. Wind setups across the channels may also occur, however, these are generally small due to the narrowness of the channels.

For each lake a record of historical static water levels exists, formed from the average of levels recorded at a number of gauges around each lake. Due to the slope of the water surface in a channel, a similar static water level data set for a channel cannot be representative. Without static level series for the channels, applying the combined probability method used to calculate peak stillwater level frequencies for the lakes was considered to be inappropriate.

As an alternative, since four of the water level gauges on these channels have periods of record exceeding 60 years, it was decided that univariate frequency analysis of peak instantaneous levels recorded at gauge sites on the upper connecting channels would be conducted. The maximum instantaneous water level for each year was selected from each connecting gauge record adjusted for Basis of Comparison conditions. The Environment Canada computer program CFA1 was used to analyze the frequencies of these annual peak level series. The LPIII distribution was selected for the analysis in each case. To assist in describing this process, a plot of the fitted distribution for the record at the gauge at Port Lambton on the St. Clair River is shown in Figure A3.1.3.

The estimation of 100 year flood levels at ungauged locations along the upper connecting channels utilized a number of sources of information. Values had been calculated for the lakes at the upstream and downstream confluence of each channel, as described above. In addition to the calculated 100 year level(s) at the Canadian gauge location(s) along the channel, levels calculated for the U.S. gauge locations were consulted (USACE 1988). Water surface profiles that have been determined for the connecting channels by the U.S. Army Corps of Engineers (USACE 1988) were also used. Based on all of these sources of information, interpolated 100 year levels were determined for the ungauged portions of the upper connecting channels.

Calculation of the St. Lawrence River flood levels is described later in this section (Section A3.1.3(b)).

iii) Serial Correlation and Independence of Events

An underlying assumption of computing frequencies of extreme annual events is that the recorded extreme in each year is an independent event. For the Great Lakes, annual extreme events are not truly independent because the large storage capacity of the lakes in relation to the outflow capacities of the channels results in persistence in the system. This persistence is evident in the serial correlation or auto-correlation that exists between successive highest annual levels.

The U.S. Army Corps of Engineers found that, of all of the Great Lakes, the series of annual maximum monthly mean levels for Lake Michigan-Huron had the highest degree of auto-correlation. To test the significance of this auto-correlation, two separate sample population series were constructed based on even year and odd year data to eliminate the yearly dependence present. The study concluded that the overall impact of reducing the dependence and re-introducing it in the form of a complete annual series did not appear to significantly alter the frequency relationships.

U.S. and Canadian agencies currently analyze Great Lakes data assuming highest annual events are independent and that serial correlation does not significantly alter results.

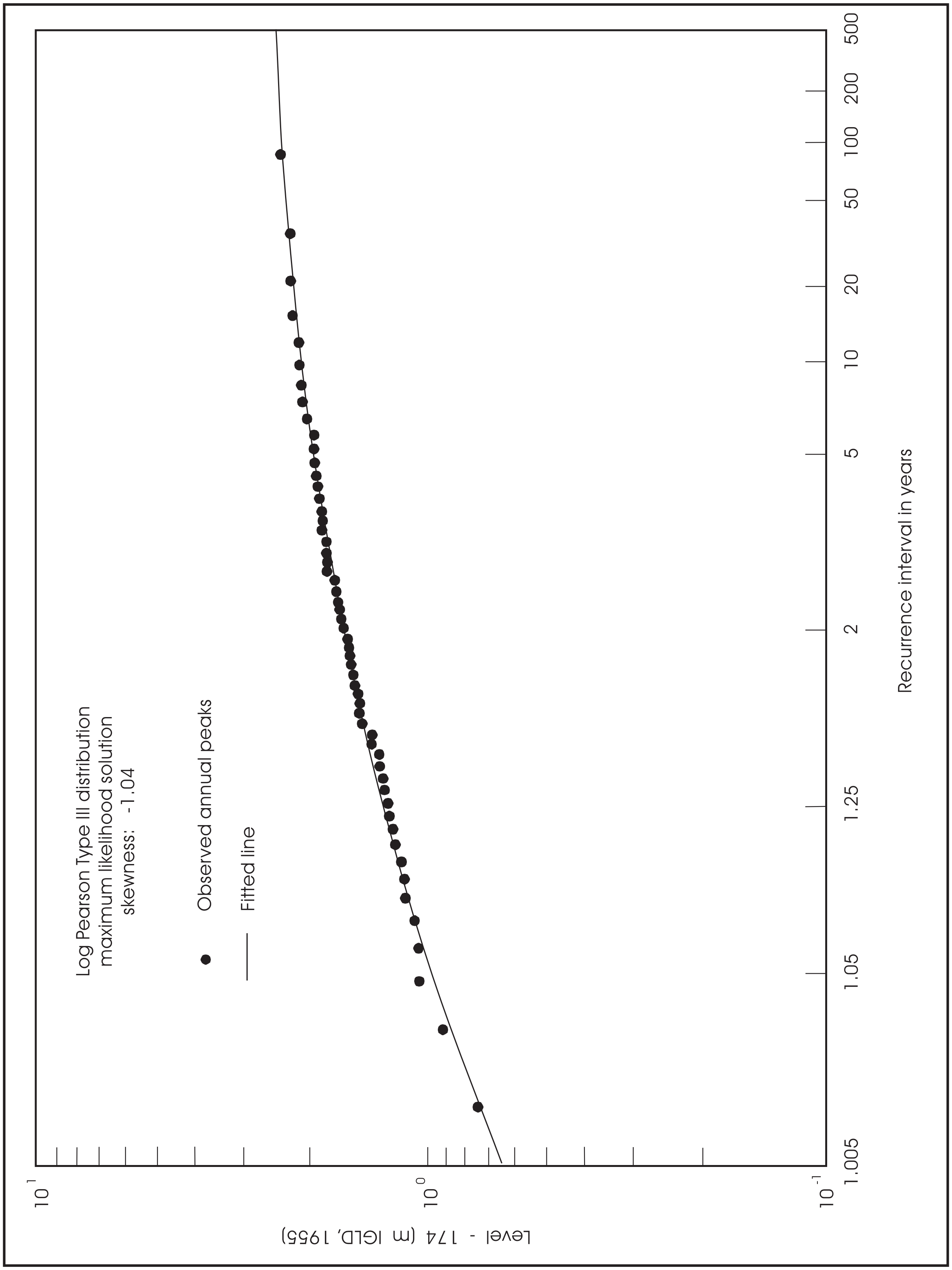
For the combined frequency analyses, it was assumed that the monthly mean lake level and the wind setup are independent events and that they occur randomly throughout time. This is not the case as wind setup values are less during the summer months (i.e., time of lowest winds) when mean monthly lake levels are typically at their peak. In order to address this concern for an International Joint Commission study, Atria (1993) subgrouped the lake levels and the wind setups into four seasons: winter (i.e., January to March), spring (i.e., April to June), summer (i.e., July to September) and fall (i.e., October to December). The resulting maximum 100 year flood levels for the nine sites studied, were, on average, 0.07 m less than the MNR (1989) values.

iv) Climate Change

An inherent assumption of the preceding water level frequency analyses is that the regional climate of the Great Lakes Basin has and will not change appreciably over the long term. A second generally accepted assumption is that the earth is undergoing a global warming trend, attributed to an increase in carbon dioxide and certain other gases in the atmosphere (i.e., greenhouse effect). The impacts of this warming on precipitation and evaporation in the Great Lakes Basin, however, is less certain.

For the purpose of planning shoreline facilities and protection works over the next 50 years, Bishop (1987) carried out a review of studies undertaken to evaluate the predicted climate change in the Great Lakes Basin and of studies of the impacts of the predicted climate changes on water levels. Under a scenario of doubled atmospheric concentrations of carbon dioxide, some climate models predict likely decreases in net basin water supplies and consequently lower mean annual lake levels. The high level of uncertainty, however, in predicting changes in precipitation, wind patterns and relative humidity under this scenario is such that water supplies to the basin could also increase. The review concludes that over the next 50 years water levels are unlikely to appreciably exceed the modern records.

Figure A3.1.3: Port Lambton Annual Peak Stillwater Levels



b) St. Lawrence River

The following description of the St. Lawrence River flood level calculations is taken from a report (Environment Canada 1993) undertaken by Environment Canada's Water Planning and Management Branch in consultation with MNR and Conservation Authority staff.

High water levels in the St. Lawrence River combined with periodic high winds can cause flooding and erosion along the shoreline. During the past 30 years, concentrated efforts have been taken to regulate Lake Ontario. Past data and experience combined with modelling techniques have enabled the International Joint Commission's International St. Lawrence Board of Control to regulate the levels of the Lake Ontario-St. Lawrence System within a limited range. Unfortunately, such a high degree of regulation can create a false sense of security for many riparian owners. The added possibility for significant development along the shoreline in the form of permanent residences and subdivisions has compounded the overall potential threat of flood damage. The purpose of the study was to establish the 100 year flood levels for the St. Lawrence River from Kingston to the Ontario-Quebec border recognizing the unique water management system which governs levels and flows within the Lake Ontario-St. Lawrence River System.

Methodologies described in the 1989 MNR report on Great Lakes System flood and water related hazards were adopted for this analysis. The results obtained in other previous studies were also taken into consideration in determining the 100 year flood level. These earlier works included the U.S. Army Corps of Engineers Revised Report on Open-Coast Flood Levels (1988) and the Flood and Fill Line Study of the St. Lawrence River from Dundas-Stormont boundary to the Ontario-Quebec border carried out by M.M. Dillon Limited (1979) for the Raisin Region Conservation Authority. Consideration was also given to the various Power Entities' (e.g., Ontario Hydro, New York Power Authority and Hydro-Quebec) operating procedures.

A3.1.4 Basis of Comparison (June 1988)

The purpose of Section A3.1.4 is to provide shoreline managers with information related to the estimation of water level frequencies and the need to first adjust the observed data to a constant set of conditions or regulations and diversions referred to as the Basis of Comparison. These adjustment procedures ensure consistency in the calculation of water level frequencies throughout the Great Lakes - St. Lawrence River System.

Over the years a number of changes in the regulation of the Great Lakes - St. Lawrence System and changes in diversions into and out of the lakes have taken place which have had measurable effects on flows and levels in the system. In order to estimate water level frequencies in the Great Lakes - St. Lawrence System from measured flow and level records, the observed data must first be adjusted to a constant set of conditions of regulation and diversions.

The general conditions are as follows:

1. A constant diversion of 160 m^3/s (5,600 cfs) into Lake Superior by way of the Long Lac and Ogoki diversion. This diversion was authorized under the exchange of notes, dated October 14 and 31 and November 7, 1940, between the United States and Canada and has averaged approximately this amount since that date.

2. Lake Superior regulated in accordance with Plan 1977, which is the currently authorized plan used by the International Lake Superior Board of Control for determining releases from Lake Superior.

3. A constant diversion of 90 m^3/s (3,200 cfs) out of Lake Michigan at Chicago. This is the maximum allowable diversion at Chicago by decree of the U.S. Supreme Court, dated June 12, 1967.

4. 1962 outlet conditions for Lake Huron. This represents the current conditions, which have existed since the completion of the 27-foot navigation channel dredging in 1962.

5. A constant diversion, by way of the Welland Canal, of 260 m^3/s (9,200 cfs) out of Lake Erie and into Lake Ontario. This is the current average diversion.

6. 1953 outlet conditions for Lake Erie. In its 1953 report on the Preservation and Enhancement of Niagara Falls, the International Joint Commission considered it essential that the relationship existing at that time between the Niagara River flow and the Chippewa-Grass Island Pool level be maintained following the commencement of operation of the Chippewa-Grass Island Pool Control Structure and power diversions as permitted by the 1950 Niagara Treaty. The rating curve for the outlet conditions was updated in 1987.

7. Lake Ontario regulated during the period 1900–April 1960 in accordance with Plan 1958–D without discretionary deviation. For the period from April 1960 to the present, Lake Ontario was regulated in accordance with Plan 1958-D with discretionary deviations as they occurred. Minor adjustments to the discretionary deviation values were required during high water periods to preclude violation of the St. Lawrence River low water profiles.

8. Recorded conditions for the Ottawa River and local inflow to the St. Lawrence River.

The levels and outflows to be used as a Basis of Comparison for each lake were obtained by routing through the system the coordinated net basin supplies employing the constant conditions previously listed.

For Lake Superior, the Basis of Comparison levels and outflows were obtained by routing through the lake the coordinated net basin supplies (i.e., adjusted for a constant, 5,600 cfs diversion into the lake by way of the Long Lac and Ogoki diversions) in accordance with the present regulation plan known as Plan 1977.

Because of the nature of the control in the St. Clair and Detroit Rivers, the problem of routing supplies through Lakes Michigan-Huron, St. Clair and Erie was fairly complex. The St. Clair River and Detroit River flows are dependent not only on Lake Huron levels, but also on the water levels in the lower river and Lake Erie. Therefore, a method of successive approximations of supply routing was used.

A3.1.5 References

Atria Engineering Hydraulics Inc., 1993. Hydrologic impact of regulation scenarios on flood risk levels for selected sites of the Great Lakes - St. Lawrence System. A report prepared for Environment Canada, Great Lakes - St. Lawrence Study Office.

Bishop, C.T. 1987. Great Lakes water levels: A review for Coastal Engineering design. National Water Research Institute, NWRI Contribution 87–18. Burlington, Ontario.

Environment Canada 1993. St. Lawrence River Flood Levels. Inland Waters Directorate, July.

Fay, D. and Moulton, R. 1990. Calculation of flood levels along the ontario shoreline of the Great Lakes. Proc. Flood Plain Management Conference. Regal Constellation Hotel, Toronto, OMNR, March 12-14.

Ministry of Natural Resources (MNR), 1989. Great Lakes system flood levels and water related hazards. Conservation Authorities and Water Management Branch, OMNR, February.

Ministry of Natural Resources (MNR), 1982. HYDSTAT computer program for univariate and multi-variante statistical applications. Conservation Authorities and Water Management Branch, OMNR, October.

M.M. Dillon, 1979. St. Lawrence River Shoreline Study. A report prepared for the Raison Region Conservation Authority.

U.S. Army Corps of Engineers (USACE), 1988. Revised report on Great Lakes open coast flood levels. Report prepared for Federal Emergency Management Agency by U.S. Army Corps of Engineers. Detroit, Michigan.

Welander, P. 1961. Numerical predictions of storm surges. Advances in Geophysics 8:316-379.

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APPENDIX A3.2

WATER LEVEL INFORMATION SERVICE

WATER LEVEL INFORMATION SERVICE

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A3.2 WATER LEVEL INFORMATION SERVICE

The purpose of Appendix A3.2 is to provide some general information and direction on securing water level information from the various gauging stations operated by the Canadian Hydrographic Service.

Table A3.2.1
Water Level Information Service

The Canadian Hydrographic Service, Central and Arctic Region, operates a network of water level gauging stations on the Great Lakes and St. Lawrence River. A voice announcement of the present water level at the following locations can be obtained by telephone:

Lake Superior at Thunder Bay	807-344-3141
Lake Superior at Rossport	807-824-2250
Lake Superior at Michipicoten	705-949-1886
Lake Superior at Gros Cap	705-779-2052
St. Mary's River above the lock at Sault Ste. Marie	705-949-2066
St. Mary's River below the lock at Sault Ste. Marie	705-254-7989
Thessalon	705-842-2215
North Channel at Little Current	705-368-3695
Georgian Bay at Parry Sound	705-746-6544
Lake Huron at Collingwood	705-445-8737
Lake Huron at Tobermory	519-596-2085
Lake Huron at Goderich	519-524-8058
St. Clair River at Point Edward	519-344-0263
St. Clair River at Port Lambton	519-677-4092
Lake St. Clair at Belle River	519-728-2882
Detroit River at Amherstburg	519-736-4357
Lake Erie at Bar Point	519-736-7488
Lake Erie at Kingsville	519-733-4417
Lake Erie at Port Stanley	519-782-3866
Lake Erie at Port Dover	519-583-2259
Lake Erie at Port Colborne	905-835-2501
Lake Ontario at Port Weller	905-646-9568
Lake Ontario at Burlington	905-544-5610
Lake Ontario at Toronto	416-868-6026
Lake Ontario at Cobourg	905-372-6214
Lake Ontario at Kingston	613-544-9264

St. Lawrence River at Brockville	613-345-0095
St. Lawrence River above the lock at Iroquois	613-652-4426
St. Lawrence River below the lock at Iroquois	613-652-4839
St. Lawrence River at Morrisburg	613-543-3361
St. Lawrence River at Cornwall	613-930-9373
St. Lawrence River at Summerstown (Lake St. Francis)	613-931-2089

When you call the telephone number for a particular station, the equipment at the gauging station will ask you to press "1" for English or press "2" for French, on the keypad of your touch-tone phone. If you do not have a touch-tone phone, the message in English will start after a few seconds delay and the message in French will follow. The present water level is give in metres relative to chart datum at that station. Then the message gives the high and low water levels recorded during the previous 12 hours, followed by the elevation of the chart datum. You can press "1" or "2" at any time during the message to have it start again or press "0" to end the call. Call the CHS office in Burlington at 905-336-4844 during business hours or by fax at 905-336-4844 to report any problems or to obtain additional information.

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APPENDIX A3.3

SURGE MODEL

SURGE MODEL

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A3.3 SURGE MODEL

General Description

The computer program SURGE, obtained from the Atmospheric Environment Service Canada, provides a means of calculating wind setup on a lake for any user defined hourly wind velocity series. The model describes the lake as a single layer of grid squares each of the same breadth and width. Each grid square has a corresponding depth based on bathometric information. The program calculates the average water surface position in each grid square for each time period in the defined analysis period.

The program models the change in water surface position resulting from given surface wind stress using the following hydrodynamic equations:

$$\frac{\delta U}{\delta t} = fV - gH\left(\frac{\delta Z}{\delta x}\right) - BU + \tau_{sx}$$

$$\frac{\delta V}{\delta t} = -fV = gH\left(\frac{\delta Z}{\delta y}\right) - BV + \tau_{sy}$$

$$\frac{\delta Z}{\delta t} = -\left(\frac{\delta U}{\delta x}\right) - \left(\frac{\delta V}{\delta y}\right)$$

where:

t	=	time
x,y	=	the horizontal coordinates (x clockwise from y)
U,V	=	the corresponding components of the vertically integrated current
Z	=	the upward displacement of the water surface from a mean (initial) level
H	=	depth of the basin for this mean level
B	=	a bottom stress coefficient
f	=	the Coriolis parameter (equal to twice the angular velocity of the earth's rotation times the sine of latitude)
g	=	the gravitational acceleration
τ_s	=	wind stress at the lake surface divided by the water density.

The wind stress is calculated within the program from wind velocity (i.e., speed and direction) data supplied by the user. The wind stress relation used in the model assumes the wind stress is equal to the square of the wind velocity times a drag coefficient.

The program output provides a water surface elevation map for each grid on the lake on an hour by hour basis for the period defined by the user.

The SURGE model is an adaptation of a one-layer, free-surface circulation model with a single Richardson Lattice developed by T.J. Simons and D.C.L. Lam of the National Water Research Institute, Environment Canada at the Canada Centre for Inland Waters. The original model is described in "Documentation of a Two-Dimensional X-Y Model Package for Computing Lake Circulations and Pollutant Transports" by Simons and Lam, 1982. The modified model, SURGE, was prepared by the staff of the Great Lakes Environmental Research Lab of NOAA in Ann Arbor, Michigan, and by staff of the Atmospheric Environment Service of Environment Canada in Downsview.

Additional Information:

The surge model was adapted for the micro computer by Linda Mortsch, Atmospheric Environment Service. For information contact L. Mortsch (905) 336-6417.

References:

Simons, T.J., Lam, D.C.L., ND: Documentation of a two-dimensional X-Y Model package for computing lake circulations and pollutant transports. Unpublished (available from L. Mortsch).

Murty, T.S., 1984: Storm surges - meteorological ocean tides. Ottawa: Department of Fisheries and Oceans, Canadian Bulletin of Fisheries and Aquatic Sciences, 212: 33-45.

To return an interactive session type:

SERIE	- for Lake Erie surge calculations
SONT	- for Lake Ontario surge calculations
SHUR	- for Lake Huron surge calculations
SMICH	- for Lake Michigan surge calculations
SSUP	- for Lake Superior surge calculations

For hard copy output:

. use a control print screen command.

You will be prompted for wind speed (i.e., in knots) and direction (i.e., degrees) forecasts and later for the type of output you wish (i.e., a map of surge heights for a particular hour or a time series of surge heights at particular locations).

Function of various programs/files:

INTFCS.EXE	prompts for time frame of forecast, wind speed and direction forecast, and user information creates files GENINF.15, WAVE.8 and WAVE.10
WAVE.8	wind data
WAVE.10	time interval between wind data points, length of forecast/hindcast
GENINF.15	length of forecast/hindcast period, time interval between wind input, start time
SURGEM.EXE	computes surge height creates file SURGE.HT
MSPLAY.EXE	prompts for type of output (i.e., map or time series) generated hard copy output if CNTL PRINT SCREEN is used
XX.DAT	contains bathymetry data for each of the lakes
XXX.DRG	contains data on time increments and drag coefficient
Note:	when running the model it is advisable to allow a gradual build up of the winds; the Ontario Weather Centre forecasters used a 48 hour 'run up time' for the Great Lakes. This was done to minimize the effects of a 'cold' start which is initially reflected by amplified seiching of the lake because of the wind pulse.

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APPENDIX A3.4

HYDSTAT MODEL

HYDSTAT MODEL

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A3.4 HYDSTAT MODEL

The program HYDSTAT can be used for both single variate and multi-variate probability distributions. For single variates, it fits seven probability distributions to a given set of data and identifies the best fitting. For multi-variate functional relationships where:

$$Z = f(I_1, X_2, X_3......X_n) \text{ and } X_1, X_2, X_3.....X_n$$

are independent variables each with their own probability distributions, the program obtains the probability distribution of Z by solving the multiple integral formed by the product of the probability density functions of X_1, X_2, $X_3.....X_n$. The program requires the parameters of the probability distribution of each variable or sets of data from which the best fitting will be selected and the functional relationship between the variables. There is no limit on the number of independent variables that can be handled.

The program is suitable for solving a wide range of statistical problems in water resources.

More detailed information can be found in *"HYDSTAT Computer Program for Univariate and Multi-variate Statistical Applications*" prepared by the Ministry of Natural Resources (1982). It reviews the appropriate theory, describes the computational procedure and contains appropriate documentation including a user's manual and program listing.

A3.4.1 Update on HYDSTAT V2.0 1993

In response to concerns that HYDSTAT only used goodness of fit tests to determine the best statistical fit to a data set, graphical plots have been added to the program. An additional goodness of fit test has been added; the Akaike selection criteria has been added for reference purposes. An additional 8th distribution has been added: The General Extreme Value distribution (EV3). This version was developed by Dr. Ander Chow of Atria Engineering Inc., for the federal government.

To obtain graphical plots of the data, one must set a flag on the first input card.

COLUMN	VARIABLE	TYPE	DESCRIPTION
6-10	NVAR	INTEGER	Number of Variables.
16-20	GRAF	INTEGER	GRAF = 1 for plots, = 0 no plots.

Graphics reset commands:

These graphics should work on most PCs, but will perform best on VGA or higher. Upon completion HYDSTAT resets the graphics to text mode. If the program terminates abnormally, programs have been supplied for resetting the graphics to text mode. Enter:

TEXTC80 Resets to colour, 80 column mode.

TEXTC40 Resets to colour, 40 column mode.

TEXTBW80 Resets to monochrome, 80 column mode.

Install:

An install batch file has been provided for installing the software on hard drive. Put the disk in the disk drive, select this drive and type install.

Automenu:

Menus have been provided for use with Automenu. If you have AUTOMENU.COM in your path, determine the disk and directory where your AUTOTEMP.BAT file is. Modify the file AUTO.BAT to add the drive and directory to the autotemp command.

You may want to modify the menu file SPF.MDF to call your own favourite text editor.

Automenu also allows for printing the graphics to a Dot Matrix or Laserjet printer. Use the appropriate Menu item. When running the program press PRINTSCREEN to print the graphics.

A3.4.2 HYDSTAT Model Overview

In water resources, statistical methods are used to predict the magnitude and frequency of occurrence of events, such as flood flows and flood levels. Determination of the parameters of appropriate statistical distributions is often the most critical part of these analyses. Depending upon the nature of the problem, one or more independent variables may be involved (8), and a multi-variate or combined frequency analysis is required. Examples of distributions involving one or more independent variables are discussed below.

a) Univariate Distributions

A set of observations consisting of one variable such as maximum mean daily river flows can be considered as an example of a single variable or a univariate distribution (7).

b) Multi-Variate Distributions

If two or more independent variables, X_1, X_2, X_3...X_n, occur jointly to cause Z, and if each of the variables has its own frequency distribution, the frequency distribution of Z is a multi-variate distribution of N variables.

Resultant lake levels arising out of increases due to wind setup are examples of a two-variable or bivariate distribution (2). The probability of lake levels occurring is a function of both the probability of a calm water level and the probability of winds causing wind setup rises. The probability of resultant lake levels is the combined probability of both variables. An example is given in Appendix III.

Direct runoff and river flows can be considered as multi-variate frequency distributions since they are a function of antecedent moisture conditions, groundwater conditions and other independent basin characteristics which occur randomly. The frequency of occurrences of all independent variables contributing to river flows could be combined to treat river flows as a multi-variate frequency distribution.

Runoff due to snowmelt and rainfall is a function of the independent variables of temperature, rainfall and wind speed. Each of the variables has its own frequency distribution. These can be combined to develop a multi-variate frequency distribution for total runoff. This would represent the total probability of spring runoff occurring from all possible combinations of the variations which could cause it.

In many cases where rivers outlet to lakes in low-lying areas, the river level for a particular flow depends on the level of the lake. A high lake level and low river flow could result in the same river level as a low lake level and a high river flow at a point on the river. The frequency of river levels depends on the combined frequencies of river flows and lake levels.

At the confluence of rivers, the tributary inflows can, in some cases, be treated as independent variables, perhaps due to upstream regulations or, in large watersheds, climatic variables such as storms occurring on one tributary but not the other. The flows and levels downstream can be treated as multi-variate frequency distributions.

The flows from a reservoir can be a function of the three random variables of rainfall, snowmelt and reservoir level, and therefore, can be treated as a multi-variate case and distribution.

In order to deal with water resource problems involving both univariate and multi-variate probability distributions, the program HYDSTAT was developed.

HYDSTAT uses the statistical distributions commonly used in hydrology. These are normal, log-normal, Gumbel, log-Gumbel, Pearson Type III, log-Pearson Type III and 3-parameter log-normal. The accepted procedure selecting which distribution to use for a set of data is to test a number of distributions and select the statistical distribution which best fits the data. The minimum chi-square and least-squares value goodness of fit tests are both calculated in the program.

The choice of the best fitting distributions, based upon the minimum chi-square or least-squares values, is somewhat arbitrary. Since the magnitudes of predicted values depend largely upon such choices, sufficient care and good judgement must be exercised in choosing the appropriate distribution(s) and in utilizing the results derived from the application of the program.

To use the multi-variate analysis component of the program, it is necessary to input the functional relationship of the variables $Z=f(X_1, X_2, X_3...X_n)$ and either the statistical parameters of the individual variables of $X_1, X_2, X_3...X_n$ or a sample population for each variable from which these can be calculated. The probabilities of discrete values of Z are calculated by solving the multiple integral formed by the products of the probability distributions of $X_1, X_2, X_3...X_N$ using the convolution formula and numerical integration.

Reference

Ministry of Natural Resources (MNR), 1982. HYDSTAT computer program for univariate and multi-variant statistical applications. Conservation Authorities and Water Management Branch, OMNR, October.

TECHNICAL GUIDE FOR
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APPENDIX A3.5

FLOOD ALLOWANCE FOR WAVE UPRUSH:

15 METRES FOR GREAT LAKES;

5 METRES FOR CONNECTING CHANNELS

FLOOD ALLOWANCE FOR WAVE UPRUSH:
15 METRES FOR GREAT LAKES; 5 METRES FOR CONNECTING CHANNELS

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A3.5 FLOOD ALLOWANCE FOR WAVE UPRUSH: 15 METRES FOR GREAT LAKES; 5 METRES FOR CONNECTING CHANNELS

The purpose of Appendix A3.5 is to provide technical direction on the calculation, use and application of wave uprush values in the determination of the applicable flood allowance for a given shoreline site.

A3.5.1 Flood Allowance for Wave Uprush

Wave uprush levels, including wave setup, were calculated at 43 sites (MNR 1989) using the procedures given in Chapter 7 of the *Shore Protection Manual,* 1984 (USACE 1984). These procedures have been superseded by the work of Stoa (1978a) and implemented by Dewberry and Davis (1990). This updated methodology has been used to recalculate the horizontal and vertical uprush levels.

a) Great Lakes

The flood allowances for wave uprush, provided for the Great Lakes shorelines (15 metres) were calculated as follows:

i) Onshore and offshore nearshore bottom profile data were obtained for the 163 Great Lakes erosion monitoring stations established in 1973. The profile data was that of Boyd (1981). The calculation of wave uprush levels were limited to those stations having a bluff height of less than 3 metres above lake datum. In total, 88 stations between Wasaga Beach on Lake Huron to Outlet Beach on Lake Ontario were selected. At some of the profile locations, the measured onshore elevations did not extend far enough inland to contain the predicted uprush. Therefore of the 88 profiles selected for wave uprush analysis, only 43 profiles provided sufficient information to determine the extent of wave uprush. The location of the 43 sites is depicted in Figures A3.5.1 to A3.5.3. A typical profile is shown in Figure A3.5.4.

ii) For each profile, a beach slope, on which the wave uprush occurs, and a nearshore slope were characterized using the Geometrical Analysis of Basic Shore Situations described in Dewberry and Davis (1990) (see Figure A3.5.5). The work of Stoa (1978a) was then used within the Dewberry and Davis model (1990) with a 7 second wave period, wave heights up to approximately the 10-year storm wave, and the 100 year flood level to determine the uprush at each site.

iii) For low bluffs, wave uprush may travel over the bluff crest similar to a wave bore. For these cases, the New England Methodology (FEMA 1989) was used to calculate the distance of horizontal uprush past the bluff crest along the flatter slope (Figure A3.5.6 and Figure A3.5.7).

iv) For the 43 sites examined, the calculated horizontal offset due to uprush was as great as 25 metres with approximately 10% of the calculated values greater than 20 metres and approximately 20% greater than 15 metres. Where no detailed, site-specific study is carried out, a standard flood allowance value of 15 metres was selected as a reasonable estimate that would provide a safe setback for most sites.

The following is the step-by-step procedure used:

- **Step 1** Plot onshore-offshore nearshore bottom profile data (i.e., the data from 1973 were selected as being adequate for this analysis).

- **Step 2** Estimate a shore slope, and a nearshore approach slope using the methodology of Dewberry and Davis (1990) (Figure A3.5.5); the point of intersection is considered to be the toe of slope.

Figure A3.5.1: Wave Uprush Profile Locations: Lake Huron

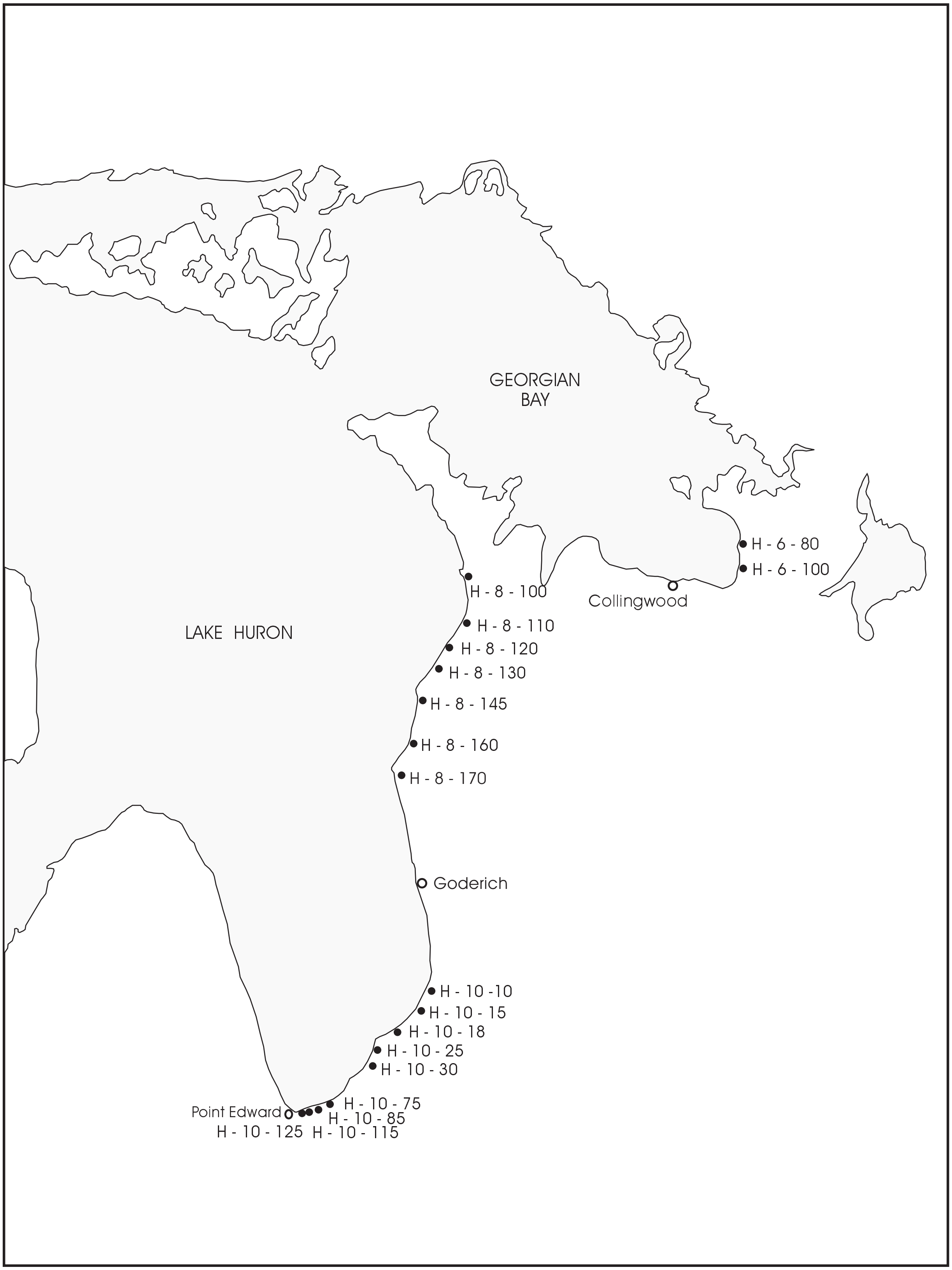


Figure A3.5.2: Wave Uprush Profile Locations: Lake Erie

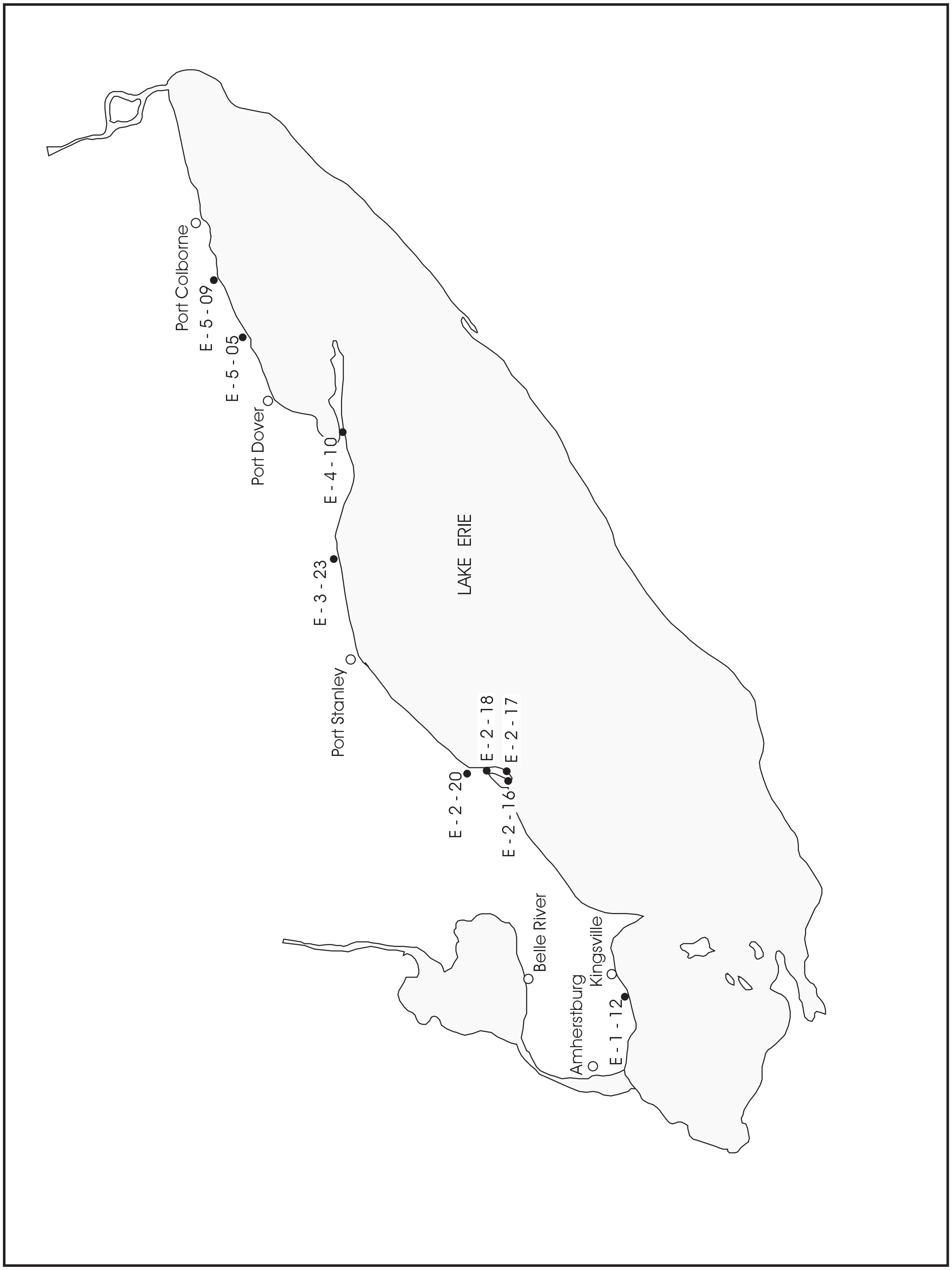


Figure A3.5.3: Wave Uprush Profile Locations: Lake Ontario

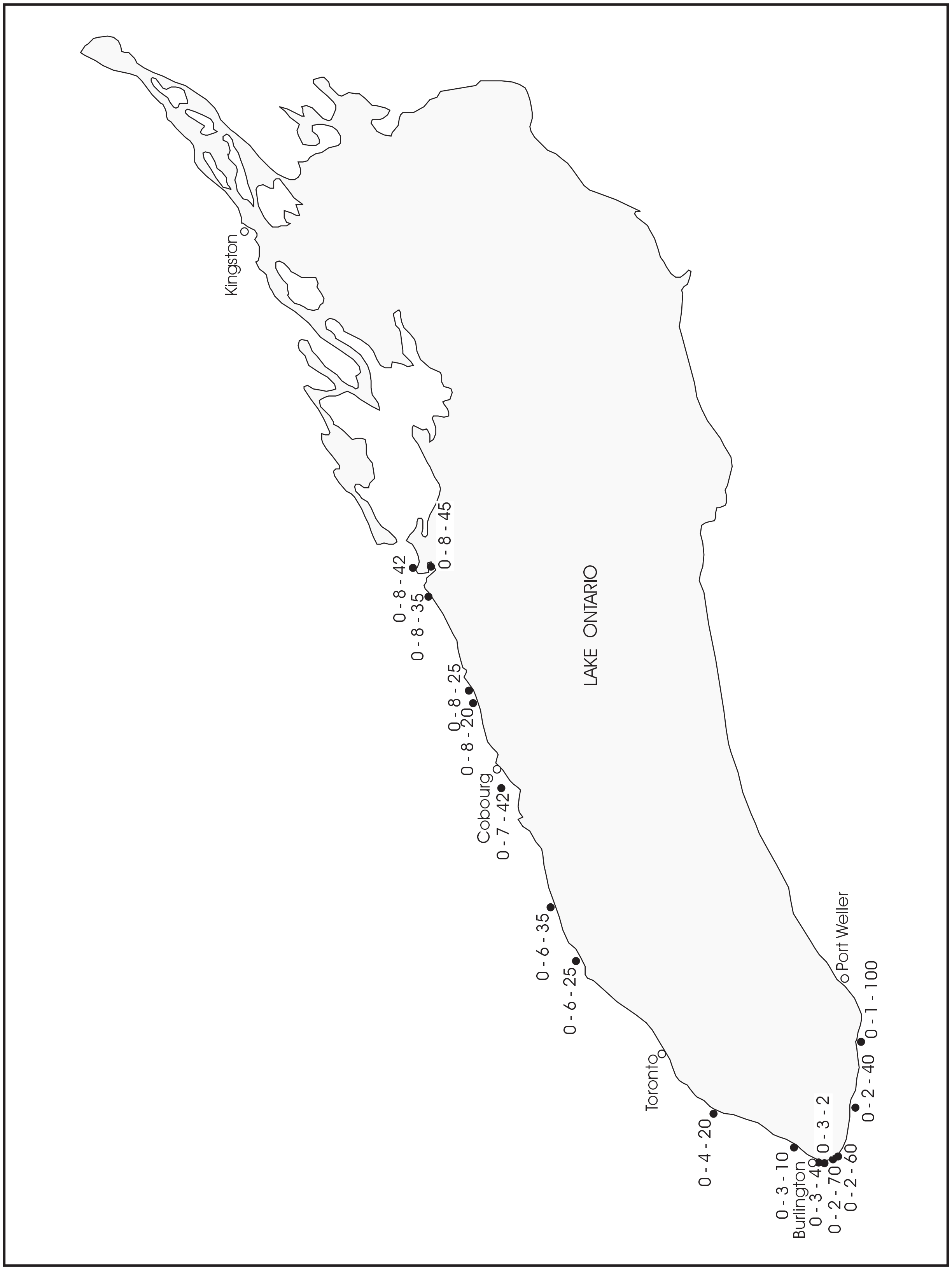


Figure A3.5.4: Typical Profile for Wave Uprush Calculation

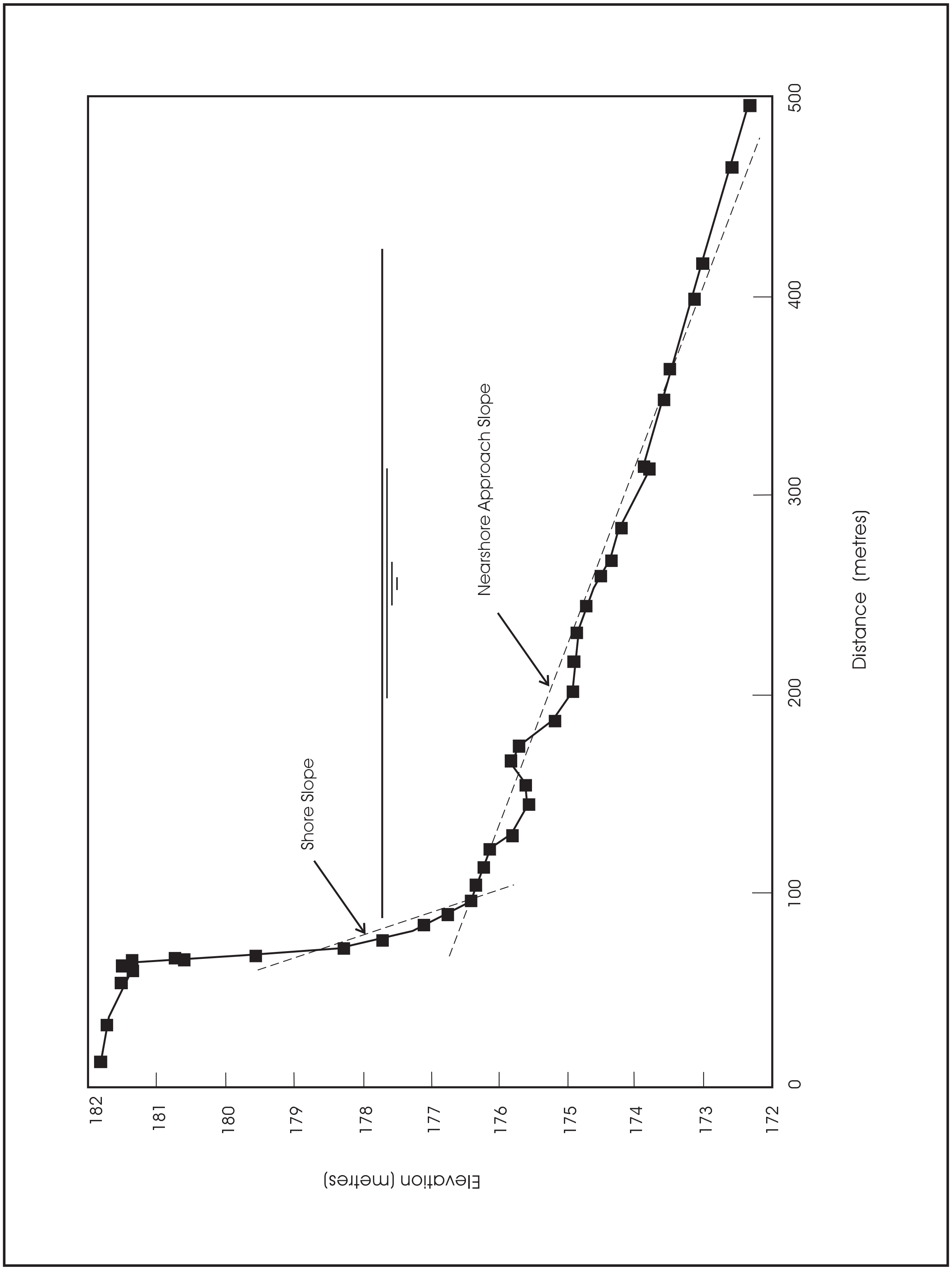
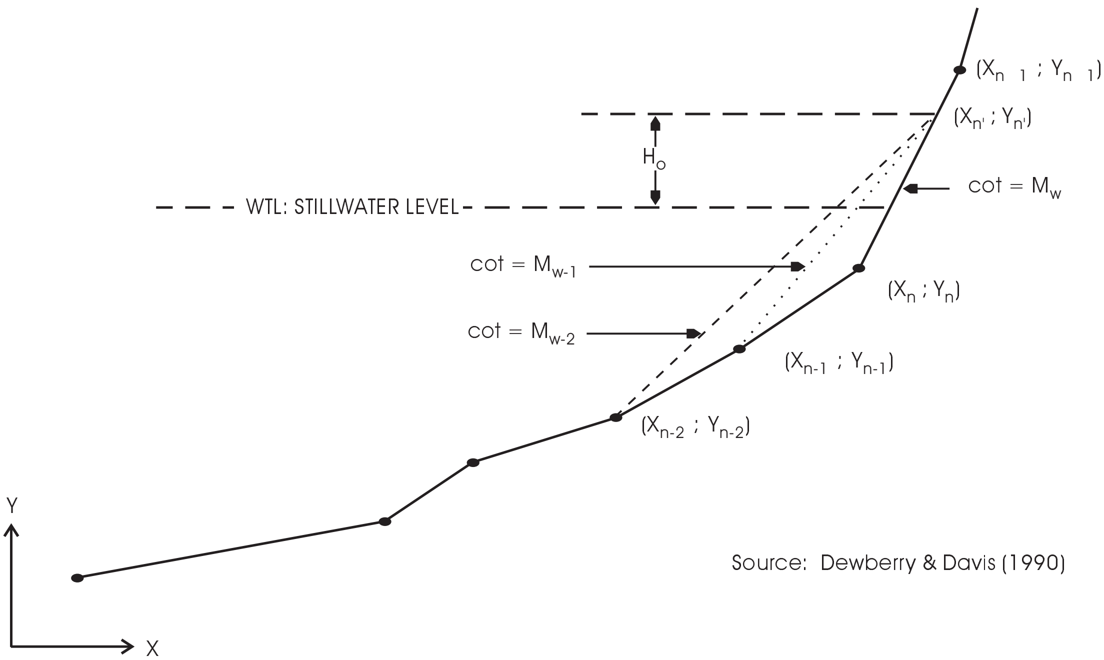


Figure A3.5.5: Outline for New Geometrical Analysis of Basic Shore Situation



Source: Dewberry & Davis (1990)

Analysis for Lakeward Extent of Shore Slope

1. M_w = Cotangent of emergent profile segment (i.e., including stillwater intercept). If segment extends to elevation exceeding $(WTL+H_o)$, determine coordinates at that elevation, $X_{n'}$; $Y_{n'}$, and use in place of $X_{n\ 1}$; $Y_{n\ 1}$ in the following.
2. Add first fully submerged profile segment to emergent one, and determine overall slope of combination, namely $M_{w-1} = (X_{n\ 1} - X_{n-1})/(Y_{n\ 1} - Y_{n-1})$.
   If $M_{w-1} \leqslant 1.2\ M_w$ consider Shore Slope to include present segment, and proceed to next step; otherwise, Shore Slope extends lakeward only to X_n ; $Y_{n'}$
3. Add next lakeward profile segment and determine new overall slope M_{w-2}.
   If $M_{w-2} \leqslant 1.2\ M_w$, admit this segment to Shore Slope and repeat tentative extension; otherwise, do not.

Analysis for Lakeward Extent of Nearshore Approach Slope

1. M_{s-1} = cotangent of profile segment immediately lakeward of Shore Slope limit. Add next lakeward profile segment and determine overall slope M_{s-2}.
   If $M_{s-2} \leqslant 1.2\ M_{s-1}$ and $M_{s-2} \leqslant 15$, admit second segment to Nearshore Approach Slope, and proceed to next step; otherwise, Nearshore Approach Slope is limited to single segment.
2. Add next lakeward profile segment and determine overall slope M_{s-3}.
   If $M_{s-3} \leqslant 1.2\ M_{s-1}$ and $M_{s-3} \leqslant 15$, admit segment to Nearshore Approach Slope and repeat tentative extension; otherwise, do not.

Figure A3.5.6: Uprush on Low Bluffs (New England Methodology) (after FEMA 1989)

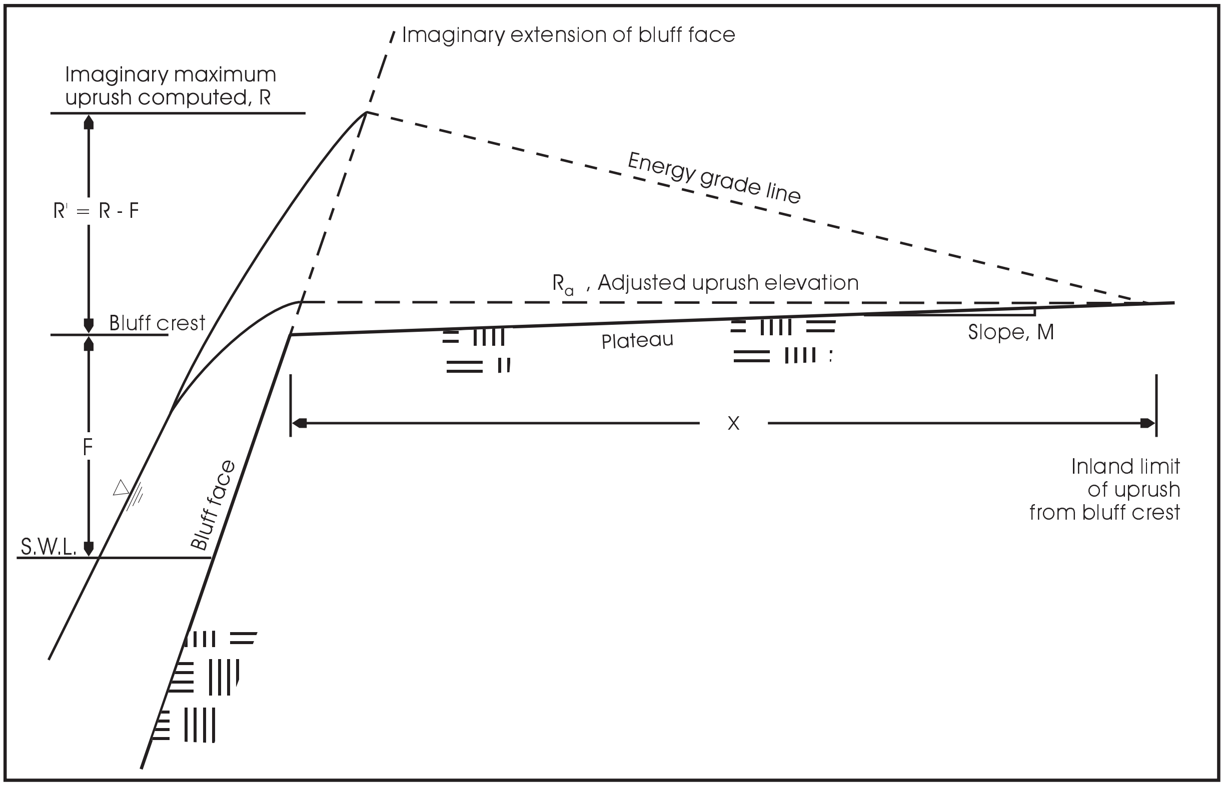
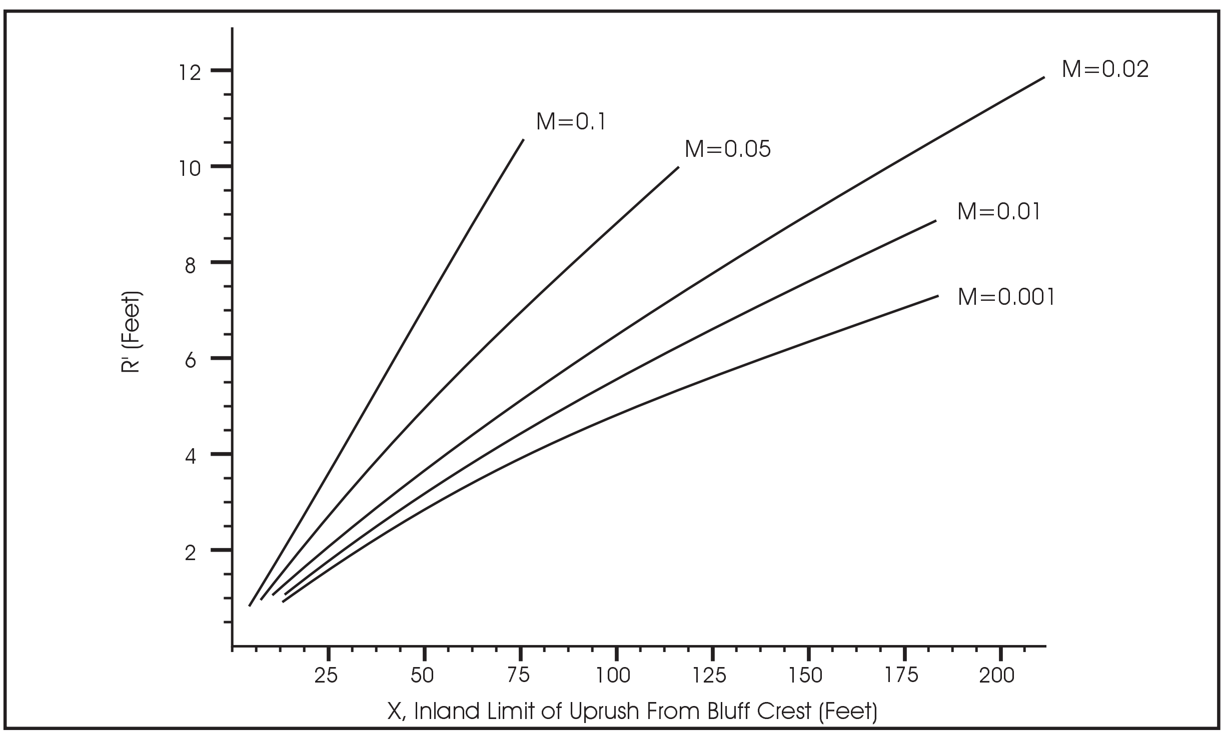


Figure A3.5.7: Computation of Uprush over Low Bluffs (New England Methodology) (after FEMA 1989)



- **Step 3** Neglecting refraction and shoaling effects, use Stoa (1978a) as contained in the model of Dewberry and Davis (1990) to estimate the vertical extent of wave uprush.

- **Step 4** Perform a sensitivity analysis to determine the unrefracted deep-water wave height (H_o') that produces the maximum vertical wave uprush for each profile.

- **Step 5** From the profile geometry and the maximum vertical extent of wave uprush, determine the maximum horizontal extent of wave uprush for each profile.

- **Step 6** For those low bluffs that are overtopped, the New England Methodology (FEMA 1989) was used to calculate the horizontal extent of the wave uprush past the crest of the bluff (Figure A3.5.6). An imaginary maximum vertical extent of wave uprush (R') was calculated assuming that the bluff face extends indefinitely (i.e., beyond the bluff crest). Using R' and plateau slope M, Figure A3.5.7 was entered to determine the inland limit wave uprush from the bluff crest.

  Since the New England Methodology (FEMA 1989) results in some large values for the horizontal extent of wave uprush for large values of wave uprush, a maximum horizontal extent of wave uprush of 20 m inland from the crest of the bluff was assumed and used in place of any values predicted larger than 20 m.

Results of the depth-limited wave uprush calculations for 43 sites along the Great Lakes are provided in Table A3.5.1.

b) Connecting Channels

For shorelines of connecting channels, sheltered from waves generated on the lakes, the largest waves experienced will most likely be those generated by ships or boats passing nearby. Sorenson (1967) described the physical characteristics of ship generated waves for ships of varying size, speed and distance from shore. From this data, a reasonable wave condition (H = 0.9 m; T = 3.8 s) for connecting channels was estimated and wave uprush calculations were conducted for a range of slopes (i.e., 1:2 to 1:30). A horizontal allowance of 5 m from the 100 year flood level was selected as a reasonable allowance for wave uprush.

Table A3.5.1
Summary of Wave Uprush Calculations

Profile	100 Year Flood Level (m IGLD)	Uprush (m)	Horizontal Allowance (m)	Comments
Lake Huron				
H-06-80	177.8	0.89	10.3	Beach
H-06-100	177.8	0.37	17.7	Beach
H-08-100	177.4	0.38	8.8	Beach
H-08-110	177.4	0.54	7.7	Beach
H-08-120	177.4	1.52	25.1	Low Bluff
H-08-130	177.4	1.13	5.0	Beach
H-08-145	177.6	1.49	6.4	Beach
H-08-160	177.4	0.80	10.9	Beach
H-08-170	177.5	0.33	9.0	Beach
H-10-10	177.7	1.08	11.4	Beach
H-10-15	177.7	1.92	6.8	Beach
H-10-18	177.7	0.53	2.3	Steep Bluff
H-10-25	177.7	0.46	8.5	Low Bluff
H-10-30	177.7	0.82	11.6	Curved bluff
H-10-75	177.8	1.86	6.4	Bluff
H-10-85	177.8	2.76	6.7	Steep bluff
H-10-115	177.8	1.25	2.8	Steep bluff
H-10-125	177.8	1.18	4.6	Steep bluff
Lake Erie				
E-01-12	175.5	3.00	21.1	Low bluff
E-02-16	175.2	0.98	11.6	Low bluff
E-02-17	175.3	1.11	17.0	Low bluff
E-02-18	175.1	0.80	8.9	Beach
E-02-20	175.1	1.74	6.7	Beach
E-03-23	175.4	1.46	18.9	Beach
E-04-10	176.1	0.39	9.4	Beach
E-05-05	176.3	4.87	22.0	Low bluff
E-05-09	176.6	4.30	22.6	Low bluff
Lake Ontario				
O-01-100	76.0	2.87	25.3	Low Bluff
O-02-40	75.9	1.55	10.0	Beach
O-02-60	75.9	0.53	8.2	Beach
O-02-70	75.9	1.05	9.6	Beach
O-03-02	75.9	0.41	5.3	Beach
O-03-04	75.9	1.43	12.2	Beach
O-03-10	75.8	2.70	7.5	Bluff
O-04-20	75.7	1.00	8.3	Beach
O-06-25	75.5	1.99	6.0	Low Bluff
O-06-35	75.5	0.51	8.1	Beach
O-07-42	75.7	0.46	15.2	Beach
O-08-20	75.7	2.38	8.8	Beach
O-08-25	75.7	0.62	6.9	Low Bluff
O-08-35	75.6	1.40	9.2	Beach
O-08-42	75.6	0.27	4.0	Beach
O-08-45	75.6	1.93	0.5	Steep Bluff

A3.5.2 References

Boyd, G.L., 1981. Canada/Ontario Great Lakes Erosion Monitoring Programme. Final Report 1973-1980. Bayfield Laboratory for Marine Science and Surveys. Ocean Science and Surveys. Department of Fisheries and Oceans. Unpublished Report.

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Federal Emergency Management Agency (FEMA), 1989. Guidelines for specifications for wave elevation determination and V Zone mapping. Federal Emergency Management Agency, Federal Insurance Administration, Washington, D.C., Third Draft, July.

Ministry of Natural Resources (MNR), 1989. Great Lakes System Flood Levels and Water Related Hazards. Conservation Authorities and Water Management Branch.

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Stoa, P.N., 1978a. Reanalysis of wave runup on structures and beaches. Coastal Engineering Research Center, Technical Paper TP 78-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

U.S. Army Corps of Engineers, 1984. Shore Protection Manual. 4th ed. Coastal Engineering Research Centre, U.S. Government Printing Office. Washington D.C.

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APPENDIX A3.6

ICE

AND

SHIP GENERATED WAVES

ICE AND SHIP GENERATED WAVES

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A3.6 ICE AND SHIP GENERATED WAVES

The purpose of Appendix A3.6 is to provide background information on ice and ship generated waves. This appendix first gives and overview on ice formations, ice forces, how ice breaks up including ice jams and ice piling, and provides general information on ice conditions on the Great Lakes and connecting channels (i.e., St. Mary's, St. Clair, and Detroit Rivers).

It then provides information on the ship waves process, methods of calculating ship generated waves, and a summary of the various studies that have been done in Canada and the United States. In addition a list of pertinent references for both ice and ship generated waves has been included at the end of this appendix.

A3.6.1 Ice

Ice is often seen as a natural protector of shoreline environments by acting as a protective layer. Shore ice and ice piling along the shoreline or river bank can reduce the influence and overall destructive impact of wave action.

Ice can also be damaging to shorelines and cause erosion of beaches. It can cause severe flooding, particularly during spring breakup. During spring, damage occurs most often when ice jams, frazil dams or other ice initiated restrictions increase water levels and deflect or increase water flows onto shoreline areas.

The ice cover, itself, can also cause changes to the hydraulics of the *Great Lakes - St. Lawrence River System* posing local and regional flooding threats. This usually occurs where a portion of the system which is normally open channel flow is restricted or closed such that the flow becomes closed to conduit flow, resulting in flooding.

a) Ice Types and Ice Formation

The formation of an ice cover on lakes, rivers or streams depends on such factors as flow velocity, turbulence, surface disturbances (e.g., wind) and temperature. Successive days of below zero temperatures are required for lakes, rivers and streams to form an ice cover. Two types of ice can be formed:

- static; or
- dynamic or drift ice.

When the static and dynamic ice join and cover a significant area an ice cover is formed. Once ice covers the lake, river or stream, it can insulate the water below from the much colder below freezing air.

Static ice is formed during quiet conditions as the water is supercooled (i.e., the water temperature is about -0.05°C resulting from the mixture of air and water) and an ice cover is formed over the water surface. An example of static ice is shorefast ice.

Shorefast ice forms along the shoreline and is usually frozen to the substrate. "It usually remains in place for most of the winter and gradually grows, both vertically and horizontally" (MNR 1988). Where foreshore slopes are relatively flat, a thin layer of ice can accumulate landward whereas, ice can be relatively thick where foreshore slopes are steep. Shorefast ice is produced by several mechanisms working simultaneously including:

- the accumulation of wave spray and wave uprush;
- precipitation; and
- the accumulation of drift ice pushed by onshore winds.

Dynamic or drift ice is formed in turbulent conditions such as currents, wind waves, or fast flows. The ice crystals (i.e., frazil) are suspended in the turbulent, supercooled water. They then join and float to the surface or accumulate under the ice cover and are often referred to as slush ice or frazil ice, from which other types of ice are formed.

There are various types of drift ice including:

- slush ice;
- frazil ice;
- anchor ice;
- pan or pancake ice;
- floe ice; and
- composite ice.

These ice types, as listed in the order above, also represent stages of formation. In other words, the formation of each type, with the exception of slush and frazil, is dependant on the formation of the preceding types. The significance and formation are as follows:

- Slush ice — Snow blowing or falling into flowing water will accelerate the water's chilling rate, forming an unconsolidated mass of floating slush ice. Slush ice is extremely difficult to control and tends to restrict water flow and cause jamming (MNR 1988).
- Frazil ice — Frazil ice is a quick forming mass of frozen water particles that forms in supercooled water where there is no ice cover.
- Anchor ice — The frazil particles may freeze to objects which are on the bottom to form anchor ice.
- Pan or Pancake ice — Floating frazil ice forms 'flocs' which freeze together. Due to frequent collisions these flocs form floating, circular 'pancakes' with a slightly elevated rim, hence the name pancake ice. Diameters usually range from less than a metre up to 10 m.
- Floe ice — If the current is slow enough, pancake ice will come to rest and unite to form a floe. A floe is a mass of floating ice. Floes are more common in early winter and they are large (e.g., 50-100 metres in diameter), circular and capable of holding snowfall. If the water is turbulent, then chunky, uneven ice forms and does not remain on the surface. This can create hanging dams. If the hanging dams are large enough they may block the flow of water, raise water levels and cause flooding.
- Composite ice — As the name implies, composite ice is made up of a mixture of types whereby various types ride up on top of each other to form an ice layer several layers thick. Freezing rain, snow and water can bind the layers together. Composite ice becomes more abundant as winter progresses.

b) Ice Forces

Ice forces typically fall into the following categories (Wuebben 1983):

- Static ice forces — Arise from an ice sheet touching a structure subject to thermal expansion and contraction or subject to steady wind or water drag forces.
- Dynamic ice forces — Arise from ice sheets or floes that move against a structure due to water currents or wind.
- Vertical ice forces — Are due to a change in water level and require the adhesion of floating ice to structures.

c) Connecting Channel and Lake Ice

Since a connecting channel is turbulent, frazil ice is initially formed. Lake ice is often created in static conditions, although wind and currents can also cause frazil ice to form. The compressive strength of lake ice can often be higher than river ice. On lakes the forces due to wind action on the ice are usually higher than on rivers.

For rivers and lakes the ". . . times of ice formation and breakup depend mainly on the climate but are also affected by flow regime and channel geometry" (Ashton 1986). Generally, ice cover on lakes forms earlier than on rivers in the same climate, and break-up occurs later. Also, ice cover on lakes is relatively unaffected by changes in water level, whereas on rivers, the ice can be broken up by changing water levels and discharges. The duration of the ice cover on small lakes is practically the same at different points on the lake, but on large lakes the duration can differ by several weeks at different locations (Ashton 1986).

i) Lake Breakup Process

In a lake the force needed to cause the ice cover to break up is provided by the wind. Lake ice therefore tends to melt in place. Away from the shore the snow cover melts and either pools on the ice or, more frequently, drains towards the shore or through cracks in the ice. After the snow has gone, the top few centimetres of ice reflect the incoming radiation which protects the underlying ice and therefore the ice remains strong. "Radiation seems to be more important than air temperature in weakening an ice cover because radiation can penetrate the ice and be absorbed within the cover . . ." (Ashton 1986).

Nearshore ice is usually thinner because of snow drifting, proximity to the ground, and support of the cover by the banks, all of which can reduce overflow and snow ice formation. However, on large lakes subject to waves and wave spray in winter, quite the opposite may be true. "Nearshore ice also ponds snowmelt runoff from the shore and is therefore darker and dirtier, significantly reducing the surface albedo (the degree to which a surface reflects light). These two features, thin ice and low albedo, mean that the shore ice will melt first, allowing the main sheet, which is usually still strong, to be moved about the lake by winds" (Ashton 1986).

ii) Connecting Channel Breakup Process

The *Great Lakes - St. Lawrence River System* has five connecting channels or rivers, they are the St. Clair River, Detroit River, St. Mary's River, Niagara River and the St. Lawrence River. These rivers all experience ice formation and ice break, to varying degrees. Breakup does not happen simultaneously everywhere along a river; like spring, it arrives at different times in different places. Rising water levels, caused by high spring runoff and rainwater for example, and melting along the banks generally frees the ice of its restraints. As the ice begins to move downstream, it breaks up and an ice run occurs. If thawing occurs over the entire ice sheet, the sheet may be weak and thin enough that even a low flow is sufficient to fragment and move the ice. "At the other extreme, water level and flow can increase enough to float the sheet free of the bed and banks and fragment it while it is still strong" (Ashton 1986).

Ice breakup can occur early in the winter season, however, the majority of breakups occur in the early spring due to frequent temperature fluctuations and subsequent freezing and thawing.

d) Ice Jams

Ice jams are the accumulation of ice in a river or stream which reduces the cross sectional area available to carry the flow and increases the water surface elevation. Ice jams can involve floes of frazil, broken ice, or sheet ice. During ice freezeup or breakup, flooding can occur when the amount and type of ice present impedes the flow of water thereby causing the water to back up and overflow.

Ice jam potential is great wherever the quantity of both ice and water together exceeds the capacity of the lake or river to transport it. "The severity of an ice jam is a function of the preceding rate of rise of water level and velocity, the amount of ice travelling with the breakup front, and the nature of the obstacle that initiates the jam" (Ashton 1986).

Total or partial breakup of an ice cover, with the potential of causing an ice jam, may take a few days or even many weeks. For example, if the breakup process takes two to three weeks the concentration of ice and water is usually not sufficient to cause jamming. However, if the breakup process is compressed into a few days it can cause jamming. Flooding follows upstream from the ice jam and on both sides of the river as water is diverted from the ice jam (Hall 1985). It is important to note, however, that not all ice breakups and freezeups yield ice jams, and not all ice jams yield subsequent shoreline flooding.

i) Ice Jams in the Connecting Channels

Wherever ice pieces accumulate in front of and/or under an ice cover, a bending force is created which causes it to breakup. Depending on the thickness and quality of the ice, a substantial local jam may result without causing the entire ice sheet to breakup. When the ice pieces in this local jam dissociates, the ". . . pieces moving downstream to the next ice cover may cause a new local jam, and this process may be repeated a number of times before the ice reaches the river's mouth. The time required to complete breakup is extended and there is less severe jamming in any one area" (MNR 1988).

There are two types of ice jams:

- the simple or floating jam; and
- the grounded or "dry" jam.

A floating jam occurs when ". . . there is relatively unobstructed flow under the accumulation, except perhaps for a short section near the toe (downstream end), . . . and the maximum water level is governed by the accumulation thickness and the hydraulics of flow under it" (Ashton 1986). Grounded or "dry" ice jams can occur when floating ice floes become grounded or blocked by obstacles, such as, bridge piers, stable intact ice cover, or constrictions, gravel bars or meanders in the river channel.

There are many factors which increase the likelihood of the riverine breakup process and thus create ice jams. Some of those factors are degree days of melting, precipitation, rise in water levels, water velocity and ice type and are described as follows;

- Degree-Days of Melting — "One 'degree day of melting' occurs when the mean of the maximum and minimum air temperatures is +1°C" (MNR 1988). The chance of an ice jam and flooding is high if there is a consistent thaw of approximately twenty degree days of melting in a very short period of time, 3 to 5 days (MNR 1988).

- Precipitation — Depending on how much melting occurred before the rain, and depending on the water content of any snow already on the ground, the effects causing ice jams will vary. The likelihood of a river ice jam and flooding is high ". . . in the event of precipitation of 12 mm or more in 24 hours, especially if this follows several days of melting, or if the ground is still frozen, or if the watershed has a large urban area" (MNR 1988).

- Rise in Water Levels — A rise in water levels is due to increased flow. The increase in flow caused Lake Erie's water levels to rise ". . . so much that the Honeymoon Bridge was forced off its foundation and collapsed onto the ice" (MNR 1982).

- Water Velocity — The breakup process also depends on water velocity or topography of the river and water depth, together with restraints such as islands, bends or ice booms (MNR 1988). Studies indicate that a sudden increase in water velocity can cause breakup.

- Ice Type — The type of ice present in the waterway can influence the likelihood of ice jams and flooding. For example, hard, blue ice is more prone to jam than other softer forms, such as slush or frazil ice (MNR 1988).

  Anchor ice, as another example, which may seem innocuous, can also increase the likelihood of ice jams and flooding. Providing that the water velocity, flow and temperature are all very low, water flowing over the surface of anchor ice freezes rapidly. Consequently, the river tends to freeze from the bottom up, this blockage impedes the flow and causes the water level to rise. A rise in water temperature can also add to the flooding problem because some anchor ice will be dissociated, and as it floats to the surface, it can jam up at restrictions in the river.

In conclusion, the sequence of weather events and the presence of natural or man made structures can initiate the jamming process.

e) Ice Piling in Lakes

Ice piling is a dynamic event. Onshore winds cause ice floes to "pick up sufficient momentum over a length of open-water fetch to produce ice piling when they ram onto the shore or shore-fast ice" (Tsang 1974). The kinetic energy from the momentum of the incoming floes is partially transferred to the breaking of the shore ice and used to overcome the friction. The rest of the kinetic energy is converted to potential energy where it is piled up on the shore. If the ice floe has greater kinetic energy than potential energy the ice will continue to go overtop of the pile where it will break up and fall to the front of the pile. However, if the ice floe which goes over the top runs into an obstacle all of the energy is then transferred to the object which can cause great damage.

Ice piling can occur by either overriding, undersliding or rafting. See Figure A3.6.1 and the following descriptions from Tsang (1974).

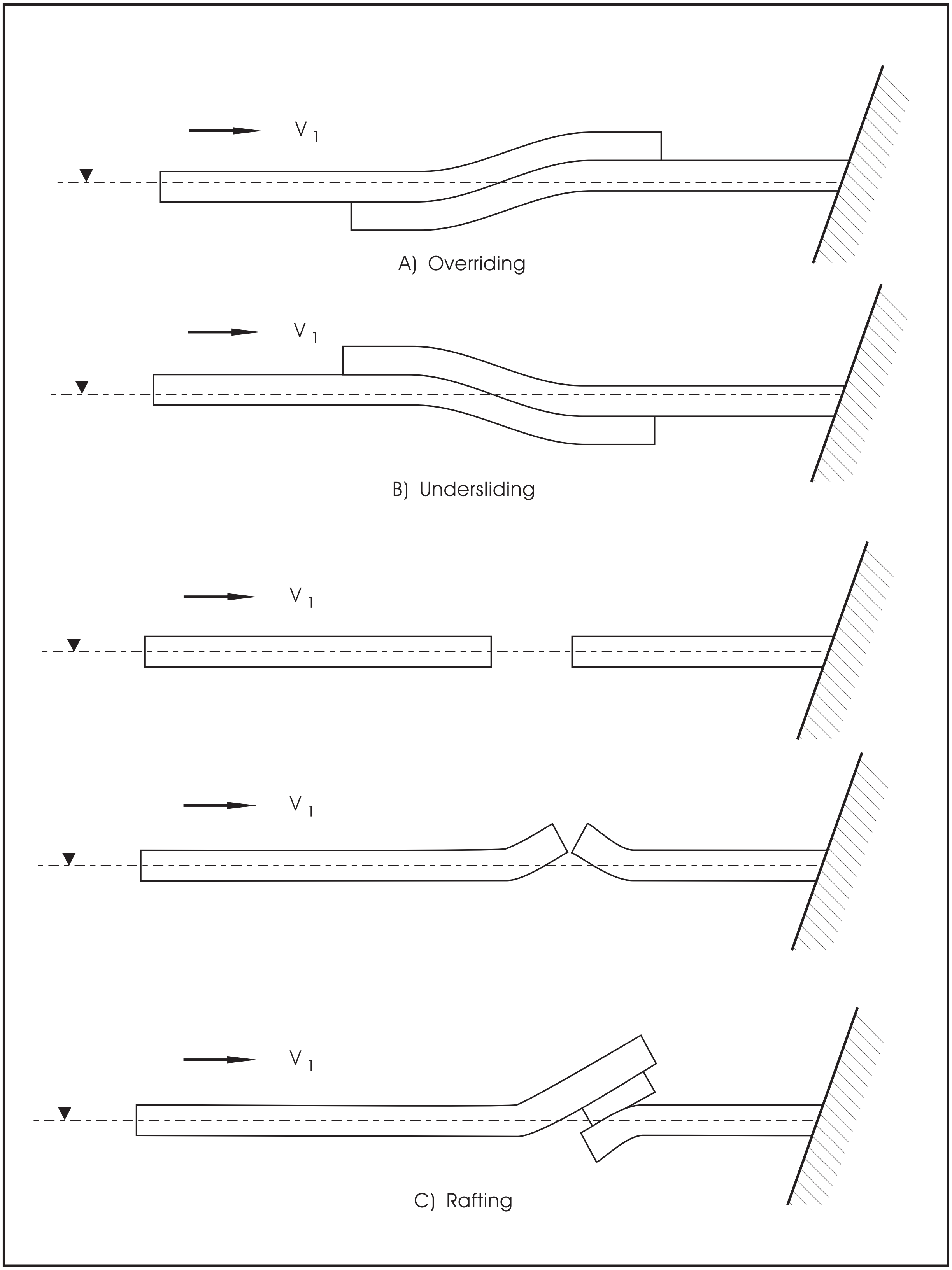
- Overriding — Overriding occurs when the upwind floe rides on top of the downwind floe. This usually occurs when the wind is strong and the water surface is rough.

- Undersliding — Undersliding occurs when the upwind floe slides under the downwind floe. Undersliding does not occur as often as overriding and when it does the piling seems to stop.

- Rafting — The drifting ice will run into the shore-fast ice and buckle and break off. The broken piece will then be overridden by the upwind ice floe and caught between the downwind ice floe. Rafting is the most common method of ice piling when the wind is not strong and there is little wave action.

An analysis by Tsang found that "a strong onshore wind is not a requirement for ice piling on lake shores. An open-water fetch is necessary for ice floes to gather sufficient speed for ice piling. A shift of wind from the offshore direction to the onshore direction over a short period allows the ice floes to use the whole open-water fetch to gather speed, and consequently promote ice piling." Winds and wave action help to breakup the ice which provide conditions for ice piling. It was also found that ice piling will not occur below freezing temperatures.

Figure A3.6.1: Three Modes of Telescoping Ice Floe



f) Ice Conditions on the Great Lakes and Connecting Channels

Figure A3.6.2 shows the estimated maximum ice cover for the Great Lakes. Through consultations with the various local implementing agencies bordering the *Great Lakes - St. Lawrence River System* , areas prone to ice jams and ice piling where identified and are illustrated in Figures A3.6.3 to A3.6.6. This information is provided for illustrative purposes only, for detailed information on the extent and magnitude of concern within a particular jurisdiction the appropriate local and federal agencies responsible for shoreline management should be consulted.

i) St. Mary's River

According to Carey (1980) the ice conditions in Reach 1 (Figure A3.6.7), from Lake Huron to the head of Lake Munuscong, are essentially as in a Lake, with extensive shorefast ice and a continuous, uniform stable ice cover. The ice cover may move vertically due to water level fluctuations on Lake Huron, and in the spring, drifting floes may move horizontally due to wind action. This shore ice forms in Reach 2 (Figure A3.6.8), and natural uplift due to water level fluctuations on Lake Huron move this ice vertically. Broken ice moves through the reach during spring breakup.

In Reaches 3,4,5 and 6 (Figure A3.6.8), thick, table ice forms a continuous cover. During spring breakup, broken ice moves through these reaches.

Reach 7 (Figure A3.6.9) is a transition zone between a generally stable, continuous ice cover downstream and generally open water upstream. A stable ice cover and shore ice form in the side channels and around the islands. Under natural conditions the Little Rapids Cut may have shore ice or may become ice covered. Broken ice passing downstream through open water progressively fills out the Little Rapids Cut with compacted ice.

Reach 8 generally has open water with shore ice. Shore ice and often a more extensive ice cover form in reach 9. This ice moves horizontally under the influence of wind during breakup (see Figure A3.6.9.)

ii) St. Clair River

According to Carey (1980), the south channel in reach 1 (Figure A3.6.10) is subject to the formation of stable shore ice, extending generally out 1.8 m (6 feet) or to the end of the shore structures. Otherwise, it becomes ice filled only after Lake St. Clair freezes over, and floe ice coming down the St. Clair River progressively covers the channel from the south end northward. Horizontal movement of the shore ice is negligible. Vertical movement of the shore ice, due to wind-induced changes in the level of Lake St. Clair, is confined to the early season when shore ice is thin. Thus, the vertical forces resulting from this movement are negligible. Also, large level changes do not generally occur due to ice jamming, since jams form upstream of the South Channel. During the spring, ice moves out of the channel along a shear zone to the offshore edge of the shore ice. Later, when the shore ice is melted, the moving floe ice generally remains confined to the deeper (>1.8 m (6 feet)) parts of the channel and does not interfere with shore structures.

Ice jams consistently form in the main channel in this reach due to the accumulation of ice floes coming downstream from Lake Huron. The frequency and severity of jamming is highly dependent on the supply of ice from Lake Huron. Pressure in the floe field forces ice pieces up on edge and produces piling and layering, so that the thickness of the jam may reach 2.4 to 3.0 m (8-10 feet). The jam stabilizes by freezing together, and when weather or river conditions allow it to break and release, the movement and turning of the ice damages structures. Level changes resulting from the jam may be as much as a 0.3 to 0.6 m (1 to 2 feet) increase in stage. This causes uplift forces on frozen structure piles. Ice conditions in reaches 2 and 3 are the same as in reach 1, but to a diminished degree.

Reach 4 is upstream from locations where ice jams commonly occur. Shore ice normally forms in this reach, but the principle form of ice in the reach is unjammed ice floes and brash floating downstream. Ice floes may be released in quantity at times from Lake Huron, or they may be sparse as a consequence of the formation of a natural ice bridge at the mouth of Lake Huron. Ships penetrating the ice bridge can increase the amount of floating ice in the reach, until the ice bridge re-forms. Little or no damage to structures generally occurs due to ice north of Fawn Island at the southern end of the reach.

Figure A3.6.2: Estimated Maximum Ice Cover for the Great Lakes

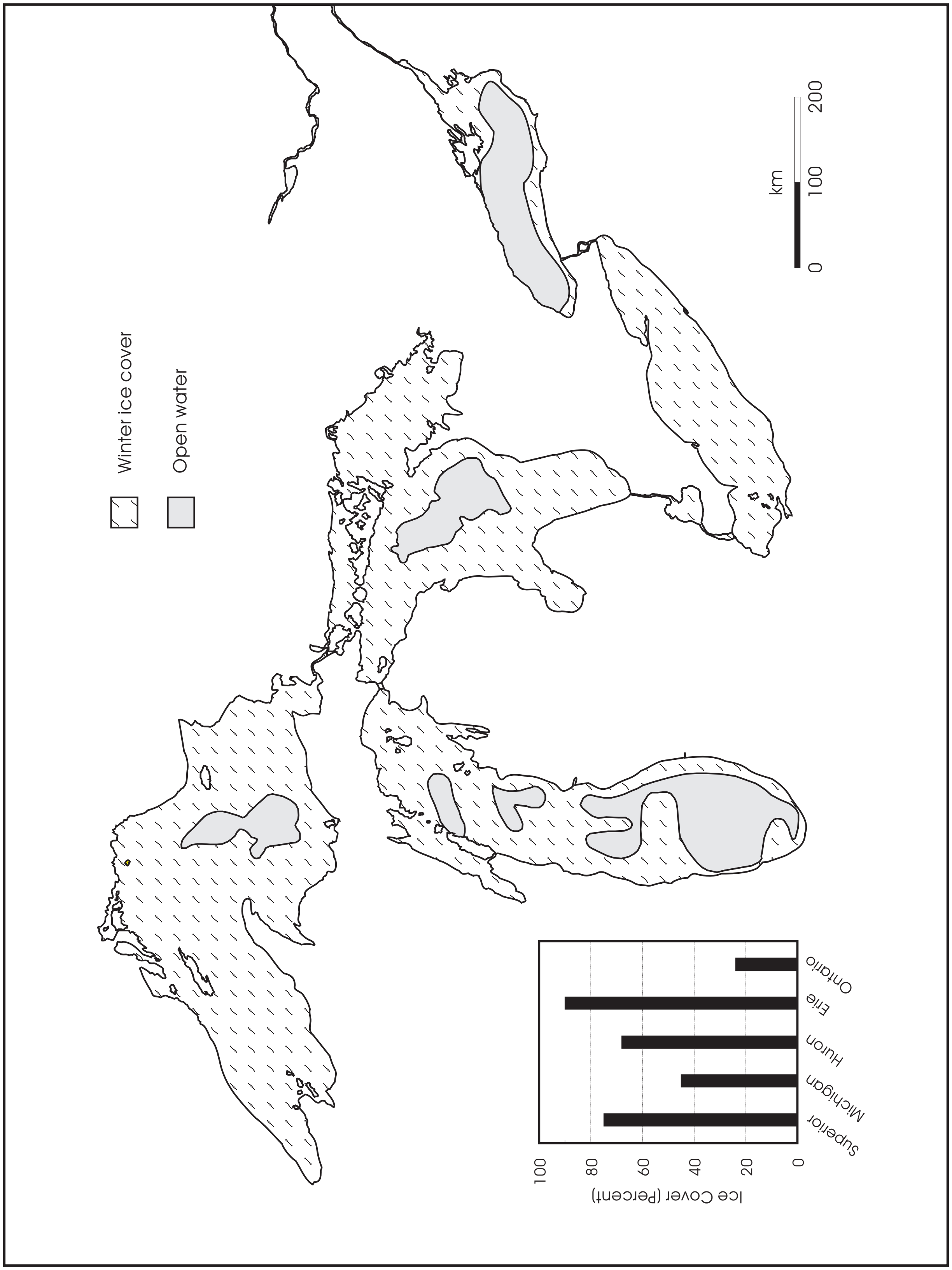


Figure A3.6.3: Ice and Ship Wave Flood and Erosion Prone Areas: Lake Superior

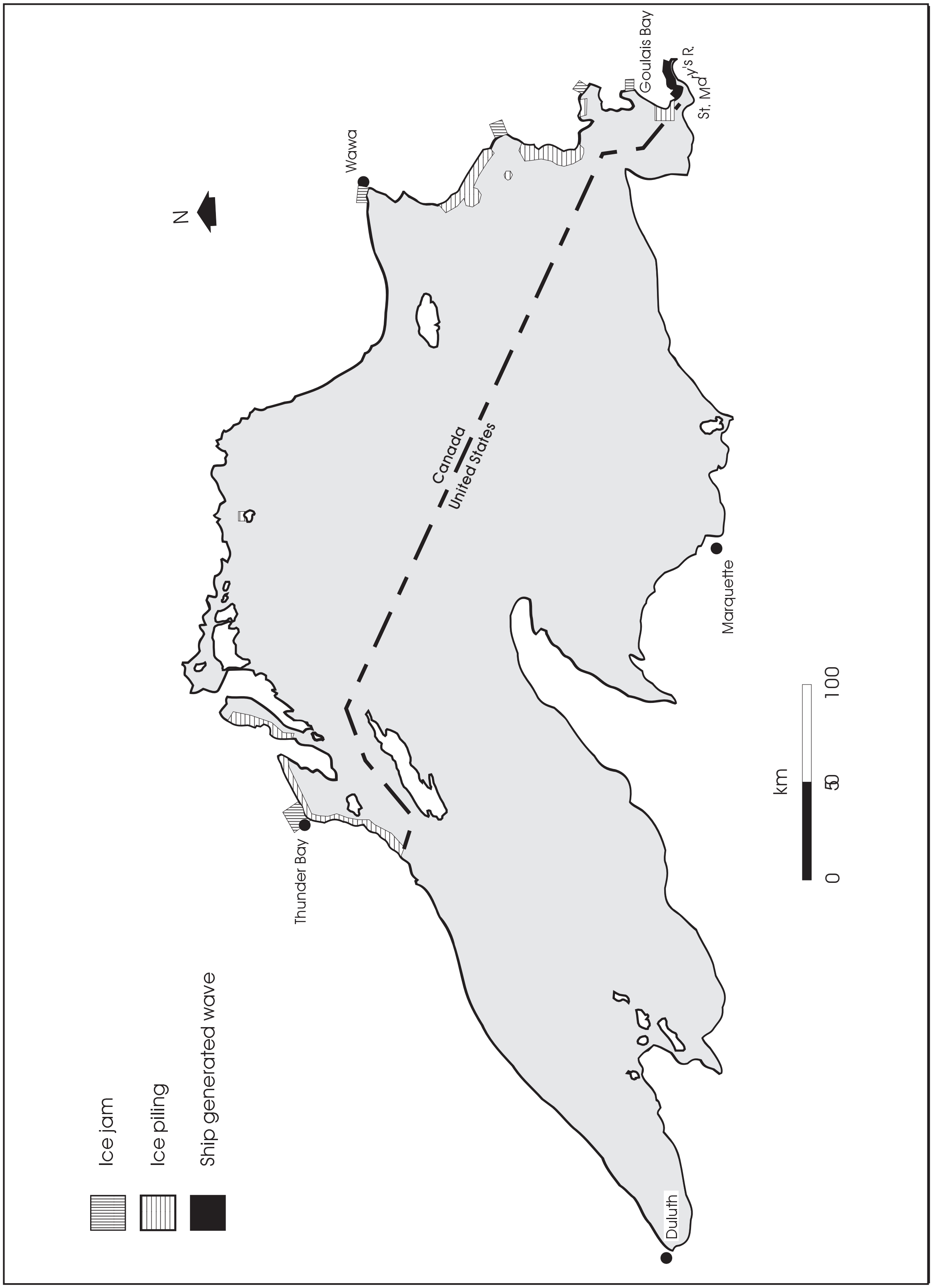


Figure A3.6.4: Ice and Ship Wave Flood and Erosion Prone Areas: Lake Huron

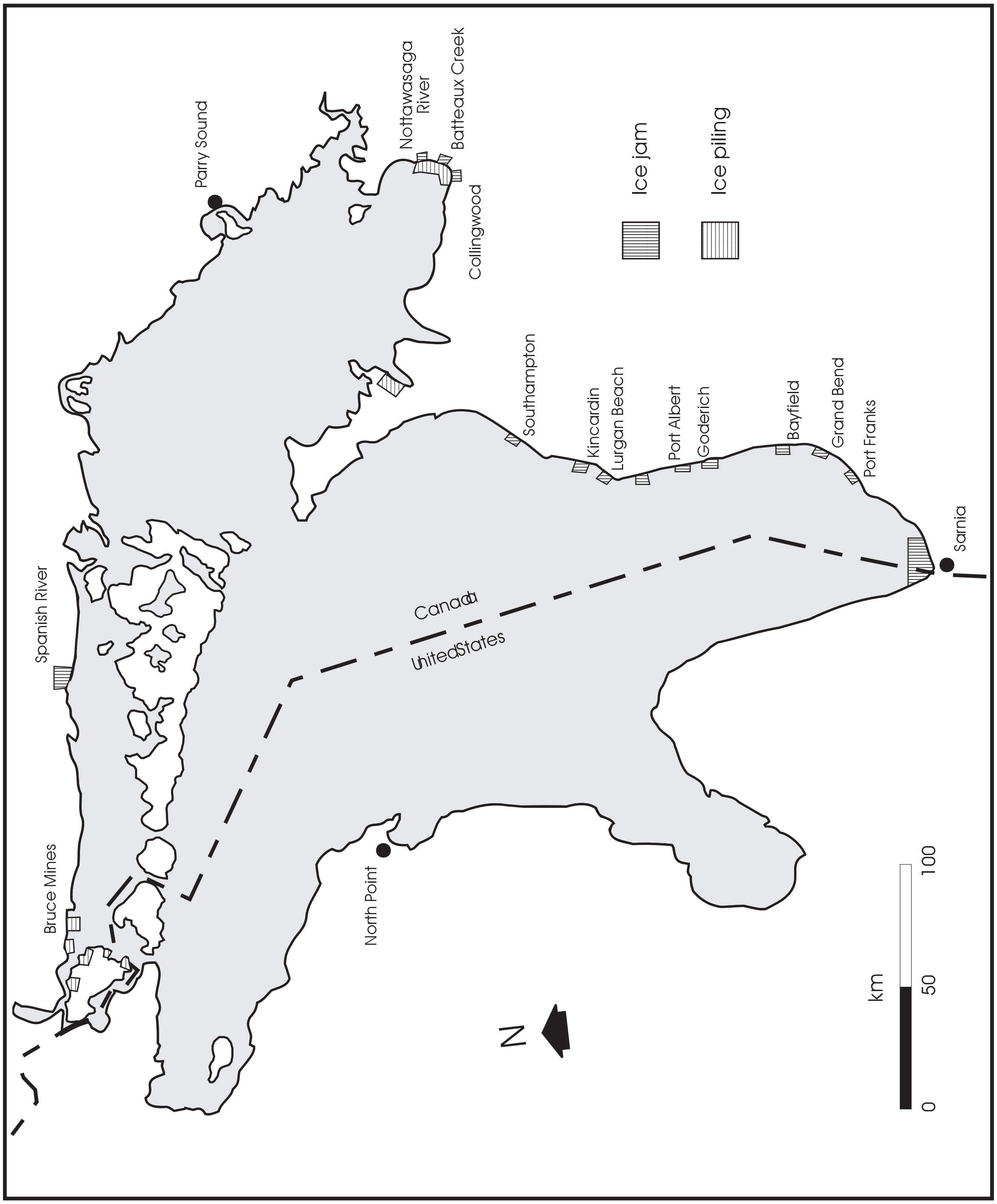


Figure A3.6.5: Ice and Ship Wave Flood and Erosion Prone Areas: Lake Erie

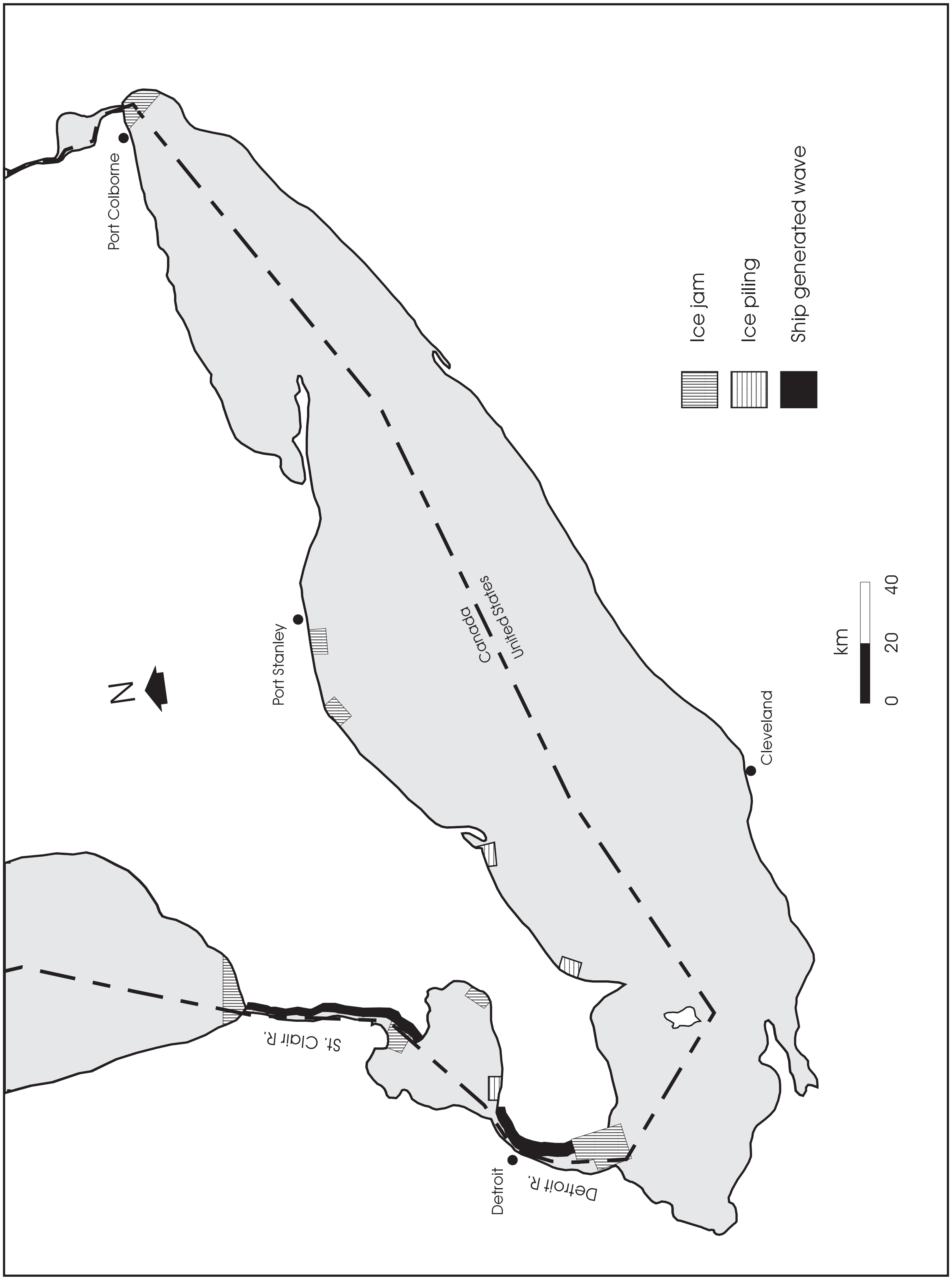


Figure A3.6.6: Ice and Ship Wave Flood and Erosion Prone Areas: Lake Ontario

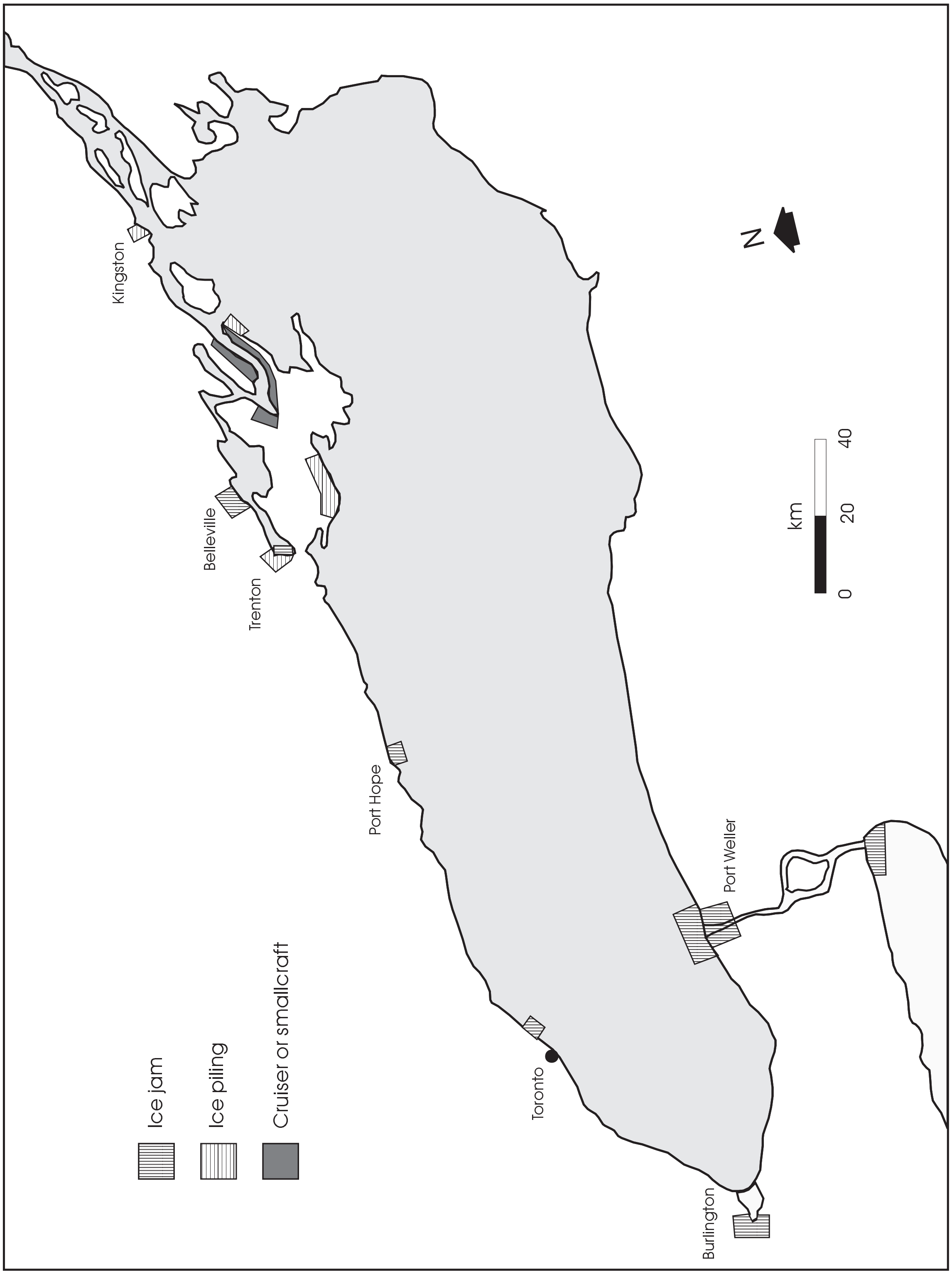


Figure A3.6.7: St. Mary's River Reaches (Reach 1)

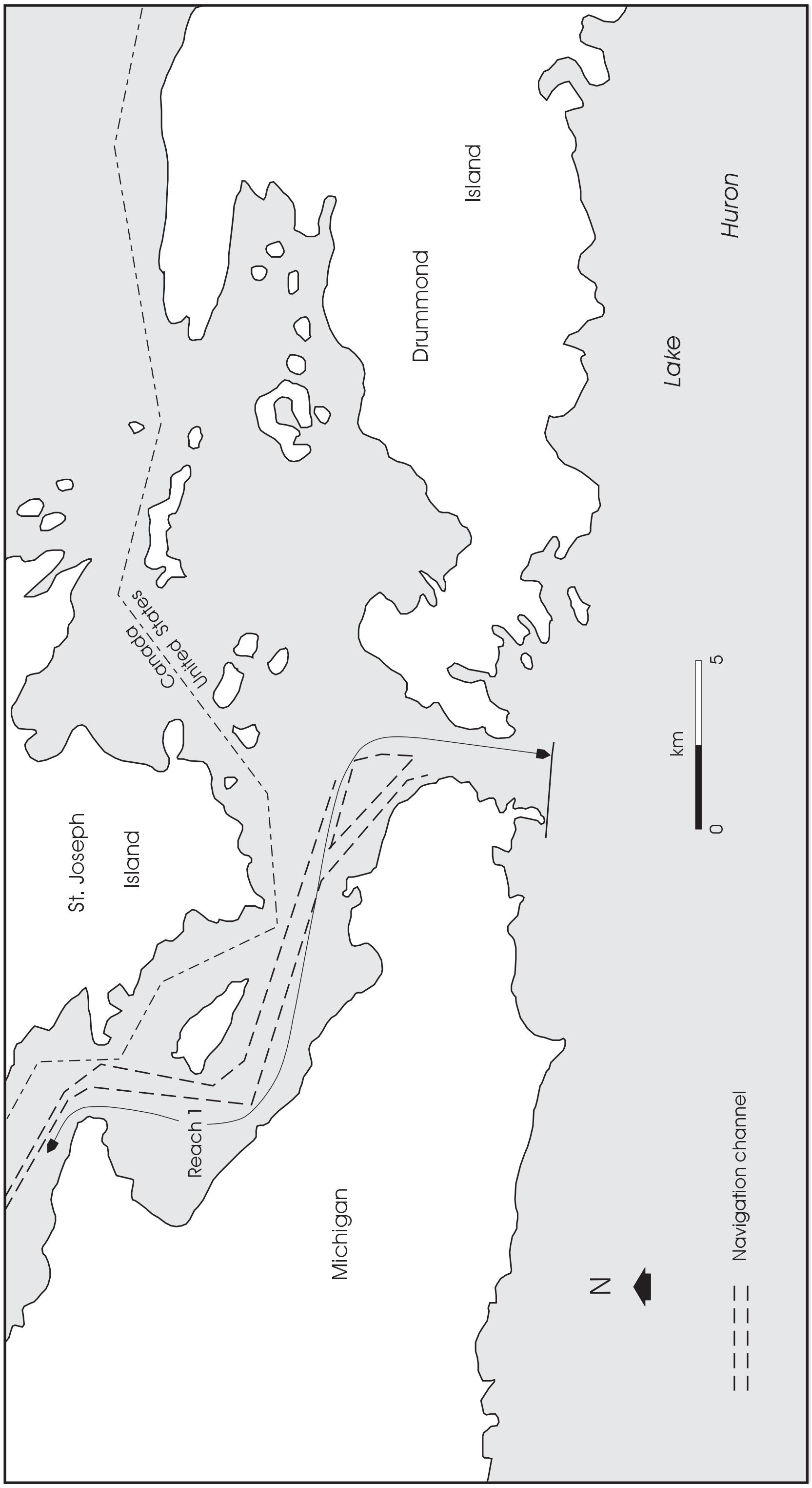


Figure A3.6.8: St. Mary's River Reaches (Reaches 1 to 7)

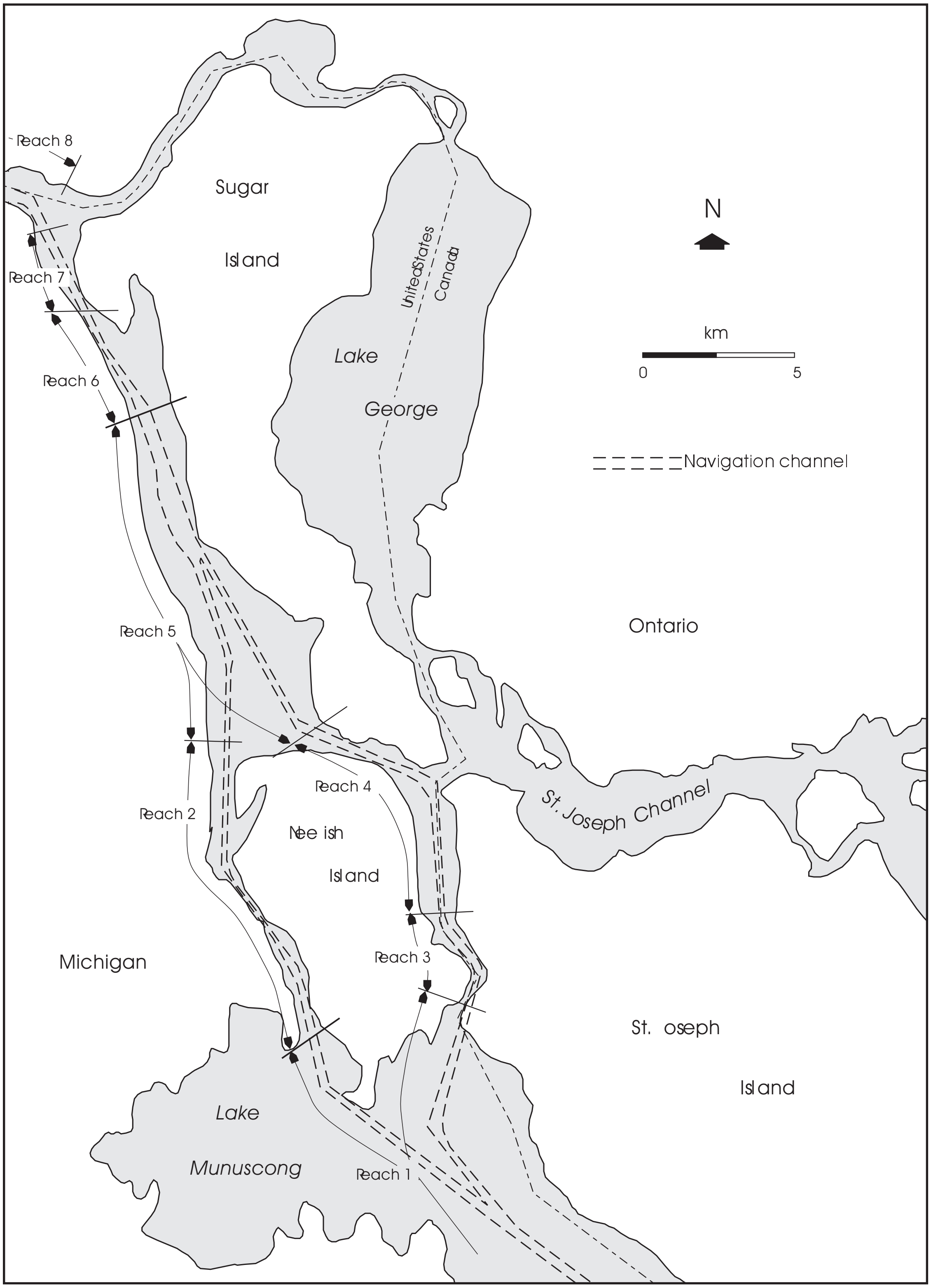


Figure A3.6.9: St. Mary's River Reaches (Reaches 6 to 9)

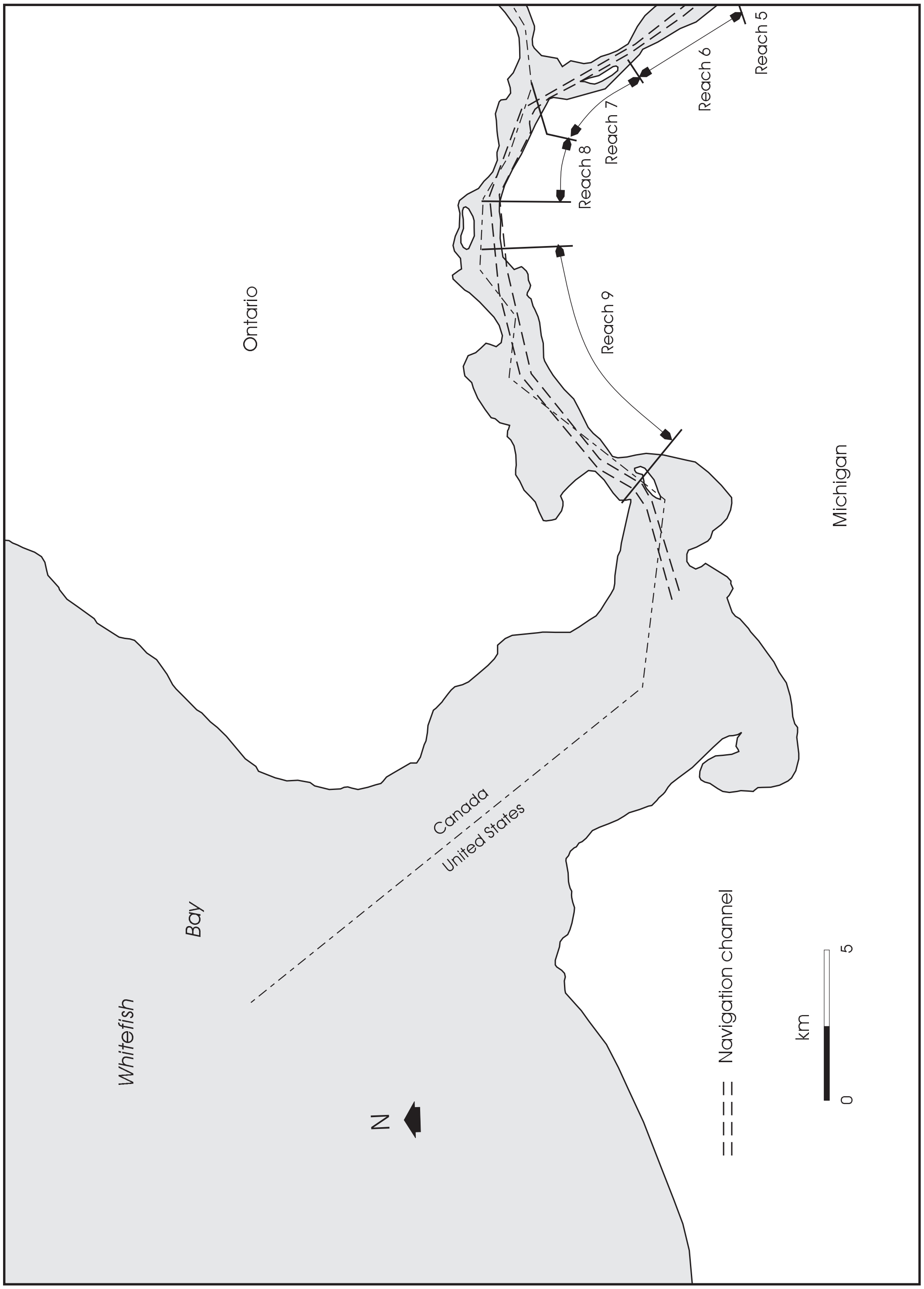
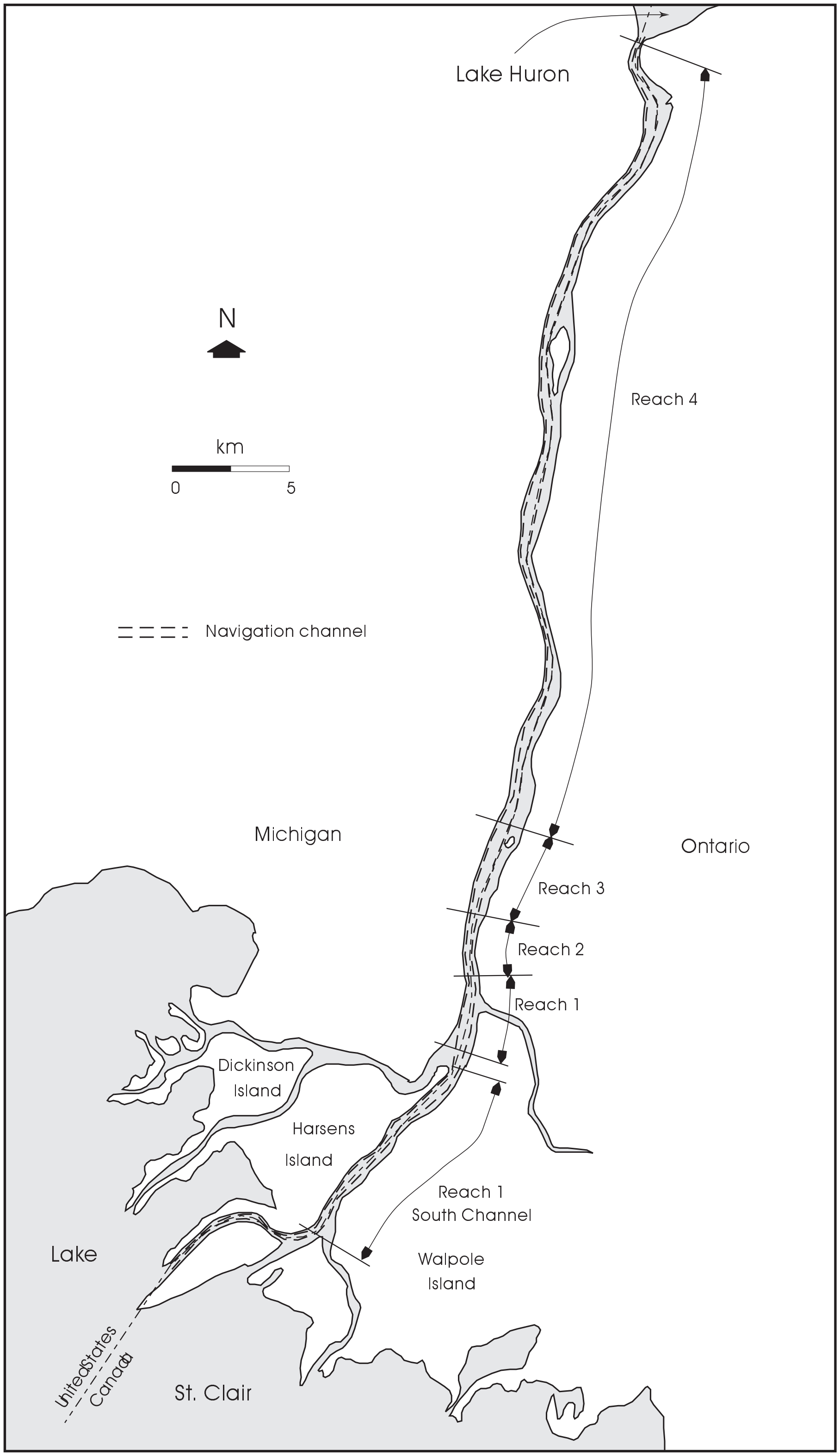


Figure A3.6.10: St. Clair River Reaches



iii) Detroit River

A level of ice cover is never formed in the main channel of the Detroit River (Figure A3.6.11). Generally, the river is ice-free except for small amounts of shorefast ice and occasional jam-ups of ice floes from Lake St. Clair in the Peach Island - Belle Island area and in the Fighting Island - Grosse Ile area.

A3.6.2 Ship Generated Waves

Ship generated waves are a common occurrence for people interacting near harbours, navigational channels, or other bodies of water where ships operate. The St. Lawrence River and Seaway, St. Clair River and St. Mary's River have encountered problems in the past and can be considered susceptible to ship waves. In these areas, the *flooding hazard* limit would need to be adjusted landward to the limit of the ship wave action. Figures A3.6.3 to A3.6.6 illustrate the areas prone to ship wave action along the Great Lakes - St. Lawrence River System. As stated previously, the information in these figures is provided for illustrative purposes only; for detailed information the appropriate local and federal agencies responsible for shoreline management should be consulted.

a) Ship Wave Process

Waves generated by commercial ships and recreational boats can be the cause of erosion on the banks of the navigation channels and can produce wave agitation problems at adjacent ship berths or marinas. The magnitude of the ship waves depends on the size of the vessel, the hull shape and the relative speed of the ship. However, the largest waves are not necessarily produced by the largest ships (Figure A3.9.12). A complete description of ship waves and their impact on the shore will involve obtaining details of the number of ship passages, the type of vessel and the ship's speed.

Surface waves, including short and long periods waves, are created as a ship moves across the water surface. The surface waves are essentially a function of:

- the speed of the ship;
- the dimensions, hull shape and draft of the ship;
- water depth of the ship;
- distance of the ship's sailing line to the shore; and
- the width and depth of the channel.

As a ship moves over the water, water is pushed in front of the moving ship creating a series of diverging and transverse waves (see Figure A3.6.13). The diverging waves are the V shaped waves while the transverse waves travel normal to the sailing line. The surface waves form a constant pattern and meet to form a locus of cusps with the angle of ϕ with the sailing line. In deep water, the angle (ϕ) is about 19 to 20 degrees (Johnson 1958) and becomes greater in shallow water. The maximum wave height, commonly referred to as H_{max}, occurs at the locus of the cusps and is defined as the maximum vertical distance between any given wave trough and the following crest. The half-period (T/2) is the time it takes for the maximum crest of the wave and the next trough to pass (see Figure A3.6.14). The wave heights decay, caused by the distribution of energy along the crest of the waves, as the waves propagate away from the ship (Sorenson 1973).

As a ship pushes through the water, a bow wave or surge is created in front of the ship. The nearshore may experience an increase in the water level (i.e., surge) as the ship initially passes alongside the shoreline or connecting channel. The water is then drawn from in front of the ship and the channel sides to fill in the moving trough created by the passing vessel (Zabilansky 1992). This phenomena, commonly referred to as "drawdown", results in a drop in the water level on the nearshore. As the speed of the ship increases the trough deepens. The water level then raises again and is gradually returned to normal after the ship waves decay (see Figure A3.6.15). The vessel speed has the greatest effect on the surge and drawdown. The distance from the navigation channel, channel and shoreline geometry, ship draft and beam are also important factors.

Figure A3.6.11: Detroit River Reaches

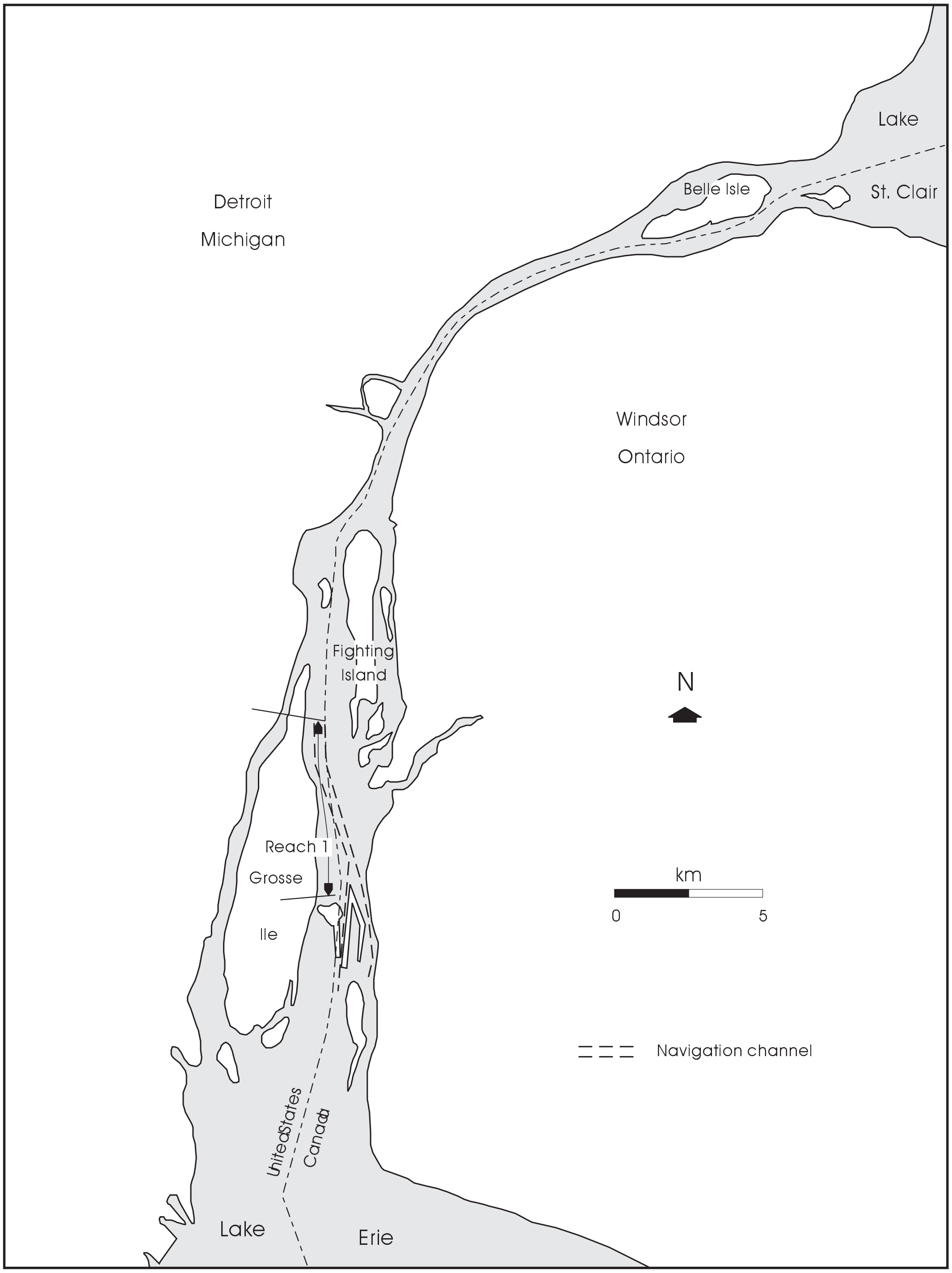
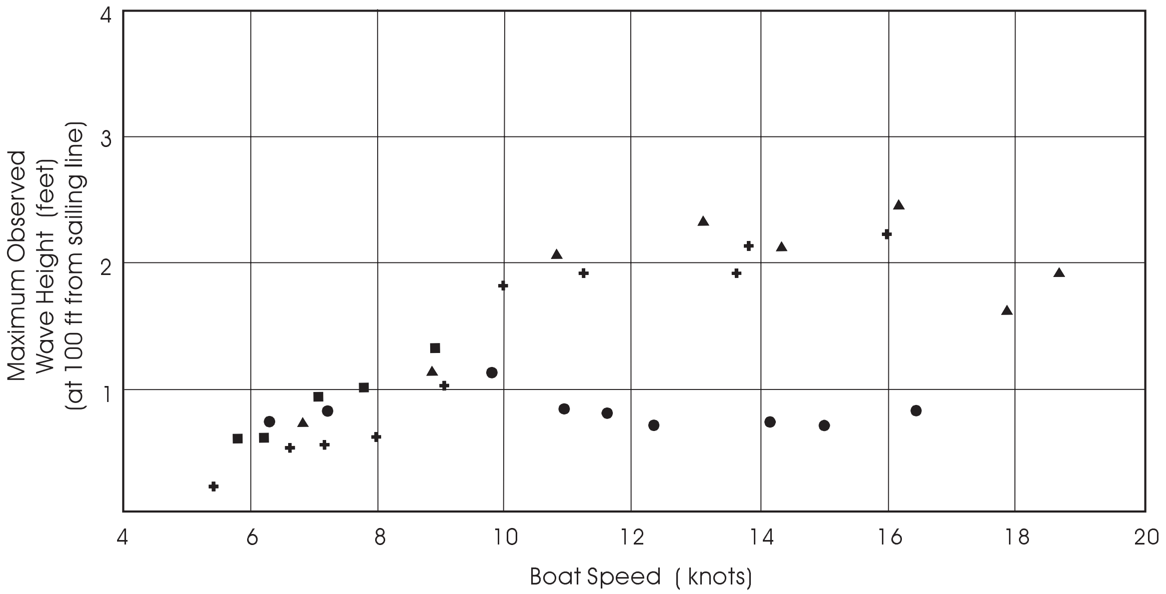


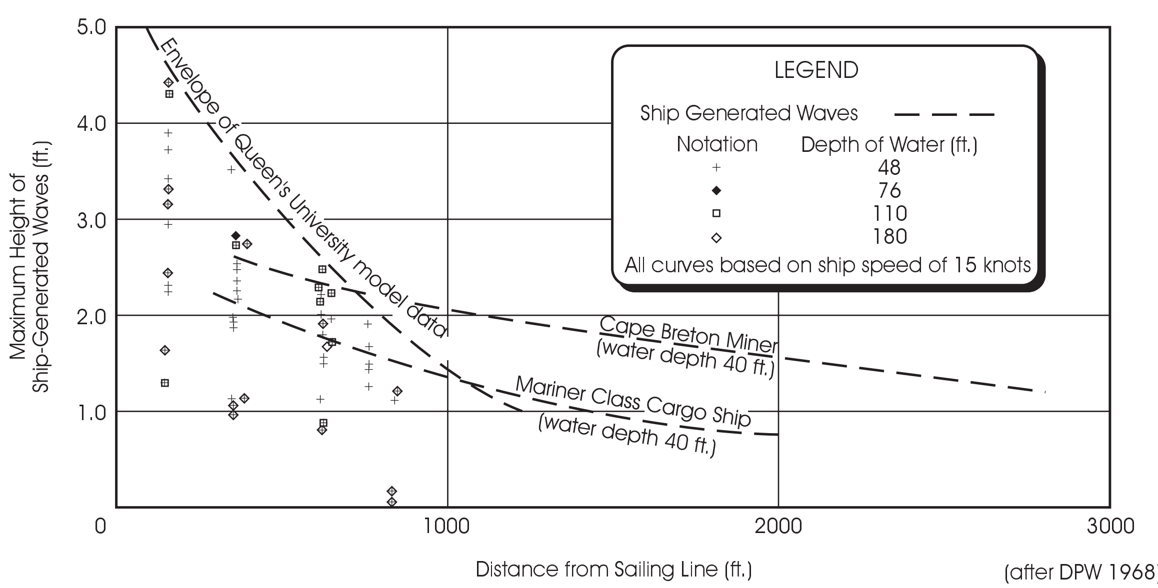
Figure A3.6.12: Maximum Observed Heights of Waves Generated by Various Vessels



Maximum observed wave periods: 2.8s and 3.8s.

	Length (ft.)	Draft (ft.)	Beam (ft.)	Approximate displacement (tons)
■ Tugboat	45	6	13	29
▲ Coast Guard cutter	40	3' 6"	10	10
● Cabin cruiser with V-bottom hull	23	1' 8"	8' 3"	3
✚ Fishing boat	64	3	12' 10"	35

(MNR 1981, after Sorenson 1967)



(after DPW 1968)

Figure A3.6.13: Typical Pattern of Wave Crests Generated by a Ship Moving in Deep Water

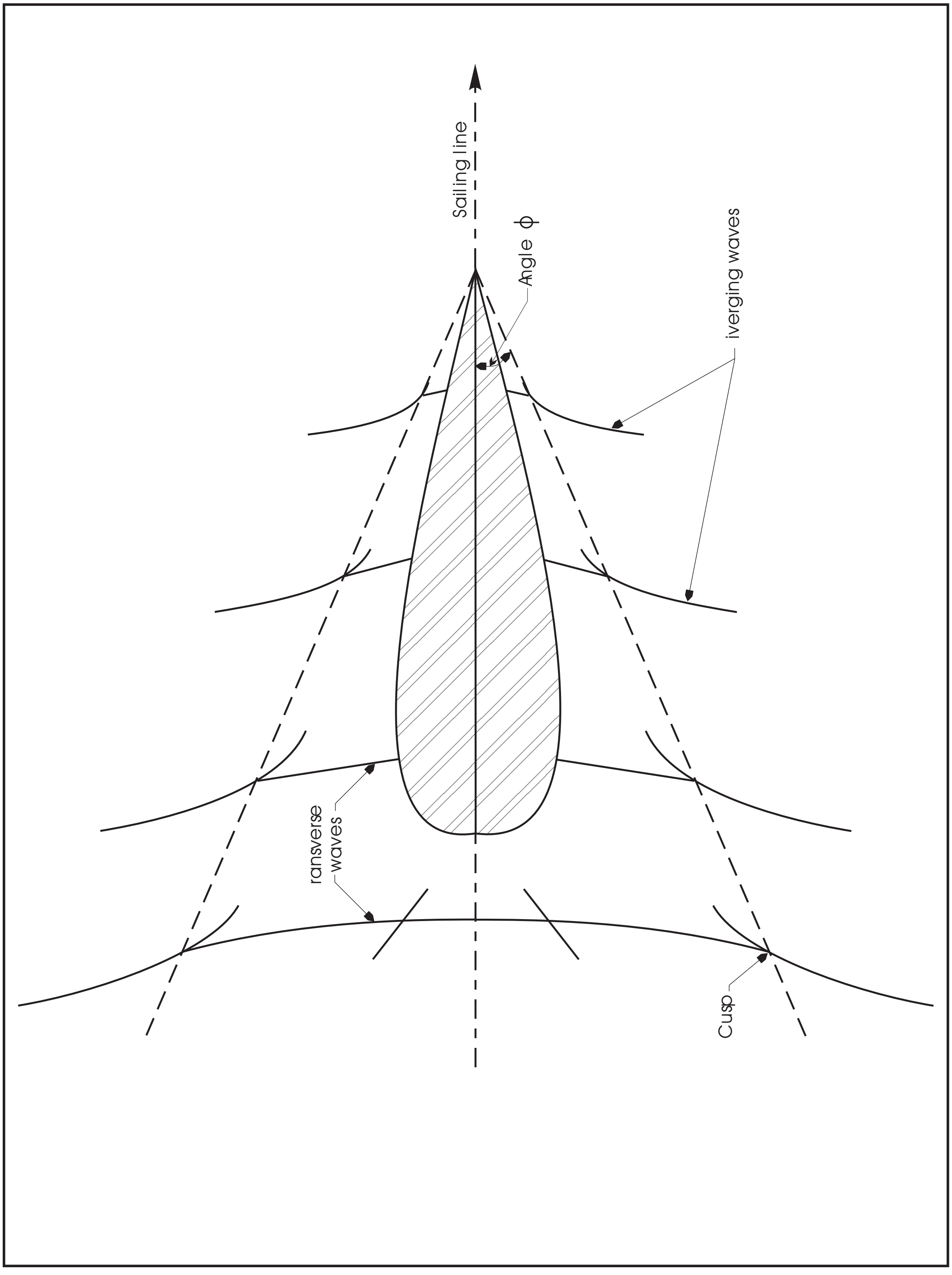


Figure A3.6.14: Typical Ship Wave Profile

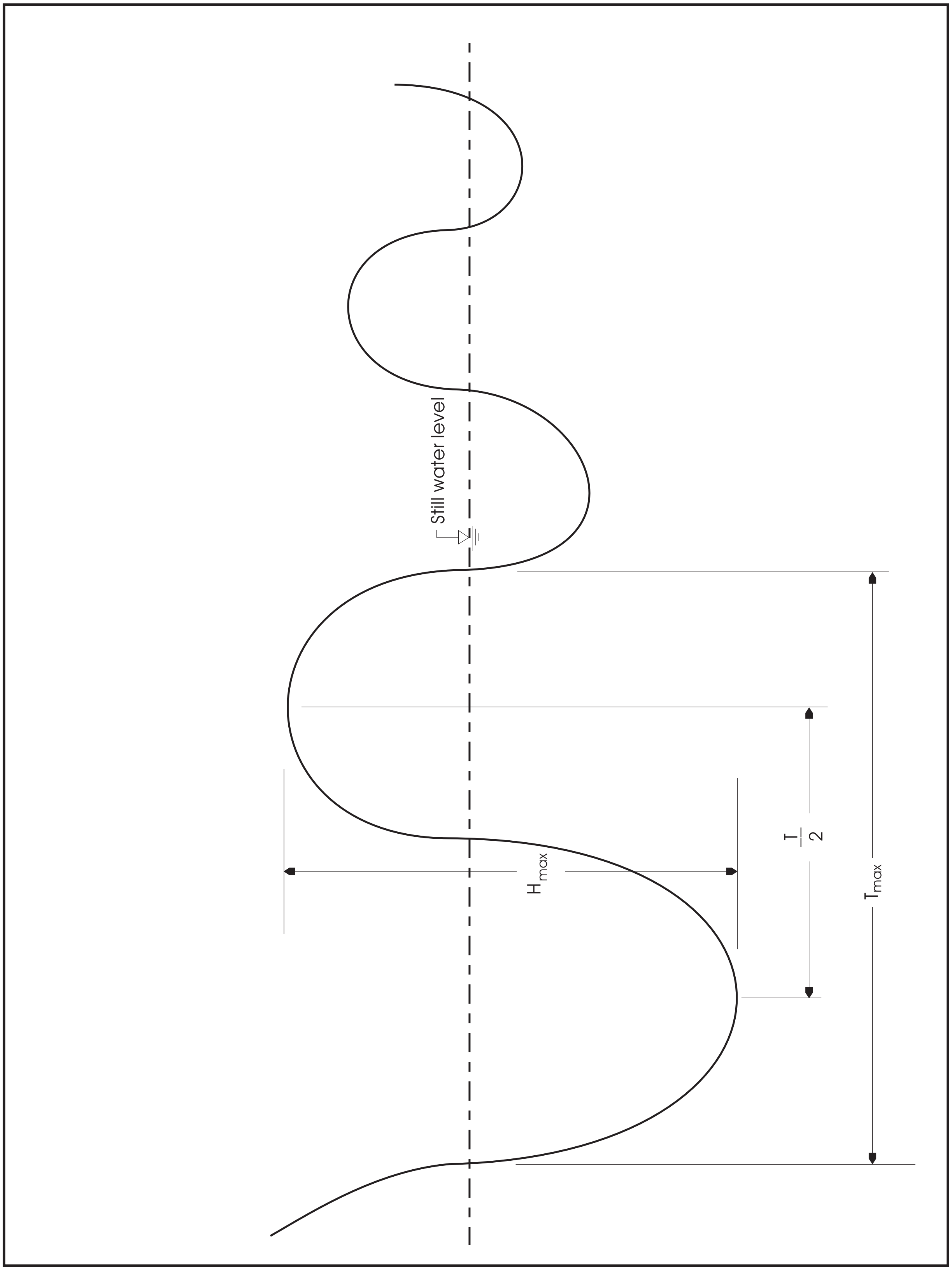
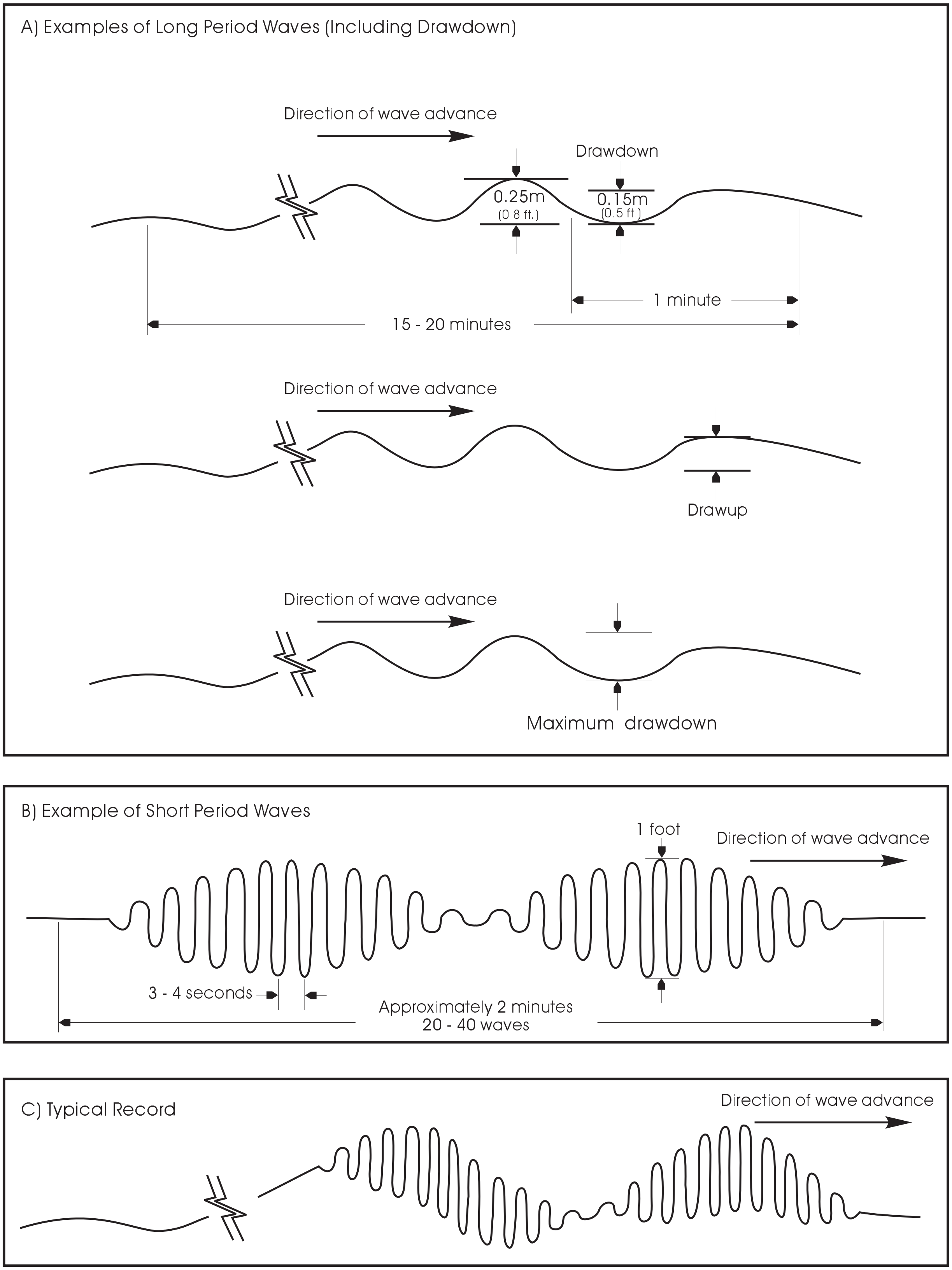


Figure A3.6.15: Examples of Wave Records



As the ship passes large and rapid changes can occur in the velocity and direction of flows, particularly in the connecting channels. Along with the changes in the channel velocity, the water level changes can occur so quickly that the pore pressure in the soils will be unable to adjust. These rapid changes can initiate sediment transport and result in shoreline erosion. If there is a decrease in the water pressure on the channel bed while the trough is moving and the trough moves faster than the pore pressure can change, a net uplift force on the soil surface will result. After the trough passes the water level rises, the opposite effect occurs and there is a net downward force. This continues and is combined with gravity acting down the slope initiating a net offshore movement of sediment (Wuebben 1983). This process does not include natural currents or water velocities.

b) Methods of Calculation

Few studies have been carried out on ship waves in the *Great Lakes - St. Lawrence River System* . Where studies do exist, they were initiated by the Federal Government to assist them in determining a method of distributing the responsibility and costs associated with erosion to the appropriate government departments. Depending on the cause of the shoreline erosion, either ship (i.e., navigation) or wind (i.e., natural) generated waves, various agencies would be responsible for repairs and costs.

Within the United States, where studies exist, they have been conducted by the U.S. Army Corps of Engineers. As in Canada, little work has been carried out specifically on ship waves. Studies that were conducted by the U.S. Army Corps of Engineers were developed to examine the displacement of sediment in the channel and its effect on the environment and as such their primary focus was on flow velocities resulting from ship traffic rather than on the actual ship waves.

i) Canadian Studies

At present the Federal Government uses an "energy method" which predicts the ship and wind wave distribution in terms of erosion energy. Field studies must be carried out at the specific sites to use this method. The energy studies cannot predict the range of possible ship generated wave heights, however, they do use the measured waves from the site to estimate the amount of erosion that can occur as a result of the ship generated waves. In essence, the energy method uses the short period wave characteristics of the ship waves in predicting the erosional energy. What the energy method does not take into consideration are the long period waves (i.e., surge and drawdown) which can be important in narrow navigation channels.

In 1992, a study was carried out by Y. Ouellet at the University of Laval for Transport Canada and Public Works Canada. Based on earlier methodologies of the 1960's and subsequent work by A.O. Ofuya for the Department of Public Works Canada (1970), the 1992 study methodology uses the relative energy of the ship and wind waves.

To provide a more detailed discussion of ship generated wave studies undertaken in Canada, a description of the general methodologies and the general procedures for their application is warranted. The following subsections provide an outline of this information as it relates to three separate studies including:

- Ofuya (1970);
- Baird (1973); and
- Ouellet (1992).

· Ofuya (1970)

The "energy method" developed by Ofuya assumes that "the quantity of material eroded by wave action is directly proportional to the total energy propagated to the shorelines by both the wind and ship waves" (Ofuya 1970). In order for erosion to occur and sediments to be removed, work must be done on the shoreline.

H_{max} and T_{max} are considered to be the most representative features of a ship wave profile (see Figure A3.9.7). The short period waves were used for the calculations.

The magnitude of the energy depends on the distance from the sailing line. The waves decay as they propagate away from the ship. Ship waves must be measured for each particular site taking into consideration the distance from the sailing line. "The wave energy propagated to the shoreline by a vessel can be expressed as a function of H_{max}, T_{max} and distance from the sailing line".

Other information required for using this method includes; bathymetry, speed of ships or cruisers, width and depth of the channel, ships' short and long period waves, wind waves, as well as a description of the marine traffic at the site (e.g., the time when ship passed the site, direction, distance of the ship from the gauge, bow and hull shape, beam, draft and length).

Ofuya's theory defines a "threshold of energy" with respect to the wave power or H^2T that is required in order for the waves to initiate erosion of the shoreline. This applies to ship, cruiser and wind generated waves. "Since H and T are independent for non-breaking waves, the choice of a threshold based on H^2T requires that H and T be selected independently."

In Ofuya's 1970 study, the threshold was only applied to the wind waves, it was assumed that the ship and cruiser wave energy was high enough to cause erosion. It was also decided that the $T_{1/3}$, significant period of the wind waves be at least of the same order of magnitude as the T_m mean period of the ship and cruiser waves.

Ofuya included an assessment of cruiser waves in his study as being separate from ship waves. Cruiser waves may be important in the implementation of the Provincial Policy depending on local conditions and as such, may need to be assessed. The importance and application of this information is to be determined on an individual site basis.

For more detailed information on this method and general conclusions of the study, Ofuya (1970) should be consulted.

- **Baird (1973)**

In 1973, Baird, using Ofuya's (1970) energy method, completed a study using the short period ship waves, however, he did not consider cruiser waves or use a threshold of energy for the wind or ship generated waves.

Baird (1973) also used field data to calculate wave energy at a specific site. This method involves measuring the maximum wave height for a series of ships at a given distance from the sailing line. The average of all of the maximum wave heights is taken for the upbound ships and then the same was done for the downbound ships. The maximum period is acquired from the field data and the number of ships passing per hour is determined from available data.

The ratio of $H_{max}^2\ T_{max}$ to Energy is calculated. From this information the average ship generated wave energy arriving per hour, per foot length of the bank is calculated. The wind wave energy is also calculated to determine the appropriate distribution of erosion between the ship generated and wind generated waves. In following the method by Ofuya (1970), the ship and wind wave energy results are used to distribute the amount of erosion which has occurred.

For a more detailed description and discussion of the study see Baird (1973).

- **Ouellet (1992)**

The 1992 study initiated by Ofuya also used the calculation of the energy in order to assess the erosional forces. The limit of threshold of erosion and cruiser waves were not used or measured under this study methodology in order to calculate energy.

The Ouellet (1992) study involved determining the average energy of the short period waves for one minute. The maximum wave height, maximum wave period and the standard difference for one minute of this section of record were determined from the field data. A standard difference of another section of a record which was not influenced by the ship wave was determined for the wind wave. The average energy of the ship waves was then compared with the total energy of the ship and wind waves. The energy produced by the ship waves and the energy due to the wind waves was calculated. The distribution of erosion was associated with the ship and wind wave energy. For a more detailed description and discussion of the study see Ouellet (1992).

ii) United States Studies

In the interests of improving and optimizing the use of their waterways, the U.S. Army Corps of Engineers have developed a numerical model to predict the forces which are created when ship traffic passes along the waterways. Their ultimate concern is to study the forces which are created and "can have an adverse impact on the natural fluvial environment. These forces are developed, in part, as a result of propeller jet and displacement or backwater flow velocities" (Hochstein and Adams 1986). As such, their analysis includes the effects of the ships propeller jet and backwater velocities along with the resulting ship waves.

The model was developed by A. Hochstein and C. Adams Jr. in 1986 to specifically analyze the environmental effects that would result if the navigation season was extended in the St. Mary's River. Although specific in nature, the model could be used in other similar locations provided its limitations were clearly recognized.

To address these limitations, the model was revised in 1992 by E. Foltyn to allow for more general applications. More inputs for the flow velocities, descriptions of the channel geometry and characteristics were allowed. The 1992 revisions improved the calculations of the ship waves from the 1986 model by using the theory that "the divergent wave height at the sailing line should be a function of the velocity head of the ship".

Saunders (1957) has shown that this wave height is:

$$H = K_w \left(\frac{B}{L_{ent}} \right) \frac{V^2}{2g} \qquad (Imperial\ \ Units)$$

where:

H = the divergent wave height above the SWL(stillwater level), ft
K_w = a shape factor of the ship; = 2.6 for cargo ship and tankers (Saunders,1957);
B = the ship beam, ft;
L_{ent} = the entrance length of ship, ie: the length from the bow of the ship to the maximum beam, ft;
V = the ship speed, relative to the water, ft/sec;
g = the gravitational acceleration, 32.3 ft/sec^2 " (Foltyn 1992).

The divergent wave length and wave period are solved by the iterative process using the wave celerity (C) equation. The divergent wave height and period are the same parameters as the short period wave height and period referred to in the Canadian studies.

$$C^2 = \frac{L^2}{T^2} = \frac{gL}{2\pi} \tanh \left(\frac{2\pi h}{L} \right) \qquad (Imperial\ \ Units)$$

where:

L = the wave length, ft;
T = the wave period, sec;
h = the depth of channel at sailing line, ft;
Tanh = the hyperbolic tangent function.

The actual maximum water particle (U_{max}) velocity is then determined from the wave length and wave period at a depth *h*.

$$U_{max} = H \frac{2\pi}{T} \frac{1}{\sinh\left(\frac{2\pi h}{L}\right)} \qquad (\textit{Imperial Units})$$

where:

H = the wave amplitude from the first equation of this section.

From the conservation of mass and energy equation the ship drawdown is then calculated. The drawdown is the same as the long period wave referred to in the Canadian studies.

$$h_1 - h_2 = \frac{V_1^2}{2g}\left[\left(\frac{A_1}{A_1 - A_s W(h_1 - h_2)}\right) - 1\right] \qquad (\textit{Imperial Units})$$

where:

h_1 - h_2 = the water surface reduction or drawdown, ft;
A_1 = the normal channel cross-sectional area without the ship,ft;
A_s = the maximum underwater cross-section area of the ship - Cx (Beam)Draft, ft;
W = the channel top width, ft;
V_1 = the speed of the ship relative to the water, ft/sec.

Assuming that the channel top width (*W*) does not change much, the drawdown (h_1 - h_2) can be solved by the iterative process.

For additional information in further calculations for the flow area and velocity distribution across the channel, jet velocity of the propeller, backwater velocity and ambient river velocities, Hochstein and Adams (1986) and Foltyn (1992) should be consulted.

A3.6.3 References

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