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https://doi.org/10.4224/40002704

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Guide for Design of Flood-Resistant Buildings

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NRC Contract No. 930582

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ACKNOWLEDGEMENTS

The Coulbourne Consulting team appreciates the help and guidance from the National Research Council (NRC) of Canada in the preparation of this guidance document. We especially thank Dr, Ahmed Attar, Dr. Zoubir Lounis, Dr. Naveed Khaliq, Ms. Marianne Armstrong, and Ms. Fiona Hill. The many discussions held jointly between the NRC and the consulting team made it possible to cover as many topics related to flooding and design for flood-resistant buildings as are covered in this report.

The efforts of the consulting team members are very much appreciated. The consulting team included Dr. David Kriebel, Mr. Randall Behm, Ms. Kimberly McKenna, and Mr. William L. Coulbourne. Mr. Alex Ferencz provided graphic support to the team. Everyone's diligence, thoughtfulness, and creativeness allowed the team to meet our objectives in a timely manner in spite of the inconvenience encountered in working during the COVID-19 crisis.

Finally, the consulting team would like to thank the many provincial, city, and town floodplain experts who provided information and background on how each one dealt with the flood hazard in their own communities. These perspectives and numerous flood loading data generation case studies from across Canada helped guide our approach to develop the guidance on designing for flood-resistant buildings.

EXECUTIVE SUMMARY

This report is written as guidance on how to design flood-resistant buildings that could be applied across Canada. This guidance could be considered in future editions of the National Building Code (NBC) of Canada. There currently is no flood load information in the NBC and thus this guidance has been developed to fill that gap in the NBC structural load provisions. Much of the guidance has been developed from flood design information that exists in the United States where guidance from the Federal Emergency Management Agency (FEMA) and the United States Corps of Engineers (USACE) already exists. There are also current flood load provisions in US building codes and standards, especially American Society of Civil Engineers *ASCE Standard 7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, and these codes and standards have been used to ensure all necessary flood load topics are covered.

The guidance is written to provide a national standard for flood design so all designers across Canada can use a standardized approach to determining flood loads based on a consistent risk in accordance with the Importance Category table in the NBC. We know there is much diversity in how provinces treat the flood hazard in terms of both the design annualized exceedance probability (AEP) and in how the flood hazard information is displayed to the public. The guidance provides a method to convert existing AEP flood elevations to a higher (and recommended design flood levels) AEP flood elevation.

The recommended flood design levels, as described above, are linked to the Importance Categories for Buildings Table 4.1.2.1 in the NBC. The recommended design levels are:

Importance Category	Recommended Flood Design AEP
Low	1:100
Normal	1:500
High	1:750
Post-disaster	1:1000

The rationale for this recommendation is covered in Chapter 2.

The primary determinants of flood loads are flood depth and flood velocity yet neither of these parameters is easily found on Canadian flood maps. Methodologies have been created to address both these issues and recommended methods for determining these parameters are presented. Examples have also been included illustrating how to use these methodologies. Formulas and provisions have been included for determining flood loads in riverine, coastal, and Great Lakes locations. The accuracy of the methods and formulas for determining flood loads are only as good as the information available to use for the development of the method or the determination of the flood load. Old flood maps, out of date hydraulic methods, lack of recent topographical information, or the use of old datums will all contribute to some inaccuracies in the methodologies and results suggested in this guidance, and to the results current practitioners might expect using current but older information.

This guidance also includes a chapter on performance-based design (PBD) for flood which attempts to present an alternative method for improving flood-resistance without necessarily adhering to the prescriptive recommendations offered here. This PBD guidance focuses on how flood resistance can be achieved by focusing on building or functional performance. This method is supported by two case studies in two different Canadian locations.

This guidance document has four chapters of technical content and an Introductory Chapter 1. Chapter 2 covers flood design conditions and considerations for riverine, coastal and Great Lakes locations. This chapter covers the important flood parameters to consider and how these parameters can be determined from available provincial data. Chapter 3 covers flood load formulas and provisions that use the flood parameters determined in Chapter 2. The formulas are explained, including the origin of many of them and additional references helpful in research are presented. Chapter 4 provides some detail on how to approach PBD for flood, as described in the previous paragraph. Chapter 5 is a summary of the information in this Guide with recommendations for NRC to consider in implementation.

Appendix A describes the methodology for scaling Mean Recurrence Intervals (MRIs) from two known MRI elevations. This method is very helpful in being able to articulate an expected design flood elevation for any MRI other than the two known elevations. Appendix B has additional information on the methodology for determining riverine velocities using HEC-RAS as the basis. Appendix C broadly covers sea level rise and climate change case studies for consideration in determining future conditions. Appendix D provides a summary of design guidance that could be used in the future development of building code proposals.

There are many possible approaches to some aspects of flood design; there are many different equations depending on the flood conditions, and there are many variations on which parameters to use. Throughout this guide, the recommended methods, formulas, or approaches are considered best practices and those that are more easily applied by practitioners.

This guide does not cover load combinations and load factors required to use for flood loads as there was insufficient information available with which to conduct this analysis within the scope and timetable of the current consulting agreement. The work on load factors and load combinations remains to be completed in order to have a more complete flood design package for the NRC.

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1 Introduction

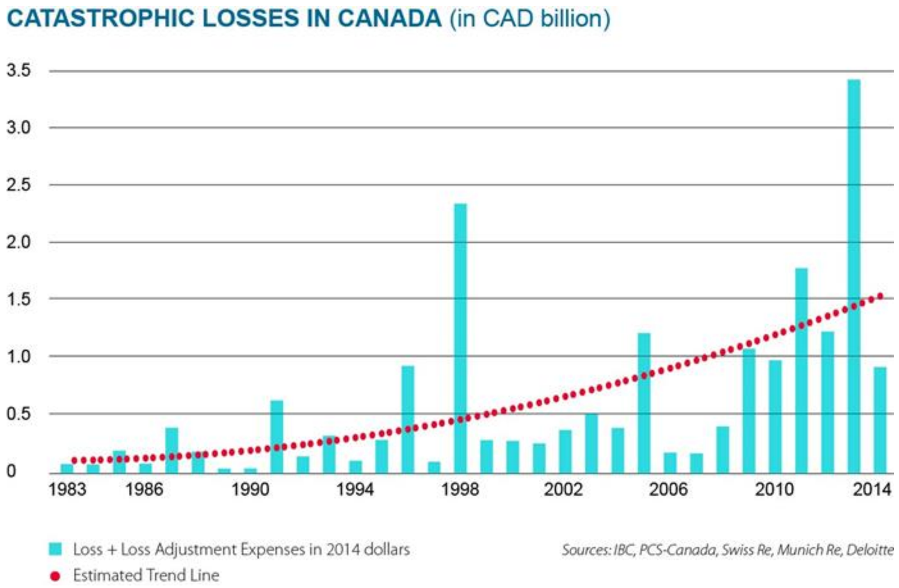
This technical guidance document has been developed for use in Canada for the design of flood-resistant buildings. Currently the National Building Code (NBC) does not discuss how flood loads should be treated in Canada, including what conditions flood loads should consider, what formulas to use to determine flood loads, and how to combine flood loads with other loading conditions such as dead or live loads, wind loads, or rain or ice loads. Floodplain mapping and associated bylaws or ordinances currently are developed at the provincial level; there are no national standards that guide how flood loads should be determined and there is no uniform flood risk used in Canada since the flood risk is established at the provincial level. This guidance document is intended to address these issues at a national level.

1.1 Background and Purpose

Flood damage across Canada has been a long-standing problem causing increasing damage across the country. During a period of 46 years from 1970-2016, the insured and uninsured losses caused by floods, storm surge, and hurricanes have exceeded $9.6 Billion. The five largest events have occurred within the last 10 years. There are 1.7 million homes at risk of flooding (19% of the total homes in the country, and 700,000 of these homes are at high risk, defined as having a 70% chance of being flooded over a 25-year mortgage period) (Rol, 2017). Estimated annual flood losses are $700 million (Rol, 2017). The large number of homes at high risk have an average flood Mean Recurrence Interval (MRI) of only 20 years – a very high risk of flooding indeed (see Table 2-6 for flood risk probabilities).

Figure 1-1 illustrates significantly increasing losses in Canada caused by extreme weather events. The spike in losses in 2013 was caused by flooding in Alberta and Ontario. (Swiss Re Media Release Aug 21, 2013 “Catastrophes cost global insurance industry more than USD 20 billion”). There is increasing evidence of the effects of climate change on flood losses; accordingly, the National Research Council (NRC) of Canada formed a group to study the flood problem from the perspective of building codes and standards impacts, and study the impact of disparate flood regulations and flood mapping requirements across the country.

The purpose of this technical *Guide on Design of Flood-Resistant Buildings* is to develop requirements for the design of new buildings to resist flood-related loads and for the design of building materials and systems to resist flood damage, including operational disruptions as defined by performance metrics. It is intended that these requirements be used nationally so that flood design across the country has a consistent risk basis. A separate technical guidance document titled *Guidelines for Improving Flood-Resistance for Existing Buildings* is developed using the nationally applicable flood design criteria and flood load formulas for existing buildings.


CATASTROPHIC LOSSES IN CANADA (in CAD billion)

Sources: IBC, PCS-Canada, Swiss Re, Munich Re, Deloitte

Figure 1-1 Catastrophic losses in Canada 1983-2014

1.2 Objectives

The objective of this technical guidance is to make available to design professionals an approach to designing buildings to improve performance during a flood event that can be used nationally. The technical approach is intended to be easily applied to any floodplain condition within the country in both riverine, lake, and coastal areas. But in the effort to make the approach simple to use in most cases, the results will tend to be conservative. Where the user would like to improve on the design and reduce conservatism, the approaches have been described or references have been provided for pursuing a more rigorous solution to determining flood loads.

Establishing flood conditions and flood loads can be used in design is a complicated technical subject with many possible contributors to loads that can damage or destroy buildings. A primary objective of this guidance is to distill the many issues and complexities into those that are most important to building design. In working toward this objective, some issues and complexities have been shortened in coverage to make the guidance more manageable for most users.

This report will illustrate how flood conditions can be determined at any site within a floodplain, demonstrate how to calculate flood loads for any site within a floodplain, and show how to use an alternative method of flood design using performance-based design techniques to meet the building code intent while increasing the probability that performance objectives can be met. Example problems and situations have been used throughout in attempts to demonstrate the use of the principles discussed related to flood conditions, flood loads, and performance-based design.

The magnitude of past flood events, and the growing concern about increasing size and magnitude of future flood events, has captured the nation’s attention, making inclusion of flood criteria in building siting and design an important step in reducing the magnitude and impact of those events. While information is presented here as a guide, portions of this report could be considered for future editions of the NBC of Canada.

1.3 Project Plan

A project plan for this study was prepared and approved by the NRC at the beginning (July 2019) of this project. The plan included a technical approach for a series of technical topics, a proposed outline to use for each technical topic, a list of resources and references that could be used for each technical topic, and a timeline for the development of the topics including dates of milestones.

The four technical topics that have been researched and described for this technical report include: (1) flood design conditions and considerations, (2) flood load formulas and provisions, (3) performance-based flood design, and (4) improving flood resistance of existing buildings (being delivered as a separate technical report to the NRC). Each of the four primary technical areas include the following elements:

- Currently published and available information that is relevant to the technical report topic.
- The information needed for flood-resistant design.
- The information that must be researched and/or developed to fill in the gaps between the information that is available and the information necessary to prepare flood design recommendations
- The content of the deliverable in general terms, including descriptions of the type of content (e.g. text, tables, illustrations, photos, screen captures, etc.).
- The recommended guidelines that should be included in the National Building Code of Canada and the associated commentary that supports the recommendations.
- The detailed timeline for the deliverable including NRC and team review times, including the 50%, 90%, and final deliverable times.

The project plan included a list of possible resources and references for each topic. There is substantial information about flood design available from the US building codes and engineering standards including those published by the International Code Council, the American Society of Civil Engineers, the US Federal Emergency Management Agency, the Association of State Floodplain Managers, and the US Army Corps of Engineers. In addition, building codes used in Europe, Australia, the UK, and New Zealand were reviewed; textbooks and engineering practice manuals used for riverine and coastal flood design were also reviewed. Flood conditions and methods used in Canada were reviewed and are included in the final technical guidance.

1.4 Expected Use of the Guidance

The technical guidance provided in this report is intended to be used for the design of flood-resistant buildings across Canada. This guidance could then be used for training programs for practitioners in the form of webinars or seminars, and the guidance could be subdivided into focused technical topics for more in-depth design information. Provinces and municipalities could convert the guidance into flood ordinances or into instructional material for local building officials.

1.5 Guidance Organization

This technical report has five chapters (including this introductory chapter) that cover flood design issues. Chapter 2 describes flood design conditions and considerations for both riverine and coastal floodplains. This chapter provides the methodologies for determining the many flood design parameters, and it covers the rationale for using certain design methods. It provides important background information on determining flood velocities by an approximate method (with significant technical information in Appendix B), and it describes how to use current flood return periods and corresponding water levels to determine water levels for flood events that are not mapped (with significant technical information in Appendix A). Examples of determining various flood parameters are provided.

Chapter 3 provides the flood load formulas and provisions that are necessary to determine flood loads for the flood parameters described in Chapter 2. This chapter draws a distinction between flood actions in fluvial or riverine areas, and flood actions in coastal or Great Lakes areas. In fluvial areas, flood loading is the result of hydrostatic actions resulting from static fluid pressures, as well as hydrodynamic actions related to the velocity of moving flood waters. In coastal areas, these same loads apply but additional loads are imposed by the cyclic actions of wind-generated waves. In both riverine and coastal areas, flood-related loads may also be imposed by floating or submerged debris, including ice, trees, vehicles, and other types of debris. Examples of determining flood loads for the example flood parameters in Chapter 2 are provided.

Chapter 4 describes performance-based flood design, and how it differs from the prescriptive design approach included in this Guide. The chapter discusses performance objectives and acceptance criteria for design. Performance-based design for various use and occupancy types is described as well as possible return periods to use for design. Examples of using the method are provided.

Chapter 5 contains specific recommendations for what practitioners need to consider when developing flood loads on buildings. This chapter is a summary of the detailed information provided in the other chapters.

1.6 Scope and Limitations

The scope will address flooding that occurs in mapped flood hazard areas that occur in any province and/or city including riverine, lake, coastal and ice-affected flooding. The scope is limited to those conditions that are developed into a flood hazard map and are thus available to the public for use in flood design. Present and future flood conditions are considered. Flood sources to be considered include overland flood due to high river stages including ice jam effects, elevated coastal (and Great Lakes) water levels, groundwater effects of saturated soils, and above-ground stormwater that flows outside defined channels. These are all consider as long as flooding from these sources is included in flood hazard maps.

The loading provisions described in this report apply to all building classifications in the NBC. The report is not intended for the design of: (1) flood protection structures such as levees, dikes, or flood walls, (2) shore protection structures such as bulkheads and seawalls, (3) port and harbor structures such as piers, docks, or wharves, (4) transportation structures such as railways, roadways, and bridges, or (5) floating structures that may include floating docks or floating buildings. These conditions require consideration of other forces related to water or waves that are not covered in this report.

2 Design Flood Conditions and Considerations

2.1 Background and Purpose

This chapter provides design guidance for buildings in flood-prone areas, irrespective of flood mapping history and current local regulations. The intent of the chapter is to offer a process for how to arrive at a national flood design standard that is usable and useful for any province or any jurisdiction within a province. The design flood conditions and considerations discussed in this chapter are useable and useful for both new and existing buildings. Some flood design parameters may require modification for use in existing buildings, and where that is the case, those modifications are discussed in the companion report on *Guidelines for Improving Flood-Resistance for Existing Buildings*.

2.2 Objectives

The objectives of this chapter are to:

1) Identify and characterize flood parameters of importance for flood-resistant design (including flood damage mechanisms and flood forces). This topic will include the many elements of flood-resistant design that must be considered.

2) Discuss design flood and flood frequency considerations for flood hazard mapping and flood-resistant design, including building importance. This will include how to determine the flood event, defined by its Mean Recurrence Interval MRI or the return period) or Annual Exceedance Probability (AEP), to use for design.

3) Discuss regulatory mechanisms to address flood: flood zone designation (floodway, flood fringe), by-laws and land use regulations, plans, guidelines, and building regulations. There is a range of possible regulatory steps that can be taken to address flood design on a national or regional level.

4) Discuss sources and use of “actionable science” to incorporate climate change into future conditions used in flood mapping and flood-resistant design.

2.3 Scope Limitations

This chapter covers riverine, lake, and coastal flood conditions and design considerations. The scope is limited to these conditions that are developed into a flood hazard map and are thus available to the public for use in flood design.

Flood hazard maps generally delineate the lateral extent and vertical elevation of flood waters for certain defined flood events adopted by the Provincial or local Approving Authority. In riverine or fluvial locations, the mapped flood hazard zone includes both the floodway, defined as the portion of the floodplain that conveys most of the flow during flood conditions with highest flow velocities, and the flood fringe, defined as the portion of the flood hazard area outside the floodway, which may have still-water flooding or flooding with relatively low velocities.

In this report, riverine conditions refer to freshwater rivers and streams in which flood elevations are dictated by rainfall-runoff, snowmelt, or ice jam conditions. Any overland water flow event that fills up stream and river channels, overflows the stream and river banks, causes flooding that can affect buildings and infrastructure, and that is used to develop a flood map for some storm frequency is covered by this chapter. This includes ice jams, as long as the effect of the ice jam is included in the mapped flood elevation. Situations such as storm water backup, drainage ditch overflow, sanitary sewer back up, or other results of flooding are not included in this chapter, unless those conditions are part of the flood study that was used to develop the provincial flood maps

In coastal regions and the Great Lakes, flood hazard zones are typically categorized according to heights above a designated high-water reference level allowing for the combined effects of elevated tides or lake level, storm surge, and the effects of waves. Mapping of elevations based on wave effects may include the actions of wave crests above the tides and surge level, as well as the effects of wave runup on a sloping flooded shoreline. Coastal conditions refer to saltwater bay and ocean coasts, influenced by tides, storm surge, and waves. Lake conditions refer to freshwater shorelines of the Great Lakes or other large lakes, not influenced by tides but influenced by wind setup, i.e. storm surge, and waves similar to ocean coasts.

Tsunamis are not covered by this technical report. The user of this report should consult with local floodplain authorities and the National Building Code of Canada (NBC) to determine if tsunami design should be considered for a particular project site. The American Society of Civil Engineers (ASCE) load standard ASCE 7-16 has significant flood design information for tsunamis that should be researched for design guidance for Canadian locations.

2.4 Flood Parameters Important for Flood-Resistant Design

There are many flood characteristics that are important in building design to resist flooding. Riverine flood design requires information about the extent of expected flooding, the expected elevation of flooding, the rate of rise of flood water, the velocity of flood waters, how flood forces act together, and how to resist the forces created by flooding. Coastal flood design, in addition to the information noted above, requires information about tide levels, expected wave effects on top of storm surge, water fetch distances in front of coastal buildings, and how to resist the forces created by coastal flooding. Similar information is required for design on lake shorelines, except that tides are not a factor. One difference for lakes is that the lake level may also vary with inflow-evaporation-outflow so that the entire lake level may rise or fall over time scales ranging from seasons to years.

Flood water can cause a variety of damages when it comes in contact with a building:

- Flood water can surround a building, and unless the building is strong enough and does not leak, the building or elements of the building could float, or the walls could collapse from hydrostatic pressure of the standing water against the building walls.
- Flood water can move along a river channel with some velocity, and the force of the moving water can move a building off its foundation.
- Flood water can rise rapidly due to excessive rainfall or snowmelt and cause a flash flood with very high velocity.
- Flood water can cover a flat floodplain with water that recedes very slowly due to saturated soil or very small topographical relief.

- Flood water can pick up debris as it flows, including ice, and force that debris into buildings causing collateral damage.
- Coastal flood water comes ashore with waves and, depending on the storm that creates the flooding, can inundate the coast with storm surge that floods coastal buildings.

These flood damage mechanisms and the forces and loads they create will be covered further in the sections that follow.

2.4.1 Reference Datum for Flood Events

The preferred approach for determining flood effects at a building site is to reference flood water elevation to a geodetic datum, defined as elevation above a geoidal surface based on a nationwide convention. Natural Resources Canada has established the Canadian Geodetic Vertical Datum of 2013 (CGVD2013) as the reference standard across Canada, replacing the older CDVD28 datum. Most flood maps show flood water elevations above the CGVD2013 datum (or CGVD28 datum). Most ground elevations based on recent digital elevation models are also given relative to the CGVD2013 datum (or CGVD28 datum). Elevations of flood water or land based on other local datums, including river stage and coastal or lake chart datum, can then be converted to CGVD2013.

Riverine flood elevations are sometimes specified as a river stage, which refers to river level above some locally defined reference datum. For example, the City of Winnipeg reports water levels in the Red River above a zero-reference elevation defined for a river gage at James Avenue. While useful for local planning and community flood hazard management, these local river stage elevations are generally not used for structural design unless ground elevations are also defined relative to the same datum. Because river stage datums are established locally, the stage datum will decrease downstream with approximately the slope of the river surface.

In coastal or lake regions, flood elevations are often referenced to the chart datum (CD). Analogous to river stage, the chart datum is a local datum, valid only for a specific location. In coastal areas, CD is defined as a low tide reference level, typically the level of the Lower Low Water Large Tide (LLWLT) based on an average of the lowest low tide annually for 19 years. In the Great Lakes, each lake has a defined chart datum as specified in the International Great Lakes Datum of 1985 (IGLD85). In coastal and lake areas, flood levels specified relative to CD may be used directly if ground elevations are also known relative to the same CD. Otherwise, flood levels specified from the CD need to be converted to the datum used for ground elevations.

2.4.2 Flood Elevation

The primary variable used to define flood conditions is the elevation of flood waters above a reference datum. Flood elevation is typically defined on flood maps and in flood hazard studies irrespective of ground elevation and refers simply to the elevation of water, not to the flood depth. The elevation of flood water may be denoted as the **Design Flood Level (DFL).**

For building design, the DFL is generally defined by either: (1) a return period or Mean Recurrence Interval (MRI), defined as the average interval in years between events that equal or exceed the given elevation, or equivalently by (2) an Annual Exceedance Probability (AEP), defined as the probability that an event will be equaled or exceeded in any given year. In general, MRI and AEP are given by the inverse of the other, e.g. MRI = 1/AEP, so that a 100-yr MRI event would have an AEP of 1:100 or 1% per year. The flood elevation for a given MRI or AEP, is defined specified as an elevation in metres (feet) above some vertical datum.

Another term in use when describing flood elevations is the still water level (SWL). This generally refers to a water level that would be attained in the absence of any wave action. In riverine conditions, the DFL and SWL are usually synonymous. In coastal areas, the phenomenon of wave setup may cause the average water level to rise due to wave action, so that the DFL would be above the SWL based on wave setup effects.

In riverine areas, the DFL is primarily determined from hydraulic modeling conducted as part of a flood hazard study. This is based on some design rainfall intensity over a watershed and subsequent routing of flood waters downstream. The design river discharge and flood stage or elevation are then established for specific river transects or cross sections from hydraulic modeling as a function of ground slopes, cross section geometry, flow expansions and contractions, and ground roughness.

For coastal areas, flooding will include several components that are not present in riverine flooding. These conditions include the stage of the astronomical tides (for ocean coasts) or pre-storm lake level (on Great Lakes), storm surge from wind setup and lowered barometric pressures, and an increase in mean water level due to breaking waves known as wave setup. In coastal areas, the Design Flood Level is given by

$$DFL = \text{design tide or lake level} + \text{ storm surge} + \text{wave setup effects} \qquad \text{(Eq. 2-1)}$$

Design flood levels are generally determined through coastal hydrodynamic and wave modeling to establish storm surge and wave effects. These are then coupled with tide levels. Most coastal flood studies reviewed for this study use the higher high water, large tide level (HHWLT) as the base elevation for coastal flooding. The HHWLT is the average of the highest tides measured in a 19-year period, and flood hazard studies therefore assume that severe coastal storms will be correlated to times of exceptional high tides. Because the local chart datum (CD) is based on the lower low water, large tide LLWLT, the HHWLT elevation reflects tidal range for each location and must be obtained for each site. For Great Lakes locations, the reference elevations are generally taken as some long- term elevated lake level above the lake chart datum.

2.4.3 Future Climate Effects

As used in this report, the DFL should also include future climate change effects (FC) that effectively alter the flood level over the lifetime of a structure. While most historical flood hazard studies did not include climate change effects, many modern flood studies incorporate future climate scenarios and their effect on flood elevations.

In riverine areas, climate effects may increase or decrease rainfall, snow fall, or evaporation, change ice jam conditions, and produce changes in river discharge and water surface elevation. Similar changes may occur in large lake systems. In riverine situations, climate effects may change river discharge but generally do not have a linear effect on the flood stage or the DFL. For this reason, the DFL should be updated to include climate effects in flood hazard studies and mapping programs.

In coastal regions, the primary effect of future climate change is global or regional sea level rise. This will allow both high tides and storm surge to reach higher elevations over time. In addition, however, vertical land movement may occur due to readjustment of the earth's crust or locally due to subsidence of sediments. Vertical land movement may vary on a regional or even local basis. Specific sea level change or land motion effects should be established locally based on methods adopted by an Approving Authority. In most coastal studies, the DFL is updated by linearly adding relative sea level rise effects to some previous (or current) DFL. The effects may not be truly additive, and non-linear coupling of sea level change with storm surge and wave effects can be established through modeling studies.

2.4.4 Flood Construction Level

In many locations, a **Flood Construction Level (FCL)** is defined by an Approving Authority as a regulatory elevation to govern building siting, floor elevation, and construction standards. The FCL is often higher than the DFL due to the inclusion of transient flood effects due to waves or due to inclusion of freeboard as

$$\text{FCL} = \text{DFL} + \text{wave effects} + \text{freeboard} \qquad \text{(Eq. 2-2)}$$

where:

DFL = Design Flood Level or elevation
FCL = Flood Construction Level or elevation
Wave effects = additional elevation to account for transient wave action above the DFL
Freeboard = additional elevation added as a safety margin

Wave effects will be discussed in subsequent sections of this report. These generally include the extra elevation of wave crests above the still water level due to the passing of each wave, and/or the added elevation of wave runup as waves surge up a beach or structure slope.

The freeboard, defined as an air gap between the uppermost water surface and lowest habitable portion of a structure, may account for uncertainties in flood level estimates, or may simply be prescribed by an Approving Authority as an additional safety margin. In some cases, freeboard has been used as a substitute for more rigorous estimation of climate change effects.

Any wave effects or freeboard will elevate the FCL and will influence the permitted elevation of buildings in flood zones. The wave effects and freeboard do not change the water depth, however, and do not alter flood loads related to hydrostatic pressures. Wave effects may add additional flood loads but are not considered to change the flood water depth. Freeboard may affect flood loads, as an air gap may reduce or eliminate some flood loads compared to a situation with no freeboard.

2.4.5 Flood Depth

The primary variable used to define flood forces on a structure is **Design Flood Depth, d_f.** In this report, the design flood depth is defined as the difference between the design flood elevation (DFL) for a given MRI or AEP and the ground elevation (G), which may include effects of erosion or scour, as

$$d_f = \mathrm{DFL} - \mathrm{G} \qquad \text{(Eq. 2-3)}$$

where:

d_f = design flood depth
DFL = design flood level or elevation
G = ground elevation relative to datum used to establish DFL and adjusted for effects of erosion and scour

For computation of flood loads, information is required on ground elevations at the site. This information would presumably be available from GIS maps, from the Geological Survey of Canada, from Provincial/Territorial or municipal mapping programs, or from a site survey conducted by a professional land surveyor. Additional consideration would then be required to estimate the general erosion and localized scour at the building site that may occur during a flood event.

The definition of the design flood depth is illustrated for a typical river transect in Figure 2-1. The flood hazard zone in riverine or fluvial locations includes both (1) the floodway, defined as the portion of the floodplain that conveys most of the flow during flood conditions with highest flow velocities, and (2) the flood fringe, defined as the portion of the flood hazard area outside the floodway, which may have still-water flooding or flooding with lower velocities. In riverine areas, it is assumed that either no waves exist in the riverine flood condition, or they are not large enough to consider

In many flood maps, the floodway and flood fringe are commonly defined as greater than or less than 1 m in depth and greater than or lesser than 1 m/s in velocity for a specified AEP or MRI at a specified location (see Glossary of Terms). As depicted, several provinces define the floodway as the river channel that will convey a 20-yr MRI, or 1:20 AEP, flood event.

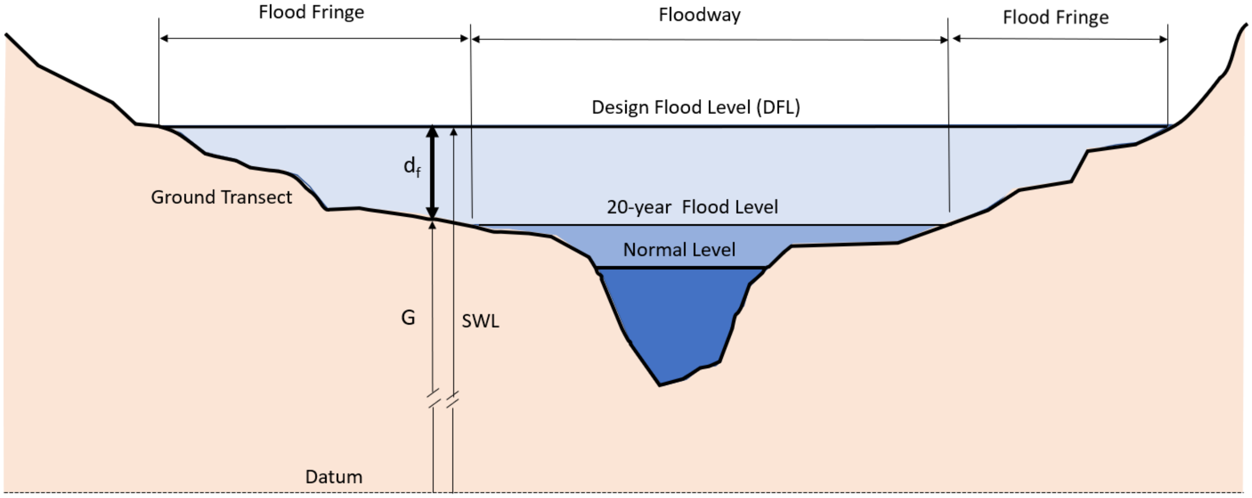


Figure 2-1 Definition of flood depth in riverine conditions

In coastal regions and the Great Lakes, the DFL is typically based on the effects of storm surge above a high pre-storm reference water level, either from long-term elevated lake levels or from a high tide level. As depicted in Figure 2-2, the design flood level (DFL) and local ground elevation establish the design flood depth, but wave effects may reach higher elevations. In the US, mapping of elevations based on wave effects is common and includes both the elevation of wave crests above the still water elevation and the elevation of wave runup on a sloping flooded shoreline. In Canada, only wave runup is included in mapped flood elevations, but is not included consistently in all Provinces or in all areas within a Province.

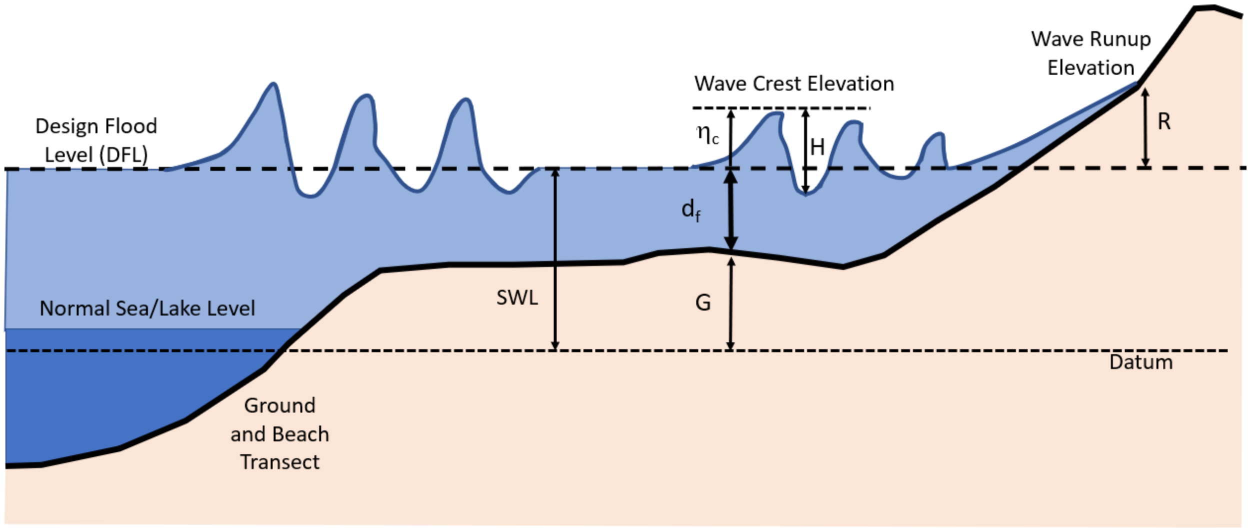


Figure 2-2 Definition of flood depth and wave effects in coastal conditions

2.4.6 Flood Velocity

Another primary variable that defines flood forces is the velocity of the flood water, V. As water passes an object, there is a hydrodynamic force exerted on the water-facing building wall and there is a drag force created as water flows around the object. The magnitude of the drag force is a function of the shape of the object and the velocity of the water. Flood water velocity can be large enough to push a building off its foundation and undermine its supporting soil and foundation.

In riverine conditions, unidirectional flows prevail, and the downstream flow velocity is a primary characteristic of riverine flooding. The flow velocity usually varies across a flow transect perpendicular to a river channel. Velocities are usually highest in the floodway, lower in the flood fringe, and sometimes the velocity is zero or nearly zero in areas of the flood plain that provide flood storage without any substantial conveyance of the flow. However, it is important to assess velocities in the flood fringe, as sometimes the flood fringe can exhibit velocities high enough to cause substantial damage to buildings.

In coastal and lake flood zones, it is usually assumed that there is little net flow velocity. On an open coast, storm surge usually causes water levels to rise and fall over a matter of hours without imparting significant flow velocities. This may not always be true, for example in areas of flow constrictions where surge may flood and ebb through an inlet or narrow region producing high localized velocities or in areas of river mouths, and designers should be aware of any localized increase in velocity. In areas subjected to waves, the back-and-forth wave orbital velocities are usually not considered to be part of the net flow velocity. Loads due to these cyclic flows are treated separately as wave loads and not as part of the velocity-induced hydrodynamic loads.

2.5 Review of Flood Mapping Practices

The development and promulgation of flood hazard maps differs considerably from Province to Province, and may differ within a Province, for example in Ontario where regional Conservation Authorities may establish flood hazard maps. Six provinces have been chosen for review and use in this design guidance. Those six provinces are: Alberta, British Columbia, Ontario, Newfoundland, Quebec, and Saskatchewan. These six provinces were chosen because of the range of MRI (or AEP) that are being used, and their handling of future conditions. The various elements addressing the treatment of the flood hazard for design purposes in these six provinces is shown in Table 2-1.

Table 2-1 Flood Hazard Information for Six Select Provinces

Province	Design Flood (AEP)	What Data are Shown on Flood maps	Climate change included in Maps
Alberta	1:100	Inundation	No
British Columbia	1:200	Inundation and depth	Yes, newer maps
Ontario	1:100	Inundation	No
Newfoundland and Labrador	1:20 and 1:100	Inundation	Yes, newer maps
Quebec	1:20 and 1:100	Inundation	No
Saskatchewan	1:500	Inundation	No

One aspect that changes by Province is the MRI or AEP of the design flood event. Most Provinces map the 100-year MRI (1:100 AEP) flood level. But some also map the 1:20 AEP, and others map 1:200 or 1:500 AEP flood events. In order to achieve more consistent national flood risk across Canada, it is likely that more uniform standards will be required in the future. In the US, the Federal Emergency Management Agency (FEMA) conducts flood hazard studies and maps the 1:100 and 1:500 year flood elevations in all coastal and riverine flood hazard zones nationwide.

2.5.1 Riverine Flood Mapping

A wide variety of riverine flood maps can be found across Canada. Some display actual elevations above datum while others simply show colour coded regions that would be inundated by specific flood events. Most maps do not provide flood depth information. In order to use this information for structural design, the user needs to locate a site on the map, find the projected water surface elevation for the flood AEP of interest, interpolate the ground elevation from a topographical map for the site, and subtract the flood elevation from the ground elevation at the site.

Figure 2-3 gives a sample of a flood map for the Campbell River in British Columbia. The 1:20 and 1:200 AEP flood elevations are noted on the map. At the location indicated, the 200-yr flood elevation is 11 m, including 0.6 m of freeboard. With a ground elevation of 9 m obtained from mapped ground elevation contours, the map would indicate a flood depth of 1.4 m once freeboard is removed.

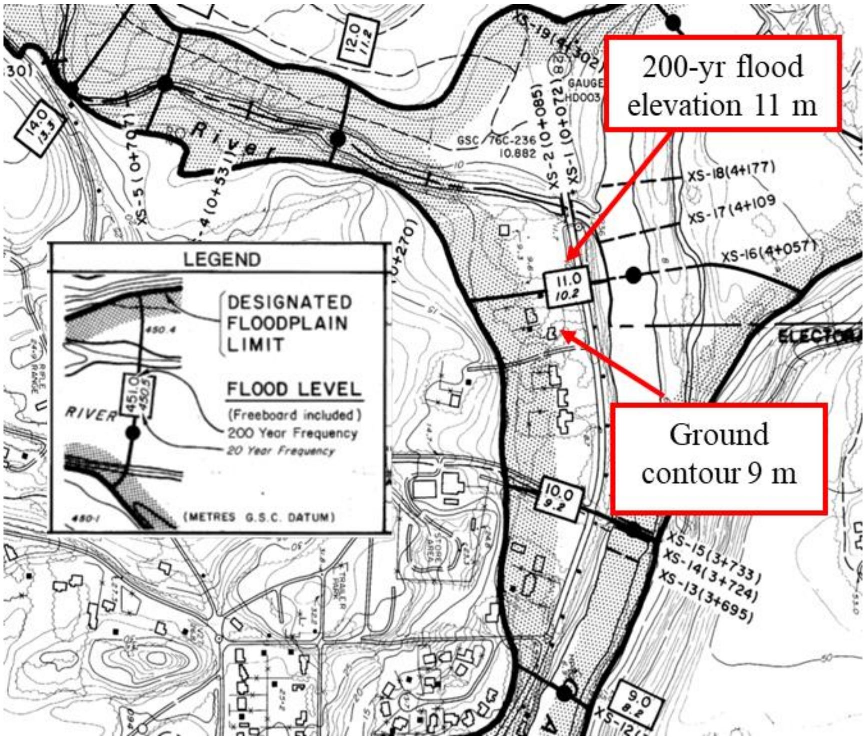
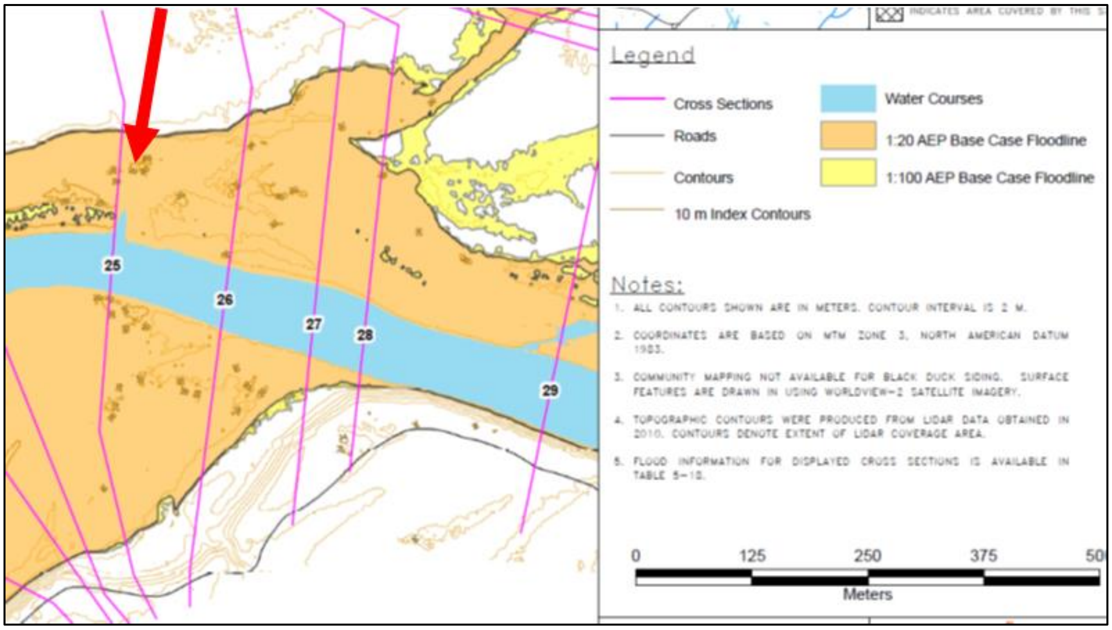


Figure 2-3 Partial Section of Campbell River, British Columbia Flood Map (source: http://www.env.gov.bc.ca/wsd/)

Figures 2-4 through 2-6 show additional examples for Harry's River in Newfoundland. Figure 2-4 shows a typical format where transects or cross sections are noted along the length of the river channel. Figure 2-5 then shows a representative flood map, with the extent of the floodplains for the 1:20 and 1:100 AEP floods indicated. Neither format indicates ground elevations or flood depth, so a user would need to obtain ground elevations from a separate source. This would likely come from either a site survey or from the digital elevation model. The depth profile for one transect, Transect 25, is then shown in Figure 2-6. The flood depths vary from 1 m at the intersection of the channel and the overflow bank (marked Point 1) to 3 m in the flood fringe (marked Point 2).

Another example from the Fort Saskatchewan Reach flood map in Alberta is shown in Figure 2-7. The information available for this location consists of orthophoto and contour maps with the 1:10, 1:50, and 1:100 AEP events shown on the maps. The legend illustrates that the 1:100 AEP line is red; the contour lines are in white. The flood study that included this map (North Saskatchewan River Study, 2007) indicates the 1:100 flood elevation at cross section XS 8 is 604.33 m. The 600 m contour intersection with the cross-section location is shown with a white arrow in Figure 2-8. At that location, the 1:100 AEP water depth is approximately 4.3 m.

Figure 2-4 Harry's River; Cross Sections overlaid on aerial imagery. Cross Section 25 identified by red arrow (source: Hydrotechnical Study of Stephenville Crossing/Black Duck Siding, 2012)



Notes:

1. ALL CONTOURS SHOWN ARE IN METERS. CONTOUR INTERVAL IS 2 M.
2. COORDINATES ARE BASED ON MTM ZONE 3, NORTH AMERICAN DATUM 1983.
3. COMMUNITY MAPPING NOT AVAILABLE FOR BLACK DUCK SIDING. SURFACE FEATURES ARE DRAWN IN USING WORLDVIEW-2 SATELLITE IMAGERY.
4. TOPOGRAPHIC CONTOURS WERE PRODUCED FROM LIDAR DATA OBTAINED IN 2010. CONTOURS DENOTE EXTENT OF LIDAR COVERAGE AREA.
5. FLOOD INFORMATION FOR DISPLAYED CROSS SECTIONS IS AVAILABLE IN TABLE 5-10.

Figure 2-5 1:20 and 1:100 Floodplain boundaries for Harry’s River. Cross Section 25 identified by red arrow (source: Hydrotechnical Study of Stephenville Crossing/Black Duck Siding, 2012)

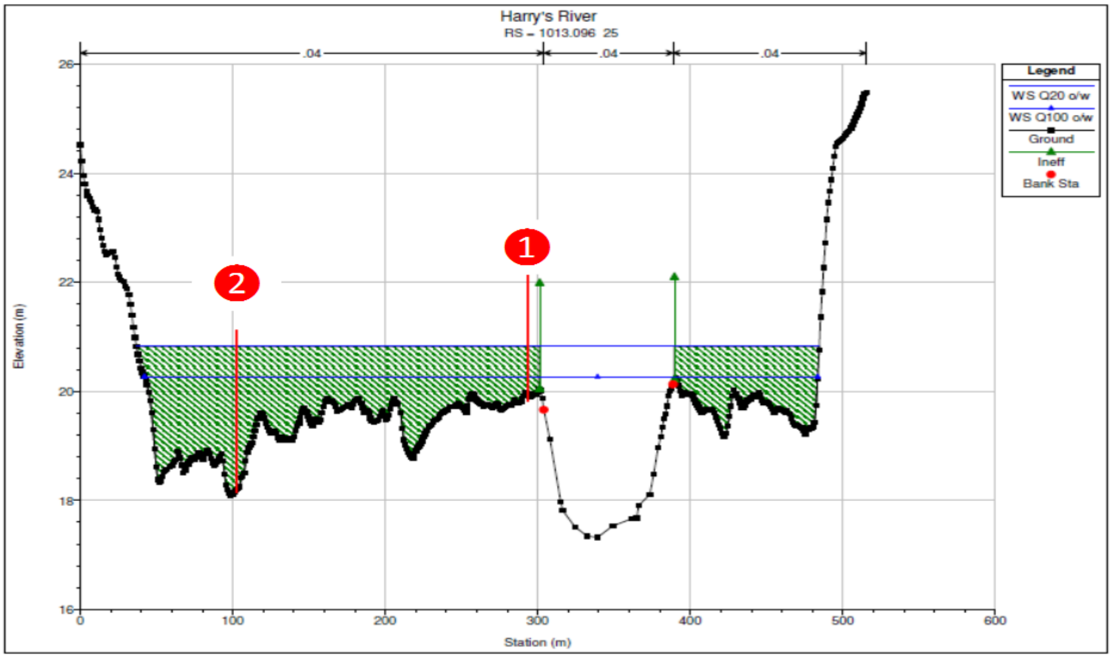


Figure 2-6 Cross Section 25 on Harry’s River, Newfoundland (source: Hydrotechnical Study of Stephenville Crossing/Black Duck Siding, 2012)

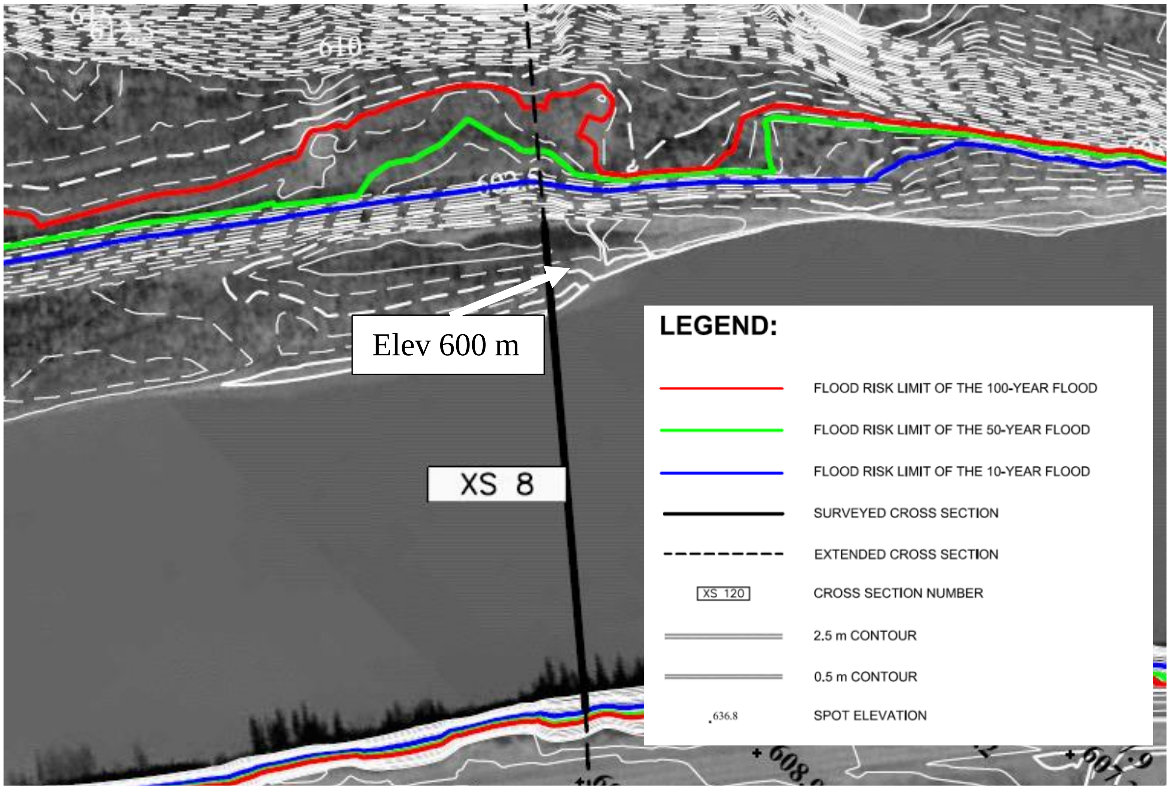
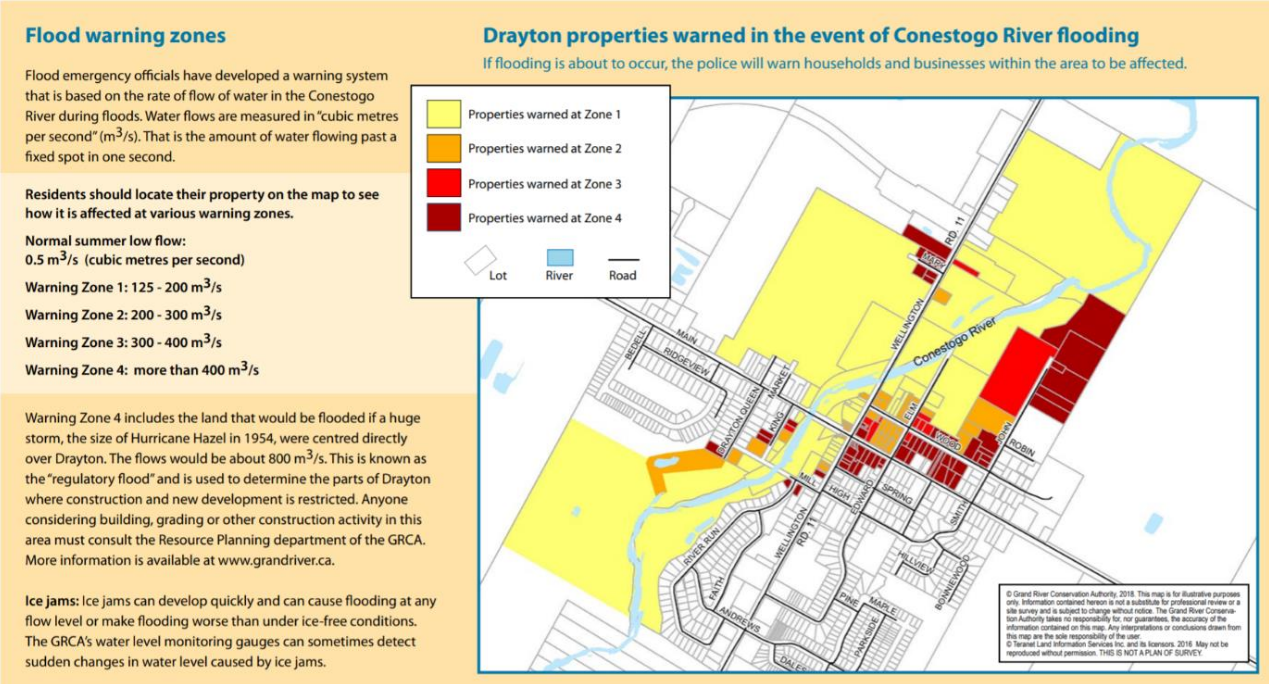


Figure 2-7 Cross Section 8 of Fort Saskatchewan Reach on North Saskatchewan River, Alberta (North Saskatchewan River Flood Study, 2007) Contour 600 m shown with white arrow

In Ontario, flood maps are produced by local Conservation Authorities. Two examples from the Grand River Conservation Authority are shown in Figure 2-8 and 2-9. A practice found in Ontario is to use a historic flood event, the 1954 Hurricane Hazel flooding in Timmins, as a design storm event. The rainfall intensities are shifted over other parts of Ontario, under the assumption of alternate storm tracks for Hazel, and the resulting river discharge is computed.

For selected communities, flood warning maps are produced that offer different warning levels depending on river discharge and location, as shown in Figure 2-8. This information is useful for siting decisions but difficult to use for building design as there is no indication of flood elevation and results are not tied to any particular MRI or AEP.

Some other flood maps in Ontario do map flood events defined by an MRI or AEP. An example is given in Figure 2-9 which shows the extent of the 1:100 AEP flood event. With the addition of property parcel and building information, the map can also show existing buildings subjected to the 100-year flood.



Flood warning zones

Flood emergency officials have developed a warning system that is based on the rate of flow of water in the Conestogo River during floods. Water flows are measured in "cubic metres per second" (m^3/s). That is the amount of water flowing past a fixed spot in one second.

Residents should locate their property on the map to see how it is affected at various warning zones.

Normal summer low flow:
0.5 m^3/s (cubic metres per second)

Warning Zone 1: 125 - 200 m^3/s

Warning Zone 2: 200 - 300 m^3/s

Warning Zone 3: 300 - 400 m^3/s

Warning Zone 4: more than 400 m^3/s

Warning Zone 4 includes the land that would be flooded if a huge storm, the size of Hurricane Hazel in 1954, were centred directly over Drayton. The flows would be about 800 m^3/s. This is known as the "regulatory flood" and is used to determine the parts of Drayton where construction and new development is restricted. Anyone considering building, grading or other construction activity in this area must consult the Resource Planning department of the GRCA. More information is available at www.grandriver.ca.

Ice jams: Ice jams can develop quickly and can cause flooding at any flow level or make flooding worse than under ice-free conditions. The GRCA's water level monitoring gauges can sometimes detect sudden changes in water level caused by ice jams.

Drayton properties warned in the event of Conestogo River flooding

If flooding is about to occur, the police will warn households and businesses within the area to be affected.

© Grand River Conservation Authority, 2018. This map is for illustrative purposes only. Information contained hereon is not a substitute for professional review or a site survey and is subject to change without notice. The Grand River Conservation Authority takes no responsibility for, nor guarantees, the accuracy of the information contained on this map. Any interpretations or conclusions drawn from this map are the sole responsibility of the user.
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Figure 2-8 Flood warning map Ayr, ON (source: Grand River Conservation Authority)

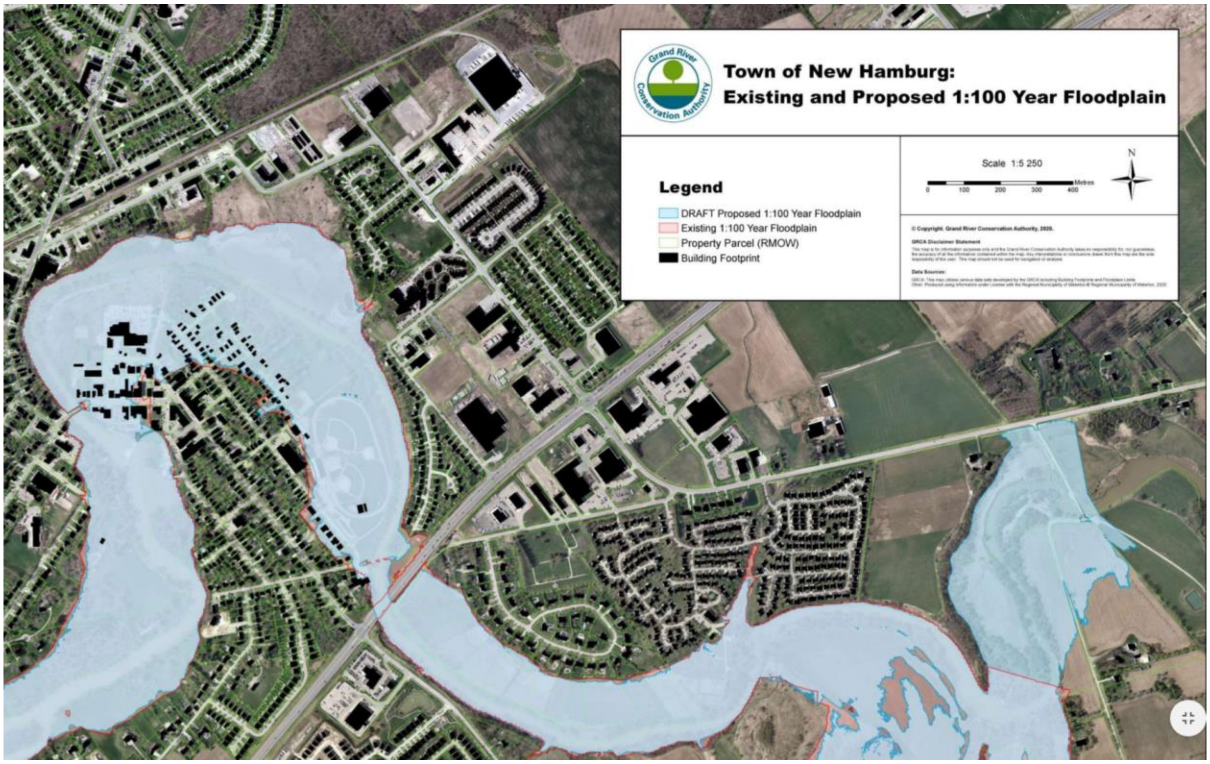
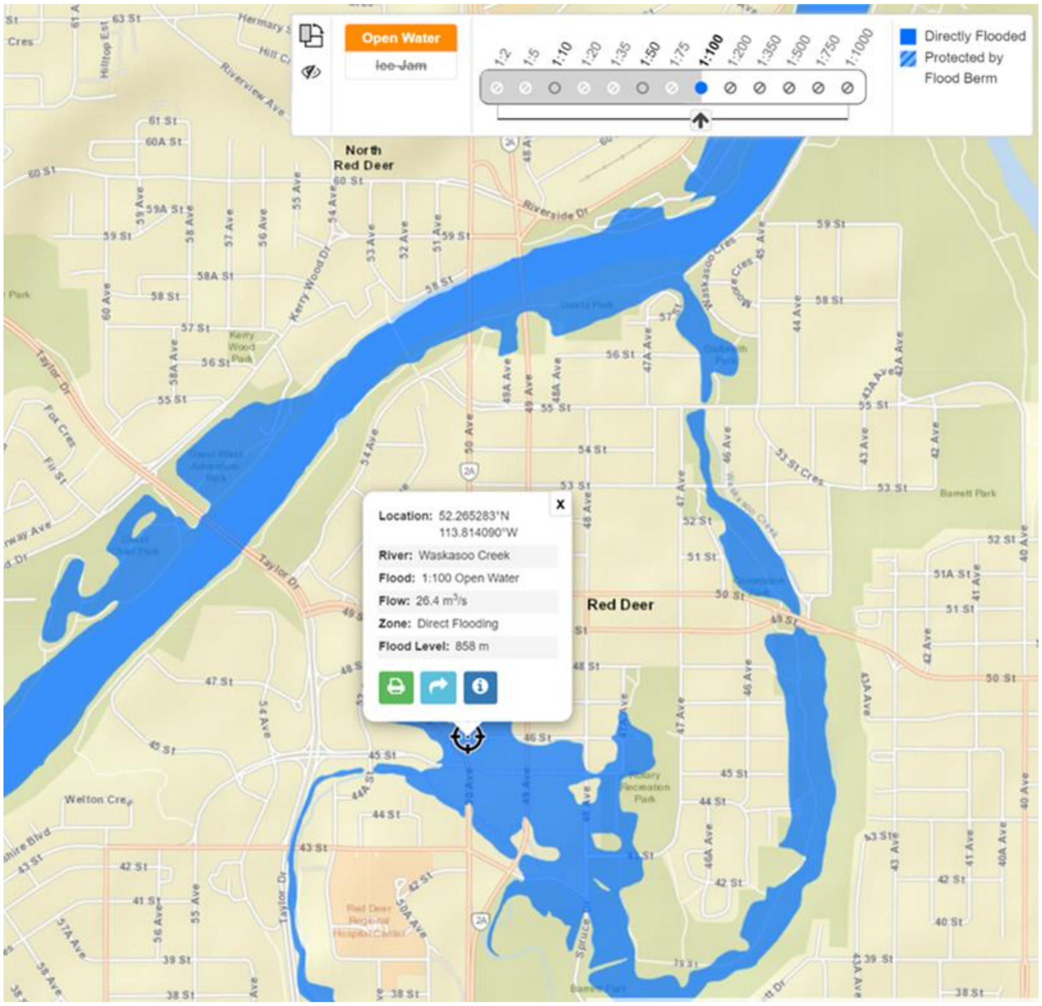


Figure 2-9 Draft flood map, New Hamburg, ON (source: Grand River Conservation Authority)

Some newer flood maps in other Provinces provide users with additional information, often in a convenient online format. Figure 2-10 shows a sample of new flood maps from Alberta, focusing on Red Deer. With the bar at the top, a user can select 1:10, 1:50, or 1:100 AEP flood events to display the extent of flooding. Clicking on a point on the map then results in a menu box giving location and flood elevation. Draft products from the same web site (https://floods.alberta.ca) extend the flood MRI options ranging from 2-yr to 1000-yr events.



Location: 52.265283°N 113.814090°W
River: Waskasoo Creek
Flood: 1:100 Open Water
Flow: 26.4 m^3/s
Zone: Direct Flooding
Flood Level: 858 m

Figure 2-10 Floodplain map with parcel information (source: www.floodsalberta.ca)

Some flood hazard maps also show information about flood velocity. Many of the current provincial flood maps use the 1:20 AEP flood to designate the extent of the floodway and based on hydraulic modeling, define the floodway as that flood area that has depth of greater than 1 m and velocity of greater than 1 m/s. By comparison, the extent of the 1:100 AEP flood designates the extent of the floodplain in which the depths are less than 1 m and velocity is less than 1 m/s. Some new flood maps show the flow as a visual colour scale of defined within some range of velocity in metres/second. Figure 2-11 illustrates such a range of velocities.

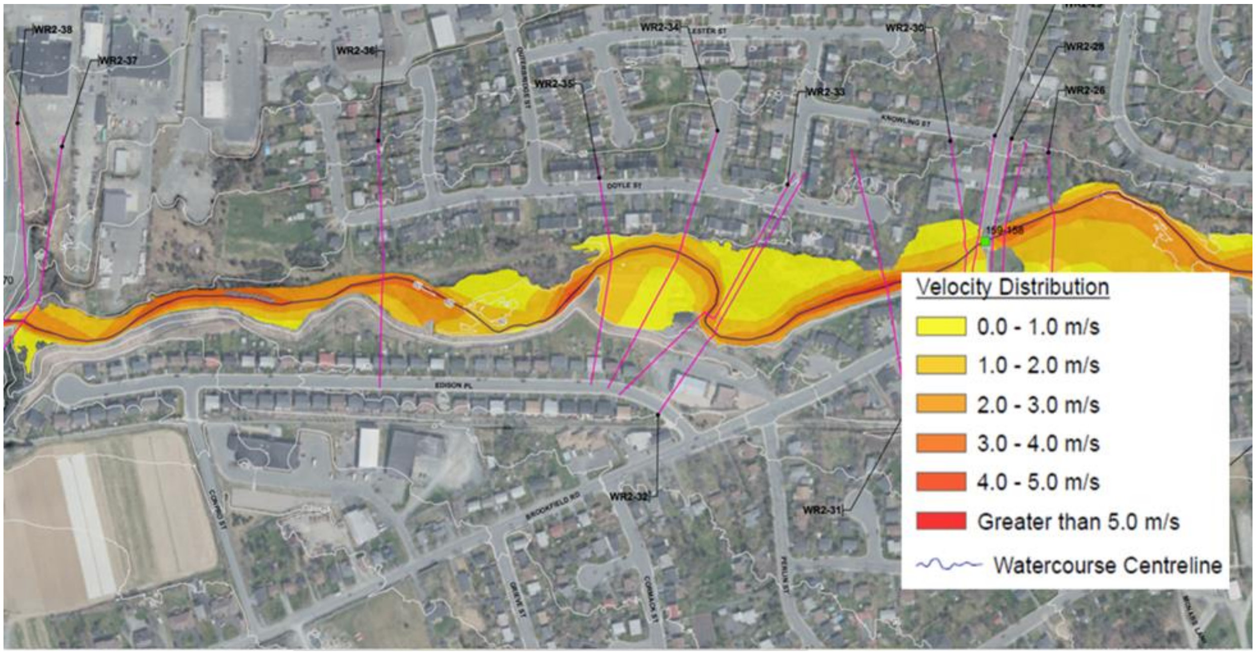


Figure 2-11 Portion of 1:100 AEP flood map showing velocity distribution (source: Newfoundland and Labrador Waterford River Area, Map by CBCL Limited Consulting Engineers, 2017)

As will be described in Chapter 3 on flood loads, hydrodynamic loads are generally proportional to the square of the velocity (V^2), as illustrated in Figure 2-12. So, when there is a wide range of possible velocities delineated only by colour differences in 1 m/s increments, large errors could be made in the calculation of the loads using a velocity scale This rapid increase in loads, makes accurate velocity determination in riverine conditions important.

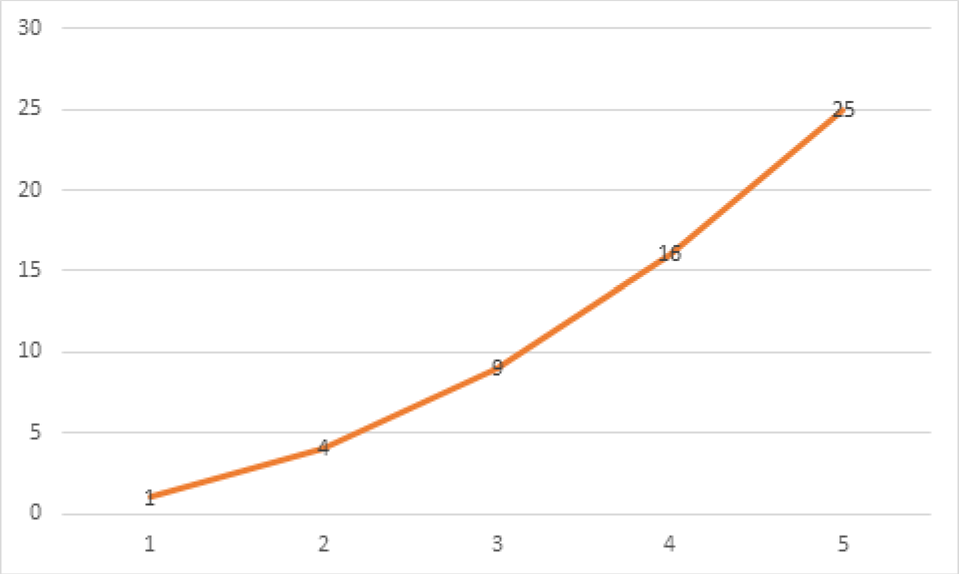


Figure 2-12 Graph of Flood Load Increase vs. Velocity Increase

2.5.2 Coastal and Lake Flood Mapping

Flood mapping practices in coastal areas and large lakes exhibit similar variability across the Provinces. A sample map from Buffalo Narrows, SK is shown in Figure 2-13. In this case the extent of flooding the 1:500 AEP flood event from Churchill Lake (right) and Peter Pond Lake (left) is shown and flood elevations are clearly given. The flood hazard study used to develop the map could not be located, but elevations presumably do not include wind-generated surge or wave effects and rely primarily on rainfall from a 500-year storm event

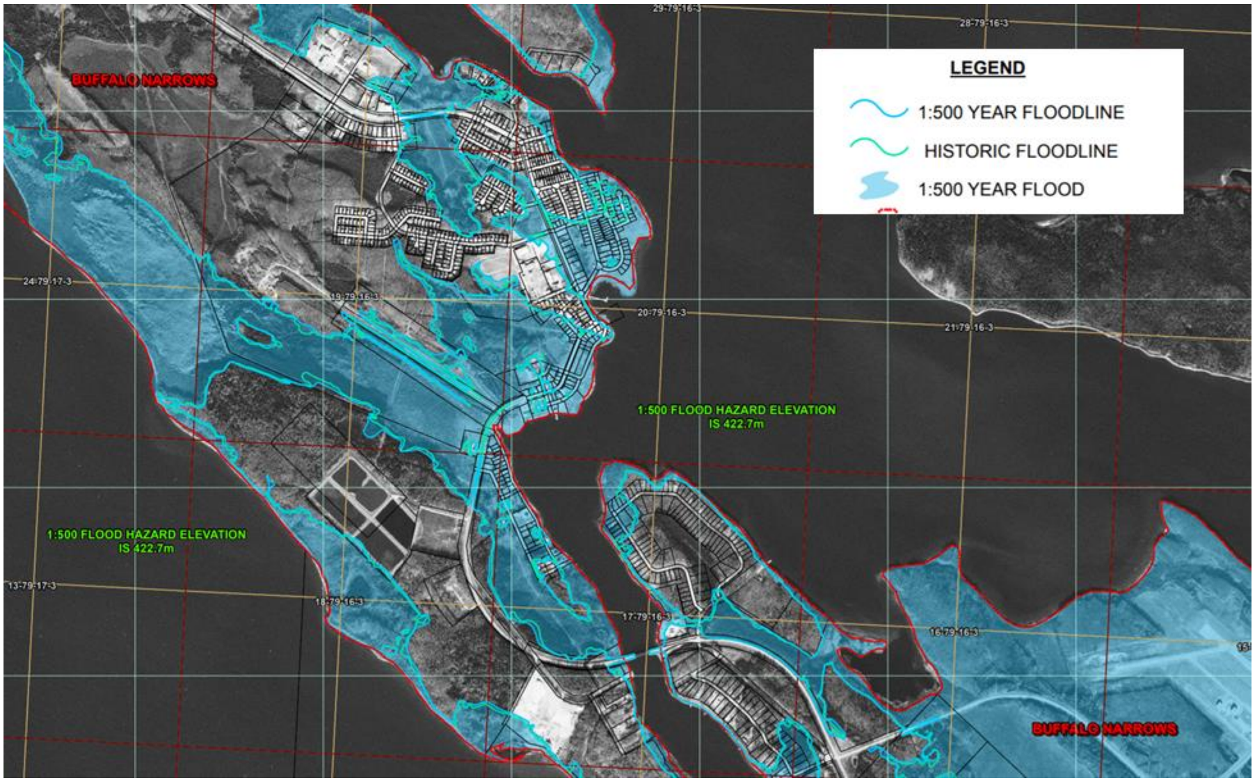


Figure 2-13 Portion of flood map from Buffalo Narrows, SK (source: Saskatchewan Watershed Authority)

A sample of a coastal flood map from British Columbia, produced in 1997, is shown in Figure 2-14. The 200-year coastal flood elevation is given and plotted to define a zone of coastal flooding. In this case, relative to geodetic datum, the mapped 200-yr coastal flood elevation of 3.8 m included a higher high water tide of 1.9 m, a 200-yr storm surge of 1.15 m, and a wave runup provision of 0.7 m. Additional freeboard was not included. A similar map for riverine areas, shown earlier in Figure 2-3, specifically states that freeboard was included. The resulting coastal flood elevation of 3.8 m was then mapped as a smooth curve overtop of the more irregular topography.

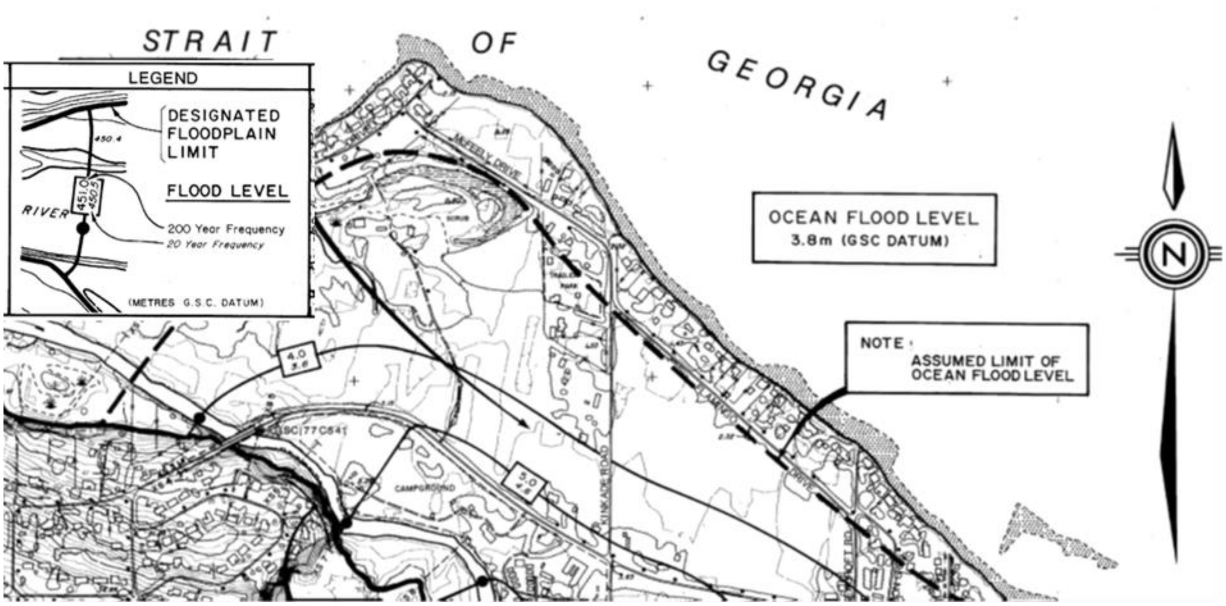
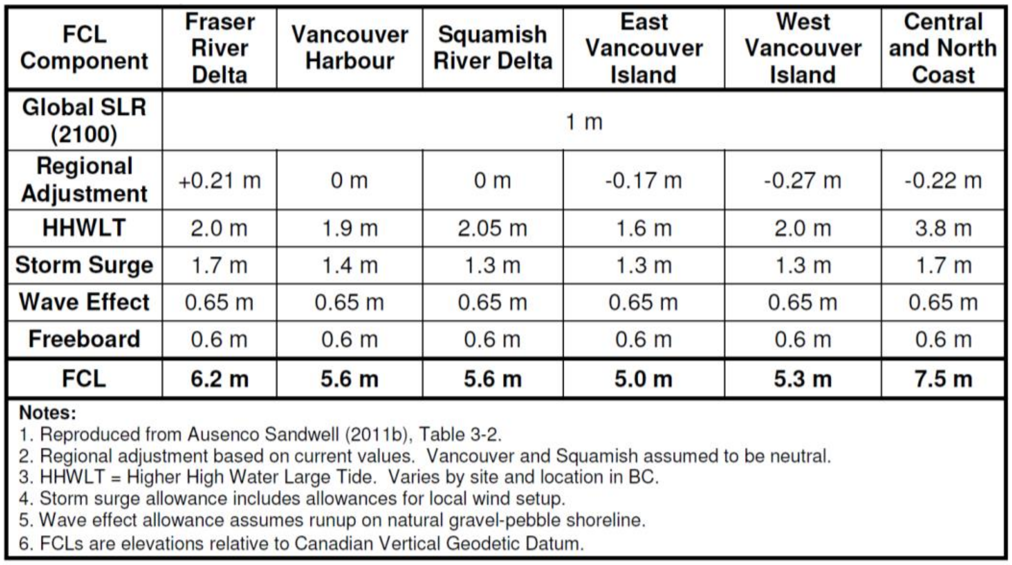


Figure 2-14 Partial Section of Vancouver Island Flood Map (Little Qualicum River) (source: Hay and Company Floodplain Map, 1997)

A more recent flood study for British Columbia, *Coastal Floodplain Mapping Guidelines and Specifications* prepared by Kerr Wood (2011), includes additional recommendations for climate change. Table 2-2 lists the various components of recommended Flood Construction Levels. In this case, the values include a scenario with a global sea level rise of 1 m by year 2100. This is then adjusted with regional scenarios for sea level rise and vertical land movement. The HHWLT, storm surge, and wave runup effects are then added. The FCL then also includes 0.6 m of freeboard.

Table 2-2 Preliminary FCL estimates for locations in British Columbia (source: Kerr Wood Leidel, 2011)



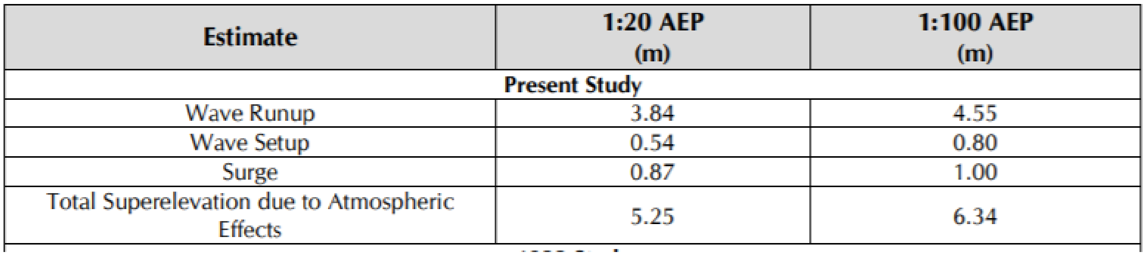
FCL Component	Fraser River Delta	Vancouver Harbour	Squamish River Delta	East Vancouver Island	West Vancouver Island	Central and North Coast
Global SLR (2100)	1 m					
Regional Adjustment	+0.21 m	0 m	0 m	-0.17 m	-0.27 m	-0.22 m
HHWLT	2.0 m	1.9 m	2.05 m	1.6 m	2.0 m	3.8 m
Storm Surge	1.7 m	1.4 m	1.3 m	1.3 m	1.3 m	1.7 m
Wave Effect	0.65 m	0.65 m	0.65 m	0.65 m	0.65 m	0.65 m
Freeboard	0.6 m	0.6 m	0.6 m	0.6 m	0.6 m	0.6 m
FCL	**6.2 m**	**5.6 m**	**5.6 m**	**5.0 m**	**5.3 m**	**7.5 m**

Notes:
1. Reproduced from Ausenco Sandwell (2011b), Table 3-2.
2. Regional adjustment based on current values. Vancouver and Squamish assumed to be neutral.
3. HHWLT = Higher High Water Large Tide. Varies by site and location in BC.
4. Storm surge allowance includes allowances for local wind setup.
5. Wave effect allowance assumes runup on natural gravel-pebble shoreline.
6. FCLs are elevations relative to Canadian Vertical Geodetic Datum.

Additional examples of coastal flood studies and flood mapping were found in Newfoundland and Labrador. In one flood study, by AMEC (2013), flood elevations for AEPs of 1:20 and 1:100 were developed for the Ferryland location. Combined storm surge and high tide levels would result in water levels of 1.97 m and 2.23 m, respectively for the 1:20 and 1:100 AEP flood events. In addition, wave run-up was estimated. Numerous coastal transects were considered and runup was found to add between 0.5 m and 4.9 m to the storm surge and high tide levels. Runup was very site specific and dependent on both the site exposure to waves as well as on the local ground slope.

Another study, by Hatch (2012) developed flood maps for the Stephensville area of Newfoundland. Water level components are shown in Table 2-3, and include the wave setup, surge due to atmospheric effects, and wave runup. In this case, wave runup was found to be the largest component. In subsequent analysis, tide effects were added. But unlike many other studies that add surge and wave effects to the HHWLT, a joint probability analysis was conducted to develop estimates of how the 1:20 and 1:100 AEP surge events would phase with time-varying astronomical tides at the given AEP level.

Table 2-3 Coastal flood components for Stephensville, NL (source: Hatch, 2012)



Estimate	1:20 AEP (m)	1:100 AEP (m)
Present Study		
Wave Runup	3.84	4.55
Wave Setup	0.54	0.80
Surge	0.87	1.00
Total Superelevation due to Atmospheric Effects	5.25	6.34

Because the runup was the largest component of the flood levels, two sets of flood maps were produced. Figure 2-15 shows the mapped flood elevations for a base case without wave runup. This format is useful as it most accurately reflects the flood depth, when ground elevations are also considered. Figure 2-16 then shows the mapped elevations with wave runup. This format is useful as it conveys a better idea of the total extend of the flood hazard, in this case due to wave impacts.

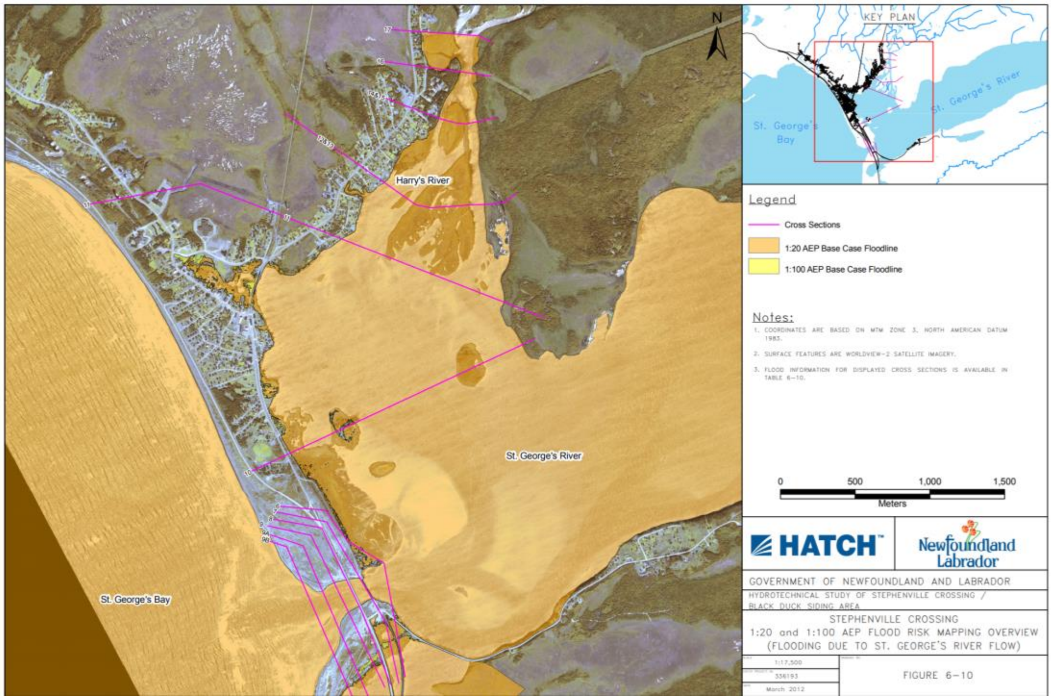


Figure 2-15 Illustration of base flood elevations (source: Hatch, report to Government of Newfoundland and Labrador, 2012)

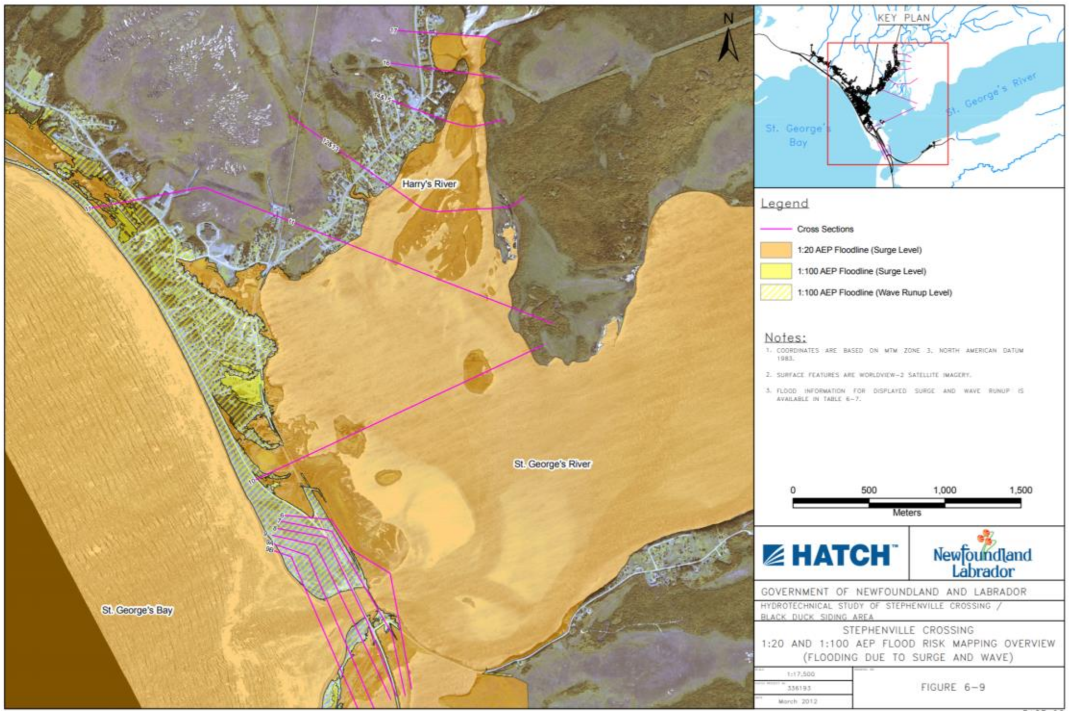


Figure 2-16 Illustration of flood elevations with wave runup (source: Hatch, report to Government of Newfoundland and Labrador, 2012)

2.6 Description of Coastal Wave Effects

As noted in the previous examples, shorelines of large lakes, wind-generated waves provide a potential source of flooding and of flood loads. As illustrated in Figure 2-17, waves can increase flood levels along a shoreline through the process of wave setup. Wave setup is an increase in the mean water level inshore of the wave breaker zone caused by the action of breaking waves. Wave setup may be on the order of 10 to 20% of the breaking wave height, so 2 m high breaking waves could elevate the mean water level approximately 0.2 to 0.4 m. Wave setup is an average water level over time and does not fluctuate with the passing of each wave. It is therefore often added to the SWL from tides and storm surge to determine the design flood level (DFL) at a site.

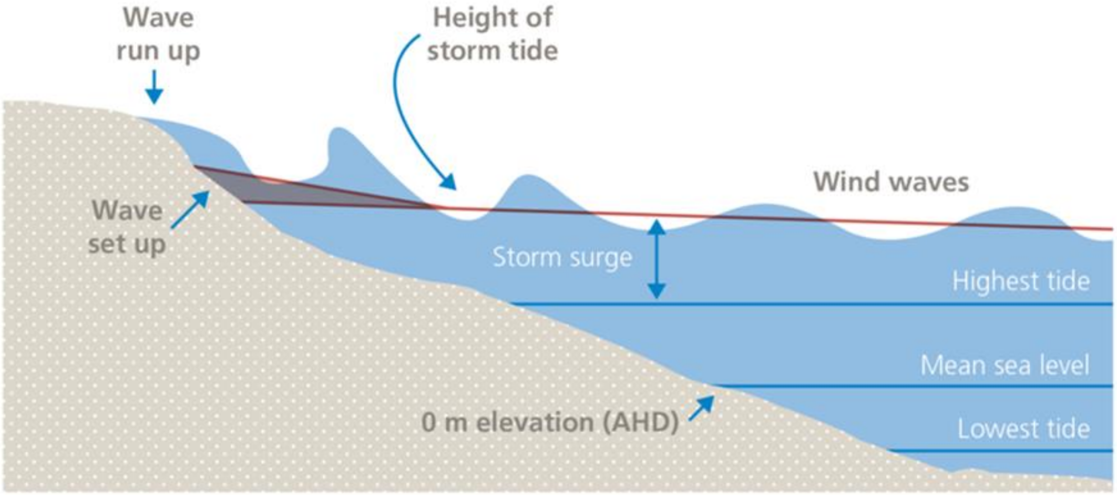


Figure 2-17 Schematic of the surf zone showing principal contributors to the coastal water level and shoreline elevation (source: State of Victoria, Dept. of Sustainability and Environment 2012)

Some of the transient effects of waves are illustrated in Figure 2-18. During a major flood event, flooding may allow waves to propagate from the open water body (ocean, bay, lake) across the flooded ground landward of the normal shoreline. Wave crests in this region increase the elevation reached by flood waters with each passing wave. As waves reach a ground slope or a shore protection structure, waves also rush up the shoreline slope to allow water to reach both a higher elevation and farther inland than would occur if waves were absent. The vertical elevation reached by waves on a shore slope above the storm surge elevation is the wave runup. The permitted elevation of buildings in flooded coastal zones should consider the higher of the wave crests or wave runup.

One critical factor in predicting wave effects, is that wave setup is usually implicitly included in any formula used to predict wave runup. Methods of estimating runup are empirical and as such have traditionally defined the runup elevation relative to the SWL. Therefore, the two effects are not computed independently. Because runup reaches much higher level than the wave setup, runup is usually used to delineate wave effects at a shoreline.

In some cases, wave runup may overtop a beach, bluff, or coastal protective structure. If the area landward has a lower elevation, the overtopping flows may lead to a retention of water (ponding) and flooding at elevations much higher than the lake, bay, or ocean water levels.

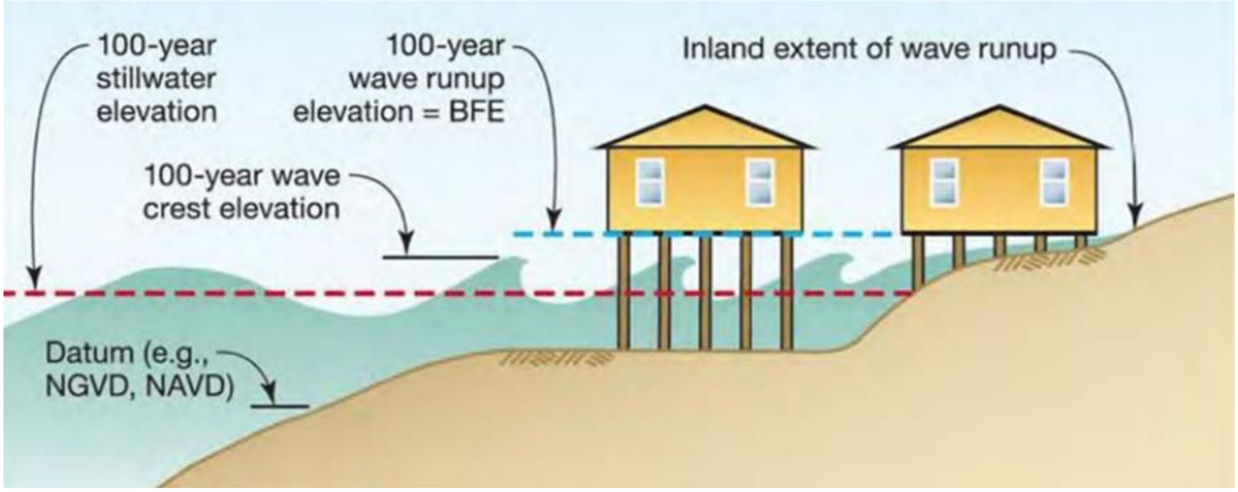


Figure 2-18 Determining building first floor elevation based on wave conditions (source: FEMA Coastal Construction Manual 2011)

In the US, wave effects are routinely displayed on flood hazard maps for all coastal areas and for the Great Lakes shorelines. Methods for including waves in flood hazard mapping are outlined in many FEMA documents, e.g. FEMA (2011), FEMA (2014), FEMA (2019).

In general, FEMA adopts a 100-year tide plus storm surge (or lake level plus surge on Great Lakes) along with wave setup to determine the DFL. Areas flooded by the 100-year DFL with wave heights (trough-to-crest) greater than 0.9 m (3 ft) are designated as Coastal High Hazard Areas or Coastal V zones. Areas subjected to the 100-year DFL with waves less than 0.9 m (3 ft) are designated as High Risk Coastal Flood Hazard areas or as Coastal A zones. FEMA then determines a Base Flood Elevation (BFE) by adding wave crest or wave runup to the DFL. Communities may also adopt local freeboard standards

At present, waves are not included consistently in Canadian flood hazard studies or flood hazard mapping. Many areas do not include any wave effects, and the mapped flood elevation is the SWL from astronomical tides plus a storm surge (usually 100 year MRI). Because wave effects can reach much higher elevation, the lack of inclusion of wave effects can dramatically underestimate the elevation reached by flood waters and flood loads.

In areas with overland flooding subject to wave action, the FCL for a coastal location should be equal to or greater than the design flood elevation (DFL) based on the crest elevation of waves. Therefore, the FCL should be equal to the sum of elevations of HHWLT + storm surge + wave crests above surge as illustrated in Figure 2-19.

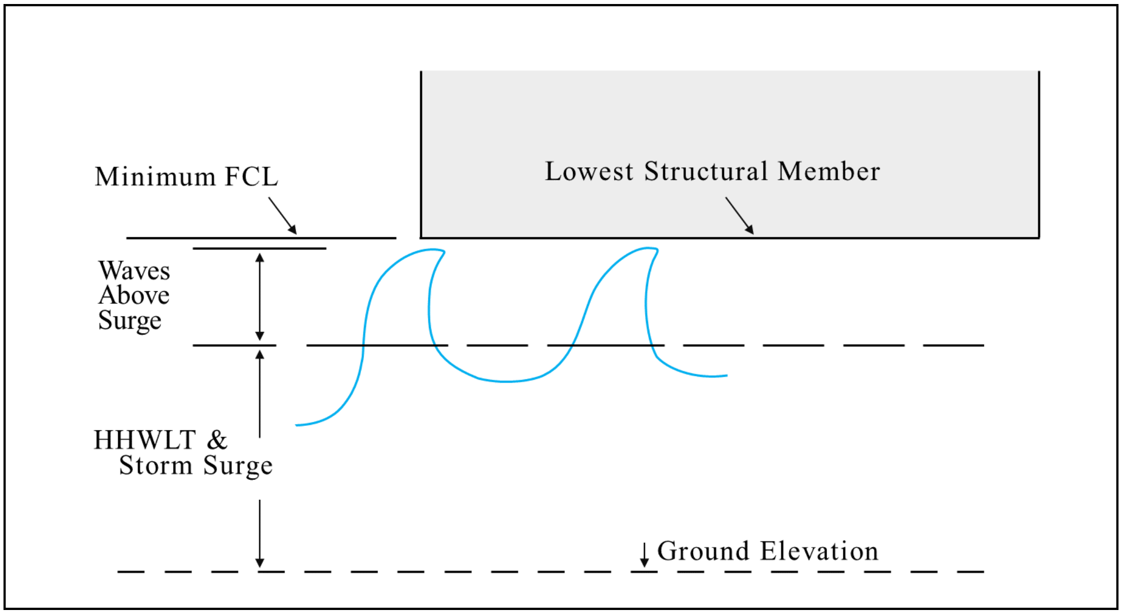


Figure 2-19 Relationship between DFL and FCL for coastal flood conditions

Only one study was found, however, that included wave crest or wave amplitude effects, by Northwest Hydraulic Consultants (2014) for the City of Vancouver. Wave amplitudes were modeled across the shoreline and flooded overland areas. A "Wave Effects Boundary" was defined and it was suggested that a 0.3 m vertical allowance could be adopted in establishing Flood Construction Levels (FCL). Zones where wave impacts could affect structures were identified as "Wave Effect Critical Areas." It does not appear that the results of the study were ever included in official flood hazards maps published by the Province of British Columbia.

As discussed earlier, flood maps showing the FCL in British Columbia (BC) appear to include the vertical wave runup above the SWL. Runup is a probabilistic quantity owing to the randomness of runup wave-by-wave in a sea state, and the 50% (median) runup was adopted as representative for natural shorelines while the 2% exceedance value was adopted for sea dikes. For most of BC, it was concluded that a 0.65 m vertical allowance could be applied.

Similar methods applied in studies in Newfoundland found much great runup values, varying from transect to transect from a low of 0.5 m to a high of 4.9 m depending on the degree of exposure and (presumably) shoreline slope of each transect. Values of runup were added to the SWL to establish the mapped flood elevation. Inclusion of runup was only found for two Newfoundland communities, so that wave effects are not included in the majority of coastal areas.

The most consistent inclusion of wave effects is in Ontario. based on guidance from the Ministry of Natural Resources (2001). These guidelines suggest that the flood hazard limit should be based on the 100-year water level (lake level plus surge) plus an allowance for wave runup and other water related hazards, as shown in Figure 2-20. The allowance can be based on a site-specific technical analysis. However, in the absence of a site-specific analysis, the guidelines suggest the use of a standard 15 m horizontal setback from the 100-year still water level shoreline. For the St. Lawrence River and connecting channels, a 5 m horizontal distance is specified.

The use of a standard horizontal allowance is appealing due to its simplicity. It also aligns with some of the known variation of wave runup with beach or shoreline slope. For a mildly sloping beach with 1:30 slope, the 15 m horizontal zone would translate into a 0.5 m vertical runup elevation. Similarly, a 1:15 ground slope would correspond to 1 m of vertical runup, a 1:10 ground slope would correspond to 1.5 m of vertical runup a 1:7.5 slope would correspond to a 2 m runup, and a 1:5 slope would correspond to a 3 m runup. These may be viewed as realistic values of runup for the Great Lakes with a realistic variation with ground slope.

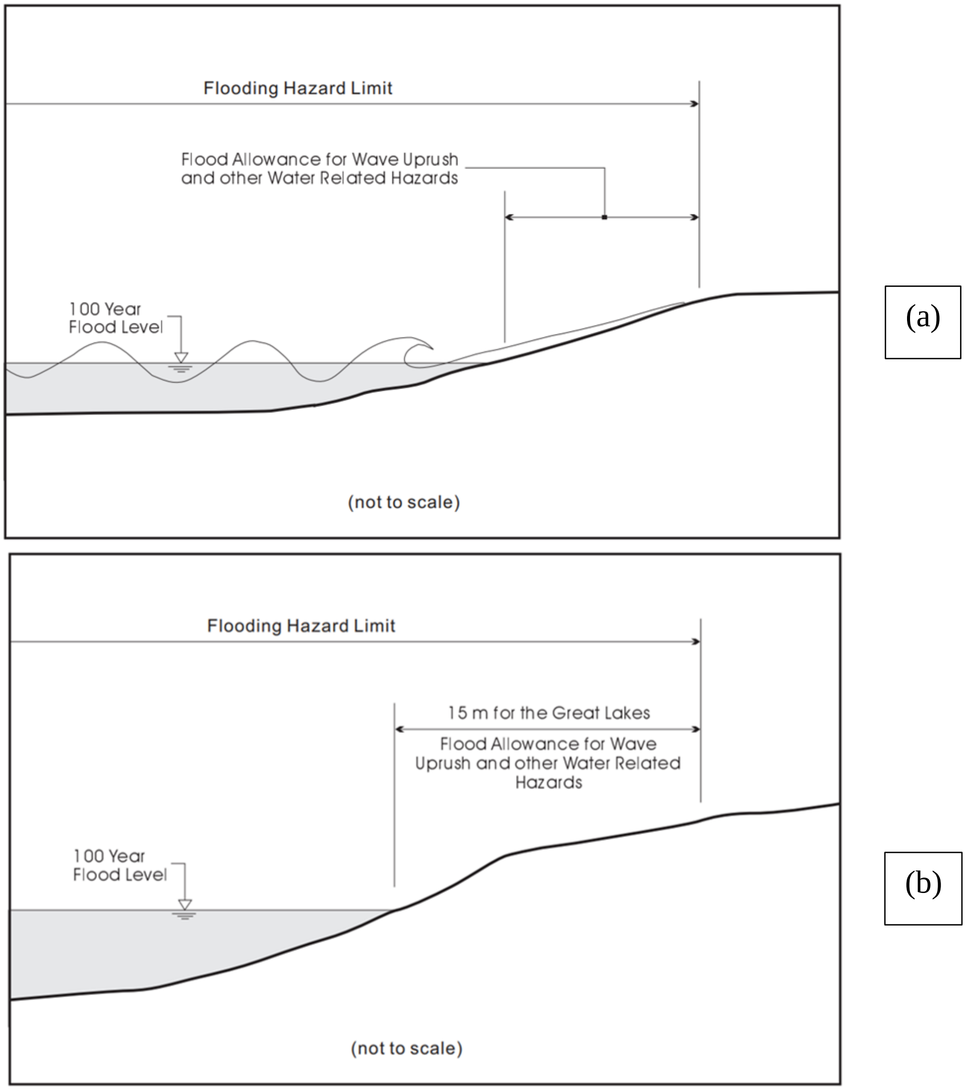


Figure 2-20 Wave runup guidance (source: Ontario Ministry of Natural Resources, 2001)

2.7 Other Factors for Flood-Resistant Design

While the primary factors in structural design for floods are the flood elevation, flood depth, flood velocity, and waves conditions along lake or coastal shorelines, other factors must also be considered for improving flood resistant design.

2.7.1 Flood Duration

The length of time a flood impacts a building will have a substantial influence on the degree of damage caused to the building. Of course, a long duration flood coupled with significant flood depth will cause more damage than long duration flooding that is not deep enough to overtop the flood protection system of the building.

Long term flooding will affect the ability of the materials below the flood level to resist the flood and those materials can potentially introduce harmful bacteria in the materials. Items and/or equipment located in below-flood level locations will be inundated with flood water and may have to be either removed or replaced. Flood durations also impact building access and function, especially if the function is critical to the community (e.g. hospital, police, fire protection). Long duration flooding, especially to buildings that are closed up, can create harmful mold and bacteria growth. Studies on the long-term effects to building materials after flooding are available (FEMA 549, Hurricane Katrina MAT Report, 2006). Information on materials that are considered flood-resistant can be found in FEMA's Technical Bulletin collection (Technical Bulletin No. 2 - https://www.fema.gov/media-library-data/20130726-1502-20490-4764/fema_tb_2_rev1.pdf).

2.7.2 Flood-Borne Debris

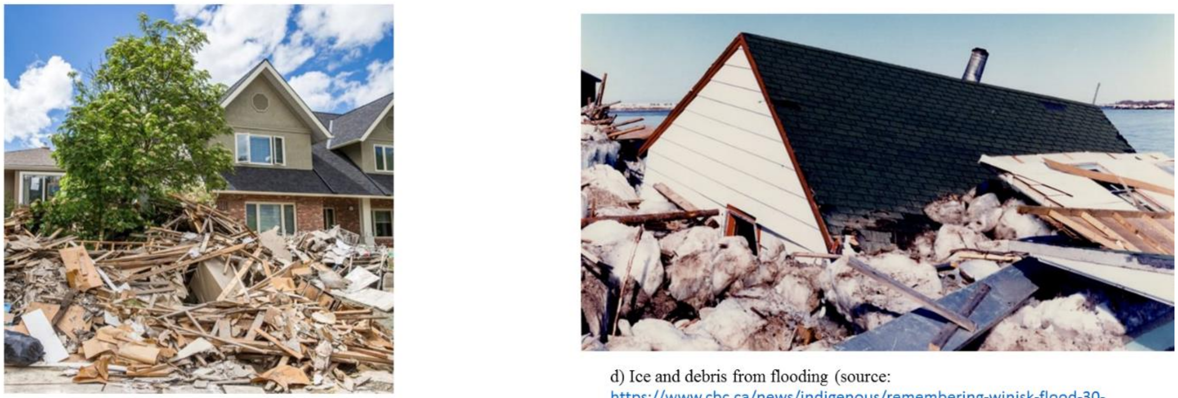
Flood water can carry debris from upstream (if riverine flooding) or coastal sites (if coastal flooding). This debris may impact other buildings causing collateral damage to those buildings. Sometimes the damage is caused by components of other buildings (as shown in Figure 2-21 a), automobiles (as shown in Figure 2-21 b), wood debris (as shown in Figure 2-21 c), or ice (as shown in Figure 2-21 d).

The difficulty in designing for flood-borne debris is that the size and weight are unknown and unpredictable. The uncertainty about where debris might originate, and if the debris occurs, how fast it might travel and on what part of the building it might impact is large. In general, the uncertainty associated with the flood hazard is very large, and the engineer should assess the extent of damage likely caused by such debris.

Riverine locations are most likely to experience impact from wood or steel building components, especially from stairs or exterior building components, which come loose and float downstream. Wood in forested areas can easily float. Automobiles and small boats can easily float and may impact buildings downstream. In contrast, coastal locations are likely to experience debris from docks and piers, larger vessels, including fishing and recreational boats and barges, and debris that has been torn off other buildings.

a) House floated into house (source: https://www.canada.ca/en/environment-climate-change/services/water-overview/quantity/floods/events-ontario.html)

b) Car floated into house (source: https://www.thestar.com/news/canada/2019/04/16/quebecs-beauce-region-hardest-hit-by-springtime-flooding-province-on-alert.html)



c) Wood debris from flooding (source: https://www.calgary.ca/uep/water/pages/flood-info/flooding-history-calgary.aspx)

d) Ice and debris from flooding (source: https://www.cbc.ca/news/indigenous/remembering-winisk-flood-30-years-later-1.3588024)

Figure 2-21 Photos of debris carried by flood waters

Ice debris is especially important to areas of Canada given the long duration freezing temperatures that occur in winter. Ice can be pushed together creating a damming effect and possibly raising the overall flood depth. Ice jams can create large stresses on buildings and other structures. Ice that continues to be pushed into itself can cause a crushing of any material resisting this debris, such as a building. Since ice is a collection of frozen water, any sudden large temperature increases could cause a flash flood downstream, and significantly increase the velocity of the water in the river.

Ice jam data from the (2013) AMEC report in Newfoundland and Labrador were reviewed for two bodies of water to determine how much effect ice jamming had on flood levels. Data were provided for ice jam levels compared to open water levels. Generally, the open water levels were higher than ice jam levels at the upper reaches of both streams/rivers. Then as the flow moved downstream, the ice jam levels were higher than those in open water. At the furthest extent downstream, the open water levels again were higher. A study of 50 data points on the two bodies of water suggested a mean increase in water levels with an ice jam of 0.95 m when the mean level was added to one standard deviation. In the absence of a specific ice jam study, this one study suggests adding 1 m of flood depth to those areas subject to ice jamming.

In general, it is assumed in this report that the effect of ice jams on flood elevation is included in any flood hazard study used to develop flood plain maps. If this is not the case, and ice jams are not included in the water level mapping at a particular site, it would be prudent to gather as much data as possible about the effects on flood levels of ice jams in order to determine a locally-appropriate water level increase in the design flood level. Similar provisions would apply to any modification of flow velocity due to ice jam effects. Direct loads due to accumulation or impact of ice on a structure are treated as debris loads and methods outlined in Chapter 3 can be used to estimate loads.

Debris loads on buildings should be treated as an extraordinary, unusual load. As such, debris loading should not be added to other flood loads as a normal design condition, but rather should be handled as a load for which collapse prevention is designed. Most debris loads are impact loads and are sudden and of short duration. Most often, these loads may be applied as concentrated loads acting on one critical structural member unless ice or debris damming causes a large more uniform load to be applied to the structure.

2.7.3 Scour and Erosion

Scour and erosion describe processes in which soil around or under buildings and building foundations is lost. Scour is most often associated with the loss of soil from a particular flood event at a building support column, pile or building corner, thus jeopardizing the structural integrity of the building support system. Erosion is most often associated with the loss of soil over a site, or along a riverbank or coastline, as water washes along these stretches of land. In both cases, soil is lost which lowers the ground elevation at or near the building, and this lost soil is replaced with water during a flood. Thus, water depths increase where scour and erosion occur, and thus flood loads increase as the water depths increase.

In the US, the best erosion information has been provided by studies conducted along the coastline prior to and after large storm events, especially hurricanes and nor’easters. Usually either state universities with coastal processes research programs or state governments with coastal environmental protection programs conduct studies on the loss of soil that occurred from these storms. Sometimes this information is collected by LiDAR methods, while other times it might be collected by surveying transects along the coast. This information is then accumulated over time, and long-term erosion projections can be made on the expected loss of soil over some time frame. The recommended time period projection is at least 50 years which represents the expected life of most normal-use buildings. The expected life could be longer for some critically important buildings, such as hospitals. Government buildings might also expect to have a long service life.

Scour depths caused by the flow of water around a building foundation element are a function of water velocity, soil type, and building element size and shape. The loss of soil expected from scour is usually a one-time occurrence, that is, loss of soil during one event. After that event, the soil lost by scour is usually replaced, and then the building foundation element is fully supported once again by soil around or under it, once the soil is re-consolidated by either machine or nature. Figure 2-22 illustrates the concepts of erosion and scour for pile foundations and illustrates the differences.

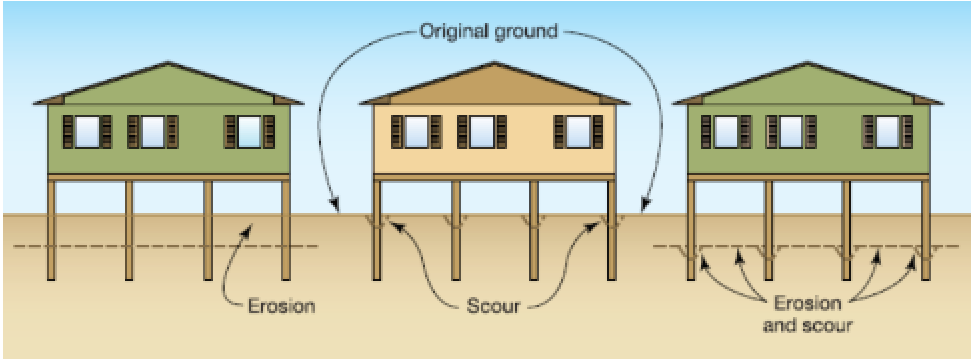


Figure 2-22 Illustration of difference between scour and erosion (source: FEMA Coastal Construction Manual, 2011)

There are examples of erosion on Canada’s coastlines as well. Figures 2-23 a) and b) illustrate some of the coastal erosion issues that must be addressed in Canada.

a) Eroded shoreline of Berrien County, MI, US 1997 (Source: Baird Erosion Theme Report, 2010)

b) Erosion of Lake Erie Shoreline (Source: Derek Williamson – presentation to National Research Council-Canada, 2020)

Figure 2-23 Examples of erosion along Canada’s coastlines

Erosion rates are a function of soil types and water levels in lakes and storm surge and wave history on the open coastlines. There are shorelines where the primary soil material is bedrock, where the primary material is cohesive, and where the primary soil is sand. The erosion processes and the erosion timelines for these soils can be very different depending on water levels and storm events that impact the shoreline. While no typical erosion rates were found for Canadian locations, there are studies that have been completed for certain locations where erosion is known to be causing a problem for developed land. Figure 2-24 illustrates a small (approximately 90 m) projected shoreline for Lake Michigan over a 50-year period, illustrating the differences in erosion in an extremely wet and extremely dry scenario.

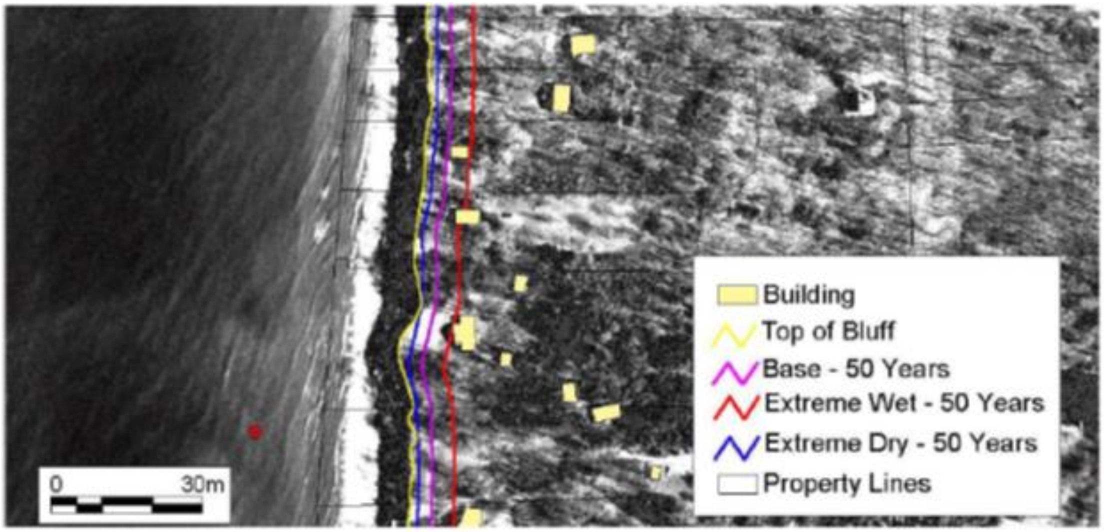


Figure 2-24 Projected loss of bluff on Lake Michigan in 50-year period (source: Baird Erosion Theme Report, 2010).

2.7.4 Permafrost

Permafrost is soil that has frozen layers of ice imbedded within it caused by extremely low temperatures. The areas of permafrost typically are found in the far northern sections of the country. The NBC Structural Commentaries-User's Guide: Part 4 of Division B Commentary K (NBC 2015) has a very general map of the likely permafrost locations shown here as Figure 2-25. Zone 2 on the map is the area of continuous permafrost, Zone 1 is the area of discontinuous permafrost and the area south of Zone 1 is the area where permafrost does not normally exist.

Figure 2-25 Map of Permafrost Zones in Canada (Source: Figure K6, Commentary K NBC Structural Commentary, Part 4, Division B 2015).

There is evidence that temperatures within the permafrost are rising as part of a warming climate. Temperature increases have been recorded from +0.10C to 0.90C per decade. Northern Quebec has had temperature increases of as much as 1.00°C recorded (Canada's Changing Climate Report, 2018). Increasing temperature in the permafrost can release hydrocarbons, allows the frozen earth to become soft and the melting lenses of ice in the earth causes subsidence. An ASCE report indicated that as much as 20 cm of surface subsidence in the Northwest Territories could occur (ASCE Cold Regions Engineering, 2019). For purposes of this chapter on flood design, subsidence is the primary future condition that users of this guidance should assess as to the impact of ground lowering, which might lead to increasing flood depths.

2.7.5 Water Velocity Caused by Failure of Flood Protection Structures

There are many flood protection structures built in floodplains, including dams, levees, seawalls, floodwalls, and shore protection devices designed to slow down coastal erosion. There are two important flood considerations related to these types of structures. Some structures are designed to hold back water and to not allow the water being held to be released downstream in such a way that the release of water causes flooding and flood damage downstream. The other consideration is that some structures cause water to flow around them and, in these cases, water can be diverted onto other adjacent properties causing damage. Also, water flowing around objects frequently causes scour to occur at their foundations and thus cause significant damage to these structures.

Water that has been stored or trapped behind a flood protection structure such as a dam or levee will release with significant force if the flood protection structure fails. Communities and individual property owners who have structures downstream of flood protection structures should consider the impact to them should the structure fail. Failures can occur in dams by cracking or overtopping the dam; failures can occur in levees by having water undermine the toe of the levee and "blow out" a section of the earthen structure. In some cases, flood control measures have to be taken by releasing some of the water behind a dam, which can cause significant flooding.

It is recommended that for those property owners or communities that are potentially in harm's way downstream of a dam or levee, engage a consultant who can help define the probable velocity of flood water for a given dam break or levee breach condition. While the result is a "modeled" result, perhaps using numerical modeling, the predicted velocity of flooding is much better than having no idea at all of how much impact there might be from such a flood condition.

For flood protection structures such as dams and levees, the structures themselves should be designed and certified to provide protection to some flood protection elevation that is consistent with a post-disaster structure type (see Table 3-4). Given the level of importance of these flood protection structures, the design event MRI should be much higher than the flood design levels used for buildings downstream of this protection.

2.8 Design Flood and Flood Frequency Considerations

In order to create a uniform risk basis for flood-resistant design across Canada, it would be useful for each province to adopt the same flood frequency basis to which their flood hazard maps will conform. At present, risk bases have been established separately for every Province and are provided in Table 2-4 (from the *Assessment of Canadian Floodplain Mapping and Supporting Datasets for Codes and Standards Technical Report* issued by the NRC in 2017). It is important to managing the flood risk nationally to have all provinces use the same standard (same AEP or MRI). This section will discuss a proposed national standard that could be considered in the future editions of NBC.

Table 2-4 Level of Severity (return period) of the Designated/Design Flood (source: NRC, 2017)

Province/Territory	**Supporting information (Category: Floodplain maps)**
Alberta	100-year flood
British Columbia	200-year flood
Manitoba	100-year flood for old maps; 200-year flood has been approved but no maps have been created with this level; historic floods
New Brunswick	Historic flood events
Newfoundland & Labrador	20- and 100-year floods for floodway and flood fringe delineations, respectively
Nova Scotia	20- and 100-year floods for floodway and flood fringe delineations, respectively
Ontario	100-year flood and historical storms (Hurricane Hazel and Timmins)
Prince Edward Island	10-, 25-, 50-, and 100-year storm surge
Quebec	20- and 100-year floods for floodway and flood fringe delineations, respectively
Saskatchewan	500-year flood
Northern Territories	100-year flood
Nunavut	Maps do not exist
Yukon	Unknown for the new initiative; some communities were mapped with 200-year flood

The use of a return period to define the design flood event is a statement of probability. A 100-year MRI flood event is a flood that has a 1% annual chance of being equaled or exceeded in any given year, or a 1:100-AEP flood. Since this is a statement of probability, it does not mean there is a 100% probability of this event magnitude occurring at all, nor does it mean that there is no chance for a very unlikely event to occur. In fact, the total probability of a structure encountering a flood event depends on how many years the structure is in place. Table 2-5 illustrates the probabilities that certain AEP events (in decimals) will be equaled or exceeded over various time periods (1 year to 100 years).

Table 2-5 Flood Encounter Probability for Event AEP vs. Time Period

	Time Periods						
Event AEP	1 yr	10 yrs	20 yrs	30 yrs	50 yrs	70 yrs	100 yrs
1:10	0.10	0.65	0.88	0.96	0.99	1.00	1.00
1:20	0.05	0.40	0.64	0.79	0.92	0.97	0.99
1:50	0.02	0.18	0.33	0.45	0.64	0.76	0.87
1:100	0.01	0.10	0.18	0.26	0.39	0.51	0.63
1:200	0.005	0.05	0.10	0.14	0.22	0.30	0.39
1:500	0.002	0.02	0.04	0.06	0.10	0.13	0.18
1:750	0.0013	0.01	0.03	0.04	0.06	0.09	0.12
1:1000	0.001	0.01	0.02	0.03	0.05	0.07	0.10
1:2000	0.0005	0.005	0.01	0.01	0.02	0.03	0.05
1:3000	0.0003	0.003	0.007	0.01	0.02	0.02	0.03

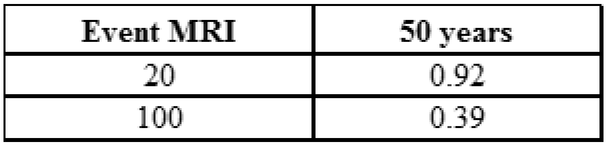
The formula used to determine these encounter probabilities is $P_n = 1-(1-P_a)^n$ and is provided in the American Society of Civil Engineers load standard ASCE 7. Table 2-6 can be helpful in selecting a flood return period to use, based on what probability of occurrence or risk is considered acceptable. It must be pointed out that each flood event MRI has an associated flood elevation. Each successively greater MRI has a higher flood elevation. The 3000-year MRI flood event would likely be catastrophic, even though it has a very low probability of occurrence (only a 3% chance of occurrence in a 100-year period).

The consideration for adopting a certain design flood MRI should balance the cost of flood mitigation with the cost of flood recovery and the impact to people and businesses. Important factors for selection of the design MRI include:

- The number of people or businesses displaced from their home or place of work;
- The overall economic impact to a community;
- The number of buildings likely damaged;
- The importance of the facility to community resiliency and a return to normal operations;
- The impact to community critical facilities with a requirement to remain operational before, during, and after events
- The accessibility of buildings where access is not extremely difficult (not too high, with too many stairs, etc.);
- The risk of flood damage to be commensurate with risks for other hazards, especially if hazards can occur simultaneously; or
- Other community metrics important to flood recovery.

If the most common flood return periods currently used in Canada are viewed in light of encounter probabilities over a 50-year building life, using the probabilities illustrated in Table 2-5, the expected probability of flood elevations represented by 1:20 AEP and 1:100 AEP to be equaled or exceeded in 50 years are highlighted in Table 2-6.

Table 2-6 Expected Probabilities for 20 and 100-year MRI flood events in 50 year time period, highlighted from Table 2-7



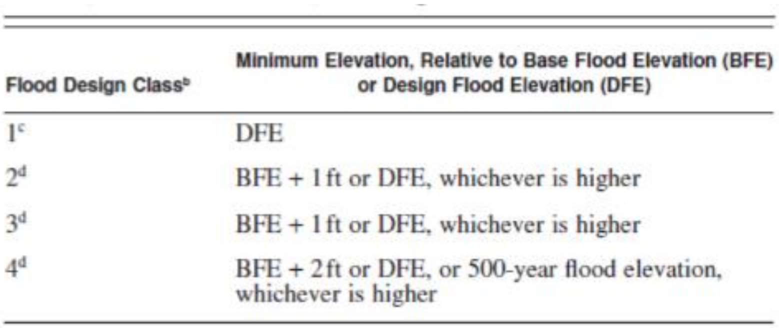
Event MRI	50 years
20	0.92
100	0.39

This suggests that there is a 92% chance, nearly a certainty, of a building being exposed one or more times to the 20-year event in a 50-year period. There is a 39% chance that buildings built to the 1:100 AEP flood elevation will be flooded at least once in a 50-year life. This establishes a likely scenario of very significant damages to buildings built at or below the 1:100-year flood elevation. It would suggest a major impact to a community generally built at or below this 1:100 AEP flood elevation. It also suggests that existing buildings built before the adoption of flood maps will be extremely vulnerable if the design flood occurs.

ASCE 24, Flood-Resistant Design and Construction Standard, establishes minimum design flood elevations to be used for various building types and risk categories. These minimum requirements are then adopted by reference by the US building codes, and thus become part of the code. There is a defined elevation for each flood class provided in the ASCE 24 standard, where flood class is defined as "a classification of buildings and other structures for determination of flood loads and conditions, and determination of minimum elevation requirements on the basis of risk associated with unacceptable performance.“ The required minimum elevation for buildings are defined from the BFE (Base Flood Elevation, in the US this is the 100-year flood) plus some freeboard, as shown as Figure 2-26 (Table 2-1 copied from ASCE 24-14).

The DFE in Figure 2-26 is defined by ASCE 24-14 as the design flood elevation (analogous to the DFL) where the design flood is "the flood associated with the greater of: (1) area within a floodplain subject to a 1% or greater chance of flooding in any year, or (2) area designated as a flood hazard area on a community's flood hazard map or otherwise legally designated". Footnote (c) indicates that Flood Class I structures shall be allowed below the minimum elevation if the structure meets the wet floodproofing requirements (meaning water is allowed to flow into the structure through flood vents). Footnote (d) indicates that for non-residential buildings or non-residential areas of mixed-use buildings, the lowest floor shall be allowed below the minimum elevation if the structure meets dry floodproofing requirements. The report on flood resistance of existing buildings has more information about both wet and dry floodproofing.

Flood class is similar to Risk Category used in the ASCE 7-16 standard, except the Risk Category include a broader range of hazards as "a categorization of buildings and other structures for determination of flood, snow, ice, and earthquake loads based on the risk associated with unacceptable performance." The risk is the risk to human life, health and welfare associated with damage or failure by nature of their use or occupancy. The Risk Categories are I, II, III, and IV, where I is low risk to human life, II is the default category (but include all residential and most commercial buildings), III is primarily high occupancy buildings and hazardous substances, and IV are those structures important to the community and considered critical such as hospitals, police and fire stations, emergency operations centres and other similar structures.



Flood Design Class[b]	Minimum Elevation, Relative to Base Flood Elevation (BFE) or Design Flood Elevation (DFE)
1[c]	DFE
2[d]	BFE + 1 ft or DFE, whichever is higher
3[d]	BFE + 1 ft or DFE, whichever is higher
4[d]	BFE + 2 ft or DFE, or 500-year flood elevation, whichever is higher

Figure 2-26 ASCE 24-14 Table on Minimum Elevations

In the US, there is the same disparity between risk-based design for various hazards. The design basis for design wind speeds is 1:700 AEP for Risk Category II buildings, including most residential and commercial buildings. The design wind speed level for the most critical buildings is 1:3000 AEP. Yet most communities in floodplains still adopt flood maps that only represent the 1:100 AEP event. Ironically, those in coastal communities have designs for new buildings that are expected to resist a 1:700 AEP wind event but only a 1:100 AEP flood event. Flood design professionals in the US are increasingly of the opinion that the 100-year MRI standard simply carries to much risk, or has too high a probability of occurrence (per ASCE 7-22 Task Committee of Flood Loads, 2021)

The design categories adopted in the US codes are similar to the Importance Categories for Buildings from the National Building Code (NBC) of Canada, as described in Figure 2-27 (Table 4.1.2.1 copied from the NBC).

Use and Occupancy	Importance Category
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including: • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings	Low
All buildings except those listed in Importance Categories Low, High and Post-disaster	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is: • as an elementary, middle or secondary school • as a community centre Manufacturing and storage facilities containing toxic, explosive or otherhazardous substances in sufficient quantities to be dangerous to the public if released	High
Post-disaster buildings are buildings that are essential to the provision of services in the event of a disaster, and include: • hospitals, emergency treatment facilities and blood banks • telephone exchanges • power generating stations and electrical substations • control centres for air, land and marine transportation • public water treatment and storage facilities, and pumping stations • sewage treatment facilities and buildings having critical national defence functions • buildings of the following types, unless exempted from this designation by the authority having jurisdiction: o emergency response facilities o fire, rescue and police stations, and housing for vehicles, aircraft or boats used for such purposes o communications facilities, including radio and television stations	Post-disaster

Figure 2-27 Building Importance Categories as listed in Table 4.1.2.1 from the National Building Code of Canada (2015)

The establishment of a design flood event return period must consider:

- Importance category of buildings listed in Table 4.1.2.1 of the NBC;
- Community (province) tolerance for flood losses, community disruption due to flooding, and ability to recover from flooding, and the role public education has in understanding the flood hazard and flood risk;
- Location in spatial and vertical directions of community critical facilities compared to the selected design flood return period;
- Flood history (depth and frequency) in a community.

Most Canadian provinces have included freeboard in the flood elevation for a defined flood event. The inclusion of freeboard essentially increases the flood return period by some arbitrary amount. For purposes of determining the appropriate flood design return period, freeboard should not be included, and it should then be considered to be added to the flood elevation explicitly for other purposes but not to effectively increase the design MRI.

Guidance on selecting a nationally applicable flood return period follows with the use of some examples (not intended to reflect either current or real situations in any province).

> **Example 1:** A community has extensive flood history, a history of flood damage and flood recovery efforts, and flood maps that define the extent of flooding for the 1:100 AEP event. In the past 20 years, several floods have either reached the 1:100 AEP flood extent or exceeded the elevation and extent affecting primarily ‘ordinary” buildings including residences. The probability of a 1:100 AEP event being equaled or exceeded is 39% over 50 years (see Table 2-7). This is a very high probability. A risk reduction should be considered. A reduction to a 10% probability in 50 years would require a design flood event to have a 1:500 AEP.
>
> The determination of the appropriate flood event should consider the consequences of various flood levels, including their impact on future community economic and social development patterns, on the cost of real estate, the impact on the tax base of the community, and the subsequent damage caused by those various flood levels. Flood damage usually is of two varieties – inundation or physical structural damage up to and including collapse. Depending on duration, inundation damage usually means building contents get wet and are frequently destroyed. Structural damage usually means some type of damage to the building frame caused by flood velocity, extreme flood depth, waves if near the coast, or damage by flood-borne debris. If significant flood damage could occur from a particular flood event, consideration should be given to increase the design flood elevation to reduce future damage.
>
> **Example 2:** The community has extensive flood history as in example 1. The only difference in community facilities is the presence of a regional medical centre with significant patient care centres located within this community and in the designed 1:100 AEP floodplain, although the centre is located near the edge of the flood fringe.
>
> In addition to physical damage to the medical centre, there must also be consideration for how the centre continues to operate both during and after the flood. There must be

consideration given to tolerance for downtime of some parts of the centre, including power supply to the centre. If protection methods are likely to be provided by barriers of some sort, then the overtopping of any portion of a barrier renders the flood protection system useless, so protection must be guaranteed. While guarantees are not certain, near certainty can be attained if either sufficient height of the site is provided or if the flood protection level is established sufficiently high.

While “sufficiently high” is not a defined flood level, a flood with an estimated high return period will reduce the probability of a flood occurring within the service life of the medical facility. In the Canadian classification system, it would seem that both High and Post-disaster buildings would be those buildings that must have the greatest protection from flooding. This suggests a 95% probability (a very high probability) that no damage will occur during a very high flood level. This would possibly translate into a 1:1000 AEP event for a 50-year service life or a 1:2000 AEP event for a 100 year service life (5% probability of being equaled or exceeded in accordance with Table 2-7).

2.8.1 Recommended Flood Frequencies

It is recommended that the NRC consider different return periods for different classes of occupancy as shown in Table 2-7. These return periods will establish the minimum elevation required for the FCL for each Importance Category type.

Table 2-7 MRI Recommendation for Importance Categories from NBC

Importance Category	Recommended Minimum Flood Design AEP
Low	1:100
Normal	1:500
High	1:750
Post-disaster	1:1000

For normal buildings, this would raise the design MRI to 500-years, or a 1:500 AEP. One rationale for this is that use of the 500-yr design level reduces the 50-yr encounter probability to 0.10, which may be interpreted as a 10% risk that the design event will be equaled or exceeded in 50 years. Use of the 1000-year event cuts the risk in half, to about 5%, for post-disaster buildings. It is believed that these risk levels are more appropriate for reducing flood hazards when compared to the much higher risk of 39% associated with design for a 100-year event in a 50-year period.

The committee working on revising ASCE 7 for the 2022 edition is likely to recommend similar flood design levels as suggested by Table 2-7. Preliminary but unpublished work suggests that the higher return periods shown in Table 2-7 are more likely to allow structural reliability targets in section 1.3.1 of ASCE 7 to be achieved which will also reduce the current flood load factors. Current flood design levels using the 1:100 AEP as the DFE are not at all close to ASCE 7 reliability targets.

2.8.2 Freeboard Recommendation

Freeboard provides an additional margin of safety in the flood elevation and is usually added on top of the mapped flood elevation. Some provincial flood maps include freeboard in the flood elevation, usually 0.3 m or 0.6 m. The way this margin of safety is applied does not change the design flood return period; it simply is added elevation. Since freeboard amounts are ‘arbitrary’ and not based on flood models, the amount of freeboard cannot be equated to a return period directly.

The amount of ‘safety’ provided by freeboard is a function primarily of the terrain and the flatness of the flood plain. A large expansive floodplain will not see a significant increase in flood elevation for a much larger return period since the water will spread out further and create more floodplain area or extent (Figure 2-28 (a)). A floodplain that has stream banks with steep topography will likely experience significant depth for an increase in flood elevation (Figure 2-28 (b)).

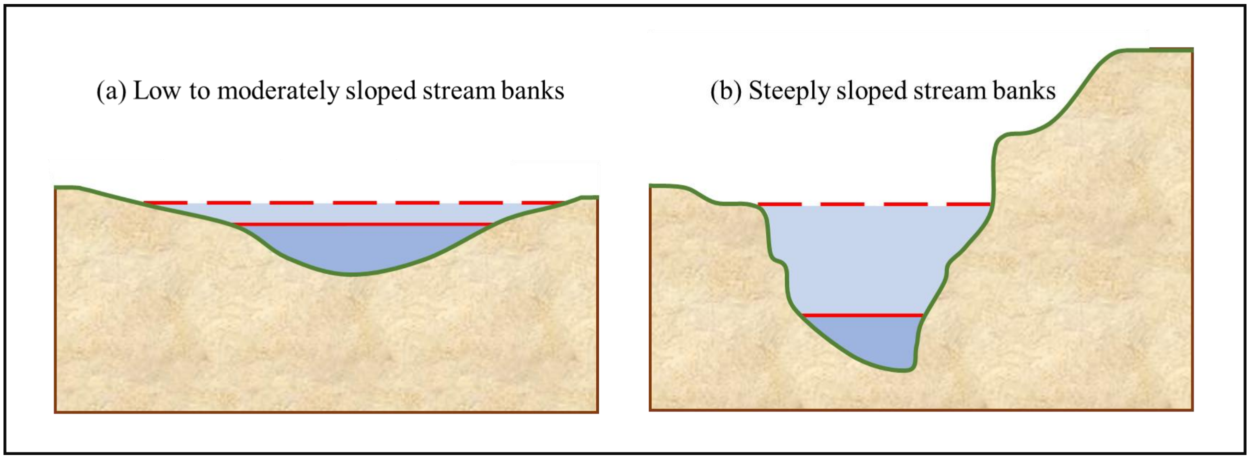


Figure 2-28 Flood Elevation Differences Compared to Topographic Differences

The recommendation of this guidance is to not include an arbitrary margin of safety or freeboard in the Design Flood Level or DFL. If additional safety from flooding is desired, the best way to achieve this is the increase the MRI of the design flood event. Table 2-8 illustrates this idea by using Cross Section 25 from Harry’s River in Newfoundland and Labrador (see Figure 2-6). The 1:20 elevation for Harry’s River at Cross Section 25 is 20.91 m. The 1:100 elevation is 21.44 m (Hatch, 2012). Other values have been estimated using methods discussed subsequently and outlined in Appendix A.

Table 2-8 Return Period Elevations for Harry’s River, Cross Section 25

Location	Elevations (m)						
	1:20	1:100	Freeboard	1:200	1:500	1:750	1:1000
Cross Section 25	20.91	21.44	0.30 (0.60)	21.67	21.97	22.10	22.20

Adding freeboard to the 1:100 AEP elevation increases the flood elevation to either 21.74 m or 22.04 m depending on the level of freeboard that is added. The 1:500 AEP event has a predicted elevation of 21.97 m and the 1:750 AEP event has a predicted elevation of 22.10 m, so that the inclusion of freeboard is the same as adopting a higher MRI flood level. But adding the same 0.3 or 0.6 m freeboard in other parts of the country may produce much different levels of risk, or different effective MRI values, depending on local conditions. Adopting the elevations associated with the higher return periods, rather than including freeboard, has the benefit of a defined return period (or AEP) that provides a consistent risk across other locations in the country. The elevations that have added freeboard have reduced the risk of flooding, but the risk is not consistent with other flood return periods used throughout the country.

2.9 Performance-Based Design Approach

Performance-based Design (PBD) requires a Design Flood Elevation. Chapter 4 on Performance-Based Design for Flood will cover this topic in some detail and discuss considerations in using a PBD method. For the purposes of this chapter on Design Flood Conditions and Considerations, the user should be aware that multiple design flood return periods might be used in PBD for a single location in order to address flood resistant design appropriate for a large and/or critical facility, irrespective of the building code-established flood design return period used based on the use and occupancy classification.

The design flood conditions to use for PBD include the establishment of the design flood elevation, the inclusion of freeboard, if any, and the treatment of future conditions. Table 2-5 of flood probabilities vs. event AEP is used to help select a design flood event (in terms of AEP).

2.10 Estimating Flood Frequencies from Available Design Flood Information

Design standards for the MRI or AEP of mapped flood elevations differ for each province, and a goal in development of national flood standards is to develop a prescriptive approach that will allow adjustment of values mapped in each Province to achieve a consistent national design standard. For estimating flood loads, consistent methodologies are needed for both flood elevation and flood velocity at the recommended levels of 100-500-750-and 1000-year MRI's (1:100, 1:500, 1:750 and 1:1000 AEP's).

One goal in developing national flood standards is to standardize the design flood event. Increasing the design standard to a 500-year MRI has the effect of increasing the width of the floodplain, and increasing the design flood elevation, thus not only including more structures in the design floodplain but also increasing their design elevation to prevent them from "getting wet" and experiencing flood losses.

A second goal in developing national flood standards is to develop consistent load factors that can be applied to flood loads. To achieve a consistent structural reliability, the choice of load factors is strongly influenced by the choice of the AEP (MRI) of the design event. To illustrate, the ASCE 7 flood loads in hurricane-prone coastal zones are now based on a 100-year design event with a load factor of 2.0 to reflect the potential for hurricane loads much larger than the 100-year design standard. In contrast, ASCE 7 tsunami design is based on a "maximum considered" tsunami having approximately a 2,500-year MRI, but then uses a load factor of 1.0.

Given the desire to have national standards for the AEP of the design flood event but given the disparity in mapped AEP flood events in each province, an approach is needed to allow a designer to start with flood values mapped in a province and then to adjust flood values to other (probably higher) MRI events. More specifically, it is useful in the building code to provide a prescriptive method to allow a user to start with, say, a 1:100 AEP mapped flood elevation but to then rationally estimate the flood elevations of the 500-year or larger flood events.

The details of a proposed method for scaling flood elevations from one MRI to another is given in Appendix A. This prescriptive approach to scaling flood elevations is, by necessity for broad application, simplified and approximate but justifiable based on the availability of data. It would be appropriate for a prescriptive approach to also be conservative and perhaps estimate the higher MRI events "on the high side." It is noted that the building code would also contain a provision to allow any end-user to conduct their own flood study with the approval of the Authority Having Jurisdiction to more accurately define flood events with other MRI's should they wish to apply the prescriptive method.

2.10.1 Assumptions

It is assumed that, depending on the province, the end user can identify flood elevations, S, for at least two MRI values from flood hazard maps. This requirement for flood elevations at two MRI values is attainable in most Provinces but may not be possible in all Provinces. In those cases, alternate methods may have to be developed to achieve the goal to using mapped flood information to extrapolate to higher MRI values.

It is further assumed that all elevations are referenced to a defined vertical datum, either the Canadian Geodetic Vertical Datum of 2013 (CGVD2013) or the older Canadian Geodetic Vertical Datum of 1928 (CGVD28). For coastal sites, elevations may be referenced to a local Chart Datum (CD), usually defined as the Lower Low Water, Large Tide (LLWLT), or to Mean Sea Level (MSL). In the Great Lakes, elevations are generally referenced to the International Great Lakes Datum of 1985 (IGLD85).

2.10.2 Prescriptive Method

The proposed prescriptive method in Appendix A uses the published flood elevations for two MRI values to extrapolate to higher MRI conditions. Often, extreme value data can be plotted in a format showing water levels, S, as a function of the logarithm of the MRI, as indicated in Figure 2-29. For cases where data (or numerically generated results) are available for several MRI values, an extreme value probability distribution (GEV, Gumbel, log Normal, etc) can be used to fit the data and to extrapolate to higher MRI values. Because 100-year values are commonly estimated in flood studies, fitting an extreme value distribution would be the preferred approach to estimate the 500, 750, and 1000-year recommended values.

For cases where only two values are known, for example at the 20- and 100-year levels, the only unambiguous fit of the data is a straight line. As shown in Appendix A, and in Figure 2-29 below, a straight line fit often works quite well even if more data points are available.

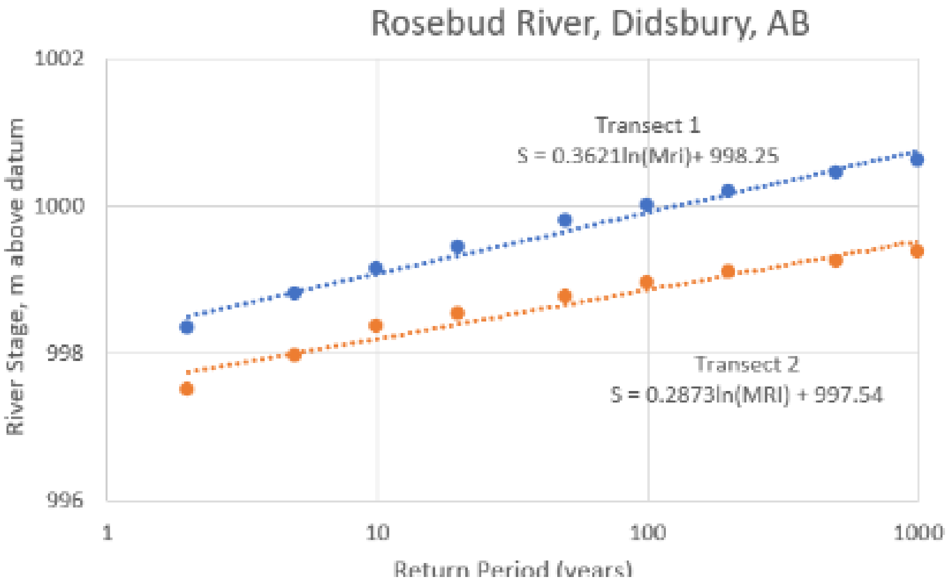


Figure 2-29 Graph of Flood Return Period Data for two transects, data from Rosebud River, Didsbury, AB (source: NHC, 2006)

The prescriptive method requires at least two data points. One is the lowest return period data available (usually the 10- or 20-year MRI data) and the other is the reference design level elevation used for the maps (usually the 100-year data). Return periods and flood elevations plotted in a format with the return period on the X-axis in a log scale, often plots as a straight line. This straight line has an associated logarithmic equation that is easy to use for determining other return periods and their associated flood elevations.

For simpler application, a scaling factor, denoted by α in Equation 2-4, provides an easy to use method for estimating elevations for return periods other than for those used to plot a straight line. In this case, with two reference levels known, the flood level for some desired MRI can be estimated as

$$S_n = S_{R1} + \alpha * (S_{R2} - S_{R1}) \qquad \text{(Eq. 2-4)}$$

where:

S_n = flood elevation of return period of interest
S_{R1} = flood elevation of lowest reference return period
S_{R2} = flood elevation of highest or mapped reference return period
α = scaling factor

The scaling factor, α, is effectively a multiplier for the difference in water levels between the two known points, with the result added to the lower reference level. Recommended values of α can be obtained in one of two ways as outlined in Appendix A: (1) using theoretical relationships from a straight line fit and (2) using empirical values obtained from flood studies that publish water levels at multiple return periods.

As shown in Appendix A, theoretical scaling values may be derived and summarized in Table 2-9. To produce this table, the 100-year MRI value is assumed for the upper reference level in Equation 2-3, while results in each column are shown for different lower reference values frequently found in Canadian flood maps. It is noted that for cases where 100-year values are not known, in B.C, for example where 20- and 200-year levels are mapped, Table 2-9 would not be appropriate and different theoretical factors would have to be derived using methods outlined in Appendix A.

Table 2-9 Theoretical values of MRI scaling parameters with 100-year upper reference

	Reference Level			
MRI	2 year	5 year	10 year	20 year
2	0.00			
5	0.23	0.00		
10	0.41	0.23	0.00	
20	0.59	0.46	0.30	0.00
50	0.82	0.77	0.70	0.57
100	1.00	1.00	1.00	1.00
200	1.18	1.23	1.30	1.43
500	1.41	1.54	1.70	2.00
750	1.52	1.67	1.88	2.25
1000	1.59	1.77	2.00	2.43
2000	1.77	2.00	2.30	2.86

As an example of how to apply these values, assume that for a river site a flood map indicates that the 20-year level is S_{20}=106.4 m while the 100-year level is S_{100}=107.2 m. For the 20-year lower reference level, from the last column in Table 2-9, the 500 year flood level would be estimated with α = 2.00, so that $S_{500}=106.4 + 2.00\ (107.2-106.4) = 108.0\ m$.

Appendix A also considers empirical scaling parameters for Great Lakes, coastal, and riverine areas based on use of 50 to 60 data points for each region. In many cases, values from Canada are difficult to locate so that data from the US in areas adjacent to Canada were used. With empirical parameters from 50 to 60 sites, values of α can be back-solved from published water level information, and results reported in terms of mean values and standard deviation of the scaling parameter. Upper confidence limits can then be developed to build some additional conservatism in to recommended values. An advantage of the empirical values of α is the deviation of the extreme value distribution from a straight line can be implicitly included.

A summary of selected results for the empirical scaling parameters is given in Table 2-10. For sake of brevity, only results using a 10 year lower reference and a 100 year upper reference are shown. Results are given in Appendix A for additional conditions. All results are reported here at the 95% upper confidence limit. Theoretical results are also shown for the 10 and 100 year combination of reference levels.

Results show that coastal sites are most similar to the theoretical values. The analysis considered US locations in the northeast and northwest for US states adjacent to Canada. The degree of agreement between the empirical and theoretical values indicates that a straight line fit of data work quite well in these coastal locations. Great Lakes sites show a similar agreement up to about the 200-year levels, after which empirical values are below the theoretical. This indicated that the data falls below the straight line fit, as in effect extreme water levels start to flatten out at higher MRI values. Riverine conditions, based on 60 locations in Canada (though dominated by Alberta locations), show that results fall slightly above the straight line fit.

Table 2-10 Scaling Factors for Great Lakes, Coastal, and Riverine Locations

MRI (years)	Theory	α Great Lakes	α Coastal Sites	α Riverine Sites
10	0.00	0.00	0.00	0.00
20	0.30	0.32	0.30	0.37
50	0.70	0.70	0.69	0.74
100	1.00	0.99	1.00	1.00
200	1.30	1.29	1.29	1.30
500	1.70	1.67	1.69	1.71
750	1.88	1.84	1.86	1.91
1000	2.00	1.96	1.99	2.04
2000	2.30	2.25	2.28	2.35

As an example of how to use these scaling factors, consider the same river locations used earlier, this time with a 10-year water level of 106.1 m and the 100-year level of 107.2 m. If the design (for a post-disaster building) requires a water surface elevation for the 1000-year event, then using the empirical values from Table 2-10, with α = 2.04, the 1000-yr level is then estimated as

$$S_{1000} = S_{10} + 2.04\times(S_{100}-S_{10})$$

Substituting values gives:

$$S_{500} = 106.1 + 2.04\times (107.2-106.1) = 108.34\ m$$

2.11 Guidance for Estimating Flow Velocity

Because flood velocity is not usually depicted in flood hazard maps, flow velocities are generally not available unless a detailed flood study has been published, or unless a user conducts a detailed numerical model study. In this section, approximate methods are presented for estimating flow velocity.

2.11.1 Guidance for Riverine Velocity

Appendix B provides details of a simplified method to estimate riverine velocity from mapped flood elevations. Examples shown in the appendix illustrate how far off the 1 m/s limitation can be from estimated actual velocities.

The method in Appendix B adopts standard methods for open channel flow, as outlined in the HEC-RAS manual (HEC-RAS, 2016). HEC-RAS is a computer program commonly use to establish riverine flood elevations and velocity methods based on a specified river discharge for a given MRI or AEP. The method outlined in Appendix B applies the same methods but makes use of the fact that flood elevation is generally known from flood hazard maps.

In open-channel riverine flows, if a transect is drawn across the flow channel to define the cross-sectional area of the flow, then the mean velocity for the cross section, V, is given by the Manning's equation, which empirically relates the flow velocity to the slope of the energy grade line as

$$V = (K/A)\ S^{1/2} \qquad \text{(Eq. 2-5)}$$

where K is the conveyance that quantifies the flow the carrying capacity of the cross section as

$$K = A\ \frac{k}{n}\ R_h^{2/3} \qquad \text{(Eq. 2-6)}$$

where:

V = mean velocity, in m/s (ft/sec)
K = conveyance of the cross section, with unit of discharge in m^3/s (ft^3/s)
S = slope of energy grade line (m/m or ft/ft), approximated by slope of water surface
A = cross sectional area of transect, in m^2 (ft^2)
R_h = Hydraulic radius (m or ft), given by $R_h = A/P$ or approximately by $R_h \sim A/W$
P = Wetted perimeter of the cross-sectional area (m or ft)
W = Width of the cross section (m or ft)
k = Coefficient associated with units (k=1 in SI units, k=1.486 in US Customary units)
n = Manning's roughness coefficient (typical values are given in Appendix B)

The Manning's equation is properly applied using the slope of the energy grade line, but in practice this is not available from flood maps. As an approximation, assuming that the flow does not vary dramatically over distance from up- to down-stream, then the principles of uniform flow may be used to approximate the slope of the energy grade line with the slope of the water surface. This is useful as the water surface elevations are mapped and surface slopes may therefore be estimated from the flood maps.

As discussed further in Appendix B, Equations 2-5 and 2-6 may be applied to subsections of the transect as illustrated in Figure 2-30. With this approach, the floodway or main channel normally has by far the largest portion of the flow and conveyance, often with low values of Manning's *n*, giving the highest velocities. Shallow overbank areas in the flood fringe will often have smaller values of conveyance but larger values of roughness, resulting in smaller velocities.

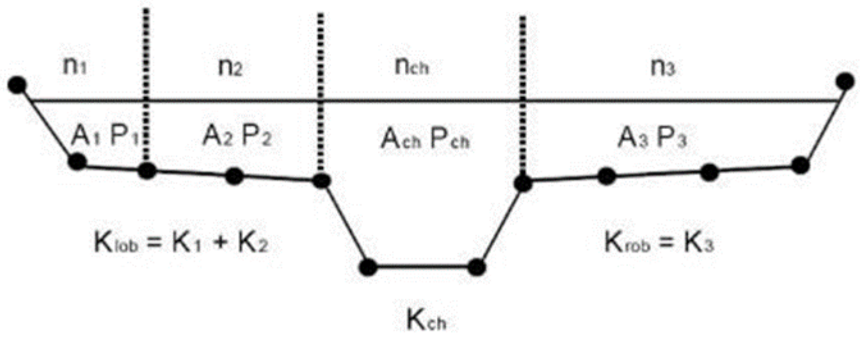


Figure 2-30 Subdivision of cross-sectional area to define main river channel (floodway) along with left and right overbank sections (flood fringe) (source: HEC-RAS Manual, 2016)

The proposed method, based on established principles of open channel flow in rivers, allows for a rather simple estimate of flow velocity based on: (1) water surface slopes obtained from flood maps, (2) a cross sectional area of flow obtained from GIS mapping or traditional land surveying, and (3) a description of ground roughness through selection of Manning's *n* values. The method is suitable as a prescriptive method in a guide document as it can be summarized in just a few equations, all based on well-known open channel flow expressions learned by almost every civil engineering university graduate.

2.11.2 Guidance for Coastal Flood Velocity

There is some guidance in FEMA's technical literature and in the US Army Corps of Engineers *Coastal Engineering Manual* on estimating flow velocities in coastal flood zones. Flow velocity in coastal flood conditions is created as water rises due to storm surge and flows across submerged land areas and into bays and estuaries. This water flows past buildings and building foundations; it frequently flows twice – once coming onshore and once returning to the sea.

In general, there is no well accepted method of estimating these flow velocities other than through detailed hydrodynamic modeling. Unlike riverine flows, mean flows in coastal regions are not always correlated to flood depth. Shallow water flow velocities during storm surge conditions are often maximum before or after the peak water level occurs, and velocities may be nearly zero at the time of peak water. Storm surge propagation is modeled as a shallow water long wave, and the correlation of speed and depth depends on the local bathymetry/topography and whether the surge acts as a progressive or standing long wave.

Given the general need for modeling, prescriptive guidance in design codes is limited. The guidance in FEMA's Coastal Construction Manual, also adopted in ASCE 7 (2016), gives range of design velocities as:

$$V = d_f / (1\ sec) \qquad \text{(Eq. 2-7)}$$

$$V = (g\ d_f)^{1/2} \qquad \text{(Eq. 2-8)}$$

The upper bound is based on a depth Froude number of unity and corresponds, in hydraulics, to a critical flow condition. In reality, storm surge flow is generally much slower so that the upper bound value is thought to be overly conservative. Methods of reducing the calculated velocity are being considered in revisions to the flood load methods in ASCE 7-22. A provision which has been developed based on modeling data from hurricane flood conditions in the US is to use

$$V = 0.5\ (g\ d_f)^{1/2} \qquad \text{(Eq. 2-9)}$$

This value was established as an upper bound to extensive numerical model data for shallow water flows and is still generally conservative. This provision has been accepted in a peer-reviewed ballot in and, will appear in the next revision of ASCE 7.

2.12 Regulatory Mechanisms and Examples

2.12.1 Flood Zone Designations (flood fringe and floodway, other)

National regulation of construction in floodplains requires standards clearly described at the national level that must be followed by building contractors and owners when new buildings are built or when major renovations are made to existing buildings. The reason for national regulations is to ensure risk consistency of flood design across the country, and to minimize the cost of rebuilding after flooding and loss of function of High and Post-disaster buildings located in or near the floodplains. The only way that flood losses will decrease over time is if the area of the built environment subject to flooding is reduced, or if buildings located in the ‘wet space’ are designed to resist damage from both flood forces and inundation.

The current method in Canada of describing the floodplain is to distinguish between the floodway and flood fringe. The floodway in some provinces is the extent of the 1:20 AEP flood event; the flood fringe is defined as the extent of the 1:100 AEP flood event (in most provinces; some describe the floodplain as the 1:200 AEP event; one province defines the floodplain as a 1:500 AEP event). While the depth and velocity limits might help define the floodplain boundaries in general terms, these limits (usually 1 m/s of velocity and 1 m of flood depth) are not sufficiently precise to use for flood-resistant design.

2.12.2 Flood Ordinances and By-laws

Floodplain construction requirements must be placed into local legislation in order to be locally enforced. In the US, this is done by adopting a floodplain ordinance at the community level that is represented by a particular watershed or flood threat. The ordinance describes the FCL requirements including the required elevation. In the US, the flood elevations used for construction are primarily based on locally adopted flood maps developed by engineering contractors under the guidelines published by FEMA. The flood hazard may be displayed in other ways, but the flood information must be publicly available in order for it to be used by the public. The flood hazard map should be clearly identified in the local ordinance so there is no confusion about what map is required to be used for construction in the floodplain.

In British Columbia, the province has adopted a floodplain construction ordinance titled FLOOD HAZARD AREA LAND USE MANAGEMENT GUIDELINES (2004). The ordinance requires that community plans be developed and that those "Official Community Plans (OCPs) must contain general land use policy statements and maps respecting restrictions on the use of land that is subject to hazardous conditions. OCPs should include statements that emphasize the need to manage development in flood prone areas in order to reduce impacts on people and property."

There is a provision in the guidelines to allow for modification of covenants that deal with flooding. It states: "The approving officer may modify any covenant to best match the flood hazard provided the level of protection is not altered. This discretion extends to the reduction of elevation requirements, where floodplain mapping exists, by the freeboard, provided the subject property is in the floodplain fringe area and there are no major erosion or channel avulsion hazards in the immediate vicinity."

Flood mapping is included in the guidelines. The mapping is included by the statements: "As such, available floodplain mapping information is incorporated into and forms a part of these Guidelines. A floodplain map delineates the area that can be expected to flood, on average, once every 200 years (called the 200-year flood). Floodplain mapping information is available for many settled areas of the province, e. g. http://srmwww.gov.bc.ca/aib/fpm/index.html. The floodplain is delineated by the translation of the flood profile plus freeboard allowance to base mapping to produce the finished floodplain maps."

The building construction requirements for ordinary watercourses described in the guidelines are in part: "Setback – Buildings should be setback at least 30 metres from the natural boundary of any watercourse, except as noted in sections below. FCL where a designated flood level has been determined – Areas used for habitation, business, or storage of goods damageable by floodwaters should be constructed within any building at an elevation such that the underside of the floor system thereof is no less than the Flood Construction Level."

In addition, the guidelines describe restrictions of building locations as follows:

> *"Section 5.5 Depth of Flooding: Subdivision in areas of flooding depth greater than 2.5 metres requires that the applicant demonstrate how full flood proofing can be achieved and how safe ingress and egress can be achieved during the flood.*
>
> *Section 5.6 Flood Velocities Subdivision in areas where flood velocities are in excess of 1.0 metre per second requires that the applicant demonstrate how safe ingress and egress can be achieved during the flood."*

In Calgary, Alberta, the floodplain ordinance passed in 2007 and still in effect requires new buildings built in the flood fringe to:

- Be designed and constructed to prevent structural damage by floodwaters
- Have the first floor of all buildings constructed at or above the designated flood level
- Have all electrical and mechanical equipment be installed at or above the designated flood level, and
- Have a sewer back up valve installed in every building.

Ontario has development requirements in floodplains in Ontario Regulation. 97/04: CONTENT OF CONSERVATION AUTHORITY REGULATIONS UNDER SUBSECTION 28 (1) OF THE ACT: DEVELOPMENT, INTERFERENCE WITH WETLANDS AND ALTERATIONS TO SHORELINES AND WATERCOURSES. The last amendment was made in 2011. The regulations include what return flood to use for design and includes a prohibition of 15 metres from the edges of the flood fringe for development.

By-laws and ordinances that describe the regulations required for development in the floodplain must be easily accessible to the public and easy to understand. There should be a reference to the flood map used to define the flood hazard and a clear description of the building construction allowed in the floodplain in reference to the FCL.

2.12.3 Building Regulations

Building regulations should be clearly articulated in ordinances and by-laws and supported with graphics or other technical materials describing what is expected of the building performance during a flood event. The technical guidance provided in this chapter suggests there are several different building conditions that must be met in order to minimize flood damage. These conditions are:

1. Buildings are located in the flood fringe of a riverine floodplain; some have basements, and some do not. Some buildings are elevated above ground.
2. Buildings are elevated in a coastal floodplain; all are elevated above the ground.

Figure 2-31 illustrates the recommended FCL developed in these guidelines compared to various building foundations and floor levels. These floor levels compared to the FCL should be described and articulated in national and provincial guidance and design requirements for flooding.

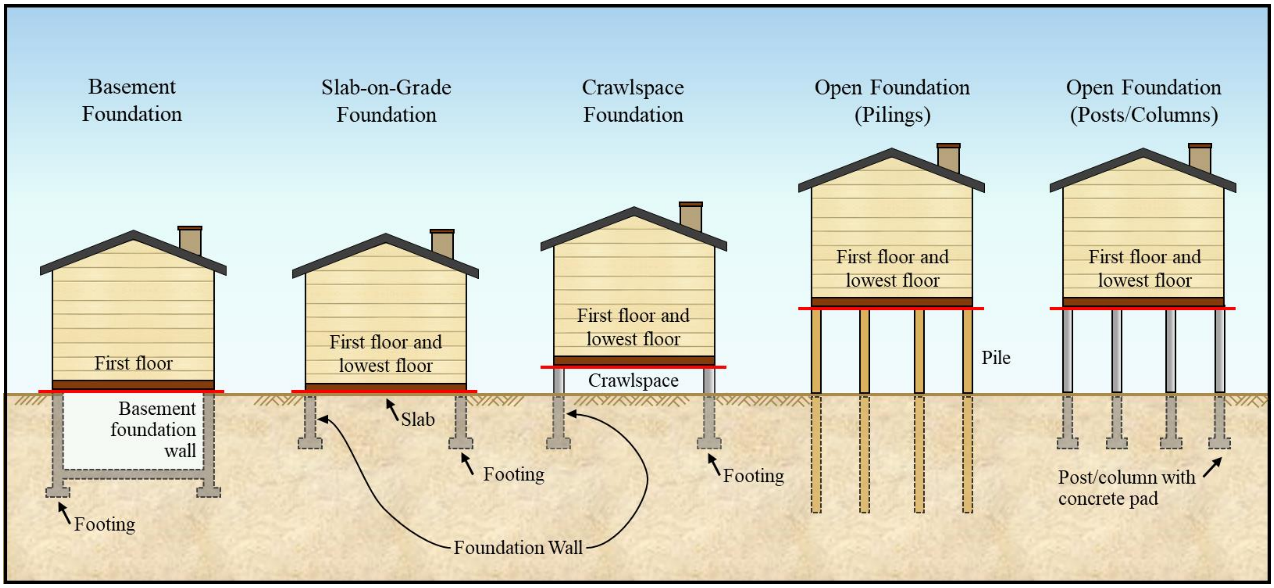


Figure 2-31 Typical Foundation Types and Minimum Recommended FCL Levels; FCL shown as red line

Several important flood design considerations are illustrated in Figure 2-31.

1. In the first sketch showing a basement, the FCL is shown just below the building's lowest floor. This means the basement is below the FCL and should be required to be flood resistant.
2. In the second sketch showing a slab, the FCL is at the top of the slab. This means the area below the FCL will get wet; that flood water could possibly erode the soil under the slab, and that flood water will possibly permeate the soil under the building slab.
3. In the third sketch showing a crawl space, the FCL is shown just below the building's lowest floor. This means the crawl space is below the FCL and should be required to either be flood-proofed or allowed to have water enter the crawl space to relieve hydrostatic pressure against the walls.
4. In the fourth and fifth sketches, the FCL is shown just under the lowest structural member supporting the elevated building. This means that the area under the FCL should remain open to allow water to freely flow under the building and around the piles or columns. Any structural members in the area under the FCL should be designed to accommodate the flood loads.

2.12.4 Flood Hazard Identification

Flood hazard identification is usually provided to the community in the form of flood maps. Those maps usually show inundation limits or the extent of the floodplain; they should also show flood elevations or flood depths across the flood hazard, and there should be some information about flood velocity on the maps. The maps should show elevations (or depths) that are linked to a specific AEP (1:100, 1:200, 1:500, etc).

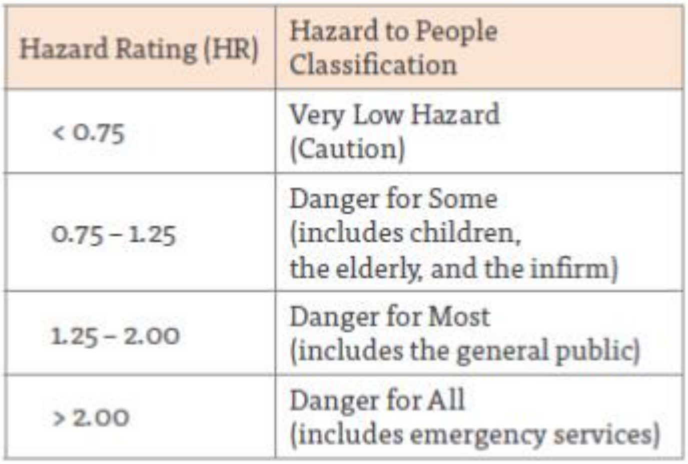
The flood hazard can also be identified in ways that address public safety. British Columbia has used a hazard rating system developed in the United Kingdom that includes consideration for flood depth, flood velocity and flood debris. If the flood area is influenced by an alluvial fan, it includes the flow debris from the fan in the consideration. Figure 2-32 shows the classification system as it could affect people. The hazard rating (HR) is determined by

$$HR = d + (V + 0.5) + DF \tag{Eq. 2-10}$$

where:

HR = hazard rating
d = flood depth (m)
V= flood velocity (m/s), and
DF = debris factor of 0, 0.5, 1 depending on the probability that debris leads to a significantly greater hazard.

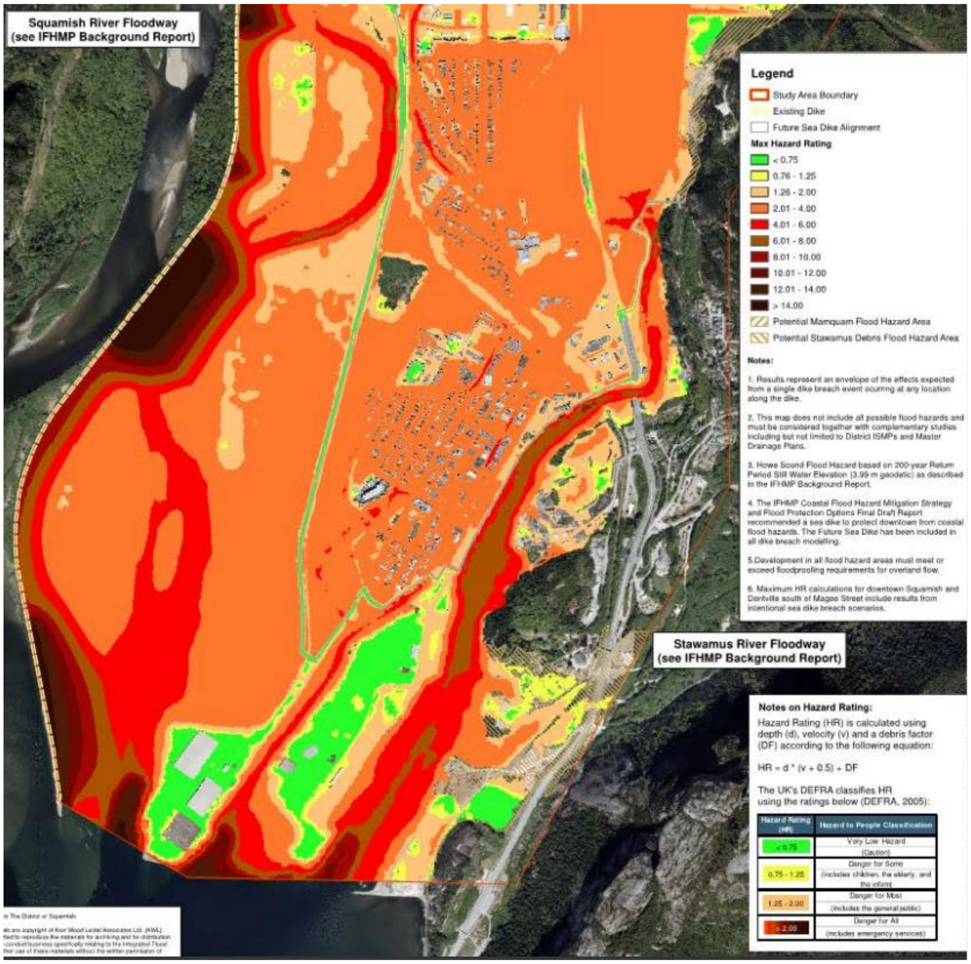
Notice that even in the flood fringe, where depths might be 1 m and velocity 1 m/s, the hazard rating is quite high with HR = 2.5 indicating "Danger for All" in the hazard rating classification.



Hazard Rating (HR)	Hazard to People Classification
< 0.75	Very Low Hazard (Caution)
0.75 – 1.25	Danger for Some (includes children, the elderly, and the infirm)
1.25 – 2.00	Danger for Most (includes the general public)
> 2.00	Danger for All (includes emergency services)

Figure 2-32 Hazard Rating Classification (source: APEGBC 2017)

This rating system was used on the Squamish River in British Columbia to illustrate the flood hazard. The identified potential hazard was a dike breach in year 2100 for a 1:200 year AEP. Thus, the hazard included future conditions and a potential catastrophic event such as a dike breach. The hazard map is shown in Figure 2-33 (APEGBC, 2017).



Notes:

1. Results represent an envelope of the effects expected from a single dike breach event occurring at any location along the dike.

2. This map does not include all possible flood hazards and must be considered together with complementary studies including but not limited to District ISMPs and Master Drainage Plans.

3. Howe Sound Flood Hazard based on 200-year Return Period Still Water Elevation (3.99 m geodetic) as described in the IFHMP Background Report.

4. The IFHMP Coastal Flood Hazard Mitigation Strategy and Flood Protection Options Final Draft Report recommended a sea dike to protect downtown from coastal flood hazards. The Future Sea Dike has been included in all dike breach modelling.

5.Development in all flood hazard areas must meet or exceed floodproofing requirements for overland flow.

6. Maximum HR calculations for downtown Squamish and Dentville south of Magee Street include results from intentional sea dike breach scenarios.

Notes on Hazard Rating:

Hazard Rating (HR) is calculated using depth (d), velocity (v) and a debris factor (DF) according to the following equation:

HR = d * (v + 0.5) + DF

The UK's DEFRA classifies HR using the ratings below (DEFRA, 2005):

Hazard Rating (HR)	Hazard to People Classification
< 0.75	Very Low Hazard (Caution)
0.75 - 1.25	Danger for Some (includes children, the elderly, and the infirm)
1.25 - 2.00	Danger for Most (includes the general public)
> 2.00	Danger for All (includes emergency services)

Figure 2-33 Squamish River Dike Breach Hazard Rating in 2100 (source: APEGBC, 2017)

2.13 Consideration for Future Conditions

An important component of a design flood depth is how to incorporate future conditions. It is more important to consider the future in some way than to ignore it just because it is difficult to predict the future accurately. The future conditions could include those that increase water depth, lower ground elevations, or some combination of both. There will likely be imprecision in the projected future conditions, so the user must be encouraged to research the possible ranges of increased water depth and/or more extensive ground lowering to select the most appropriate future condition to include for flood design. The user should also document the research and the selected future condition so the selection can be evaluated for possible modifications as needed.

2.13.1 Possible Future Conditions

There are several possible future conditions that should be considered in flood-resistant design. As we have seen, design for floods involves knowing or being able to estimate the depth of water against a building or flood structure, and the future flood depth is an unknown. Many Canadian provinces have studied the effects of changing conditions on the future flood depths that might be expected, but until now, these studies have mostly been hydraulic studies that are illustrated by flood maps with new floodplain boundaries for a variety of MRI. Most often these boundaries define the floodway (often the 1:20 AEP) or the flood fringe (often the 1:100 AEP).

The most likely future conditions to consider for flood-resistant design are:

- Sea level rise (SLR)
- Subsidence
- Erosion along the coastline affecting coastal flooding
- Increased stream erosion or stream wandering (changing course)
- Increased rainfall that overflows flood conveyance channels
- Increased density of the built environment
- Increased frequency of rain on snow events

The American Society of Civil Engineers (ASCE) has provided some direction to US Civil Engineers regarding important issues that practitioners should consider regarding climate change. In a monograph published in 2015 (*Adapting Infrastructure and Civil Engineering Practice to a Changing Climate*), ASCE suggests design professionals consider the following:

- Engineering standards and practices usually assume that the frequencies and intensities of past events will represent future events as well.
- There is large uncertainty about both frequency and intensity of future events.
- Using low-regret, adaptive strategies such as the observational method to make a project more resilient to future climate and weather extremes.
- Engineers should seek alternatives that do well across a range of possible future conditions.
- Climate models are usually better at predicting large temporal and spatial changes than small ones, and they often model only one possible future rather than a number of possible futures.

Practicing engineers owe their clients and the public designs that will work well into the future. To achieve that objective, the many unknowns inherent in designing for future conditions must be considered in some way. Answers to the following questions are an important part of the design for the future:

- How do climate change impacts affect engineering factors of safety included in existing and proposed designs?
- What are the limits in current designs and materials for extreme loads due to wind, temperature, flooding and precipitation?
- How does the increased expected climate variability affect loads and performance?
- How do behavioral responses and changing demands for services provided by infrastructure affect near-term and long-term infrastructure vulnerabilities?

Considerations for the future conditions of SLR, subsidence, and coastal erosion can be dealt with in the same way – by increasing the water depth. For SLR, the water depth will increase the amount determined by a future scenario. For planning purposes, the recommended planning period should extend to 2100. Intermediate periods should also be considered since the increase in rising water due to climate change is unlikely to be a linear function. The recommended minimum window for future conditions is 50 years. For subsidence, the water depth will increase by the amount the ground is sinking determined by a future scenario. For coastal erosion, the water depth will increase by the amount the ground is eroding at the water/land interface.

Consideration for increased stream erosion and stream wandering, for increased rainfall overflowing channels, and for increased density will likely lead to increased inundation and increased floodplain area. Therefore, future conditions need to consider how to determine larger inundation areas and incorporate more built environment into the potentially flooded area.

A Canadian example is illustrated with the work completed by the Ministry of Forest, Lands and Natural Resource Operations in British Columbia (BC). BC has developed a method for considering future coastal flood depths using a formula that includes sea level rise (SLR) and flooding at higher high water large tide levels (HHWLT). The formula is

$$FCL = \mathrm{HHWLT} + \mathrm{SLR} + \text{storm surge} + \text{wave effect} + \text{freeboard} \qquad \text{(Eq. 2-11)}$$

where:

FCL = flood construction level
HHWLT = higher high water level tide elevation
SLR = sea level rise for some defined scenario (such as 2100)
Storm surge = estimated storm surge for the selected design storm
Wave effect = estimated wave effects for the selected design storm

The consideration for SLR has been determined by scenarios and has been plotted in increasing water depth to 2200. The scenario increases are shown in Figure 2-33. It should be noted that while the SLR projection has been reduced to a straight line, the projection is not linear. In addition, the study has provided some idea of what the upper and lower bounds are on the amount of SLR over increasing time. The BC study also provided a graphic of how the floodplain boundaries might expand with SLR. This plan and cross-section view of the floodplain boundary are shown below in Figure 2-34.

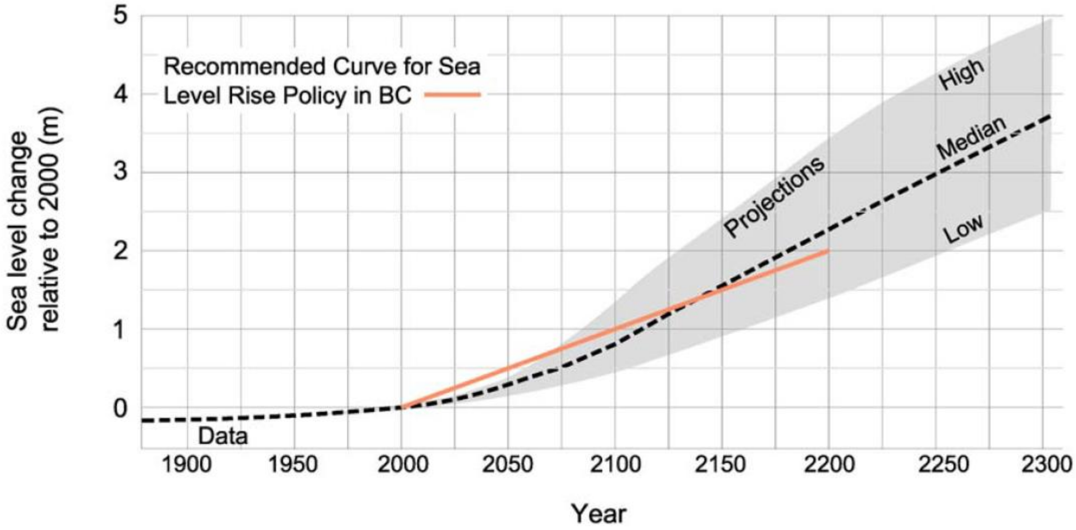


Figure 2-34 BC Sea Level Rise Projection and Recommended Increase (source: Kerr Wood Leidal, 2011)

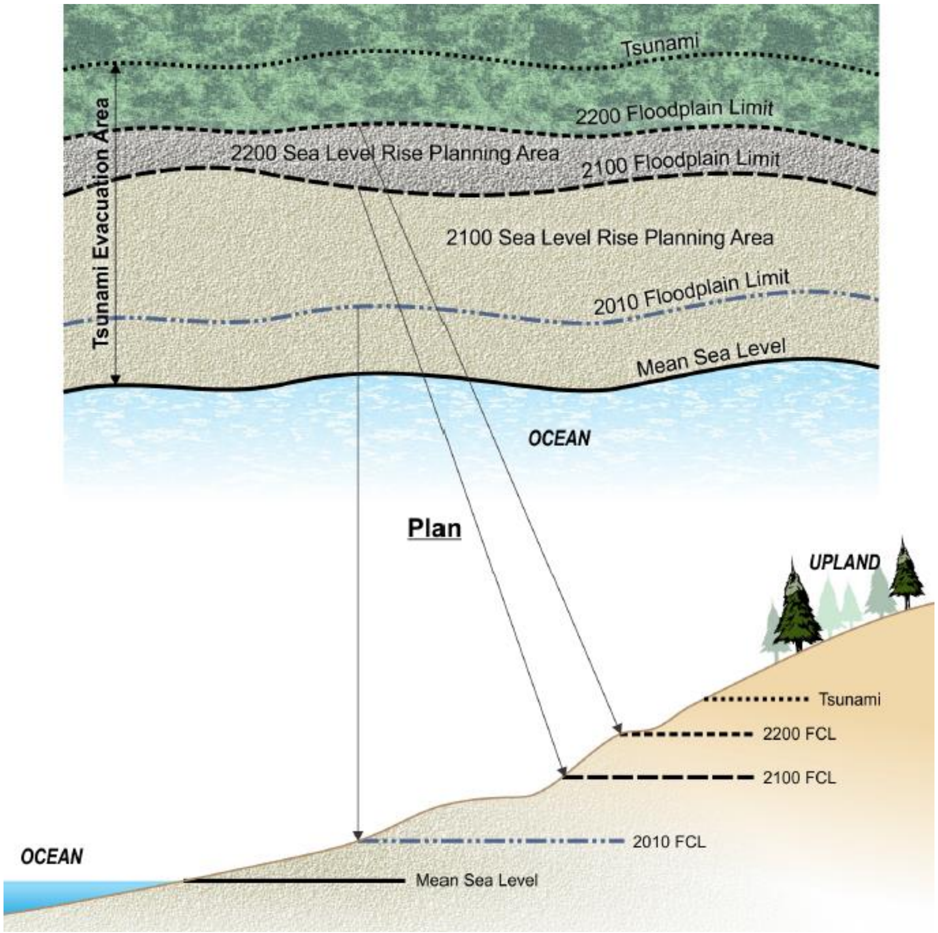


Figure 2-35 BC Sea Level Rise Study Depicting Increase in Floodplain Boundary (source: Kerr Wood Leidal, 2011)

In an engineering study conducted in 2011 (Coastal Floodplain Mapping – Guidelines and Specifications, prepared for Ministry of Forest, Lands and Natural Resource Operations by Kerr Wood Leidal Consulting Engineers, June 2011), a case study was created for the City of Campbell River that illustrated how this formula (Eq. 2-11) would work. Three transects were cut across the water/land interface and depths (elevations) of flooding were predicted based on two SLR scenarios – 2100 and 2200 and were compared to the current condition in 2010.

Some further explanation of the predicted parameters is necessary:

- The FCL is the Flood Construction Level at the point at which the water level meets the ground.
- The HHWLT is the higher highwater level tide elevation which is greater than the mean water level established at elevation 0 m. This HHWLT flood level means that the flooding is assumed to occur at the highest of tides.
- SLR is the predicted sea level rise for a given scenario. This level is an increase above the 0 datum and the HHWLT level.
- Storm surge is the increase in water level that occurs when a coastal storm comes ashore. This increase in water level is created primarily by wind pushing the water ashore and a decrease in atmospheric pressure caused by the rotating winds of a coastal event.
- Waves are induced by both wind and the act of water breaking on the shore. Wave heights may be estimated as a function of the depth of water (0.78 x depth of water or as determined by a site-specific flood study)
- Freeboard is an additional margin of safety that provides an increase of the MRI of an unknown amount. Since the elevations of the ground, tides, storm surge, and waves can all vary, adding a fixed amount of freeboard (recommended at 0.6 m) adds protection in additional height above the water level but it is an unknown level of protection.

An example of results at one of the three transects is shown below (Figure 2-35 extracted from Kerr Wood Leidal, 2011):

CASE	**2010 "Updated" Guideline[a]**	**2100**	**2200**
Reference Vertical Datum	CGVD28 = 0.0 m = 0.29 m CD		
Regional Sea Level Rise (m)	0	0.63	1.22
HHWLT	1.9	1.9	1.9
Total Storm Surge	1.5[b]	1.5[b]	1.5[b]
Wave Effect	0.4	0.5[c]	0.8[d]
Freeboard Allowance	0.6	0.6[e]	0.6[e]
FCL	4.4	5.1	6.0

Note:

a: Methodology to derive FCL applied as per the case example described in the foregoing text but with no allowance for future seal level rise (i.e. present-day conditions)

b: 1/500 yr annual exceedance probability

c: land in this area is awash – Wave Effect includes expected wave set-up

d: land in this area is flooded – Wave Effect includes expected wave crest elevation

e: allowance only – appropriate freeboard is structure dependent

Figure 2-36 Sample from one transect at Campbell River, British Columbia (source: Kerr Wood, Leidal 2011)

2.13.2 Overland Flow and Consideration of Unintended Consequences

Floods can occur when stormwater drainage channels and pipes overflow or backup, when rainwater conveyance paths are blocked, or when sewage treatment plants overflow. All of these conditions can create excessive overland flood flow and inundate buildings and infrastructure if the entry points into those structures is below the level of overland flow. These conditions are not normally considered in flood mapping unless they are part of the hydrologic study used to develop the map. In order for them to be considered in flood design, the magnitude of these conditions must be estimated and included in flood design as a “future condition”.

Some of these overland flow conditions can create unintended consequences to critical facilities. These can be considered in a performance-based flood design for a critical facility, but an estimate of flood elevation must first be made so design solutions can be developed. An estimate of the “consequence” and the probability of that “consequence” must also be made so adequate plans can be made to deal with the “consequence.”

2.13.3 Sources of Actionable Science Information

Actionable science in the context of this chapter, is science-derived information about the changing climate that is developed and reported on in a way that allows the user to take the science-developed information and predict future flood levels at some standard reporting time period (year, decade, century).

An excellent example of actionable science is a recently published Canadian comprehensive study on climate change and its impact on Canada. The report covers climate change modeling, temperature, precipitation, snow, ice, permafrost, and oceans. The URL for the report is: https://www.nrcan.gc.ca/maps-tools-publications/publications/climate-change-publications/canada-changing-climate-reports/canadas-changing-climate-report/21177
Appendix C covers three additional case studies; two in British Columbia and one in Newfoundland and Labrador. The studies illustrate a range of approaches to changing conditions, and include an approach to dikes, to subsidence, and to both sea level rise and increasing flood flows inland of the coast.

2.14 Recommendations

Figure 2-37 provides a summary of recommended flood construction levels (FCL) for riverine and coastal conditions that has been covered in this guidance. The recommended design flood depth is discussed in Section 2.10 and shown in Table 2-9 based on NBC Importance Categories. Future conditions are discussed in Section 2.19 with examples used to illustrate how future conditions might be added to design flood levels for several different types of future conditions.

RECOMMENDED FLOOD CONSTRUCTION LEVELS (FCL)

The FCL for Riverine Conditions should include:

Design flood elevation for a specified MRI +

FC (Future Conditions that affect water surface elevation or flood elevation)

The FCL for Coastal Conditions should include:

Design flood elevation for a specified MRI +

Wave effects above storm surge +

FC (Future Conditions that affect water surface elevation or flood elevation)

Figure 2-37 Recommended Flood Construction Levels for Riverine and Coastal Conditions

2.14.1 Flood Hazard Mapping

Flood maps ideally should be developed using the building-code required flood return period as the design flood elevation. Given the wide differences between provinces in the flood return period used and the dates of the existing maps, having updated flood maps using the same national flood return periods is very unlikely. The current flood maps that were reviewed for this guidance all use some form of flood modeling that is widely accepted. The most urgent future matter is to update maps to current topography and development.

2.14.2 Flood-Resistant Design for New and Existing Buildings

Flood-resistant design nationally must use a risk-consistent basis. The recommended risk basis is the design event in years (MRI) shown in Table 2-7 and copied here for ease of use. Designs using these MRIs should be for new buildings.

Table 2-11 MRI Recommendation for NBC Importance Categories for New Buildings

Importance Category	Mean Recurrence Interval (years)
Low	1:100
Normal	1:500
High	1:750
Post-disaster	1:1000

It is recognized that there is a large difference in the ability to conform to this standard for new and existing buildings. While new buildings can more easily be either elevated or floodproofed as needed to resist flooding at the FCL, existing buildings will usually require extensive structural modifications to be able to resist the loads from flooding to the FCL. In order to recognize the difficulty and cost in performing this flood resistance work for existing buildings, it is recommended that the flood design levels shift down slightly by use or occupancy category, thus for existing buildings, the MRIs used for flood-resistant design should be in accordance with Table 2-12.

Table 2-12 MRI Recommendation for NBC Importance Categories for Existing Buildings

Importance Category	Mean Recurrence Interval (years)
Low	1:100
Normal	1:250
High	1:500
Post-disaster	1:750

2.14.3 Building Guide and Commentary

Future revisions of the NBC should consider inclusion of:

1. Table 2-11 MRI Recommendation for NBC Importance Categories for Buildings
2. The general equations to use for both riverine and coastal FCL
3. A sketch showing the relationship of building floors to the FCL
4. The method to use for scaling MRI for riverine, coastal and Great Lake locations
5. A requirement that flood parameters for depth, velocity, flood debris loading, wave height, storm surge, and future conditions must be determined using a rational method either from the code or a recognized peer-reviewed source

The code should recognize that building elements covered in other portions of the code could be affected by flooding. These elements include but may not be limited too, foundations, slabs on grade, basement walls, and foundation drainage systems. Other issues such as frost depth and soil submerged under water could impact flood designs.

The building code commentary should include:

1. Discussion about the need to determine flood depth at a site of interest
2. Coverage of the interpolation method for finding flood velocity using approximations of HEC-RAS methodology
3. Discussion about flood exceedance probabilities and the recommended MRIs to use for various occupancy classes
4. Discussion of the MRI scaling method for riverine, coastal and Great Lake locations
5. Discussion of future conditions that should be considered

2.15 Example Problems Using Proposed Methods

The following three examples are intended to represent how to determine the flood hazard at the selected sites. In each case, a flood map or flood study has been used from a province to illustrate how the flood hazard is determined from the available data.

2.15.1 Riverine Example

The example site location is in Newfoundland-Labrador Province on Harry’s River. The location is on transect 25 and is shown as a red dot on Figure 2-38.

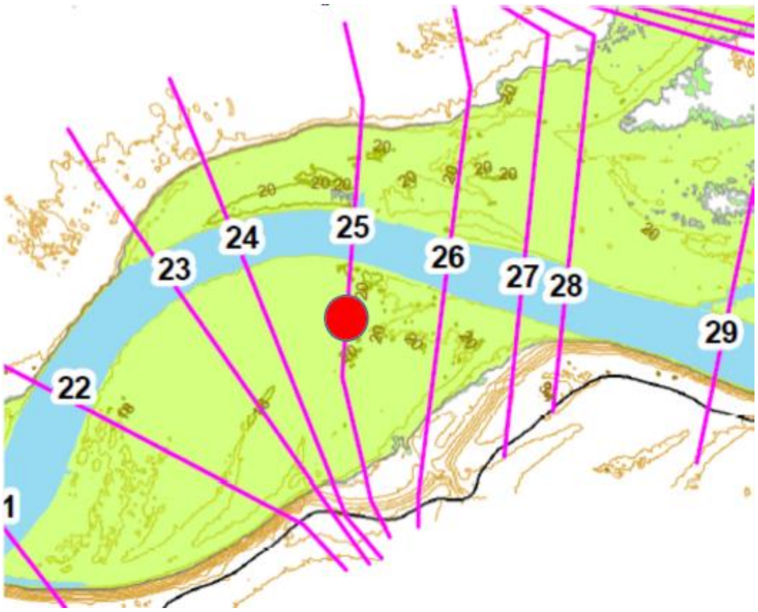


Figure 2-38 Harry’s River Transects (red dot is site location of example)

The flood profile at transect 25 is shown in Figure 2-39 and the site location is at the red line.

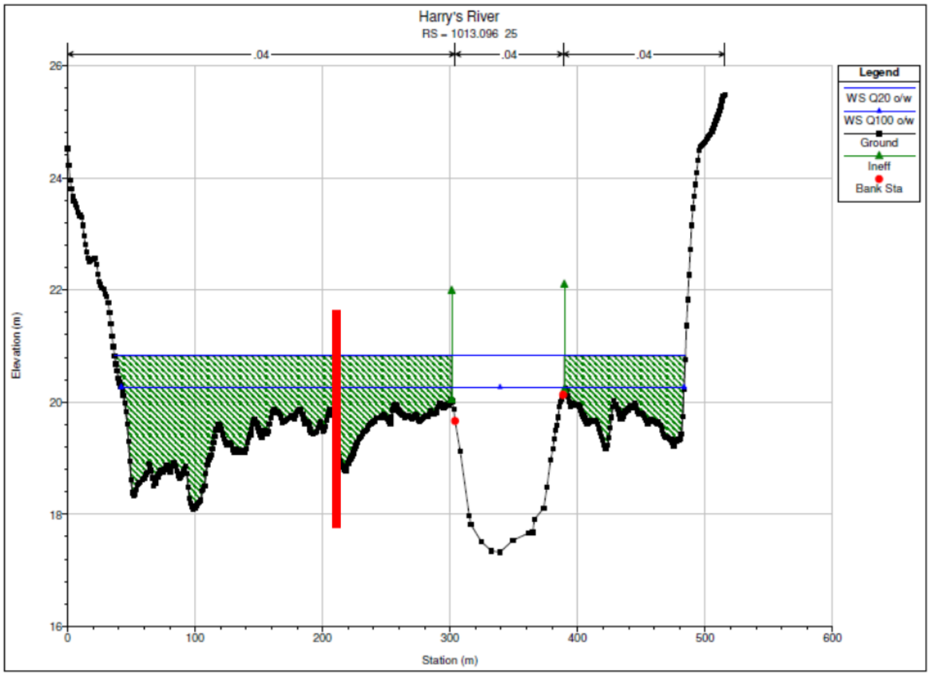


Figure 2-39 Flood Profile at Transect 25 (red line is site location)

Given:

- The flood depth at the site (according to the scale on Figure 2-39, is 2.0 m.
- The flood map for which the transect 25 is created represents a 1:100 AEP and is 21.44 m.
- The 1:20 AEP is 20.91 m from the flood study.
- Flood profiles for Transects 25 and 27 from the flood study.

Find:

- The flood depth for a 1:500 AEP.
- The flood velocity for a 1:500 AEP.
- The flood depth for a 1:1000 AEP.
- The flood velocity for a 1:1000 AEP.

Solution:

- Finding 1:500 and 1:1000 AEP elevations require the use of scaling factors from Table 2-12.
- The scaling factor for a 1:500 AEP = 1.9.
- The scaling factor for a 1:1000 AEP = 2.2.
- The predicted flood elevation for 1:500 AEP uses Equation 2-14 and = 20.91 (1:20 AEP) + 1.9 x [21.44 (1:100 AEP) – 20.91 (1:20 AEP)] = 21.92 m.
- **The predicted flood depth for a 1:500 AEP = 21.92 – 21.44 = 2.0 = 2.48 m.**
- The predicted flood elevation for 1:1000 AEP = 20.91 (1:20 AEP) + 2.2 x [21.44 (1:100 AEP) – 20.91 (1:20 AEP)] = 22.08 m.
- **The predicted flood depth for a 1:1000 AEP = 22.08 – 21.44 + 2.0 = 2.64 m.**
- The flood velocity is determined using Equation 2-7. From the flood profiles for Transects 25 and 27, the cross-sectional area (A) of the floodplain at the site and the conveyance K are determined. The slope of the energy grade line (S) is found from the elevation differences of Transects 25 and 27.
  - **The flood velocity at Transect 25 at 1:500 AEP = (K/A) x $S^{1/2}$ = (10293/485.1) x $0.00487^{1/2}$ = 1.48 m/s.**
  - **The flood velocity at Transect 25 at 1:1000 AEP = (11675/523.2) x $0.00487^{1/2}$ = 1.56 m/s.**

2.15.2 Coastal Example

The example site location is near Tyee Spit, in Vancouver, British Columbia. The satellite view using Google Earth is Figure 2-39. The red dot is the example site location.

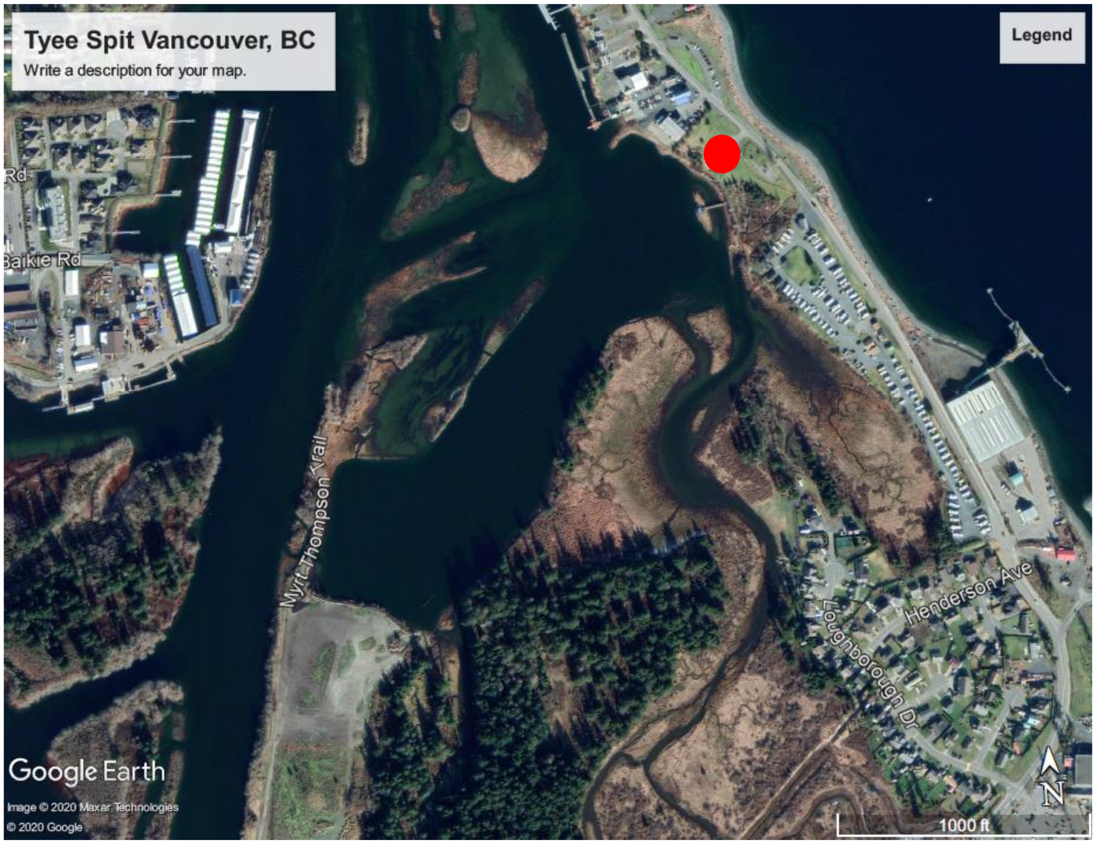


Figure 2-40 Google Earth view of example site location in Vancouver, BC

The example site location is shown on the current flood map as Figure 2-41 The red dot is the example site location.

Given:

- The flood map shown as Figure 2-41 is dated 1980 and was updated in 1990.
- The flood map represents a 1:200 AEP of 3.5 m or 11.5 ft including freeboard.
- The flood map also shows a 1:20 AEP of 3.1 m 10.2 ft including freeboard.
- While it is not clear from the map, it is assumed that the 3.5 m flood elevation includes HHWLT + storm surge +freeboard. No wave effects were included.
- The freeboard is assumed to be 0.6 m or 2.0 ft
- The ground elevation from a contour on the flood map is 1.8 m or 5.9 ft

Find:

- The flood depth for a 1:500 AEP.
- The flood velocity for a 1:500 AEP.
- The flood depth for a 1:1000 AEP.
- The flood velocity for a 1:1000 AEP.

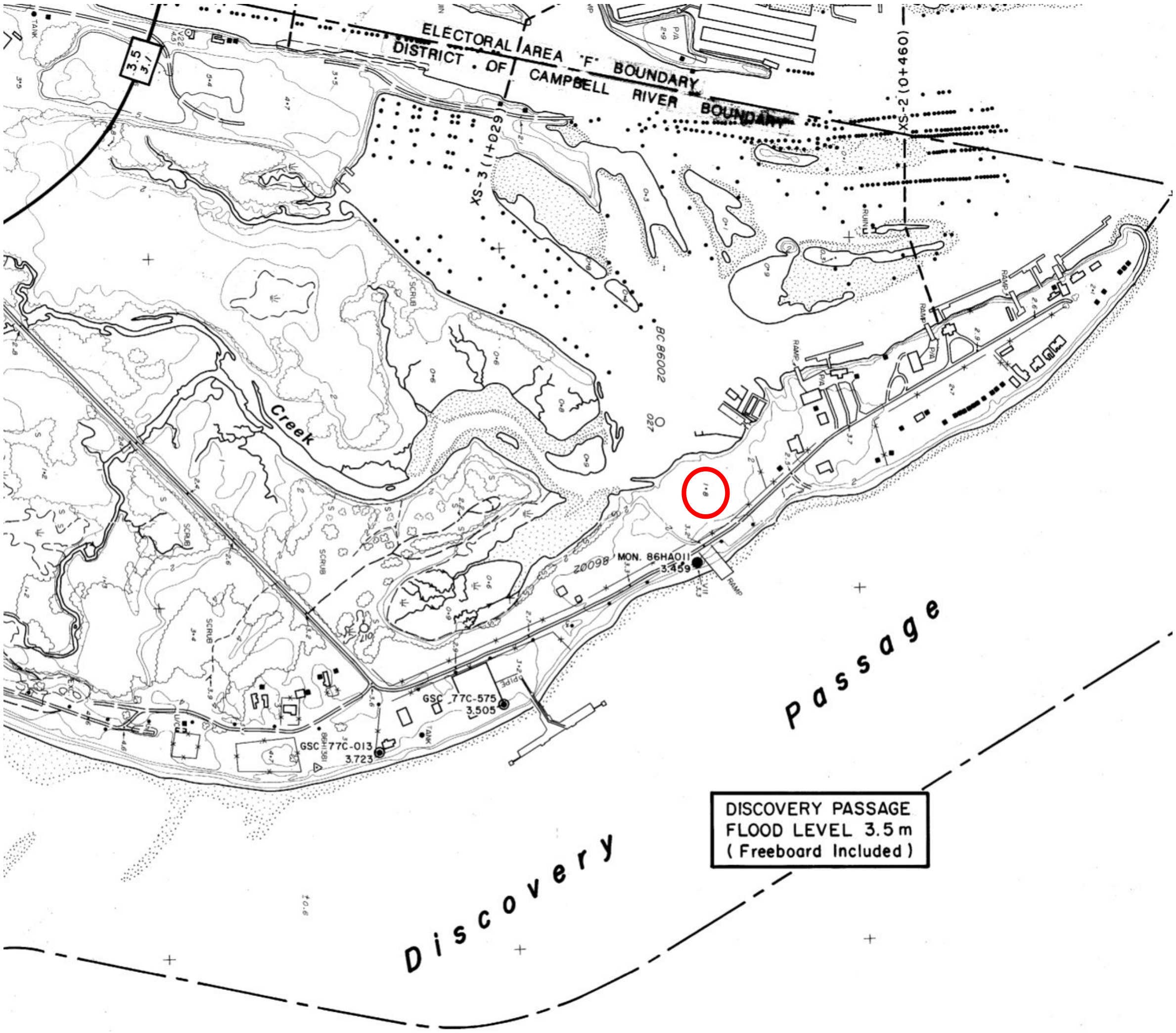


Figure 2-41 Coastal Flood Map for Tyee Spit, Vancouver, BC (red circle is hypothetical site location)

Solution:

- Remove freeboard effects to obtain the still water flood elevation. For the 200-yr MRI level, the SWL is 3.5 m - 0.6m = 2.9 m while the 20-yr level is 3.1 m – 0.6 m = 2.5 m.
- Convert 1:200 AEP elevation to 1:100 AEP and then increase to 1:500 AEP by first constructing MRI log graph using 20 and 200-year MRIs.
- The S_{200}/S_{100} ratio is 1.05, thus the S_{100} elevation = 2.9 m/1.05 = 2.8 m.
- Scaling this S_{100} elevation to S_{500} = 2.8 m x 1.12 = 3.1 m.
- **The flood depth for 1:500 AEP above ground is = 3.1 m – 1.8 m = 1.3 m (4.3 ft).**
- **The flood velocity for 1:500 AEP = 0.5 x $(9.81 \times 1.3)^{1/2}$ = 1.8 m/s (5.9 ft/s).**
- The S_{1000}/S_{100} scaling factor is 1.17.
- The S_{1000} elevation = 2.8 m x 1.17 = 3.3 m.
- **The flood depth for 1:1000 AEP = 3.3 m – 1.8 m = 1.5 m (4.9 ft).**
- **The flood velocity for 1:1000 AEP = 0.5 x $(9.81 \times 1.5)^{1/2}$ = 1.9 m/s (6.3 ft/s).**

2.15.3 Great Lakes Example

The project site is on Lake Ontario in Cobourg. The area is covered by Reach 7 in the report *Lake Ontario Shoreline Management Plan*, prepared by Zuzek, Inc in June 2020. The specific site is adjacent to a soccer field in Cobourg and is a yellow pin on the Google Earth image shown as Figure 2-42.

Figure 2-42 Lake Ontario Site of Interest, red line is 30 m setback of the dynamic beach

Given:

- The lowest ground elevation at the site from Google Earth is 75.9 m (249 ft).
- The Chart Datum elevation for Lake Ontario is 74.2 m
- The 1:100 AEP flood level from the study is 76.01 m.

Find:

- The flood depth at the site for the 1:500 AEP.
- The flood depth at the site for the 1:1000 AEP.

Solution:

- There is no information on other flood elevations for AEP events other than 1:100. Methods outlined in Appendix A can be used to estimate the water levels from the Chart Datum (CD).
- From Table A.3, the empirical scaling factor from the 100 to 500 year flood level is 1.14, while the scaling factor from the 100 to 1000 year flood level is 1.20.
- The 100-year flood elevation above CD is S_{100} = 76.01 – 74.2 = 1.81 m (5.9 ft)
- The 1:500 AEP flood level above the chart datum is S_{500}= 1.14 x 1.81 = 2.06 m (6.8 ft)
- The 500-year elevation is then estimated as 74.2 + 2.06 = 76.26 m (250.1 ft)

- **The predicted flood depth for a 1:500 AEP = 76.26-75.9 m = 0.36 m (1.2 ft).**
- The flood elevation above chart datum for the 1:1000 AEP = 1.20 x 1.81 = 2.17 m (7.1 ft)
- The 1000-yr flood elevation is then 74.2 + 2.17 = 76.37 m (250.5 ft)
- **The predicted flood depth for a 1:1000 AEP = 76.37 m – 75.9 m = 0.47 m (1.5 ft)**

3 Flood Load Formulas and Provisions

3.1 Background and Purpose

This chapter contains formulas and provisions that designers can use to estimate flood forces on structural elements and on entire buildings. The formulas for flood forces are based primarily on flood elevation, flood depth over ground, flood velocity, and on nearshore wave conditions for coastal and lake regions. Following other international building codes and standards, expressions for flood forces are developed for typical structural elements of buildings including vertical walls, vertical columns, horizontal structural members, and floor slabs. The provisions of this chapter apply to buildings located in any areas prone to flooding with the goal of providing a standard method of treating flood forces applicable across Canada.

3.2 Overview

For flood loads, a distinction is made between flood actions in fluvial or riverine areas, and flood actions in coastal areas, Great Lakes areas, or areas adjacent to other large lakes. In fluvial areas, flood loading is the result of hydrostatic actions as well as hydrodynamic actions related to the velocity of moving flood waters. In coastal areas and along lake shorelines, these same loads apply but additional loads may be imposed by the cyclic actions of wind-generated waves. In both riverine and coastal areas, flood-related loads may also be imposed by floating debris, including ice, trees, vehicles, and other types of debris.

Hydrostatic actions arise from fluid pressures under the influence of gravity and are dependent only on the depth below the free water surface. These hydrostatic pressures, when taken over the exposed surface area of a structure, impart forces perpendicular to the surface on which they are applied. Hydrostatic pressures also act below ground level in saturated soil conditions, for example on basement walls or floor slabs.

Net hydrostatic forces arise when water levels are unbalanced over opposing sides of a structure or structural element. Horizontal forces arise when water levels differ from outside to inside a building, or from one side of a building to another. Vertical forces of buoyancy are developed on any element submerged in the fluid. Vertical forces also develop when pressures act under a building that is otherwise water-tight and may develop from hydrostatic pore pressures in saturated soil conditions.

Hydrodynamic actions are fluid loads arising from a mean or net fluid velocity in moving water. These induce drag forces which are generally proportional to the square of the flow velocity. These forces typically act in the direction of fluid motion, act above ground level (assuming flow velocities are negligible within saturated soils) and usually act only in a lateral direction (assuming vertical velocities are much smaller than horizontal velocities).

Flood-related loading may also occur due to the presence of debris in the moving water, including the presence of trees and logs, ice floes, building components (typically wood), vehicles, boats, etc. Debris loads may be due to the accumulation of debris against a structure, which may give rise to increased hydrodynamic loads, or to the impact of floating debris against a structure or structural element, which may cause sudden localized impact loading. Other actions of ice related to adfreezing or ice jacking due to frozen saturated ground waters are assumed to be covered elsewhere in the National Building Code of Canada and thus are not considered here as part of the flood load.

Wave actions arise due to the fluctuations in fluid velocities and pressures associated with oscillatory wave motions around a structure. These same fluctuations allow the crest of the wave to reach elevations above still water level, applying fluid pressures to portions of the structure that may not otherwise be subjected to hydrostatic pressures. The oscillatory motions of waves produce time-varying forces that can be particularly damaging because of the reversals in the direction of loading that may flex or rock a structure back and forth. The most damaging wave actions arise under conditions where waves are breaking.

Flood loading is exacerbated by erosion and scour. Erosion is a general or broad-scale lowering of the ground surface that may occur irrespective of the structure, while scour is defined as a localized lowering of the ground surface related to the interaction of flood waters with the structure. Erosion and scour can affect the stability of the foundation and lead to footing failures. But more fundamentally for flood loading, erosion and scour result in an increase in water depth which can increase flood loads that are dependent on water depth.

The impact of flood forces on a building are additive. Hydrostatic and hydrodynamic forces must be added to obtain a total flood force on the building in riverine areas. Wave loads will then add to other loads in coastal areas. It is generally recommended that the force created by flood-borne debris not be applied to the entire structure but, instead, be applied to a significant structural element of a building (a critical column or a wall corner), and that an analysis for progressive collapse be performed to determine if and how a structure might need to have additional load paths created to resist such a collapse.

3.3 Applicability and Scope Limitations

Provisions in this chapter address forces applied by flood waters on buildings within flood hazard areas as defined by a Provincial/Territorial or municipal Approving Authority (AA). The loading provisions apply to all types of building occupancy, use, and importance categories as defined in the National Building Code of Canada (NBC).

This chapter is not intended for the design of: (1) flood protection structures such as levees, dikes, or flood walls, (2) shore protection structures such as bulkheads and seawalls, (3) port and harbor structures such as piers, docks, or wharves, (4) transportation structures such as railways, roadways, and bridges, or (5) floating structures that may include floating docks or floating buildings. These conditions require specialized consideration of forces related to water or waves that are not fully covered in this chapter.

The most important variable for determining flood loads is the depth of the flood water above ground. The depth of flood waters is addressed in Chapter 2 of this report and is based on a still water surface elevation, absent any wave effects, denoted as the **Design Flood Level (DFL)**. The DFL is determined for a design flood event with a specified mean recurrence interval (MRI) or Annual Exceedance Probability (AEP). This information can be obtained from a flood hazard map or a flood hazard study. The DFL may also include future climate change effects (FC). The **Flood**

Construction Level (FCL) is higher than the DFL due to the inclusion of freeboard or an air gap between the uppermost water surface and lowest habitable portion of a structure. In general, the FCL is useful for siting and establishing building elevation, but it is not used in structural design calculations.

Flood hazard maps generally delineate the lateral extent and vertical elevation of flood waters for certain defined flood events adopted by the Provincial/Territorial or local Approving Authority (AA). As was shown in Figure 2-1, the flood hazard zone in riverine or fluvial locations includes both the floodway, defined as the portion of the floodplain that conveys most of the flow during flood conditions with highest flow velocities, and the flood fringe, defined as the portion of the flood hazard area outside the floodway, which may have still-water flooding or flooding with lower velocities.

In coastal regions and the Great Lakes, the DFL is typically based on the combined effects of storm surge and in ocean regions on high astronomical tides, and also with the effects of wave setup. The FCL should then be elevated for the effects of waves, as depicted earlier in Figure 2-2. In the US, mapping of elevations based on wave effects is common and includes both the elevation of wave crests above the still water as well as the elevation of wave runup on a sloping flooded shoreline. In Canada, only wave runup has been included in mapped flood elevations, but even this is not included consistently in all coastal flood maps.

For computation of flood loads, additional information on ground elevations at the site are required. This information would presumably be available from GIS maps, from the Geological Survey of Canada, from Provincial/Territorial or municipal mapping programs, or from a site survey conducted by a professional land surveyor. Additional considerations would then be required to estimate the general erosion and localized scour at the building site.

As discussed in Chapter 2, the development of consistent Canadian national flood standards may result in a design flood level (DFL) that exceeds values shown on existing flood hazard maps. For example, some jurisdictions may map the 100-year flood level to define the hazard area, but the design flood may be specified as the 500-year flood event or a level associated with some other mean recurrence interval. Methods in the present chapter for computing flood loads apply to any flood level as long as the flood depth is known. This is particularly relevant because buildings that are located outside of flood hazard zones as currently defined may fall within the hazard zone of a more severe (higher MRI) flood event.

3.4 Assessment Process

Information on flood loads is based on a review of existing flood codes and guidelines adopted in other countries or in the peer-reviewed engineering literature. Primary reference sources are the following:

International Organizations

- International Organization for Standardization, (2016), Actions from Waves and Currents on Coastal Structures, Standard ISO 21650

Australia/New Zealand Guidance

- Standards Australia/Standards New Zealand (2002), Structural design actions, Part 1 Permanent, Imposed and Other Actions, Standard AS/NZS 1170.1
- Australian Building Code (2012), Information Handbook: Construction of Buildings in Flood Hazard Areas
- Australian NCC (2016), Building Code of Australia - Volume Two

Canadian Guidance

- Canadian Highway Bridge Design Code –CSA S6 (2006), Canadian Standards Association.

European Guidance

- EUROCODE EN 1991-1-6 (2005) (English): Eurocode 1: Actions on structures - Part 1-6: General actions - Actions during execution
- BSI (2015) BS 85500:2015 Flood resistant and resilient construction – Guide to improving the flood performance of buildings. BSI Standards Limited, London 2015
- BSI (2003), Maritime Structures, BSI Standards Limited, BS 6349-1:2000.

United States Guidance

- AASHTO (2020)
- ASCE 7 (2016) Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, as well as draft revisions for upcoming ASCE 7-22 (scheduled for 2022)
- ASCE/SEI 24 (2014) Flood resistant design and construction, ASCE
- FEMA (2011) Coastal Construction Manual FEMA P-55
- FEMA (2013) Floodproofing Non-Residential Buildings, FEMA P-936
- Port of San Francisco Building Code (2013)
- City of Honolulu Building Code (2017)
- US Army Corps of Engineers (1984) Shore Protection Manual
- US Army Corps of Engineers (2002) Coastal Engineering Manual, EM 1110-2-1100

3.5 General Approach to Flood Loads

During flood events, buildings located in flood hazard areas may experience one or more types of flood loads, which fall under four main categories – hydrostatic, hydrodynamic, debris, and wave loads.

In all flood loading situations, some type of hydrostatic effects will always be present, as any portion of the structure located below the flood level will experience hydrostatic fluid pressures. This includes portions of the structure located below grade and subjected to saturated soil conditions. The lateral hydrostatic effects may cancel if water levels are the same on two sides of a structure or structural element inside to outside a wall, or front to back of a building. In some cases, flood waters are purposely allowed to enter a building envelope to reduce loads on the main supporting structure.

Hydrodynamic effects may or may not be present depending on the flow conditions. In riverine situations, many flood fringe locations may serve as flood storage areas and experience standing water with little net flow, so that hydrodynamic effects may not be present. In riverine situations with moving water, primary flood loads (in addition to hydrostatic) are due to hydrodynamic drag on the entire building, on one or more structural elements, and potentially due to debris that accumulates against the structure or that drifts into the structure due to moving flood waters.

Wave effects are not considered significant in most riverine situations. However, some larger riverine locations may have very wide floodplains (on the order of several kilometres) and wind-generated waves may be important to consider. Wave effects are generally not mapped in these wide riverine floodplains, but a designer should be aware that any region where wind can act over open water for more than a kilometre or so can result in potentially damaging wave action.

In the Great Lakes and coastal regions (and potentially along the shoreline of any large lake), hydrodynamic effects could be caused by drag forces associated with net flow velocities. In many storm surge situations however, flood waters may flow in and out over a period of many hours, and mean flow velocities may be small so that hydrodynamic loads are small. Larger loads are then possible due to the action of wind waves. In some situations, waves may be non-breaking and exert small loads while in other cases depth-limited breaking waves may impose much larger loads.

The design of buildings in flood zones requires a careful investigation of combinations of the various flood load types. As described above, typical load scenarios may include:

Riverine conditions:

- Hydrostatic only with no moving water
- Hydrostatic plus Hydrodynamic drag (moving water but no waves)
- Hydrostatic plus Hydrodynamic drag plus Debris loads

Coastal and Great Lakes conditions:

- Hydrostatic only with no moving water or waves
- Hydrostatic plus Hydrodynamic drag (moving water but no waves)
- Hydrostatic plus Hydrodynamic drag plus Debris loads
- Hydrostatic plus Waves loads (coastal conditions with no net currents)
- Hydrostatic plus Wave loads plus Debris impacts
- Hydrostatic plus Hydrodynamic drag plus Waves plus Debris loads

3.6 Hydrostatic Loads

Hydrostatic loads occur whenever flood waters contact a foundation, building, or building element. Hydrostatic actions must be applied to all structural surfaces below the design flood level. It is important to also include hydrostatic pore water pressures in saturated soil conditions for any portion of the structure located below grade.

3.6.1 Hydrostatic Pressures

Hydrostatic loads originate from fluid hydrostatic pressures that depend on the effective weight of water above a point of interest that is submerged. Hydrostatic pressure may be given by

$$P_h = \rho g\, z = \gamma\, z \qquad \text{(Eq. 3-1)}$$

where:

P_h = hydrostatic pressure, in N/m^2 (or lb/ft^2)
z = depth of submergence, from SWL to point of interest (including in submerged soils)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
ρ = water density, 1,000 kg/m^3 fresh, 1,025 kg/m^3 salt (1.94 slug/ft^3 fresh, 1.99 slug/ft^3 salt)
γ = water unit weight, 9,810 N/m^3 fresh or 10,055 N/m^3 salt (62.4 lb/ft^3 fresh, 64.0 lb/ft^3 salt)

Values of fluid density and unit weight given above are standard reference values, and some adjustment could be warranted in certain local situations. For example, the ASCE 7-16 (2016) chapter on tsunami loads requires a 10% increase in fluid density when strong flows result in substantial suspended sediment and debris entrained in moving waters.

3.6.2 Buoyancy

A fundamental effect of hydrostatic pressures is that any portion of a structure or structural element (or the entire structure) that is wholly or partially submerged in water will experience a vertical buoyant force due to the effects of hydrostatic pressures around the perimeter of the structural element (or water tight structure). For any object shape, the vertical buoyant force can be given by Archimedes Principle as

$$F_B = \rho g \not{V} = \gamma \not{V} \qquad \text{(Eq. 3-2)}$$

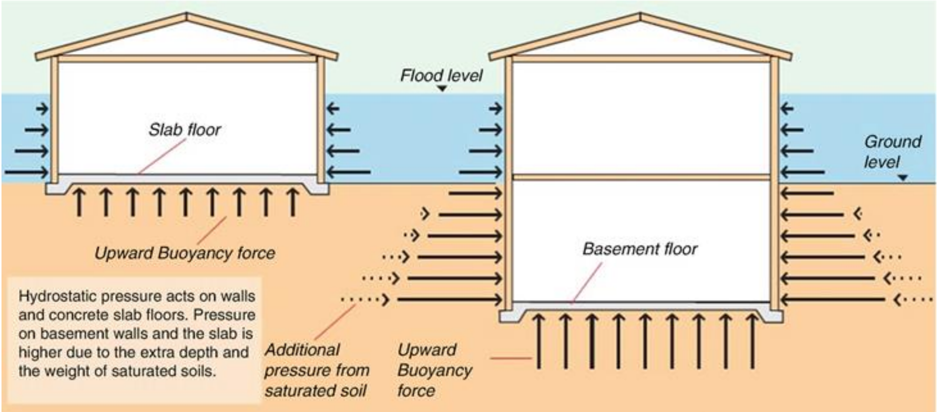
where:

F_B = vertical buoyant force due to hydrostatic pressures, in N (or lb)
$\not{V}$ = volume of water displaced by the structural element, in m^3 (or ft^3)

The submerged volume is based on the assumption that the structural element or portion of the structure under consideration is water-tight. This may include the volume of trapped air pockets that act as part of the structural element. This is the case with submerged bridge decks where air may be trapped between girders and would also apply to buildings if intact air pockets form between floor joists.

The submerged structural volume may also be wholly or partially embedded in foundation soils giving rise to buoyancy forces if the soil is fully submerged and saturated. In this case, pore water pressures in the soil are assumed to be fully hydrostatic based on depth measured below the design free water surface. Examples of below-grade structural elements for which buoyancy should be considered include basement floor slabs, footings, portions of columns extending below grade, and embedded pile foundations.

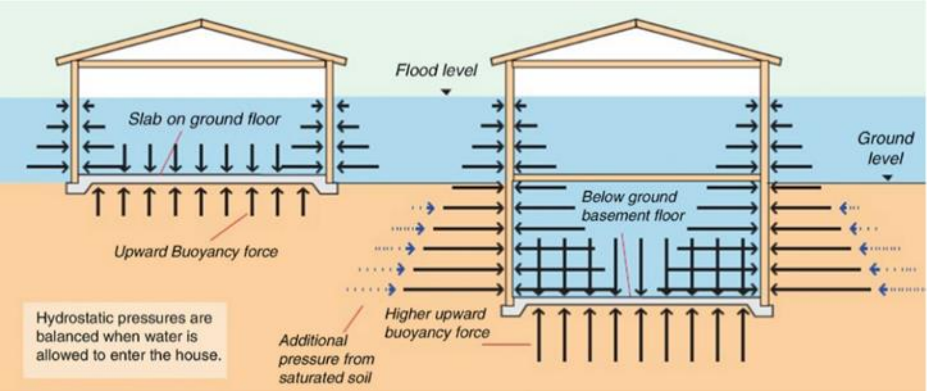
If a building is constructed with water-tight foundation walls and floor slab, the entire submerged volume of the building should be used to compute the overall buoyant force on the building. This is illustrated in Figure 3-1, from the FEMA *Coastal Construction Manual* (2011). This would apply to buildings with dry floodproofed basement floors and walls, where hydrostatic fluid pressures would be applied under the floor based on the depth of submergence below the design water level assuming saturated soils, but with air at atmospheric pressure inside the building.



Hydrostatic pressure acts on walls and concrete slab floors. Pressure on basement walls and the slab is higher due to the extra depth and the weight of saturated soils.

Figure 3-1 Buoyancy and hydrostatic forces on water-tight building (source: FEMA, 2011)

A reduction in buoyant forces occurs if provision is made for entry and exit of floodwater, as illustrated in Figure 3-2. If openings are sufficient to allow rapid interior flooding, then the water level will equalize between the inside and outside of the building. This may mostly eliminate any buoyant force on the building. However, hydrostatic pressures will still differ between the top and bottom of the floor slab so that some residual buoyant force on the floor slab will remain. Similarly, some residual buoyant force will remain on any submerged foundation elements or on wall elements.



Hydrostatic pressures are balanced when water is allowed to enter the house.

Figure 3-2 Reduction in hydrostatic loads when water is allowed to enter building (source: FEMA, 2011)

3.6.3 Vertical Hydrostatic Force

In cases when a more fundamental method of computing vertical hydrostatic forces is required, the vertical hydrostatic force can be computed as

$$F_v = (P_{bot} - P_{top})A_h = (\rho\, g\, Z_{bot} - \rho\, g\, Z_{top})A_h \qquad \text{(Eq. 3-3)}$$

where:

F_v = vertical hydrostatic force, in N (lb)
P_{bot} = hydrostatic pressure on bottom of structure or element, based on Z_{bot}

P_{top} = hydrostatic pressure on top of structure or element, based on depth to top Z_{top}
A_h = contact area over which pressures act, in horizontal plane
Z_{bot} = submerged depth to bottom of structure or element, from free water surface
Z_{top} = submerged depth to top of structure or element, from free water surface

In some cases when either the top or bottom of a structural element is exposed to air, either the top or bottom pressures would be atmospheric, i.e. $P = 0$.

When determining the vertical hydrostatic forces for an entire building, Equation (3-3) should be applied to each submerged structural element and then summed for all submerged elements of the building and foundation. In some cases, when the entire structure is submerged or when major portions of the structure are submerged, the direct application of the buoyant force from Equation (3-2) may be simpler in lieu of applying Equation (3-3) to each structural element.

Hydrostatic uplift forces can be quite large, especially for large volume structures and for dry-floodproofed structures. For this reason, the most important provisions to reduce vertical flood loading is to adopt one of two strategies: (1) elevate fully enclosed spaces above the design flood water level; or (2) allow flood waters to enter the spaces to equalize hydrostatic pressures.

3.6.4 Residual Water Surcharge

If horizontal floors are overtopped by flood waters, upward loads due to buoyancy will occur during the flood, but downward vertical loads due to the weight of water trapped above the floor may occur as flood waters recede. This may occur if walls retain water above the floor level, as might occur if flood water enters through broken windows but is then trapped at the height of the windowsill as flood waters drop. Downward vertical loads can be computed using Equation (3-3), with a flooded depth above the floor of Z_{top}, and with pressures below the floor taken as $P_{\text{bot}}=0$ due to exposure to air due to receding flood waters.

3.6.5 Lateral Hydrostatic Force

Hydrostatic fluid pressures induce lateral or horizontal hydrostatic loads when they act over a vertical structural surface. Pressures are zero at the free water surface but increase linearly with increasing depth, Z, below the surface, giving rise to the classic triangular hydrostatic pressure distribution. The net lateral force is then obtained by depth-integrating (or finding the area under) the hydrostatic pressure distribution.

On a full-depth vertical wall (extending above the water surface), the horizontal hydrostatic force per unit width (units of N/m) is illustrated in Figure 3-3 and is given by

$$f_H = \tfrac{1}{2} \rho g d_f^2 \qquad \text{(Eq. 3-4)}$$

where:

f_H = hydrostatic force per unit width along the wall, in N/m (or lb/ft)
d_f = design flood depth, based on design flood level minus ground elevation, including any added depth due to erosion or scour, as defined in Chapter 2

For a wall width, w, the total hydrostatic force (units of N) would be

$$F_H = w\, f_h = w\, ½\, \rho\, g\, d_f^2 \qquad \text{(Eq. 3-5)}$$

where:

F_H = hydrostatic force on the wall, in N or lbs
w = width of the wall perpendicular to the flow, in m (ft)

For computation of overturning moments, the point of application of the load based on the triangular pressure distribution is at a vertical location 2/3 d_f below the water surface, or 1/3 d_f above the ground level.

ASCE 7 (2016) requires hydrostatic forces to be based on the design flood depth plus an additional 0.30 m (1 ft) as an added factor of safety to account for uncertainties in defining the still water level. One reason for this provision is that flood codes in the US adopt a 100-year MRI flood event, so the addition of 0.3 m (1 ft) raises the effective MRI for design to something higher than 100 years. But the effective MRI for design then differs, sometimes dramatically, in different parts of the country or between riverine and coastal conditions. For Canada, a more robust approach would be to adopt a higher MRI design event that would give more consistent flood risk across the country.

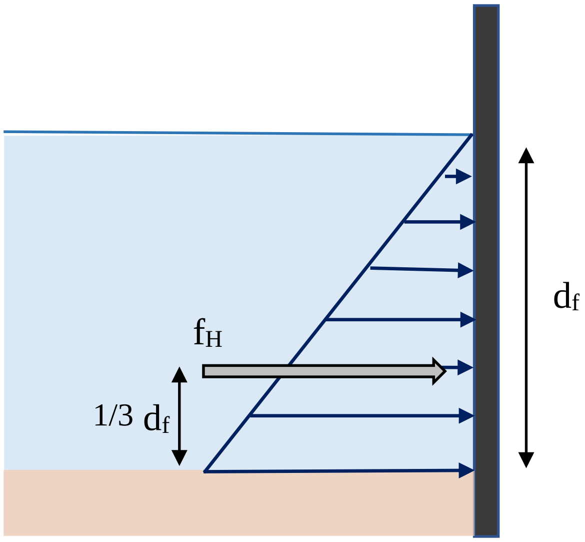


Figure 3-3 Hydrostatic pressures and force on a vertical wall

For walls that are elevated above ground level, for example when a building is elevated on piles or column foundations, hydrostatic loads can be computed using the submerged depth of the wall, d_b. As shown in Figure 3-4, hydrostatic pressures would be applied over the depth to the bottom of the building, and the horizontal hydrostatic force would then be given by

$$F_H = w\, f_H = w\, ½\, \rho\, g\, d_b^2 \qquad \text{(Eq. 3-6)}$$

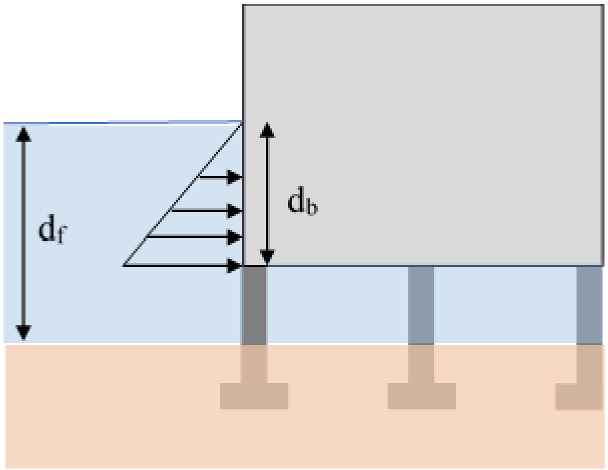


Figure 3-4 Hydrostatic pressures on wall of elevated building

If the wall extends below grade, then as shown on Figure 3-5, the hydrostatic pressure distribution continues into the saturated soil. The force per unit width and total force on the wall are then amended as

$$F_H = w\, f_H = w\, ½\, \rho\, g\, (d_f^2 + d_g^2) \qquad \text{(Eq. 3-7)}$$

where:

d_g = submerged depth below grade

The point of application would then be at 2/3 (d_f + d_g) below the water surface. This represents only the water (flood) load. The geotechnical loads due to soil effective stress would then be added, but this is not considered part of the flood load. The soil effective stress would include additional soil pressures, using the effective unit weight of the soil given as the submerged unit weight minus the unit weight of water.

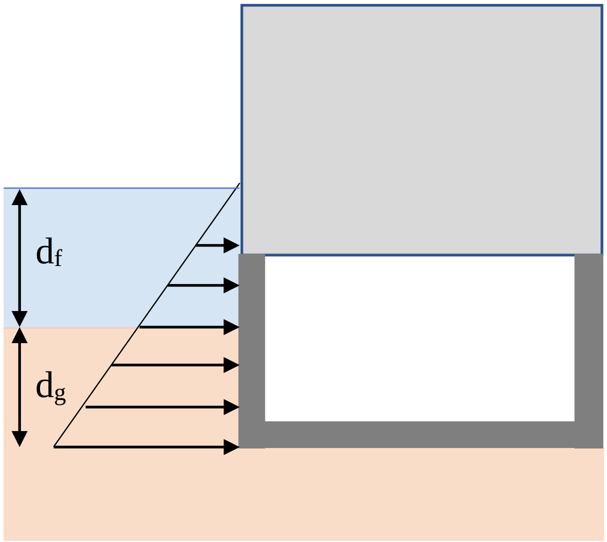


Figure 3-5 Hydrostatic loads on basement walls in submerged soil conditions

Equations (3-4) to (3-7) apply to one side of a wall, or one side of a building. But hydrostatic forces may exist on other walls and may act in other directions. For a water-tight building, horizontal hydrostatic pressures would exist on both up and downstream walls of the building. Because these would act in opposite directions, net horizontal hydrostatic loads on the building would cancel as far as global loads on the building are concerned. Loads on the building may only partially cancel if the water level on one side of the building is higher than the other. For a water-tight building, hydrostatic loads may cancel on the building as a whole but would still affect individual wall elements if the exterior side of the wall was flooded and the interior was dry.

Hydrostatic loads on wall elements are reduced if the interior space is purposely allowed to flood. Some codes, e.g. Australian Building Code (2016) limit the allowable differential in water levels from outside to inside to no more than 1 m (3.3 ft) to prevent or limit damages from lateral hydrostatic loads, thus requiring rapid free flooding to limit loads on wall panels. One method of limiting hydrostatic and other flood loads on walls is to equalize fluid pressures through use of flood vents in walls to allow for free inflow of water as flood waters rise.

Another method of reducing hydrostatic loads, and other hydrodynamic loads, is to require break-away walls and partitions. ASCE 7 (2016) and ASCE 24 (2014), as well as FEMA guidance (2011), require walls to break away, including their connections to the structure, at a pressure loading of not less than 0.48 kN/m^2, (10 lb/ft^2) but no more than 0.96 kN/m^2, (20 lb/ft^2). This requirement ensures that the break-away wall is activated, and results in flooding of interior spaces to equalize pressures, well below any level that would be damaging to the structure as a whole. Code provisions in ASCE 7-16 allow larger loads on breakaway walls if other loads (such as wind or earthquake loads) would commonly exceed 0.96 kN/m^2 (20 lb/ft^2), provided the designer certifies that the walls have been designed to break away before base flood conditions are reached, without damaging the elevated building or its foundation.

3.6.6 Special Cases

Some unique structural geometries may require further analysis. For example, methods given above would be modified for inclined, rounded, or irregular structural shapes. For these cases, hydrostatic loading may be analyzed using basic concepts covered in most fluid mechanics textbooks.

3.7 Hydrodynamic Loads

The full range of hydrodynamic loads includes any actions resulting from moving water over and above the hydrostatic actions. In riverine situations, where flood flows are assumed to be unidirectional and quasi-steady at the time of the peak flood, hydrodynamic loads arise from fluid drag forces on the building or on structural elements. In coastal (and Great Lakes) situations, these same hydrodynamic drag forces would apply if the flood water has a unidirectional and quasi-steady flow component. As noted, short period fluctuations in hydrodynamic loads are also produced by the actions of waves, but these will be treated separately. For the purposes of this chapter, hydrodynamic loads arise only from quasi-steady unidirectional flows.

3.7.1 Stagnation Pressure and Water Level Rise

As steady flow interacts with a column, wall, or building, fluid pressures on the upstream side of the obstruction will increase above the value from hydrostatic pressure based on the design flood depth d_f. The pressure rise will approach the stagnation pressure

$$P_s = ½ \rho V^2 \qquad \text{(Eq. 3-8)}$$

where:

P_s = stagnation pressure
V = flow velocity, averaged over depth, in m/s (ft/s)

If the obstruction is wide, the water surface elevation also rises at the leading edge, producing an increase in depth equal to the velocity head, $V^2/2g$. While these pressures and water level increases are not frequently used to compute loads, they may be important in some circumstances. For example, if doors or windows on the upstream side of a building fail, fluid pressures and water levels inside the building may increase above hydrostatic values for the design flood depth d_f by the amount of the stagnation pressure.

Water levels similarly drop along the sides of the object (sometimes called the drawdown) and then recover somewhat at the rear. The pressure changes are treated in most fluid mechanics texts and are often complicated to predict. However, the drop in pressure along the sides may be approximated as $-½ \rho V^2$ with a water surface change of $-V^2/2g$. Water levels and pressures at the rear are often near zero (ambient) but may depend on the shape and dimensions of the object.

The resulting situation creates unbalanced fluid pressures around the perimeter of the building. Methods of treating these pressures on each distinct wall are commonly used when defining wind loads on buildings, where pressure coefficients are defined for front, rear, and side walls, e.g. a pressure coefficient could be added to Equation (3-8) to account for variation in pressures on each wall of a building. These methods are not yet adopted in any building code for flood loads, though research papers can be found on this topic.

3.7.2 Simplified Hydrodynamic Drag

As a simplified method for estimating hydrodynamic effects when the flow velocity is low, most design codes and other guidance, e.g. ASCE 7 (2016), Australian Code (2016), and FEMA (2011), contain a method of estimating fluid forces on a building based on the assumption that water levels on the upstream side are increased by the stagnation pressure. In these cases, provided the flow velocity is less than 3 m/s (10 ft/sec), hydrodynamic loads are converted into equivalent static loads using an equivalent hydrostatic surcharge of

$$\Delta d_f = \alpha V^2/2g \qquad \text{(Eq. 3-9)}$$

where:

Δd_f = increase in water level on upstream side compared to downstream side, in m (ft)

a = shape factor or drag coefficient = 1.25 (or from Table 3-1 below as a function of building aspect ratio)

This equivalent depth is then added to the design depth, giving a total depth ($d_f + \Delta d_f$) that is used to compute lateral hydrostatic loads on the structure in lieu of a formal analysis of hydrodynamic effects. This is illustrated in Figure 3-6. With this approach, other methods of computing the drag force discussed below would not be used.

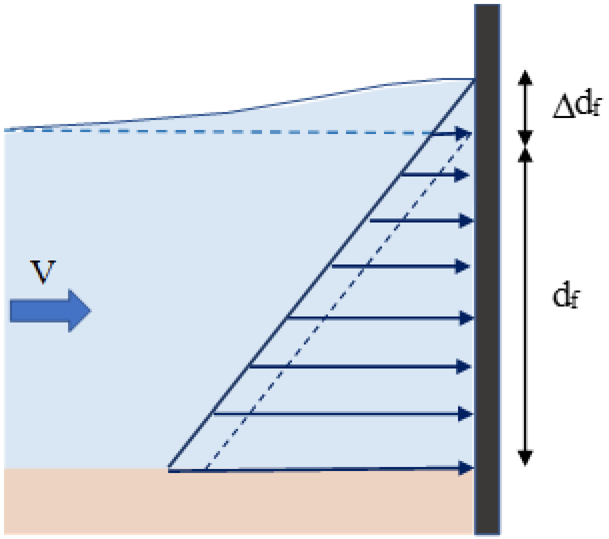


Figure 3-6 Water level rise on upstream side creating an effective increase in hydrostatic loading

3.7.3 Hydrodynamic Drag

A more fundamental method of computing the hydrodynamic force exerted by steady uniform flows on structural components is to use a standard drag force expression as

$$F_D = \tfrac{1}{2} C_D \rho V^2 A_p \qquad \text{(Eq. 3-10)}$$

where:

F_D = hydrodynamic drag force, in N (or lbs)
C_D = drag coefficient, function of structure shape and dimensions
V = velocity of flood water, in m/s (or ft/s)
A_p = the projected area of structure or structural element exposed to the moving water

For the calculation of bending moments, the drag force is often applied at mid-depth under the assumption of uniform flow velocities over depth. For additional conservatism, the load is sometimes applied at the water surface, thereby increasing the moment arm above ground to equal the full water depth. An alternative method would base the point of application on a velocity distribution accounting for the flow boundary layer, e.g. 1/7th power law or similar.

Equation (3-10) may be applied to a range of different structural geometries by redefining the projected area. For the case of vertical columns, when the design flood level is lower than the top of the column, the projected area is $A_p = w\ d_f$ where w is the column width and d_f is the design flood depth. When the design flood level is higher than the column, then $A_p = w\ hc$, where hc is the column height which is less than d_f. For the case of horizontal structural members oriented perpendicular to the flow, $A_p = w\ h$ where h is the height of the member perpendicular to the flow and w is the span-wise member length. For the case of entire buildings with walls that block the flow, Equation (3-10) may be applied to the entire building with $A_p = w\ d_f$ where w is the width of the building perpendicular to the flow, as shown in Figure 3-7.

The application of Equation (3-10) requires the selection of an appropriate drag coefficient based on the dimensions and geometry of the building or structural element. In some references, simplified standard values are adopted. The EUROCODE (2005) adopts $C_D = 0.7$ for circular members and $C_D = 1.44$ for square or rectangular members. The City of Honolulu building code (2017) specifies $C_D = 1$ for circular piles, $C_D = 2$ for square piles, and $C_D = 1.5$ for wall sections.

For entire buildings, and for a large width-to-depth aspect ratio of the structural members, FEMA (2011) adopts drag coefficient values in Table 3-1. For individual structural members, the ASCE 7 (2016) recommends Table 3-2 for selecting C_D. FEMA (2011) indicates that additional guidance on drag coefficients may be obtained from the USACE Shore Protection Manual, (1984), from FEMA (2001), or from accepted fluid mechanics texts.

Equation (3-10) is developed for a condition where the flow is normal to the building. Existing building codes do not address conditions where flow may be at an angle to the building, and FEMA guidance likewise does not include methods for treating oblique flow conditions. Research papers may be found on the subject, many based on numerical simulations for simple prismatic cylinders, but guidance for buildings in flood conditions does not seem to exist. One approach to the problem, consistent with general principles, is to apply Equation (3-10) using the projected area of the structure perpendicular to the flow to estimate the drag force in the direction of the flow. This force vector can then be resolved into components perpendicular to the walls of the building.

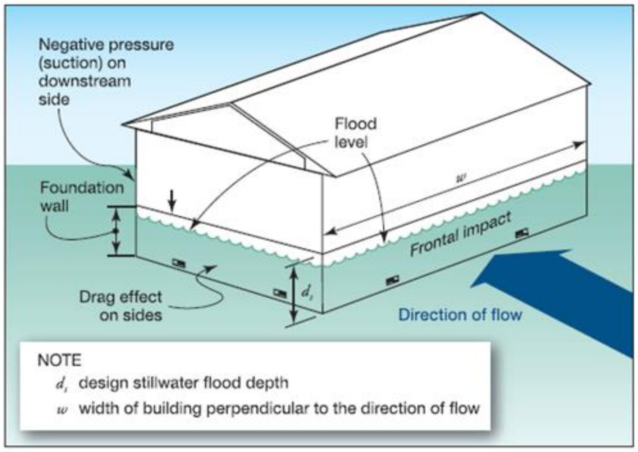


Figure 3-7 Hydrodynamic loading on a building (source: FEMA, 2011)

Table 3-1 Drag coefficients for buildings (source: FEMA, 2011)

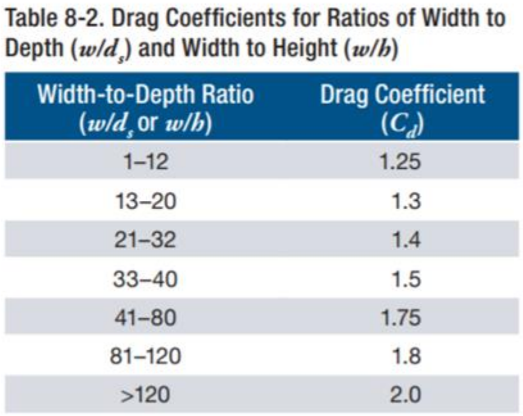
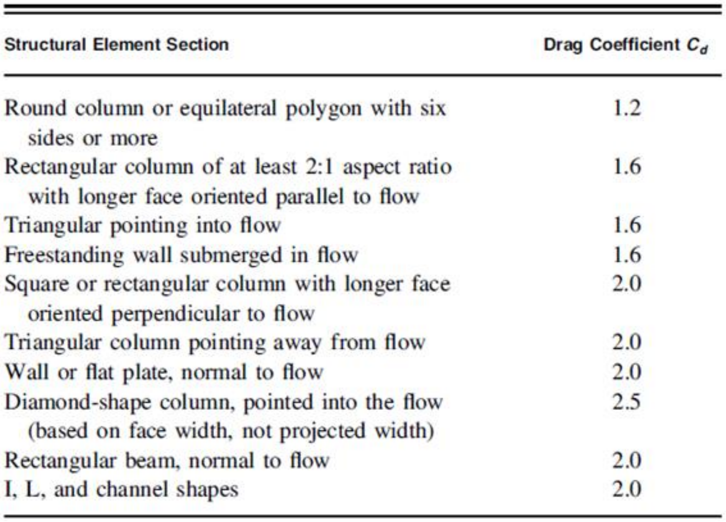


Table 8-2. Drag Coefficients for Ratios of Width to Depth (w/d_s) and Width to Height (w/h)

Width-to-Depth Ratio (w/d_s or w/h)	Drag Coefficient (C_d)
1–12	1.25
13–20	1.3
21–32	1.4
33–40	1.5
41–80	1.75
81–120	1.8
>120	2.0

Table 3-2 Drag coefficients on structural elements, from ASCE 7 (2016)



Structural Element Section	Drag Coefficient C_d
Round column or equilateral polygon with six sides or more	1.2
Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow	1.6
Triangular pointing into flow	1.6
Freestanding wall submerged in flow	1.6
Square or rectangular column with longer face oriented perpendicular to flow	2.0
Triangular column pointing away from flow	2.0
Wall or flat plate, normal to flow	2.0
Diamond-shape column, pointed into the flow (based on face width, not projected width)	2.5
Rectangular beam, normal to flow	2.0
I, L, and channel shapes	2.0

3.7.4 Hydrodynamic Lift

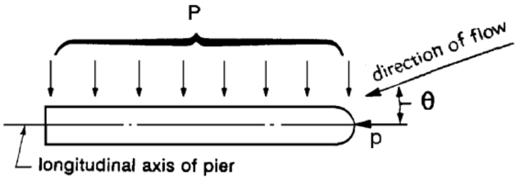
Flow around a building or building element may also create lateral forces perpendicular to the flow, which may be termed lift forces. FEMA (2011) indicates that for rigid structures, the lift is usually assumed to be small and can be neglected. Additional guidance on the subject can be obtained from the AASHTO (2012) LRFD Bridge Design Specifications, which address lateral forces on wall-like bridge piers as shown in Figure 3-10. In this case lift forces can be estimated as follows

$$F_L = ½\, C_L\, \rho V^{2}\, A_p \qquad \text{(Eq. 3-11)}$$

where:

F_L = hydrodynamic lift force, in N (or lbs)
C_L = lift coefficient, function of structure shape and dimensions

Consistent with FEMA guidance, the lift is zero for flow parallel to the wall, but the lift coefficients increase rapidly for even small angles of attack. It is not known whether these methods apply to buildings in flood conditions, and lift coefficients in Figure 3-8 should be considered interim until more appropriate guidance can be developed.



Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0 degrees	0.0
5 degrees	0.5
10 degrees	0.7
20 degrees	0.9
≥30 degrees	1.0

Figure 3-8 Lift coefficients for flow oblique to wall element (source: AASHTO, 2012)

3.8 Debris Loads

Separate from the action of water on a flooded structure, loads may also be imparted by debris carried by moving water. Two types of debris loads are recognized in the ASCE 7 and in the FEMA guidance: (1) due to an accumulation of a mass of debris against a structure and (2) due to an impact of an individual item of water-borne debris. Both types of debris loads may affect structures in riverine floodplains. Coastal flood loads are more likely due to debris impacts only as there may be little net current and oscillatory wind waves tend to disperse large accumulations of debris.

The types of water-borne debris can vary with geographic area. In many areas, logs and woody debris are common, but typical log dimensions could vary with larger logs prevalent in British Columbia compared to other areas. Ice floes are another common type of floating debris in many areas, and the typical dimensions of floating ice may vary from location to location. Man-made debris may predominate in some areas, particularly in urban areas. In coastal regions, remnants of damaged structures, including sections of foundation piles, tend to drift in waves and wash against shorelines. In riverine areas, floating cars or semi-trailers could form a common type of debris.

One complication with designing for debris impacts is that the size of the debris may be limited in size only by the water depth and draft of the floating object. Images in floods sometimes show entire homes or barges floating downstream. Ice sheets may be very large and may re-freeze with others in larger ice jams. For large debris, it is usually impractical to fully design a structure to resist the loads, so the emphasis is often on structural performance to prevent progressive collapse should debris damage one or more structural members.

Because of the variable size of debris, most design codes specify upper ranges of debris to be considered as practical for design. Most guidance in ASCE 7 (2016), for example, has involved a 4.5 kN (1,000 lb) log (or a section of a telephone pole or timber pile) with a length of 9.1 m (30 ft). Selection of typical and reasonable debris types, dimensions, and weights (or mass) should be based on discussion and consensus from Provincial or Municipal Authorities specific to the geographic area.

3.8.1 Debris Accumulations

Loads due to debris accumulations are assumed to be represented as steady loads applied statically to a structure. These loads should be analyzed for effect on any structural element that may support trapped debris, as well as on the global structural analysis.

Limited guidance exists for predicting the forces of drift accumulation. For buildings, the Commentary in ASCE 7 (2016) and the USACE (1995a) "Flood Proofing" guide suggests a uniform load of 1.48 kN/m (100 lb/ft) acting in a strip 0.3 m (1 foot) thick along the length of the structure at the waterline. More rigorous prediction of the forces due to drift accumulation can be based on the increased fluid drag forces on the drift accumulation. This is given in ASCE 7 (2016) by the standard drag force expression

$$F_{Da} = ½ C_D \rho V^2 A_a \qquad \text{(Eq. 3-12)}$$

where:

F_{Da} = drag force due to debris accumulation, in N (lb)
V = flow velocity upstream of debris accumulation, in m/s (ft/s)
A_a = projected area of the debris accumulation into the flow, approximated by depth of accumulation times width of accumulation perpendicular to flow, in m^2 (ft^2)
C_D = drag coefficient, assumed to equal 1

The EUROCODE BS EN 1991-1-6 (2005) provides a similar expression that utilizes a drag coefficient of C_D=1.33.

The expression above produces loads similar to the 1.48 kN/m (100 lb/ft) guidance when the debris depth is assumed to be 0.3 m (1 ft) and when the velocity of the flood water is 3 m/s (10 ft/s). In many cases however, with ice jams or with accumulation of logs and woody debris, the depth of debris accumulation is much greater than 0.3 m (1 ft) and is only limited by the water depth at the structure. Observations of woody debris accumulations at bridges often show depths of 1.5 to 3 m (5 to 10 ft) or more, with horizontal widths spanning between adjacent bridge piers whenever the spacing of the piers is less than the typical log length. If debris accumulation is of concern, the design professional should specify the projected area of the debris accumulation based on local observations and experience. Individual Provincial or Municipal Authorities could also specify values to be used in design.

3.8.2 Debris Impact Loads

Two methods of estimating debris impact loads are given below. The first method, from FEMA (2011) and the ASCE 7 (2016) Chapter 5 on Flood Loads is applicable to woody debris and is the simplest to apply. The second method, from the ASCE 7 (2016) Chapter 6 on Tsunami Loads, is more consistent with impact mechanics but requires additional information on the structural stiffness that may not always be available. Debris impacts produce short-duration impulsive loads, that can be quite large. In lab tests supported by FEMA, and forming the basis for the first method below, one test produced a maximum impact load of 37,000 N (8,300 lb) for a log weighing just 3,250 N (730 lb), moving at 1.2 m/s (4 ft/sec).

3.8.2.1 Debris Impact from FEMA and ACSE 7 (2016) Flood Load Chapter

In the ASCE 7 (2016) Chapter 5 on Flood Loads, debris impact loads are included in the Commentary but not in the main code provisions. As a result, inclusion of debris impact in flood conditions is not compulsory. This method, also outlined in FEMA (2011), had its origin in the City of Honolulu building code and was further developed by FEMA based on guidance from laboratory tests using logs towed into a rigid target.

The expression used in ASCE 7 for debris impacts is based on an impulse-momentum approach and is given by

$$F_{di} = 1.57\, C_{de}\, C_{bl}\, C_{or}\, R_{max}\, W\, V \,/\, (g\, \Delta t) \qquad \text{(Eq. 3-13)}$$

where:

F_{di} = impact force, in N (lbs)
W = weight of debris, in N (lbs)
V = velocity of flood water propelling debris, in m/s (ft/s)
Δt= impact duration or time to reduce object velocity to zero, in sec, taken as 0.03 sec
C_{de} = depth reduction coefficient
C_{bl} = debris blockage coefficient
C_{or} = debris orientation coefficient, recommended as 0.8
R_{max} = structural response coefficient

The size, shape, and weight of waterborne debris may vary according to region. In the absence of information about the nature of potential debris from local sources, ASCE 7 (2016) recommends a weight of 4.5 kN (1,000 lbs). Objects with this weight could include portions of damaged buildings, utility poles, portions of previously embedded piles, and empty storage tanks.

For calculating debris loads, the velocity of the waterborne object is assumed to be the same as the flood velocity. Although this assumption may be accurate for small objects, it may overstate debris velocities for large objects that drag on the bottom or that strike nearby structures.

The depth coefficient (C_{de}) accounts for reduced debris velocity as water depth decreases, as shown in Figure 3-9. For buildings in high velocity zones with depth greater than 1.5 m (5 ft), the depth coefficient is taken as 1.0. The coefficient then decreases linearly to zero at a flood depth of 0.3 m (1 ft).

The blockage coefficient (C_{bl}) accounts for the expected reduction in debris velocity because of screening provided upstream by trees or other structures. The coefficient, shown in Figure 3-10, is based on a standard log length of 9 m (30 ft) so that the coefficient is 1.0 for openings upstream greater than 9 m (30 ft) and decreases to 0.6 for flow path openings of 6 m (20 ft), 0.2 for openings of 3 m (10 ft) and is reduced to zero for opening less than 1.5 m (5 ft).

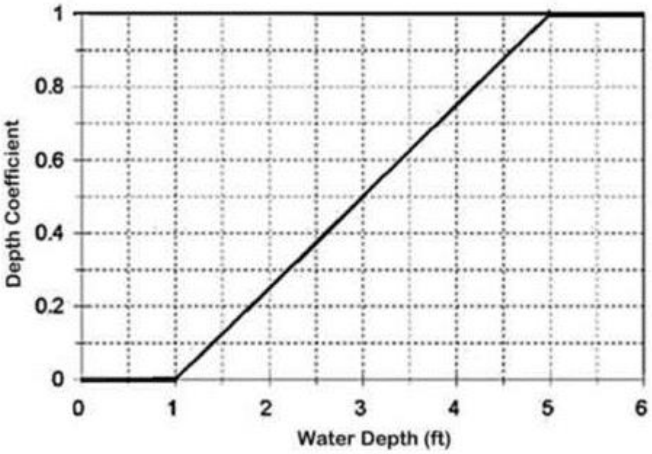


Figure 3-9 Depth reduction coefficient for debris impact loads (source: ASCE 7, 2016)

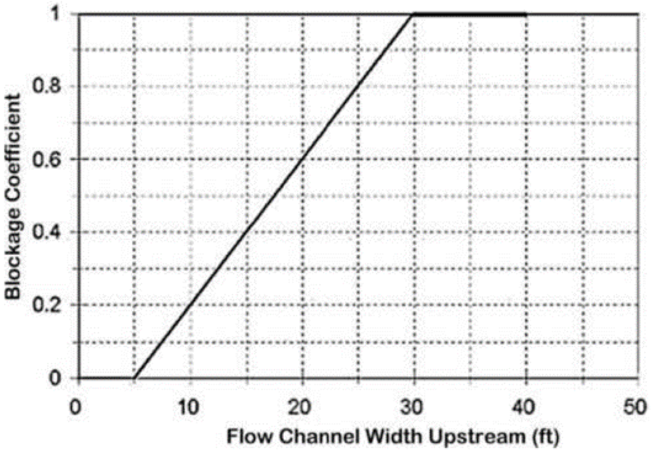
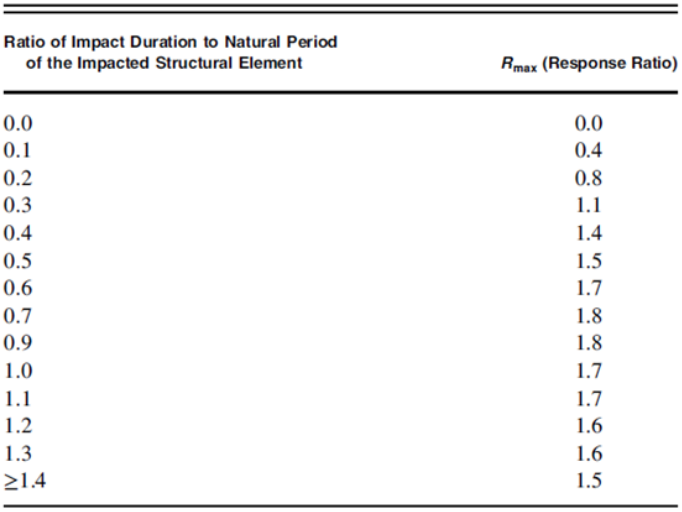


Figure 3-10 Blockage coefficient for debris impacts (source: ASCE 7, 2016)

The structural response coefficient R_{max} accounts for dynamic amplification that may occur if the impact duration of 0.03 sec is near the natural period of the structural element. Values are given in Table 3-3 and range from zero for structures with long natural periods to 1.5 for structures with a natural period near the impact duration.

Table 3-3 Dynamic amplification factor for debris impacts (source: ASCE 7, 2016)



Ratio of Impact Duration to Natural Period of the Impacted Structural Element	R_{max} (Response Ratio)
0.0	0.0
0.1	0.4
0.2	0.8
0.3	1.1
0.4	1.4
0.5	1.5
0.6	1.7
0.7	1.8
0.9	1.8
1.0	1.7
1.1	1.7
1.2	1.6
1.3	1.6
≥1.4	1.5

ASCE 7 guidance indicates that impact loads should be applied to the most critical structural member but need not be applied to other structural members simultaneously due to the small chance of multiple members being struck at the same instant. The object is assumed to be at or near the water surface level when it strikes the building. The impact loads are also impulsive in nature but are often treated as a static load in structural analysis. If warranted, the loads could be applied in a dynamic structural analysis.

3.8.2.2 Debris Impact Loads from ASCE 7-16 Tsunami Load Chapter

In the ASCE 7 (2016) Chapter 6 on Tsunami Loads, debris loads are included in the main code, making it mandatory to include debris loads in tsunami resistant building design. The approach used in ASCE 7 (2016) Chapter 6 is based on an elastic response model in which loads are governed by the elasticity and stiffness of the debris-structure interaction. The maximum instantaneous debris impact force, F_{di}, is given by

$$F_{di} = C_{or}\, V\, (\, k\, M)^{1/2} \qquad \text{(Eq. 3-14)}$$

where:

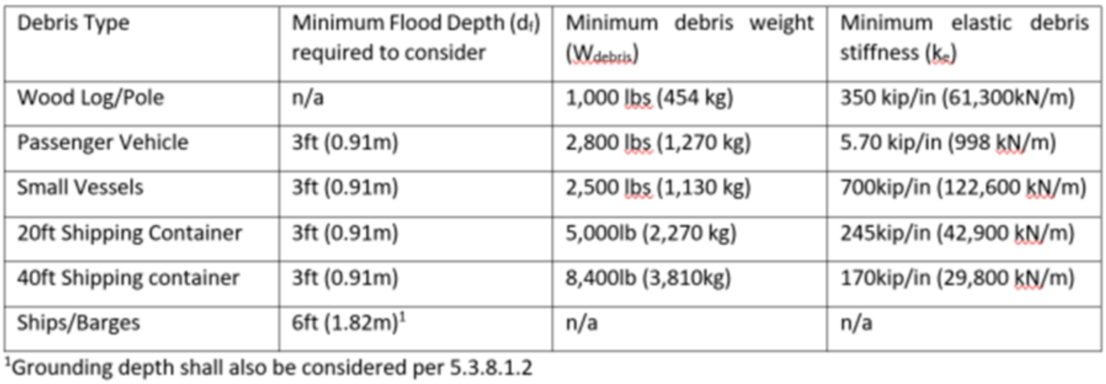
C_{or} = orientation coefficient, equal to 0.65 for logs and poles;
V = maximum flow velocity at the site occurring at depths sufficient to float debris;
k = effective stiffness of the impacting debris or of the impacted structural element(s) deformed by the impact, whichever is less; and
M = debris mass, also given as W/g

It is assumed that logs and poles strike longitudinally for calculation of debris stiffness, k, so that the axial stiffness of the log is given by $k = EA/L$, in which E is the longitudinal modulus of elasticity of the log, A is its cross-sectional area, and L is its length. A minimum mass of 454 kg (weight of 4.5 kN or 1000 lb) and a minimum log stiffness of 61,300 kN/m (350 kip/in) is specified. Values of debris mass and stiffness for other types of debris considered in tsunami loading are given in Table 3-4.

The debris impact load is applied as a short-duration impulse with duration $\Delta t = 2\, M\, V / F_{di}$. For an equivalent elastic static analysis, the impact force is then multiplied by the dynamic response factor R_{max} specified in Table 3-3. For application to floods other than tsunamis, other coefficients based on depth and blockage should also be included.

Debris impact loads are included for flood depths of 0.9 m (3 ft) or greater. For flow depths greater than 3.66 m (12 ft) as may occur in tsunamis, impact loads of large vessels are also considered but only for the highest risk category buildings.

Table 3-4 Values of debris mass and stiffness given by ASCE 7 (2016)



Debris Type	Minimum Flood Depth (d_f) required to consider	Minimum debris weight (W_{debris})	Minimum elastic debris stiffness (k_e)
Wood Log/Pole	n/a	1,000 lbs (454 kg)	350 kip/in (61,300kN/m)
Passenger Vehicle	3ft (0.91m)	2,800 lbs (1,270 kg)	5.70 kip/in (998 kN/m)
Small Vessels	3ft (0.91m)	2,500 lbs (1,130 kg)	700kip/in (122,600 kN/m)
20ft Shipping Container	3ft (0.91m)	5,000lb (2,270 kg)	245kip/in (42,900 kN/m)
40ft Shipping container	3ft (0.91m)	8,400lb (3,810kg)	170kip/in (29,800 kN/m)
Ships/Barges	6ft (1.82m)[1]	n/a	n/a

[1] Grounding depth shall also be considered per 5.3.8.1.2

3.8.3 Debris Loads Due to Floating Ice

In Canada, floating ice would form a common type of floating debris that should be considered during flood events occurring during spring ice break-up periods. Of all debris types, ice is the most thoroughly investigated, primarily because it effects all bridge piers, and thus highway and bridge design, in cold regions. Despite this, much less is known about the interaction of ice with residential and commercial structures, and no general design guidance could be located for ice loading on buildings during floods.

In general, ice loads in rivers may include the following:

- Static Ice Forces – Those forces associated with intact ice sheets subject to thermal expansion and contraction, or subject to steady pressure from winds or currents.
- Uplift or Drawdown Forces – Those forces associated with adhesion of ice to piles, acting mainly in a vertical direction due to changes in water level.
- Ice Mass Forces – Those force associated with the accumulation of ice rubble, either in riverine ice jams or in ice pile-up against a structure or shoreline.
- Ice Impact Forces – Those forces associated with floating ice sheets and floes driven into a structure by currents or wind (or waves). This is normally the critical mode of action in rivers and may be in some lake (and oceanic) situations.

Methods of quantifying these loads may be found in the CSA International (2010) *Canadian Highway Bridge Design Code,* AASHTO (2012) *Bridge Design Specifications* and in the USACE (1999) Manual 1110-2-1612 *Ice Engineering*. For the present report, it is assumed that the forces of interest during a flood would be primarily related to the action of ice as floating debris, either through ice accumulation or ice floe impacts on a structure.

The latter situation, where ice floes are propelled into a structure, is similar to other flood-borne debris impacts. In fact, such ice "debris" may be the most significant form of debris to impact a structure during its design life. Field observation on bridge piers show that ice floes can often weigh 4.5 kN to 45 kN (1,000 lbs to 10,000 lbs) or more. Typical dimensions of individual ice floes in rivers are commonly observed to have thickness of 0.33 to 0.66 m (1 to 2 feet), with typical lateral dimensions ranging from about 1 to 5 m (3 to 15 ft). However, these dimensions may be exceeded, particularly in larger river systems. Approving Authorities would likely need to establish design dimensions for ice floes based on local conditions.

Engineering design guidance for ice impact loads may be found in either empirical formulations, based largely on field tests, or in analytical formulations, based on the initial momentum or kinetic energy of a moving ice floe. While numerous approaches have been proposed in the literature, these can be summarized in three main approaches:

- Ice Crushing - Ice loads are based on the contact pressure at which ice fails locally in crushing. This method is most appropriate for large ice sheets that completely surround the structure, and which are driven by a very large driving force (wind or current). These methods are usually conservative, and they have been adopted in numerous bridge design codes. The CSA (2010) and AASHTO (2012) give the design load as

  $$F_c = C_a P_c t w \qquad \text{(Eq. 3-15)}$$

  where:

  t = ice floe thickness in m (ft)
  w = width of structural element in m (ft)
  C_a = interaction coefficient = $(1+5t/w)^{1/2}$
  P_c = crushing strength of ice, from about 400 kPa to 1500 kPa depending on properties of ice during breakup.

- Ice Impact – Ice loads are determined from the incident momentum of the moving ice sheet. These methods assume that the impact force is lower than that required for local crushing of ice, so loads would be bounded by Equation (3-15). This method is most appropriate when smaller individual ice floes strike a structure and would be predicted using methods for floating debris impacts covered in earlier sections.

- Ice Accumulation – Ice loads are based on the accumulation of ice against a structure under the action of a steady current. This approach assumes that the driving force is too small to cause local ice failure, so loads would be bounded by Equation (3-15). Numerical predictions could be made with an expression similar to Equation (3-12) but with the projected area of the ice jam against the structure and with a suitable drag coefficient.

3.9 Wave Loads

As discussed in Chapter 2, wave conditions along the coasts or along the shorelines of large lakes may lead to additional flood loads on buildings during high water levels. The possibility of large wave loads leads to requirements in many flood codes to elevate the habitable floors of coastal buildings well above the anticipated wave crest elevation or wave runup elevation in the design flood event. In addition, new development in these regions should avoid wave effects through use of horizontal setbacks to avoid the zone of wave uprush.

But because wave effects are not included in many flood hazard maps in Canada, it is possible that new development will occur in areas of potential wave impact. In addition, future sea level rise may allow waves to reach higher elevations and farther inland, thus impacting areas and buildings that have historically not been subject to wave action. Any new development or retrofit of existing buildings in coastal zones should consider possible wave impacts in order to limit future flood damages.

Wave loads are more complicated than other flood loads as they are dependent on multiple parameters defining the wave conditions, most fundamentally on wave height, H, wave period, T, local water depth, d, and bottom slope, $\tan\theta$. The action of waves on top of tide and storm surge effects is illustrated in Figure 3-11.

Waves are generated over a large water body due to over-water winds and then propagate into flooded coastal regions. In some areas, ship generated waves may also occur. As waves move into shallower water, they undergo various transformations, including shoaling, refraction, diffraction, and breaking. A complete description of wave generation and modification is given by the USACE (2002), Dean and Dalrymple (1991), Goda (1985), or Kamphuis (2010). These processes are usually estimated in a site-specific coastal engineering study.

For application in building codes, ASCE 7 (2016) permits use of simplified approximations of wave conditions in many circumstances. Among the most useful of the simplified approaches is that waves of interest to building design in flooded areas, as opposed to wave conditions needed for pier and wharf design, are often in relatively shallow water and are often breaking. Breaking waves are generally depth-limited so that wave heights are restricted and cannot be larger than some maximum value based on the local water depth.

Design methods reviewed in this chapter will assume shallow-water depth-limited breaking wave conditions. Forces computed under these assumptions should be conservative, as design will be based on the largest possible waves that can exist in a given flooded water depth.

Designers should be aware, however, that design wave heights at a site can be non-breaking and less than depth-limited values so that forces may be lower than those given here. For example, waves may be less than the depth-limited height when the wind speed, wind duration, or fetch are insufficient to generate waves large enough to break in a given water depth, or when nearby objects dissipate wave energy and reduce wave heights prior to reaching a building. In such cases, more advanced coastal engineering studies would be required to refine wave conditions at a design site to reduce the conservatism in load estimates.

An additional consideration for designers is whether to apply wave loads in a static structural analysis or in a dynamic structural analysis. All wave loads occur intermittently in random waves with periodicity related to the wave period. Typical wave periods of interest may range from 2 or 3 seconds, up to 10 seconds or more. But some wave loads also can be impulsive and occur on very short time scales in the range of 0.01 to 0.1 seconds. Most design codes allow designers to apply loads related to typical wave periods as static loads. The shorter impulsive loads are sometimes applied statically, but due to their short time duration they are often more appropriately analyzed in a dynamic structural analysis.

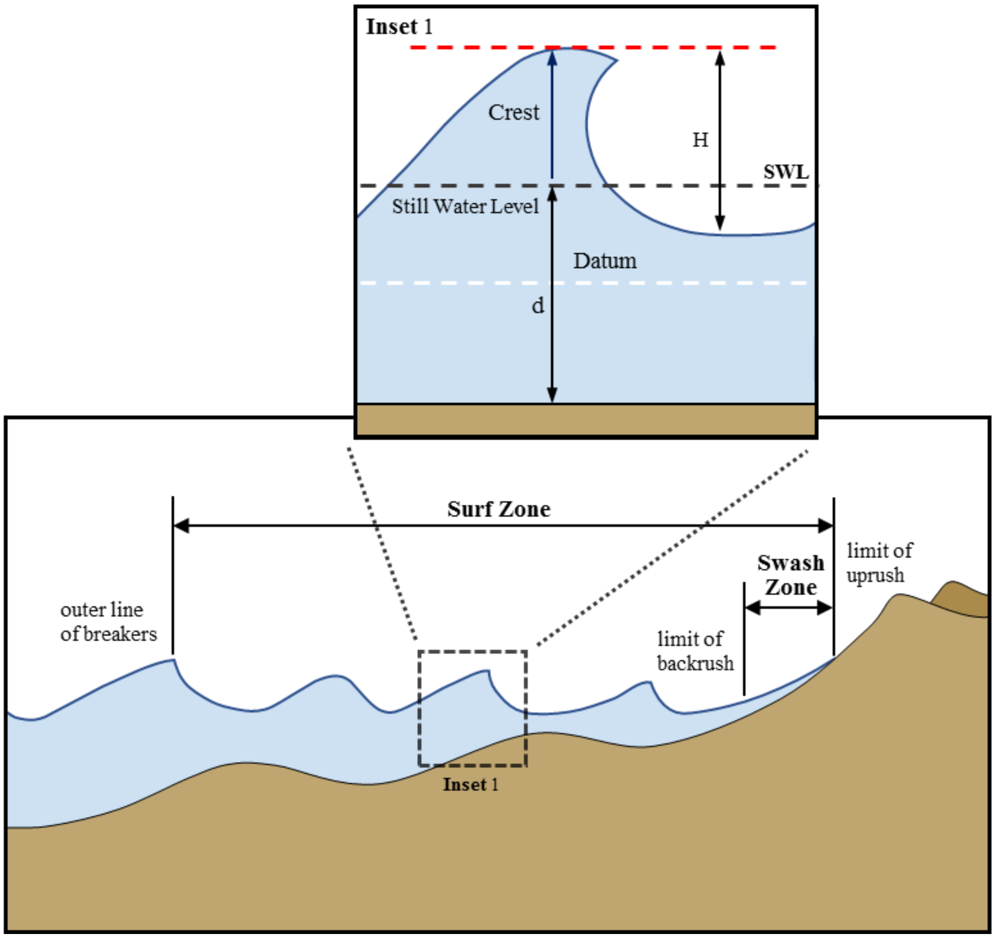


Figure 3-11 Wave properties

3.9.1 Wave Parameters and Definitions

Waves are random in nature and a structure exposed to waves will encounter a range of wave heights, H, and wave periods, T, during a design storm event. It is common to define the sea state using statistical properties of waves, and two parameters are commonly used: (1) the significant wave height, H_s, in m (ft) and (2) the peak wave period T_p in sec (s) (USACE, 2002).

The significant wave height, denoted here using the general notation H_s, is usually defined as four times the standard deviation of the random sea surface fluctuations (often given by the notation H_{mo}) but also corresponds to the average of the highest one-third of the waves in a random sea (often given by the notation $H_{1/3}$). The peak wave period, denoted here at T_p, is the wave period corresponding to the most energetic waves in the random sea.

The two parameters, H_s and T_p, are commonly obtained from wave measurement programs, from wind-wave forecasting methods in USACE (2002), or from advanced numerical wave models. Wave conditions vary with storm climatology, the degree of exposure of a coastal region, and on wave dissipation mechanisms, so no general relationships can be given for how H_s and T_p vary around the coastal regions and lakes in Canada. A site-specific coastal engineering analysis may be required to estimate these parameters. But any water body with an exposed fetch of more than 1 to 2 kilometres can generate damaging wave conditions during high wind conditions.

3.9.1.1 Significant Wave Height

For application in flooded coastal regions, where the over-ground flooded depth, d_f, is expected to be small, and waves likely started breaking farther offshore, random wave heights are likely limited by wave breaking. USACE (2002) guidance indicates that in shallow water with active wave breaking, an approximate upper-bound on the significant wave height limited by breaking is given by

$$H_{sb} = 0.6\, d_f \qquad \text{(Eq. 3-16)}$$

where:

H_{sb} = significant wave height with depth limited wave breaking, in m (ft)
d_f = local still water depth in coastal location, in m (ft)

Kamphuis (2000) has developed a refined wave breaking model that includes beach slope, based primarily on physical model tests performed at Queens University. In shallow water, the Kamphuis method gives the significant wave height with breaking as a function of the local nearshore ground slope as

$$H_{sb} = 0.56\, e^{3.5\, tan\theta}\ d_f \qquad \text{(Eq. 3-17)}$$

where:

$tan\ \theta$ = local beach slope (rise over run)

For slopes of 1:100, 1:50, 1:20, 1:10, and 1:7.5, the Kamphuis method gives H_{sb} as 0.58, 0.60, 0.67, 0.79, and 0.89 times the depth respectively. This is consistent with Equation 3-16 for milder slopes but gives larger wave conditions for steeper slopes.

3.9.1.2 Maximum or Design Wave Height

While the significant wave height defines the overall characteristics of the sea state, it is a statistical value and does not indicate the largest individual wave that may occur in the sea state. For structural design in shallow flooded regions, the maximum individual wave heights correspond to the largest breaking wave height that can occur in any water depth, H_b. In shallow water, this can be approximated using the traditional form of the Miche breaking criterion. The USACE (2002) gives this limiting wave height as

$$H_b = 0.9\, d_f \qquad \text{(Eq. 3-18)}$$

where:

H_b = maximum breaking wave height in m (ft)

More detailed guidance on the maximum individual breaking wave in shallow water includes the effects of beach slope, as well as the effects of wave period. Several methods have been proposed in the coastal engineering literature for estimating H_b for specific combinations of wave and beach conditions, see Goda (1985), USACE (2002), or Kamphuis (2010). For flood design codes, where over-ground water depth is expected to be small, many of these exist in simplified form as a function of just the local water depth. In a more general form, the depth-limited breaking wave height in shallow water can be given by

$$H_b = \kappa\, d_f \qquad \text{(Eq. 3-19)}$$

where:

κ = breaker height to depth ratio, or breaker index

For very flat ground slopes, FEMA guidance, FEMA (2011), as well as the ASCE 7 (2016) standard, base the maximum wave height on Solitary wave theory (theoretically valid only for flat to very mild slopes) with $\kappa = 0.78$. For general use, the value $\kappa = 0.9$ from Equation 3-18 is more appropriate. But it is known that the breaker index can increase as slope increases. For waves over very steep slopes, it is known that the local ratio of wave height to water depth can be $\kappa = 1.2$ to 1.4, and in some cases can be higher due to flow convergence or other local effects. Use of the shallow water significant wave height (Kamphuis, 2010), in Equation 3-17, along with a 1.5 factor to relate to maximum breaking waves to significant wave height, gives, respectively $\kappa = 0.87$, 0.90, 1.00, 1.19, and 1.34 for ground slopes of 1:100, 1:50, 1:20, 1:10, and 1:7.5.

3.9.1.3 Wave Period

In the wave transformation processes, wave period often remains the same as waves move from deeper water into the nearshore region. In general, there are no simple methods available to estimate wave period except for conducting a wind-wave analysis. Unlike wave height, wave period does not reach any limiting value in shallow water. Because wave period increases with the exposed fetch of over water winds (or with the scale of the water body), and with the wind speed, wave period is highly dependent on the degree of exposure of a site. The wave period is not mapped in any of the Provincial flood hazard maps, however.

Fortunately, the assumption of shallow water breaking wave conditions means that wave period is not always needed, as it is not used explicitly in many of the formulas for calculating flood forces. The exception is when considering the forces due to wave runup. Wave period would be required for estimating the horizontal and vertical extent of wave runup. Wave runup is generally proportional to wave period, so that long period waves can reach higher elevations farther inland than short period waves.

3.9.1.4 Wave Crest Elevations or Wave Amplitudes

A fundamental way of reducing flood loads in coastal areas is to require that habitable spaces be elevated above the flood waters. FEMA guidance, FEMA (2011), requires the bottom of the first occupied floor to be above an elevation given by the sum of the design still water level plus the wave crest amplitude. In FEMA terminology, this establishes the Base Flood Elevation (BFE) which, by its definition, includes wave effects. Figure 3-12 illustrates the relationships between the wave height and the wave crest.

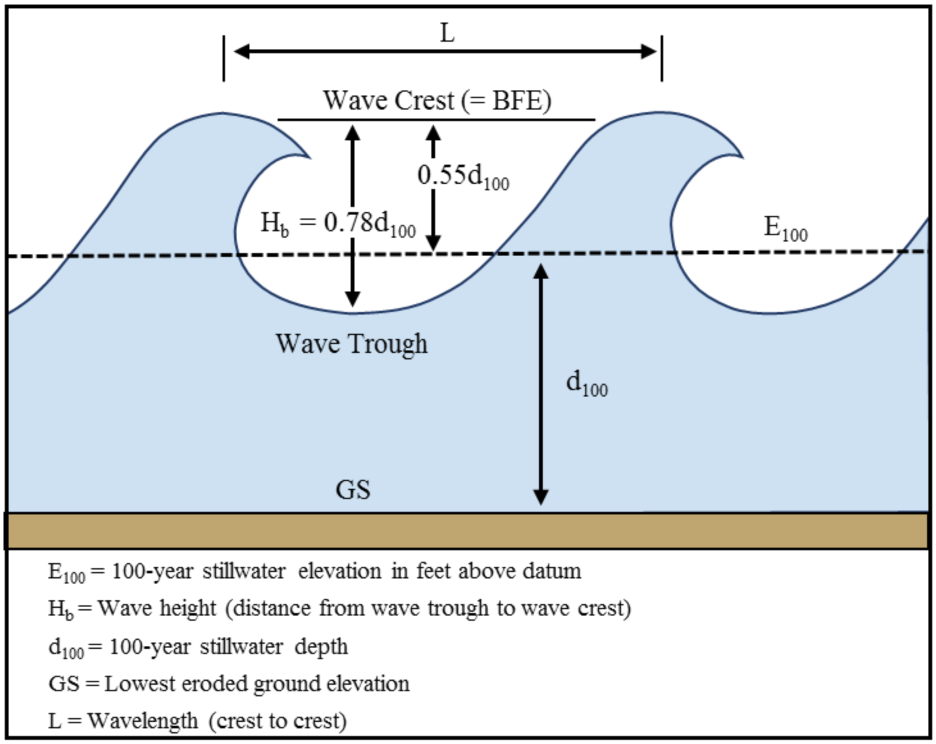


Figure 3-12 Relationship between wave height and wave crest elevation

Wave crest elevations are defined as the maximum amplitude or elevation that the crest of the wave reaches above the still water level. FEMA (2011) and ASCE 7 (2016) adopt a simplified expression for wave crest elevation for shallow water breaking waves as a percentage of the breaking wave height as

$$\eta_c = 0.7\, H_b \qquad \text{(Eq. 3-20)}$$

where:

η_c = wave crest elevation, in m (ft), above the still water flood level

Methods in the USACE (2002) show that for breaking waves in very shallow water, the crest may be more than 70% of the wave height, with values of 80 to 90% or more being possible.

The Port of San Francisco Building Code specifies the wave crest elevation as

$$\eta_c = 0.5\,(1 + 0.78\,H_b/d_f)\,H_b \qquad \text{(Eq. 3-21)}$$

If the breaking wave height is depth limited, then using $H_b = 0.9d_f$ from Equation 3-18 would give the wave crest elevation as $\eta_c = 0.85\,H_b$.

More advanced methods in USACE (2002) give the 2% exceedance wave crest elevation as a function of the local depth, d, period, T_p, and significant wave height, H_s. Results cannot be given in simplified form but limiting results in very shallow water show that the wave crest elevation is nearly equal to the water depth $\eta_c \sim d_f$. Thus, in a 1 m depth, the 2% wave crest could reach as high as 1 m above the still water level or 2 m above the local ground elevation.

3.9.1.5 Wave Runup

As waves reach a shoreline, a last transformation of wave energy causes waves to rush up the beach slope, or a structure slope, to an elevation above the still water level and to a location landward of the still water shoreline. Wave runup, R, is then defined as the vertical elevation reached by water above the still water elevation as the wave rushes up the slope, as shown in Figure 3-13.

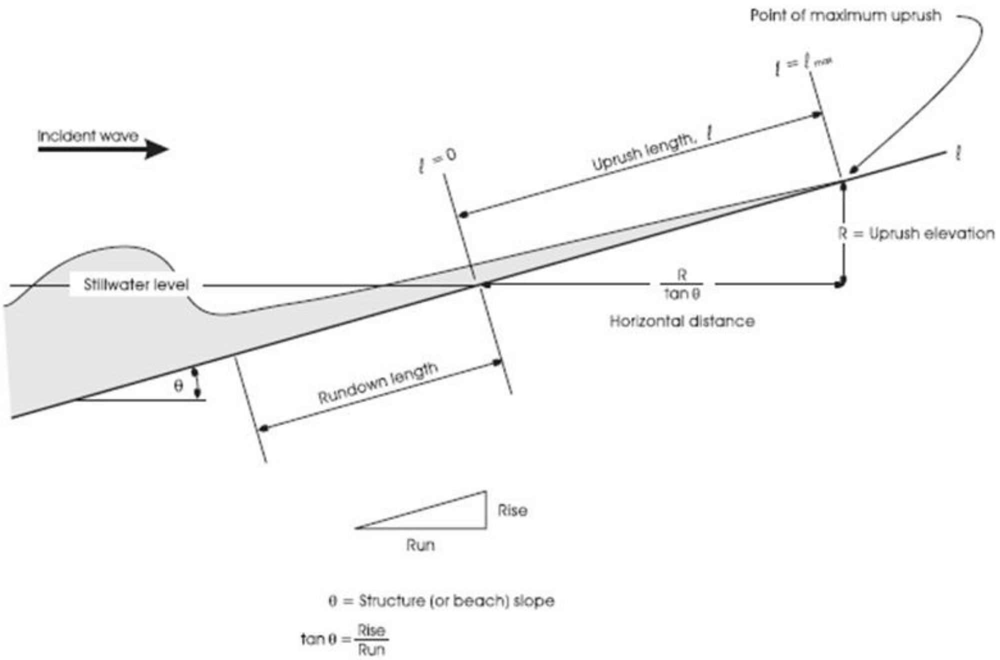


Figure 3-13 Wave runup (source: Ontario Ministry of Natural Resources, 2001)

As shown in Figure 3-13, the horizontal extent of wave runup is related to the vertical runup as

$$X_R = R/tan\theta \qquad \text{(Eq. 3-22)}$$

where:

R = vertical runup above SWL
X_R = horizontal runup excursion landward from SWL shoreline
$\tan\theta$ = average ground slope of foreshore, for slope angle θ

It is noted that the Ontario practice of mapping the uprush zone as a 15 m horizontal distance from the shoreline would imply vertical runup values of 0.5 m, 1 m 1.5 m, and 2 m for slopes of 1:30, 1:15, 1:10, and 1:7.5, respectively.

Wave runup is a complicated phenomenon that cannot be easily simplified. Runup is usually predicted using empirical expressions derived from either small-scale laboratory tests from field observations, or from numerical models that incorporate these empirical expressions of runup into a general model of surf zone wave conditions. Runup methods are reviewed by TAW (1999), USACE (2002), CIRIA (2007), FEMA (2018), EurOtop (2018). A review with specific reference to Great Lakes conditions is also given by the Ontario Ministry of Natural Resources (2001).

For most design applications, the vertical runup is defined statistically as a 2% exceedance value, $R_{2\%}$. Because the upper limit of wave uprush is a thin wedge of water, and because of high variability in random wave runup, the maximum value of runup is generally not used and a 2% level of exceedance has been widely accepted as a close measure of the largest runup events in a random sea state. Other statistical definitions are in use, for example the median or mean runup is used in British Columbia. In most coastal design situations, this would be considered too low as about 50% of wave runup events would exceed the mapped value.

3.9.1.6 Runup on Beaches

For natural sand beaches, that typically have mild slopes, the runup method of Stockdon et al. (2006) is one method recommended by FEMA (2018). Unlike most other runup prediction methods that are based on small scale laboratory tests, the Stockdon et al. (2006) method is based on field observations from a wide range of beaches from the North Sea, the US Atlantic coast, and the US Pacific coast. For dissipative beaches, Stockdon et al. (2006) give a runup expression as a linear function of the Irribarren Number as

$$R_{2\%}/H_S = 0.73\ \xi \qquad \text{(Eq. 3-23)}$$

where:

$R_{2\%}$ = vertical runup above SWL at 2% exceedance level
H_s = significant wave height
$\xi = \tan\theta / (Hs/Lo)^{1/2}$ = Irribarren Number or Surf Similarity Parameter
L_o = deep water wavelength = $gT_p^2/2\pi$.

This was developed for beach slopes between 1:100 and 1:9. Because wave conditions vary widely across the beach profile due to shoaling, refraction, and breaking, the significant wave height used in the Irribarren number is adopted from deep water conditions offshore and not from local conditions within the surf zone. Although Equation 3-23 looks straightforward, the runup depends on wave period which is not mapped by any Province or Approving Authority. As a result, Equation 3-23 probably cannot be applied without a site specific or regional coastal engineering investigation.

3.9.1.7 Runup on Coastal Structures or Steep Bluffs

For steep slopes with low porosity, including natural bluffs and coastal structures, the runup method outlined in TAW (1999) or EurOtop (2018) is suggested by FEMA (2018). This applies to steep slopes of about 1:8 to 1:1. The TAW method is based on Dutch research and design experience for application to sea dikes and is similar to the Upper Bound Limit of Wave Uprush given by the Ontario Ministry of Natural Resources (2001). The TAW method may be summarized (simplified from the original) as

$$R_{2\%}/H_S = 1.75\ \gamma_r\ \xi \qquad \text{for } \xi < 1.8 \qquad \text{(Eq. 3-24a)}$$

or

$$R_{2\%}/H_S = \gamma_r\ (4.3 - 1.6/\xi^{1/2}) \qquad \text{for } \xi > 1.8 \qquad \text{(Eq. 3-24b)}$$

where:

γ_r = reduction factor for ground roughness, structure shape, and angle of wave attack

The first expression in Equation 3-24a would normally apply but the second term gives an upper bound on runup for steeper slopes. The significant wave height used in this expression is the value at the base of the bluff or structure slope and may be influenced by depth-limited breaking. In addition, the wave period used in the Irribarren Number is the mean period, which is about 0.8 to 0.9 Tp. The correction factor, given here as γ_r, can account for many details of specific site conditions, most importantly the effect of roughness. For stone revetments, which are usually composed of two rows of primary armor stone overtop of finer bedding stone and an impermeable underlayer, the roughness coefficient is 0.50 to 0.55. For grass and concrete, the roughness coefficient is 1.0. Steep sand and gravel beaches would also have roughness values close to 1.0.

3.9.2 Wave Forces

Wave loads during floods result from water waves striking a structure after propagating over flooded ground, i.e. over an area of normally dry ground that has been inundated during a flood. As noted, this often leads to the use of shallow water assumptions for breaking waves when describing wave motions, e.g. USACE (2002) or the SPM (1984).

The results included here are not intended for structures that are built in the water and subjected to wave action under normal circumstances. Therefore, the results are not intended for docks, piers, wharves, or similar in-water structures. In rare circumstances where flood depths may be very deep and wave motions do not satisfy the shallow water limits with breaking waves, the methods outlined here will be overly conservative and overestimate wave forces. In these cases, practitioners would likely apply an appropriate wave theory, numerical wave model, or laboratory tests to justify a reduction in wave loads.

The design and construction of buildings and other structures subject to wave loads should account for the following loads, following guidance from FEMA, ASCE 7 (2016) or USACE (2002):

- Wave actions on vertical piles and columns
- Wave actions on vertical wall panels

- Wave uplift on elevated floors
- Wave actions on horizontal beams
- Wave runup or uprush on walls

3.9.2.1 Wave Action on Vertical Piles and Columns

For structures with habitable spaces elevated above the flood waters, a fundamental form of wave loading consists of wave action on vertical piles or foundation columns. This condition applies even when the main portion of the building is elevated above the wave crest elevation. Waves may be breaking or non-breaking, depending on the ratio of wave height to water depth. Guidance summarized below is applicable to breaking wave conditions and would over-estimate loads for non-breaking waves.

Wave loading on piles and columns follow from basic equations for wave forces on small-diameter cylinders as expressed in the Morison equation, e.g. USACE (2002). In general, the Morison equation expresses wave loads as the sum of drag forces, proportional to the wave-induced fluid velocity squared, and inertia forces, proportional to the fluid acceleration. In shallow water conditions, which are applicable to most flood loading conditions where waves may reach a building across flooded ground, the drag force tends to predominate (Dean and Dalrymple, 1991). In addition, wave-induced velocities become a relatively simple function of wave height and water depth.

3.9.2.1.1 Breaking Wave Loads on Piles and Columns

For breaking waves subject to shallow water conditions, guidance from FEMA and ASCE 7 (2016) suggest the maximum drag force due to breaking waves can be estimated as

$$F_b = \text{½}\, C_{Db}\, \rho g\, D\, H_b^2 \qquad \text{(Eq. 3-25)}$$

where:

F_b = breaking wave force, in N (lbs)
C_{Db} = breaking wave drag coefficient (1.75 for round piles, 2.25 for square piles)
D = pile diameter for round pile, or 1.4 times width of pile for square, in m (ft)
H_b = breaking wave height, in m (ft)

The expression above originates in the USACE (1984) based on application of advanced wave theories for shallow water breaking waves. The approximate velocity distribution from shallow water waves is depicted in Figure 3-14. Velocities are highest in the wave crest region and diminish to the ground level but are often nearly uniform over depth in shallow water. Equation 3-25 reflects the depth-integrated result of velocity-based drag forces over the immersed length of the pile from the ground level to the wave crest.

The point of application for the force vector would be at the centroid of the velocity distribution. But, following guidance in FEMA and ASCE, the breaking wave force can be applied at the still water level as a reasonable approximation. The overturning moment about the ground level is therefore found as $M_b = F_b * d_f$.

The values of breaking wave drag coefficient, C_{Db}, account for action of broken waves on a pile or column. Suggested values in breaking waves are C_{Db} =1.75 for round piles and columns, and 2.25 for square piles or columns. This does not include the impulsive action when waves curl and break directly on a pile.

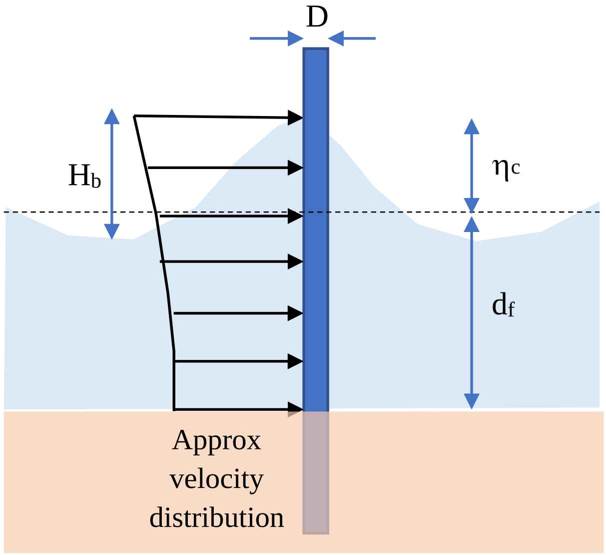


Figure 3-14 Breaking waves forces on piles

3.9.2.2 Truncated Piles and Columns

No guidance is available from either the USACE (1984, 2002) or FEMA (2011) for conditions where the pile height above ground is less than the height of the wave crest. Such conditions may occur if a first floor of a building is elevated on piles, but the wave crest elevation is above the bottom of the floor.

Because shallow water waves have horizontal fluid velocities that are nearly uniform over depth, an approximation of forces on a truncated column may be given by a simple scaling as illustrated in Figure 3-15

$$F_{bt} = ½\, C_{Db}\, \rho g\, D\, H_b^2\ \left(h_f/(d_f + \eta_c)\right) \qquad \text{(Eq. 3-26)}$$

where:

F_{bt} = breaking wave force on truncated column
h_f = height of column above ground (presumed than $d_f + \eta_c$)

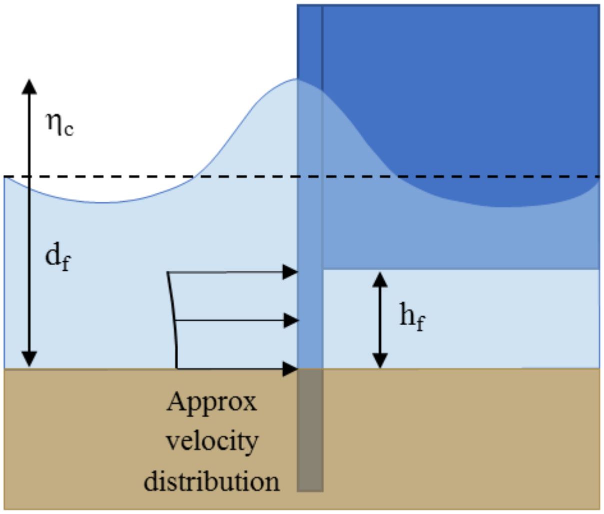


Figure 3-15 Wave forces on truncated piles and column foundations

3.9.2.3 Impulsive Slamming Load on Piles and Columns

The ISO Standard 21650 identifies an additional dynamic or impulsive wave slamming force that can occur if waves break as plunging or curling breakers directly on piles and columns. This load appears to be additive to other breaking wave forces and is concentrated in the wave crest region. This is illustrated in Figure 3-16. The ISO Standard 21650 suggests that this may be given by

$$F_S = ½ \, C_S \; \rho g \, D \, d_f \; \lambda \, \eta_C \qquad \text{(Eq. 3-27)}$$

where:

F_s= slamming force due to direct breaking wave impact, in N (lbs)
C_s = slamming coefficient = 3.1
λ = curling factor, as low as 0.1 for spilling breaking waves of low intensity and as high as 0.9 for plunging wave of high intensity.
η_C = wave crest amplitude

The occurrence of a direct wave breaking on a pile would be rare as the wave would have to exhibit a curl or plunging shape exactly at the location of the pile or column. For this reason, it may be applied to one column in a structure but likely would not be applied simultaneously to all columns. The load, while potentially large, would also occur as an impulse with a very brief duration. Given these factors, the wave slamming load might be considered only for critical structures as an added safety margin.

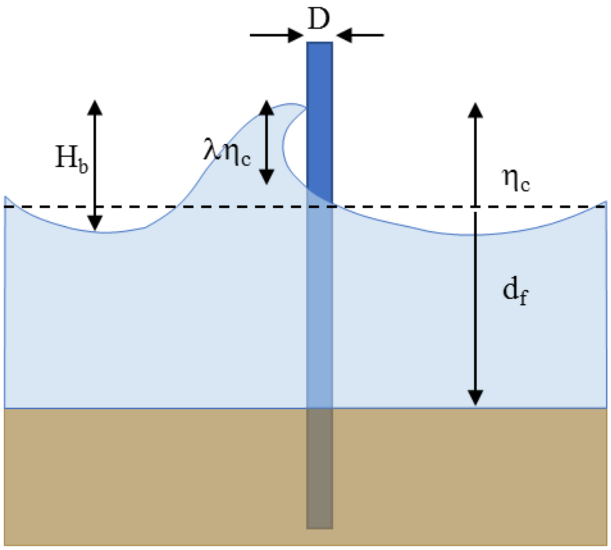


Figure 3-16 Dynamic wave slamming on a pile, adapted from ISO Standard 21650 (2016)

3.9.2.4 Wave Action on Vertical Walls

For structures with vertical wall panels subjected to waves, wave reflection occurs with superposition of incident and reflected waves at the wall. At the wall, this produces both elevated wave crests (higher than η_c) as well as elevated wave pressures compared to the incident waves alone.

Waves acting on vertical walls may be breaking or non-breaking with (as with pile foundations) higher loads associated with breaking waves. FEMA (2011) guidance indicates that post-storm damage inspections show that breaking wave loads have destroyed virtually all types of wood-frame walls and unreinforced masonry walls below the wave crest elevation. Only highly engineered structural elements were capable of withstanding breaking wave loads. Damaging wave pressures and loads can be generated by waves much lower than the 0.9 m (3 ft) wave currently used by FEMA to distinguish Coastal High Hazard Zones or VE Zones. For this reason, it is usually preferable in design to elevate the wall sections above the wave crests, rather than to design for large breaking wave loads.

3.9.2.4.1 Breaking Wave Loads on Vertical Walls

Breaking wave loads on full depth vertical walls are calculated by one of two methods. ASCE 7 (2016) adopts methods outlined by FEMA (2011), based on a USACE study of breaking wave loads by Walton et al. (1989). Future revisions to ASCE 7 will likely switch to the method of Goda (1985) which has gained wide acceptance internationally for estimating wave interaction with the vertical face of concrete caisson breakwaters. A good review of the Goda method is included in the Port of San Francisco Building Code (2013).

Loads on Full-Depth Walls from FEMA and ASCE 7-16

The pressure distribution for breaking wave interaction with a full-depth vertical wall, as adopted by FEMA (2011) and ASCE 7 (2016), is shown in Figure 3-17. A key feature is that with wave reflection, the crest of the wave at the wall (sum of incident plus reflected waves) is at an elevation of 1.2 d_f above the still water level. The resulting pressure distribution reaches a maximum at the still water line and then diminishes with reference values given by

$$P_1 = (1.2 + C_p)\rho \; g \, d_f \qquad \text{(Eq. 3-28a)}$$

$$P_2 = 1.2 \, \rho \; g \, d_f \qquad \text{(Eq. 3-28b)}$$

where:

P_1 = pressure in N/m^2 (lb/ft^2) at the design still water level
P_2 = pressure in N/m^2 (lb/ft^2) at the ground level
C_p = dynamic pressure coefficient ($1.6 < C_p < 3.5$, per Table 3-5)

The force per unit width of this pressure distribution is given by the area under the pressure distribution as

$$f_{wb} = 1.1 \, C_p \, \rho g \, d_f^2 + 1.9 \, \rho \, g \, d_f^2 \qquad \text{(Eq. 3-29)}$$

where:

f_{wb} = net breaking wave force per unit width of structure, in N/m (or lb/ft)

The total load on a wall of width w is given by

$$F_{wb} = w \, f_{wb} \qquad \text{(Eq. 3-30)}$$

The centre of pressure is just below the still water level for all values of C_p. For simplicity, the overturning moment may be computed by applying the load at the still water level.

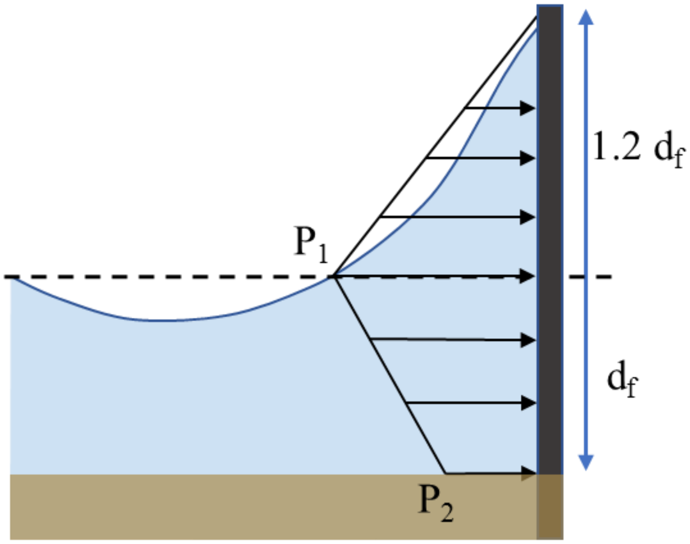
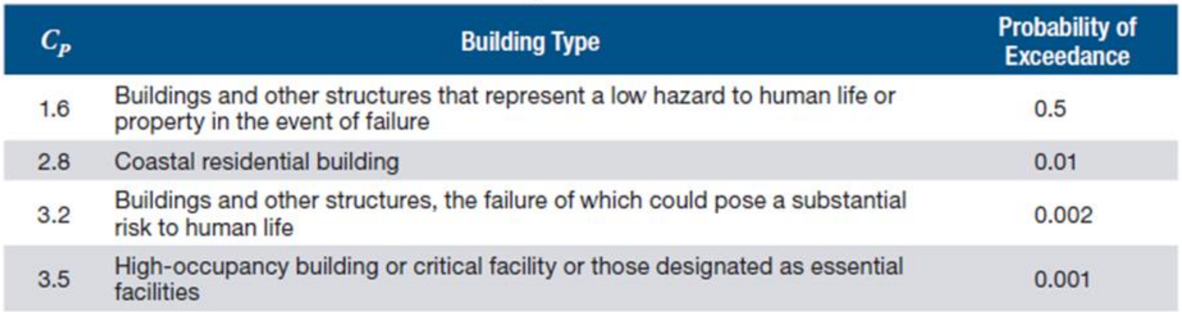


Figure 3-17 Breaking wave loads on vertical walls, adapted from FEMA

The dynamic pressure coefficient C_p has values that reflect the random nature of breaking wave impacts. In FEMA guidance (and ASCE 7), these are adopted based on the building type or risk category, as shown in Table 3-5. But as discussed by Walton et al. (1989), the recommended values of the coefficient were originally given as a function of probability of exceedance for random wave conditions and were not linked to building type. The probabilities given by Walton et al. (1989) are associated with a distribution of breaking wave pressures measured during laboratory wave tank tests in random waves. The lowest value of C_p is based on the mean of laboratory test results. For a 1% exceedance level (1 wave out of 100 in the random sea), $C_p = 2.8$. For a 0.1% probability (1 wave in 1000 waves), C_p is as high as 3.5. Not shown here, a 2% exceedance value would correspond to C_p ~2.56. As a result, use of a wave impact coefficient of $C_p = 2.56$ would be largely consistent with other design methods that utilize a 2% exceedance probability.

Table 3-5 Breaking wave impact coefficients (source: FEMA, 2011)



C_p	Building Type	Probability of Exceedance
1.6	Buildings and other structures that represent a low hazard to human life or property in the event of failure	0.5
2.8	Coastal residential building	0.01
3.2	Buildings and other structures, the failure of which could pose a substantial risk to human life	0.002
3.5	High-occupancy building or critical facility or those designated as essential facilities	0.001

As noted by FEMA, the breaking wave forces in the above equations are much higher than the typical wind forces that act on a coastal building, even wind pressures that occur during a hurricane or typhoon, and they explain some of the dramatic flood damages caused by waves. However, the duration of the wave pressures and loads is brief; peak pressures probably occur within 0.1 to 0.3 second as the wave breaks against the wall. This may destroy many types of exterior cladding but may not lead to much loading on building structural frames. Structural designers may analyze these loads using a dynamic structural analysis and not simply apply the loads statically to the structure.

Breaking wave pressures and loads from Equations 3-28 to 3-30 would be added to any other flood-induced pressures or loads. In ASCE 7 (2016) and various FEMA documents, the pressures and forces are sometimes shown with normal hydrostatic pressures included. This has led to considerable confusion. To eliminate confusion, the form given in Figure 3-17 and Equations 3-27 to 3-29 is only for the breaking wave effects and does not include the usual hydrostatic pressure or force.

Breaking wave forces given by the above equations are for waves that are normally incident to a vertical wall. Breaking wave loads on vertical walls reach a maximum when the waves are normally incident (direction of wave approach is perpendicular to the face of the wall; wave crests are parallel to the face of the wall). Obliquely incident waves produce less force on the wall.

Loads on Full-Depth Walls from Goda Method

An alternative method of computing breaking wave loads on vertical walls is to apply the widely used equations of Goda (1985) originally developed for estimating wave loads on the vertical front face of concrete caisson breakwaters. But these equations should apply to any vertical wall subjected to waves.

The Goda equations as originally proposed are in a general form applicable to any water depth and have a functional dependence on the wave length and wave period. For computing flood loads, the emphasis here is on shallow water conditions where many wave properties are simplified. As a result, equations given below are the shallow water forms of the Goda equations.

The pressure distribution due to a breaking wave on a vertical wall is calculated as shown in Figure 3-18, where pressure reaches maximum at the still water line and then remains constant to the ground level. Specific values of the wave crest elevation at the wall (with wave reflection) and maximum pressures are given as

$$\eta_* = 0.75\,(1+cos\beta)\;H \qquad \text{(Eq. 3-31a)}$$

$$P_1 \;=\; 0.5\,(1+cos\beta)\,(1.1+\alpha\,cos^2\beta)\,\rho\,g\,H \qquad \text{(Eq. 3-31b)}$$

where:

η_* = the elevation in m (ft) from the design stillwater level below which the wave pressure is assumed to act, or the elevation at which the wave pressure equals zero
H = design wave height in m (ft), taken as H_b or 1.5 H_s
P_1 = pressure in N/m^2 (lb/ft^2) at the design still water level
α= impulsive wave pressure coefficient, which can be taken as 0.8
β = wave direction relative to wall, 0^o for normal incidence

The wave-induced force per unit length on the vertical wall or structure in N/m (lb/ft) would then be calculated as the area under the pressure distribution as

$$f_{wb} = 0.5\, P_1\, \eta_* + P_1\, d_f \qquad \text{(Eq. 3-32)}$$

The total force over the wall length would be given by

$$F_{wb} = w\, f_{wb} \qquad \text{(Eq. 3-33)}$$

For overturning moments, the point of application of the wave load would correspond to the centre of pressure from Equation 3-32. For a full depth wall (not overtopped), a simple approximation is to apply the load at the still water level.

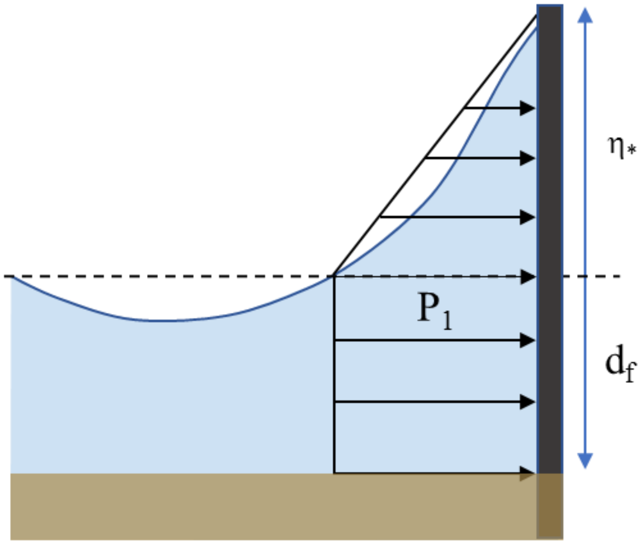


Figure 3-18 Breaking wave pressures on vertical walls, adapted from Goda (1985)

3.9.2.5 Wave Loads on Elevated Walls

The procedure in Equations 3-31 to 3-33 based on the Goda equations can be modified to estimate the force on an elevated wall, as would occur if the building was elevated on piles or foundation columns. This is based on a simple truncation of the general pressure distribution, as shown in Figure 3-19. The crest elevation η* and pressure at the still water level P_1 are assumed to remain the same. In reality, some pressure buildup would be relieved by the wave motions passing under the building.

The force per unit width is then obtained as

$$f_{wb} = 0.5\, P_1\, \eta_* + P_1\, (d_f - h_f) \qquad \text{(Eq. 3-34)}$$

where:

h_f = height of floor or wall element above ground level

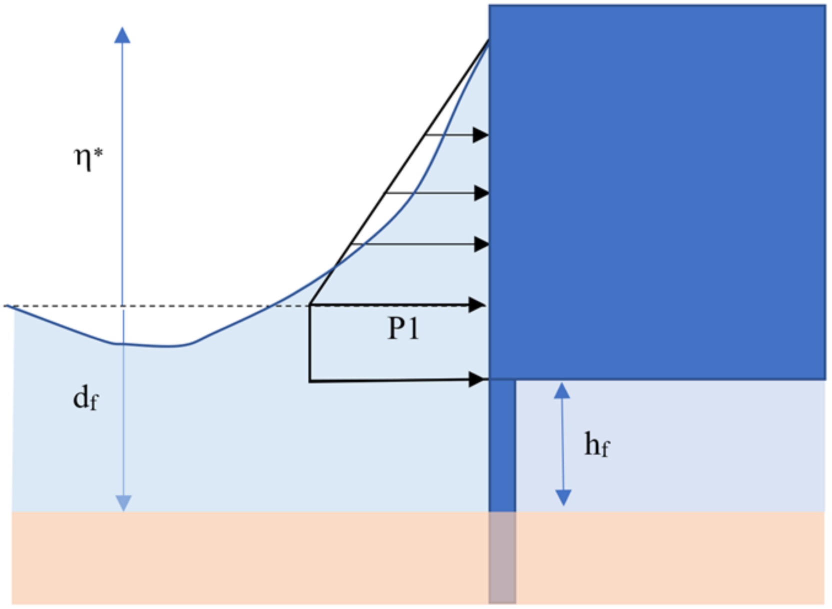


Figure 3-19 Loads on elevated wall panel, from method of Goda (1985)

3.9.2.6 Wave Slam

One related topic covered by FEMA guidance but not included in ASCE 7 is the subject of wave slam, depicted in Figure 3-20. FEMA (P-55, 2011) defines wave slam as the action of wave crests impulsively striking the elevated walls of a structure situated above the still water level. Ideally, structures would be elevated to avoid wave slam.

Wave slamming loads are impulsive in nature and, because they act on the lowest portions of the elevated structure, can result in damaged floor systems and large overturning moments on foundation piles. The wave slam can impart both lateral loads, as wave crests strike the vertical side walls of an elevated building, as well as uplift loads under the elected floor. FEMA guidance only addresses the lateral loads.

From FEMA (2011), the lateral wave slam can be calculated as

$$F_{ws} = w\, f_{ws} = w\,(0.5\, C_s\ \rho\, g\, d_f\ h) \qquad \text{(Eq. 3-35)}$$

where:

F_{ws} = lateral wave slam force, in N (lbs)
C_s = wave slam coefficient, with recommended value of 2.0
h = vertical height of wall impacted from bottom of floor joist or beam to wave crest

Wave slam should not be computed for buildings that are elevated on solid foundation walls whereas methods outlined previously would be used for wave interaction with full-depth walls. Wave slam should be considered for buildings that are elevated on piles or columns if most of the wave can go under the building and only the wave crest elevation exceeds the bottom of the floor.

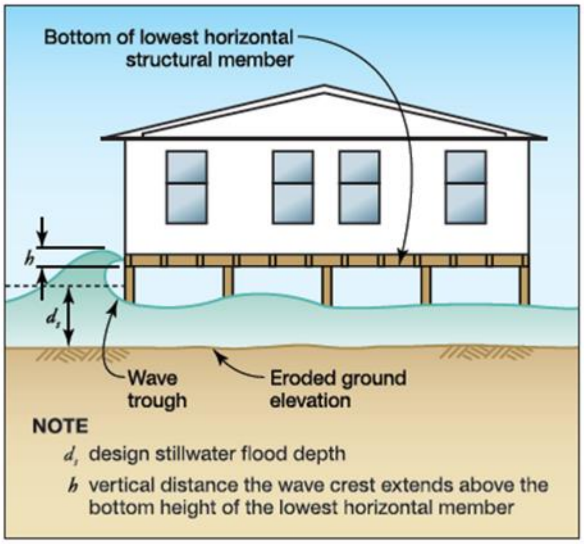


Figure 3-20 Wave slam due to wave crest hitting side wall of elevated building
(source: FEMA, 2011)

3.9.2.7 Wave Uplift Forces on Horizontal Floors

There is no well accepted method that exists for computing wave uplift forces under elevated floors of buildings. Neither FEMA nor ASCE 7-16 includes uplift loads on elevated floors. Considerable research has been done recently in the US on wave uplift under highway bridge decks. Most guidance can be found in documents from the US Federal Highway Administration (FHWA).

The FHWA (2008) reviews various methods of computing vertical wave uplift under bridge decks but ultimately recommends a simple approach. The overall load is composed of two terms: (1) a quasi-static hydrostatic uplift on the floor: and (2) a rapidly varying force that represents dynamic or impulsive effects. The forces may not occur at the same time as the impulsive term occurs due to the initial impact of waves with the underside of the floor, while the slowly varying term occurs later as the full effect of the wave crest is felt.

Based on the conditions shown in Figure 3-21, the slowly-varying or quasi-static force is given as

$$F_S = C_u \ \rho g \ A_f \ (\eta_C - Z) \qquad \text{(Eq. 3-36)}$$

while the impulsive force is given by

$$F_S = C_i \ \rho g \ A_f \ (\eta_C - Z) \qquad \text{(Eq. 3-37)}$$

where:

F_s = quasi-static or slowly varying uplift force, in N (lb)
F_i = rapidly varying impulsive uplift force, in N (lb)
A_f = floor area exposed to wave uplift, in m^2 (ft^2)
η_c = wave crest elevation, in m (ft)

Z = bottom of floor elevation, m (ft)
C_u = uplift coefficient, suggested to be 1.0
C_i = impulsive uplift coefficient, suggested to be 3.0

The area, A_f, should only include sections of a floor that get wet in a single wave, and should not necessarily include the entire floor. The designer must estimate the percentage of the total floor experiencing uplift forces based on the shape and dimensions of the wave crest.

It is not known how well the FHWA method may apply to elevated floors of buildings, as results have been developed for highway bridge decks. The quasi-static term simply reflects added hydrostatic pressures and buoyancy due to passage of a wave crest, so it is physically reasonable. A similar quasi-static force term is included in the Port of San Francisco Building Code, but with an additional coefficient that would reduce the load compared to Equation 3-37. The dynamic force term is termed "interim" by FHWA but is noted as being an important contributor to failed bridge decks in hurricane conditions. The Port of San Francisco Building Code does not include any dynamic force term.

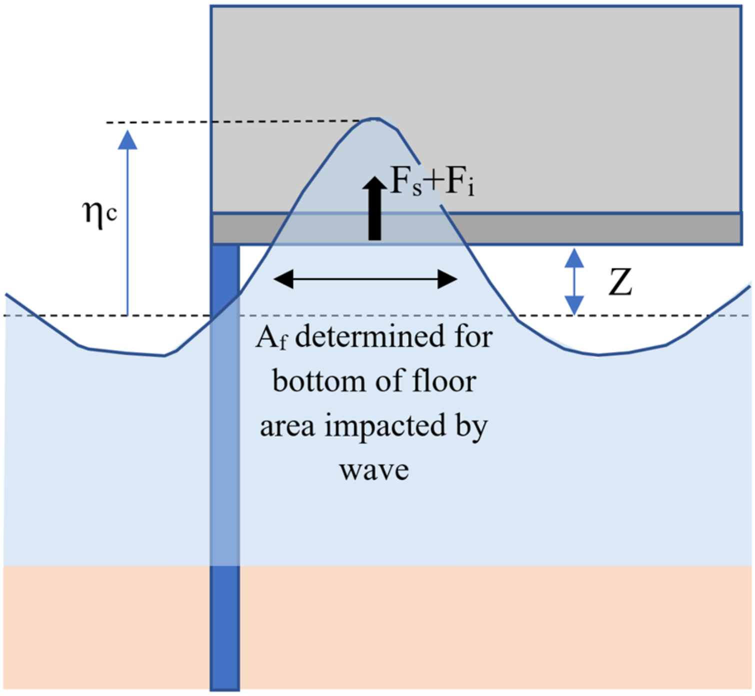


Figure 3-21 Wave uplift on floors

3.9.2.8 Wave Forces on Horizontal Beams

Horizontal beams represent a fundamental structural element, yet wave forces on beams are not included in any FEMA guidance, in ASCE 7 (2016) or in any national building code. The only reference identified for wave loads on beams is the Port of San Francisco Building Code, which addresses both breaking and non-breaking wave conditions. The method assumes the beam is located above the still water level. Wave loads on beams that are fully submerged would require an alternative analysis using the Morison equation, as outlined in USACE (1984) or USACE (2002).

Figure 3-22 shows the conditions of a wave crest impacting an elevated beam. The Port of San Francisco Building Code method first defines characteristic pressures on top and bottom of the beam, and then determines a characteristic force F_h*, as

$$P_1 \; = \rho g \;\; (\eta_C \; - (h + c)) \qquad \text{(Eq. 3-38)}$$

$$P_2 = \rho g \;\; (\eta_C - c) \qquad \text{(Eq. 3-39)}$$

$$F_{h*} \;\; = \;\; 1/2 \;\; w \, (\eta_C \; - c) \;\;\; P_2 \quad\;\; \text{if} \;\; (\eta_C \; < \; (h + c)) \qquad \text{(Eq. 3-40a)}$$

$$F_{h*} \;\; = \; 1/2 \, w \, h \, (P_2 + P_2) \quad\; \text{if} \;\; (\eta_C \; > \; (h + c)) \qquad \text{(Eq. 3-40b)}$$

where:

c = clearance of beam from still water level, in m (ft)
h = height of beach, in m (ft)
P_1 = characteristic pressure at top of beam if beam fully submerged, in N/m^2 (lb/ft^2)
P_2 = characteristic pressure at bottom of beam, in N/m^2 (lb/ft^2)
F_{h*} = characteristic force, to be adjusted, in N (lb)

The characteristic force is then adjusted as follows:

$$F_h \;\; = \; K_i \;\; 0.72 \; ((\eta_C \; - c)/H_S \;) - 2.30 \;\; F_{h*} \qquad \text{(Eq. 3-41)}$$

where:

K_i = breaking wave coefficient = 3.25
F_h = horizontal force of breaking wave on beam, in N (lb)

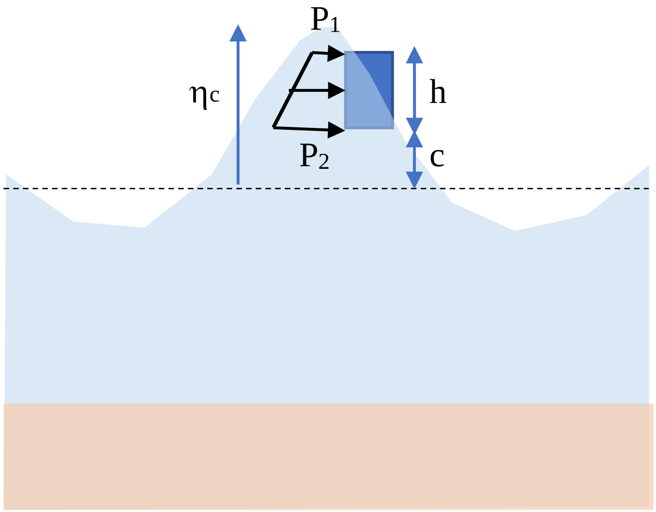


Figure 3-22 Wave forces on horizontal beams

3.9.2.9 Wave Runup Forces on Walls

Wave runup has the potential to impart forces on a structure or structural element as waves surge up a flooded beach or structure slope. In this case, the wave uprush is assumed to occur as a thin wedge of water as the wave surges up the dry beach slope. The leading edge of the surge can cause increased dynamic pressures as the surge strikes, and then reflects, from a vertical wall.

The USACE (2002) outlines a method for estimating the surge force per unit width on a vertical wall element. As indicated in Figure 3-23, the method first requires an estimate of the vertical runup, R, above the still water level. The method then requires an estimate of the breaking wave decay across the surf zone, to obtain the wave height H_{SWL} at the still water shoreline.

The wave height at the structure is then scaled from a simple triangle of wave from H_{swl} at the SWL to zero at the runup limit. With the local height d_w at the structure, the force per unit width is then based on the momentum flux at that location as

$$f_{surge} \ = \ 4.5\,\rho g\, d_w^2 \qquad \text{(Eq. 3-42)}$$

where:

f_{surge} = force per unit width due to wave uprush, in N/m
d_w = depth of wave uprush at wall, in m

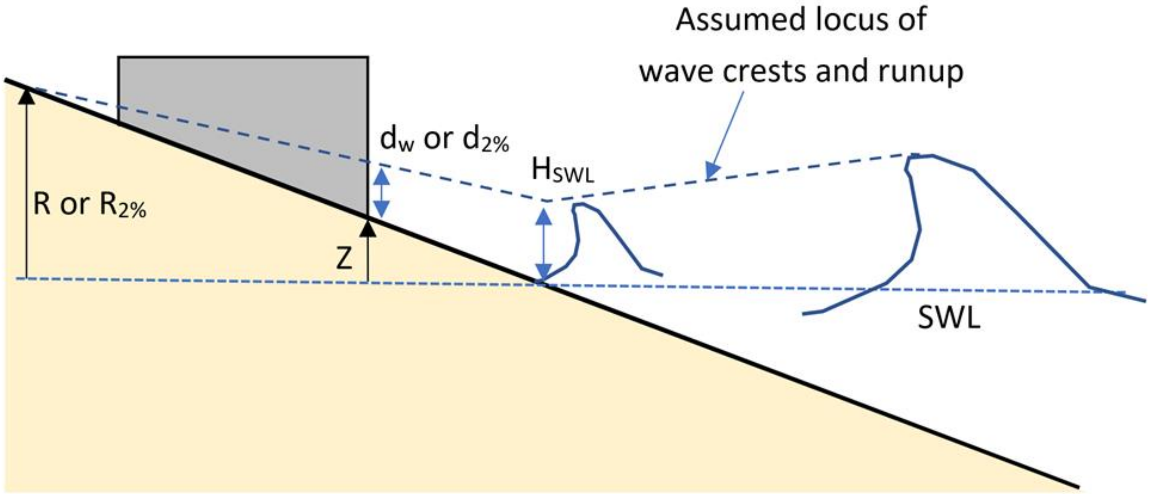


Figure 3-23 Forces due to wave uprush

Because there are no simple methods available for estimating H_{swl} without considering the details of wave transformation, one approach that may be considered would be to adopt a reference standard value for the wave height at the shoreline. A value of H_{swl} in the range of 0.3 to 1 m (1 to 3.3 ft) might be reasonable. In very sheltered waters, the lower end of the range might be appropriate. In more exposed locations, the value of 1 m should be reasonable as wave breaking limits the wave heights that can reach the shoreline.

Because some Provinces include wave runup in flood hazard maps, an alternative procedure for estimating the force of the wave uprush is to more explicitly relate the force to the runup elevation. The method given above by the USACE (2002) is based on a force per unit width on a wall is based on two terms: (1) hydrostatic pressures on the wall due to water depth d_w, and (2) the momentum flux at the wall. These are expressed as

$$f_{surge} = 0.5\,\rho\,g\,d_w^2 + (\rho U^2)\,d_w \qquad \text{(Eq. 3-43)}$$

In order to develop a more tractable solution, methods in *The Rock Manual*, CIRIA (2007), may be used to estimate the water depth and velocity of the wave uprush at the location of the structure as a function of the wave runup. CIRIA (2007) gives results for the 2% exceedance depth and velocity as follows

$$d_{2\%} = (0.33/\gamma_r)\,(R_{2\%} - Z) \qquad \text{(Eq. 3-44)}$$

$$U_{2\%} = (1.37/{\gamma_r}^{1/2})\,(g\,(R_{2\%} - Z))^{1/2} \qquad \text{(Eq. 3-45)}$$

where:

Z = elevation of ground at a point between shoreline and runup limit
γ_r = roughness coefficient (same as defined previously)
$d_{2\%}$ = depth of water at structure due to runup, expressed at 2% exceedance level
$R_{2\%}$ = runup from still water level, expressed at 2% exceedance level
Note that depth $d_{2\%}$ as defined by The Rock Manual, CIRIA (2007), is the same as depth H_w as defined by the Coastal Engineering Manual, USACE (2002)

Substituing the values from (3-44) and (3-45) into the more fundamental force Equation 3-43 gives

$$f_{surge} = [0.67/\gamma_r^2]\,\rho\,g\,(R_{2\%} - Z)^2 \qquad \text{(Eq. 3-46)}$$

The Rock Manual indicates that for stone revetment slopes, $\gamma r = 0.55$, for gravel slopes, $\gamma r = 0.80$, and for grass slopes, $\gamma r = 1.0$. In each case, it is noted however, that $R_{2\%}$ would also change with roughness and slope.

3.10 Application of Flood Loads to Structures

Methods reviewed in this chapter should allow a designer to estimate all types of flood loads on structures and structural elements. The calculation methods are sometime specific to the type of structure or structural element, and methods have been reviewed to cover elevated structures with pier or column foundations, as well as structures on grade (or partially elevated) with vertical walls (or portions of walls) exposed to flood conditions. Other methods apply to specific structural elements such as horizontal beams or floors.

An important aspect of computing flood loads on buildings is to determine which loads should be applied in a given situation. This will generally depend on flood depth, net flood velocity, and wave conditions.

For riverine conditions, two fundamental situations are common:

- Flood zones with no flow conveyance (flood storage only, standing water, no velocity) – to include the following loads:
    - hydrostatic only

- Flood zones with flow conveyance (flow velocity) – to include the following loads:
    - hydrostatic + hydrodynamic + debris (on critical structural element)

For coastal or lake conditions, a wider range of conditions are possible often depending on the conditions:

- Sheltered Flood zones with no exposed fetch for wave growth and no substantial net flows, probably in back bays and estuaries with less than 1 km of open water:
    - hydrostatic only

- Protected Flood zones with no exposed fetch for waves but with net flows, probably in narrow bays and estuaries with less than 1 km of open water but with tidal inflow or outflow, or near a river mouth
    - hydrostatic + hydrodynamic + debris (on critical structural elements)

- Exposed Flood zones with wave action but with no substantial net flows, probably on an open coast or along a bay or estuary with more than 1 km of open water:
    - hydrostatic + waves

- Exposed Flood zones with both wave action and net currents, probably on an open coast or along a bay or estuary but near a river mouth or some constriction causing currents due to tidal or surge
    - hydrostatic + hydrodynamic + waves + debris (on critical structural elements)

Ideally, when flood depth and velocity are determined based on methods reviewed in Chapter 2, flood loads can be estimated from existing flood maps. The most difficult conditions to estimate are wave conditions. Methods outlined here assume depth-limited breaking waves which is a reasonable assumption on many exposed coasts when the offshore waves are large and waves in shallow flooded areas are most likely breaking. This assumption may overestimate waves conditions on protected coasts where offshore waves may be too small to break in all flooded depths. In these and other coastal cases, wave conditions may be best determined from a coastal engineering study that can establish wave heights, crest elevation, and wave runup.

3.10.1 Examples of Flood Loads for Riverine Conditions

Loads will be determined for two building cases: one is a slab-on-grade, and one is a basement. Both building examples have plan dimensions of 12.19 m x 7.32 m (40 ft x 24 ft).

Figure 3-24 Plan view of example building

Slab-on-Grade example:

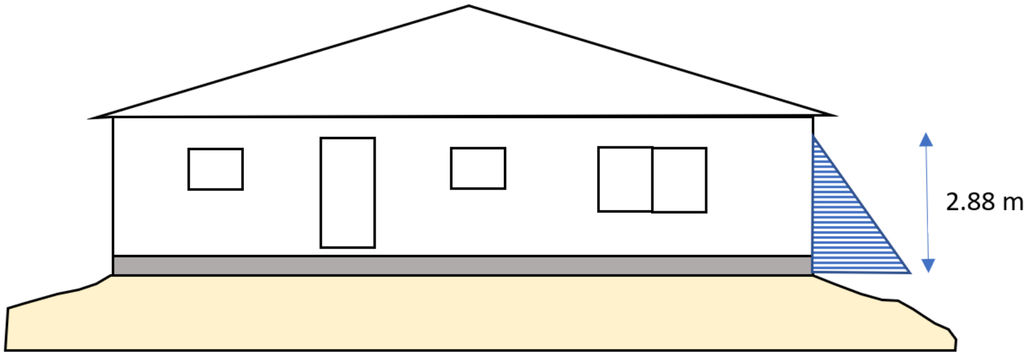


Figure 3-25 Cross section of example slab-on-grade building

Given:

- The river data from Chapter 2, Section 2.19.1 for Transect 25 of Harry's River is used to calculate loads.
- The design flood depth at the 1:500 AEP = depth at design MRI + future conditions = 2.48 m + 0.4 m (judgment on extent of future conditions) = 2.88 m (9.45 ft).
- The flood velocity at the design flood depth for the 1:500 AEP = 1.48 m/s (4.86 ft/s).
- $\gamma = 9810\ \text{N/m}^3$ ($62.4\ \text{lb/ft}^3$)

Find:

- Lateral hydrostatic forces on vertical walls
- Vertical buoyancy forces

- Hydrodynamic forces on the vertical walls
- Debris impact loads on walls

Solution:
The slab-on-grade building could be affected by lateral hydrostatic forces, buoyancy or uplift forces, hydrodynamic forces, and flood-borne debris forces.

- Lateral hydrostatic forces on the building wall are determined by using Eq. 3-4, $f_H = ½ \rho g d_f^2$
  - where $\rho g = \gamma = 9810$ N/m^3 (62.4 lb/ft^3) and $d_f = 2.88$ m (9.45 ft)
  - **f_H = 40,684 N/m (2,786 lb./ft).**
- Vertical buoyancy forces causing uplift due to submergence is determined by Eq. 3-2, $F_B = \rho g \cancel{V} = \gamma \cancel{V}$
  - where $\cancel{V}$ is the volume of the submerged portion of the building. The volume is the d_f x the plan area of the 12.19 m x 7.32 m (40 ft x 24 ft) building.
  - **F_B = 9810 N/m^3 x 2.88 m x 12.19 m x 7.32 m = 2521 kN (172,523 lb.)**
- Hydrodynamic forces on the vertical walls may be computed as an increase in depth according to Equation 3-9 when velocity is low (below 3 m/s) or by using $\Delta d_f = a V^2 /2g$.
  - where Δd_f = increase in water level on upstream side compared to downstream side, in m (ft)
  - a = shape factor or drag coefficient = 1.25 (or from Table 3-1 in the chapter that is a function of building aspect ratio).
  - The aspect ratio of plan width/water depth = 12.19 m (widest side of the building)/2.88 m = 4.23. This ratio yields a drag coefficient of 1.25.
  - Thus, Δd_f = 1.25 x (1.48 m/s)^2/(2 x 9.81 m/s^2) = 0.14 m (0.46 ft).
  - Add this delta depth to the design flood depth and hydrostatic forces are revised based on this increase in flow depth around the building.
  - **The revised hydrostatic pressure is = $½ \rho g (d_f + \Delta df)^2 = ½ \times 9810 \times (2.88 + 0.14)^2 =$ 44,735 N/m (3,065 lb./ft)**

- A comparative hydrodynamic pressure can be determined using Equation 3-10 even though the flow velocity is below the 3 m/s threshold.
  - The hydrodynamic drag force on the building is then given by $F_D = ½ C_D \rho V^2 A_p$ In order to compare results, the force must be multiplied by the depth/metre of wall length to get an equivalent force/unit wall length.
  - Therefore, F_D = ½ x 1.25 x 1000 kg/m^3 x (1.48 m/s)^2 x 2.88 m = 3,943 N/m (270 lb/ft)
- This comparative method of determining hydrodynamic pressure somewhat overstates the magnitude compared to converting the flow into increased depth and increasing the hydrostatic pressure. The increased hydrostatic **pressure due to flow velocity is 44,735 N/m (3,065 lb./ft).** The result of adding hydrostatic pressure to the conventional method for determining **hydrodynamic pressure is 40,685 N/m + 3,943 N = 44,628 N or a decrease of 0.2% (2,786 lb/ft + 270 lb/ft = 3,056 lb/ft instead of 3,065 lb/ft).**

- Debris impact loads will be determined using Equation 3-13. Since there is no information specifically about the likelihood of debris and no information about potential size/weight of the debris, this example will use the minimum recommended debris weight of 4.5 kN.
  - $F_{di} = 1.57\, C_{de}\, C_{bl}\, C_{or}\, R_{max}\, W\, V / (g\, \Delta t)$
  - Where
    - F_{di} = impact force, in N (lbs)
    - W = weight of debris, in N (lbs)
    - V = Velocity of floodwater propelling debris, in m/s (ft/s)
    - Δt= Impact duration or time to reduce object velocity to zero, in sec, taken as 0.03 sec
    - C_{de} = Depth reduction coefficient
    - C_{bl} = Debris blockage coefficient
    - C_{or} = Debris orientation coefficient, recommended as 0.8
    - R_{max} = Structural response coefficient
  - The blockage and depth reduction coefficients will both be taken as 1.0.
  - R_{max} is a function of the ratio of the impact duration to the structural element's natural period. There is no information about an assumed natural period, however, if the building was to be designed for wind loads, a small residential building would assume to be a 'rigid' building in accordance with ASCE 7, and the natural frequency would be greater than 1.0. The natural period = 1/natural frequency so if the frequency is 1.0, the period is 1.0. The ratio of impact duration (0.03 sec) to the natural period (1.0) = 0.03. Table 3 in Chapter 3 does not indicate that the values for R_{max} can be interpolated, therefore, R_{max} will assume to be 0.4 when the ratio of impact duration to natural period = 0.1.
  - **Therefore, $F_{di} = 1.57\, C_{de}\, C_{bl}\, C_{or}\, R_{max}\, W\, V / (g\, \Delta t)$ = 1.57 x 1.0 x 1.0 x 0.8 x 0.4 x 4.5 kN x 1.48 m/s^2/(9.81 m/s^2 x 0.03) = 11,369 N (2,556 lb)**

Basement example:

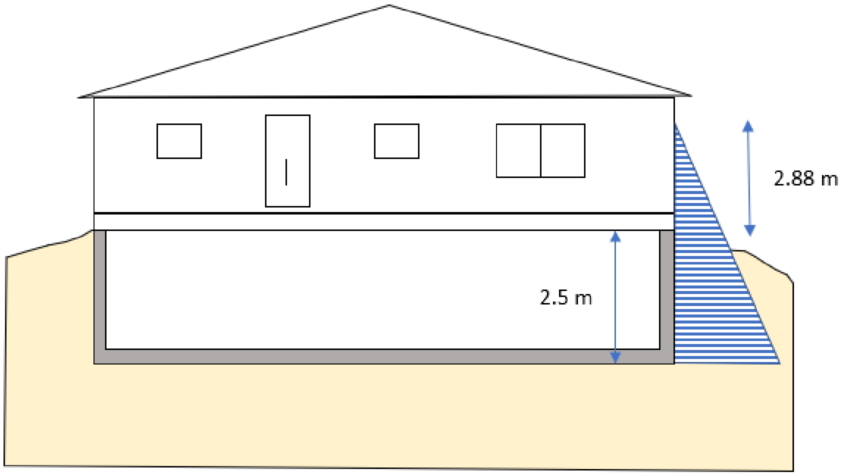


Figure 3-26 Cross section of a building with basement

The basement building could be affected by lateral hydrostatic forces, buoyancy or uplift forces, hydrodynamic forces, and flood-borne debris forces.

- The calculations for this basement building are performed the same as the slab-on-grade building.
- There are two major differences. The water depth extends to the bottom of the basement walls since the assumption (depending on soil type) is that the soil is fully saturated with water, thus the flood load at the base of the wall is very large. For this example, $d_f = 2.88 + 2.5 = 5.38$ m. In addition, the saturated soil exerts an additional pressure on the below grade portion of the wall; the magnitude depends on the soil type. For the purposes of this example, this soil load is considered a geotechnical load, not a flood load.
- The extension of the flood water to the basement slab also significantly increases buoyancy forces.
- Hydrodynamic and debris loads do not change for this basement building example.

3.10.2 Examples of Flood Loads for Coastal Condition

Given:

- From Chapter 2, Section 2.19.2, the design flood depth = 1.8 m (5.9 ft)
- The flow velocity = 4.2 m/s (13.8 ft/s)

Loads will be determined for one building case of a residential structure supported by piles shown in Figures 3-27 and 3-28.

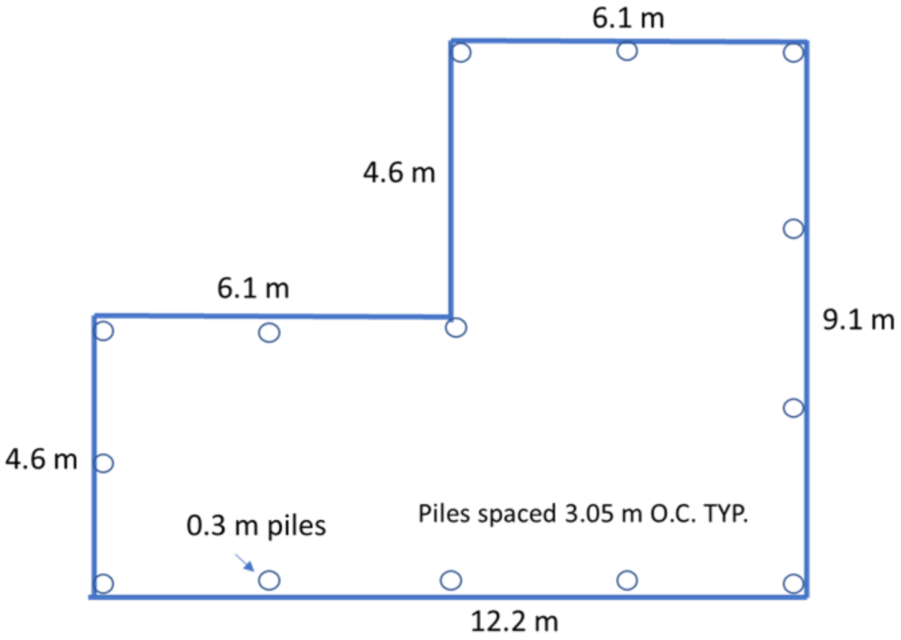


Figure 3-27 Plan view of example building on piles

Find:

- The design wave height
- Wave forces on the pile foundation
- Wave-induced force on a vertical side wall

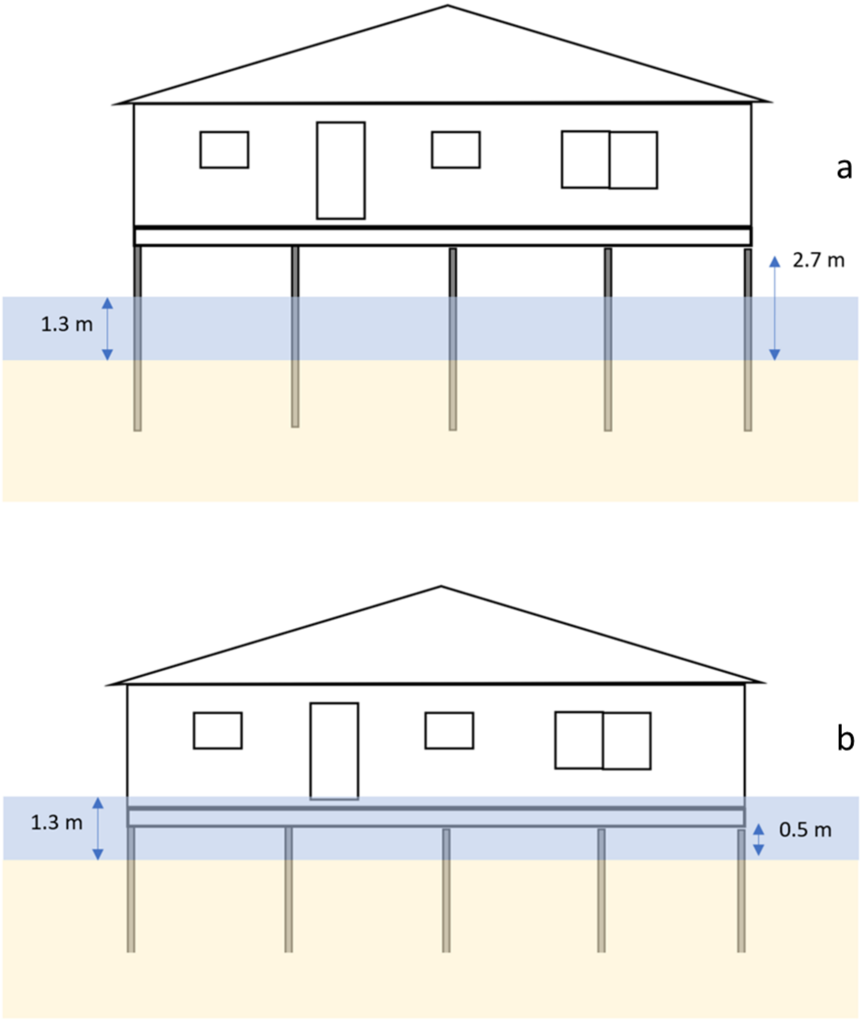


Figure 3-28 Cross section of example building on piles

Solution:

- The design wave height is determined from Equation 3-18. The design flood depth of 1.3 m used in this equation must not include wave effects.
    - **The breaking wave height is estimated to be $H_b = 0.9 d_f$, so H_b = 0.9 x (1.3 m) = 1.2 m (3.8 ft).** Note that the local fetch may not support waves this large so this may be overly conservative. A local coastal engineering analysis could be conducted to refine the wave height.
    - Given the flat nature of the example site, the ground is overtopped by floodwater so wave runup would not be an issue.
    - For determining structure elevation, the first floor should be located above the wave crest elevation. The wave crest amplitude above the still water level is $\eta_c = 0.7 H_b$, therefore, η_c = 0.7 x 1.2 = 0.8 m (2.6 ft).

- **Part A Building Elevated Above Waves (Fig 3-28a)**
  - Assuming local practice of adding 0.6 m freeboard, the flood construction level (FCL) would equal the ground elevation + design flood depth + wave crests + freeboard. The FCL is therefore at 1.8 m + 1.3 m + 0.8 m + 0.6 m = 4.5 m above datum or 2.7 m above the local grade.
- The breaking wave forces on piles are determined using Equation 3-25
  - $F_b = ½\, C_{Db}\, \rho\, g\, D\, H_b^2$
    - Assume circular piles in breaking waves so C_{Db} = 1.75
    - Assume pile diameter of D = 0.3 m
    - **Therefore the breaking wave force on each pile is F_b = ½ x 1.75 x 1000 kg/m^3 x 9.81 m/s^2 x 0.3 m x (1.2 m)^2 = 3708 N (834 lb).**
    - For design, this load would be applied to each pile at the water line.
    - For global loads, it may be assumed that waves strike the seaward row of piles simultaneously. Waves would strike other piles at other times (seconds later) so forces on each pile may not be additive. However, for a conservative solution the breaking wave loads may be added for all 14 piles. This would give an **upper bound wave load of 14x3708 = 51.9 kN (11,700 lb)**

- **Part B Building Elevated 0.5 m above Grade (Fig 3-28b)**
- For purposes of illustrating wave forces on a wall, consider a revised building elevation of only 0.5 m above grade. With a design flood depth of 1.3 m, the height of the wall above grade is h_f = 0.8 m
- The wave crest elevation on the wall and characteristic pressure are determined as follows (assume waves normally incident so angle $\beta = 0$):
  - $\eta^* = 0.75\,(1+\cos\beta)\ H$ (Eq. 3-31a)
    - η^* = 0.75 x (1+1) x 1.2 m = 1.8 m (5.9 ft),
  - $P_1 = 0.5\,(1+\cos\beta)\,(1.1 + \alpha^* \cos^2\beta)\,\rho\, g\, H$ (Eq. 3-31b)
    - P_1 = 0.5 x (1+1) x (1.1+0.8 x 1)x1000 x 9.81 x 1.2 = 22.37 kN/m^2 (467.3 lb/ft^2)
- The wave-induced force per unit length on the vertical wall or structure in N/m (lb/ft) would then be calculated as the area under the pressure distribution as:
  - $f_{wb} = 0.5\, P_1\, \eta^* + P_1\,(d_f - h_f)$ (Eq. 3-32)
    - **f_{wb} = 0.5 x 22.37 x 1.8 + 22.37 x (1.3-0.5) = 38.0 kN/m (2607 lb/ft)**
- If waves approach from the bottom in Figure 3-29 and strike the 12.2 m long exterior wall, and if exterior cladding remains intact, then the total force on the building would be:
  - $F_{wb} = f_{wb}\, W$
    - **F_{wb} = 38.0* 12.2 = 463.6 kN (104,300 lb)**

The two parts of this example illustrate the value of elevating a structure on piles to remain above the wave crests. There is almost a factor of ten increase in total force if the walls of the building are subjected to wave loads.

3.10.2.1 Example with Wave Runup Loads

Wave runup loads are to be estimated on an existing building located on a sloping backshore near the Lake Ontario site evaluated in Example 3 from Chapter 2. A beach transect and building location are given in Figure 3-29.

Given:

- From Chapter 2, Example 3, the 1:100 yr flood elevation is 76.01 m and the 1:500 yr flood elevation is 76.26 m.
- Building is built 15 m landward of the 100-year flood elevation contour
- Ground slope around building site is approximately 1:10
- 100-year wave conditions in deep water offshore of the site are Hs=6. 9 m and Tp=10.0 sec based on study by Zuzek (2020). For simplicity, assume similar conditions for 500-yr storm event.

Find:

Loads due to wave runup for a residential structure supported on a block wall foundation

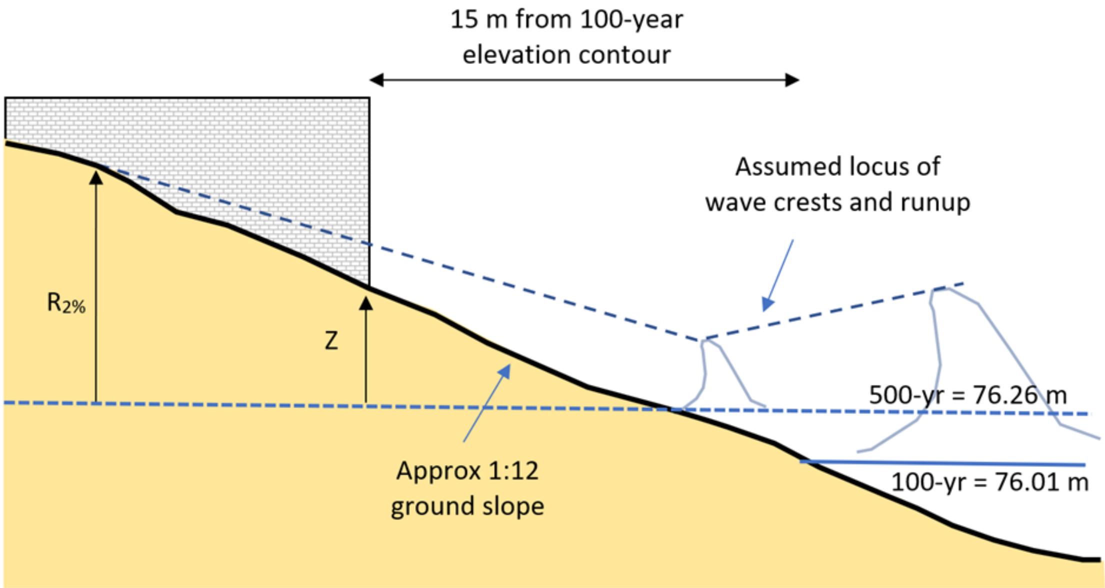


Figure 3-29 Conditions used in example problem for estimating wave runup loads on existing building

Solution:

- Part 1: Use the methods in equation 3-23 to estimate runup on natural sand and grass slopes for the 100-year conditions in central Lake Ontario.

Compute the deep water wavelength as $Lo=gT_p^2/2\pi = 9.81*(10.0^2)/(2*\pi)$ = 156 m.

Compute the Irribarren Number as $\xi = \tan\theta/(Hs/Lo)^{1/2} = (1/12)/(6.9/156)^{1/2} = 0.40$.

From Eq. 3-23, wave runup above 500-yr flood level is $R_{2\%}$ = 0.73 x ξ x H_s = 0.73(0.40)(6.9) = 2.0 m (6.5 ft)

- Part 2: Use the methods in equation 3-46 to estimate the force per unit width due to wave runup impacting the foundation wall. Assume grass slopes with roughness γ_r =1.0.

Building is located 15 m landward of the 100-yr elevation contour. The 500-yr level is 0.25 m higher or 3.0 m landward on the 1:12 slope. The ground elevation at the building is therefore Z=1.0 m above the 500-yr level.

Estimate force per unit width from equation 3.46, with roughness coefficient of 1.0, as
$\boldsymbol{f_{surge} = 0.67\, \rho\, g\, (R_{2\%} - Z)^2}$ **= 0.67x1000 x9.81x(2.0 – 1.0)^2 = 6563 N/m (449 lb/ft)**

If the foundation wall is 10 m wide, the total force on the wall is $\boldsymbol{F_{surge}}$ **= 6563x10 = 65.6 kN (44,192 lb)**

4 Performance-Based Design for Flood

This chapter discusses a topic that is gaining interest and traction in the engineering community. Performance-based design releases the normal constraints of following prescriptive methods used in engineering textbooks and promulgated in building codes and standards and allows the engineer some freedom to explore alternative methods and systems for the design of buildings and components. This alternative method focuses on the desired result instead of a required path to achieve an expected result.

4.1 Background and Purpose

In a report to the American Society of Civil Engineers (ASCE) Structural Engineering Institute Board of Governors, a Structural Engineering Institute (SEI) Task Committee noted *"Performance-based design is a process that enables the development of structures that will have predictable performance when subjected to defined loading."*

Performance-based design (PBD) turns the traditional design paradigm upside down in the sense that the required performance is the starting point for the design. Considering the desired performance of the structure and selecting the scenarios that match the owner's goals for maintaining structural function in the presence of a specific hazard, the designer works toward achieving those goals. The performance of the design is demonstrated through analysis, simulation, prototype testing, or a combination thereof. Performance-based design is founded on the premise that structural systems and the nonstructural systems they support, must meet specific performance objectives. It is also founded on the premise that the structural performance can be determined to sufficient accuracy for design purposes and that uncertainties can be taken into account.

Flood-resistant design has two primary elements used to determine the hazard – one is the spatial location of the building or building site relative to floodplain boundaries or the extent of the floodplain, and the other is the elevation of the building relative to the flood elevation. Flood damage can occur from inundation or flooding above the established elevation of the lowest portion of the building. This damage can occur to the exterior and to the contents as water leaks inside the building. Water does seek its own level so the interior water level will usually be at the same level as the water outside a building. The exact elevation of any actual flood is highly variable and extremely uncertain.

The purpose of this chapter is to describe an alternative method of achieving a minimal level of performance that is at least equal to the performance achieved if the acceptable prescriptive solutions are followed for flood design. It should be noted that as of this report date, there are no prescriptive flood design requirements in the National Building Code of Canada 2015 (NBC). Thus, there are no comparative solutions, except those that might be generated using the other technical information developed under this contract with the National Research Council of Canada (NRC).

4.2 Objectives of PBD and How it Could Be Used

Properly executed performance-based approaches enable desired performance to be attained with demonstrated confidence and reliability. Since the performance targets (or objectives) for the design are defined explicitly, communities and individual decision makers can select the performance level(s) that are appropriate and satisfy the applicable criteria. Since performance is evaluated directly as part of the design process, engineers need not be limited by requirements to conform to prescriptive solutions, thereby allowing for innovation and the use of new design solutions including new materials and systems.

The primary objective for including PBD as a chapter in this technical report to the NRC is to describe an alternative method of flood design that might achieve acceptable levels of performance for a certain class of structures without a corresponding increase in cost for the improved performance over the cost of providing flood resistance strictly in accordance with proposed prescriptive building code provisions.

This method could be especially useful for flood design for critical facilities and shelters and post-disaster buildings located in the Canadian provinces/territories. The NBC defines the Importance Categories of Buildings (Table 4.1.2.1 in the NBC) as shown in Figure 4-1 below. Buildings classified as *Normal* Importance that are high-rise or multi-family residential structures and *High* Importance or *Post-disaster* building categories would derive the most benefit from PBD. These uses normally demand the highest performance levels because of their connection to life safety, public protection and safety, and community resilience. These classes of buildings include shelters, community gathering places, hospitals, emergency operations centres, and public infrastructure such as sewage treatment and power generating stations.

Use and Occupancy	Importance Category
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including: • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings	Low
All buildings except those listed in Importance Categories Low, High and Post-disaster	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is: • as an elementary, middle or secondary school • as a community centre Manufacturing and storage facilities containing toxic, explosive or otherhazardous substances in sufficient quantities to be dangerous to the public if released	High
Post-disaster buildings are buildings that are essential to the provision of services in the event of a disaster, and include: • hospitals, emergency treatment facilities and blood banks • telephone exchanges • power generating stations and electrical substations • control centres for air, land and marine transportation • public water treatment and storage facilities, and pumping stations • sewage treatment facilities and buildings having critical national defence functions • buildings of the following types, unless exempted from this designation by the authority having jurisdiction: o emergency response facilities o fire, rescue and police stations, and housing for vehicles, aircraft or boats used for such purposes o communications facilities, including radio and television stations	Post-disaster

Figure 4-1 Building Importance Categories as listed in Table 4.1.2.1 (source: National Building Code of Canada, 2015)

An example of Canadian land use regulations suggests that buildings constructed in the overland flow area (defined as those lands abutting the floodway or flood fringe, the boundaries of which are indicated on flood maps that would be inundated by shallow overland flood water in the event of a 1:100 AEP event) must be built such that the first floor is a minimum of 0.3 m (1 ft) above the highest grade on the street abutting the parcel of interest. This land use requirement places place many buildings at risk of flooding, and if protection is required as suggested by this prescriptive design statement, adequate protection will be difficult to achieve. Figure 4-2 shows energy and financial buildings in Calgary, AB surrounded by flood water suggesting a severe curtailment of services in the current flood stage.

Figure 4-2: Energy and financial buildings surrounded by flood water in Calgary, Alberta (source: Jen Gerson/National Post).

To clarify how the PBD method is different than the design method practitioners otherwise would use in following prescriptive methods for flood design, we present the following simple example for a new building. The prescriptive flood design approach (at least as followed in the US) consists of the following steps:

- Locate the building site and determine if the site is in or out of the extent of the floodplain boundary. This usually means the limits of the design flood which in Canada could be the 1:100 AEP, the 1:200 AEP or the 1:500 AEP event depending on the provincial requirements for the site.
- If the site is in the floodplain, then construction of the lowest building floor or the Flood Construction Level (FCL) must be established at or above the elevation of the floodline at the site location. If local or provincial rules require adding freeboard or consideration of future conditions, then additional elevation would be required to comply with these rules.
- In most cases, the bottom elevation of the mechanical and electrical equipment must also be at the FCL plus freeboard and future conditions.

If the building is a commercial or industrial building, flood protection can be provided by floodproofing the exterior of the building to prevent water from entering (See Chapter 4 of

Guidelines for Improving Flood-Resistance of Existing Buildings)[1]. This method consists of sealing up openings and placing barriers across windows and doors that are below the FCL.

This prescriptive approach is deterministic in that all hazard parameters are assumed to be known and the designs that result are expected to provide buildings that will meet expected performance levels without ever stating what those performance objectives might be. It is assumed and implied that the resulting design will meet the required performance. Yet we know from past flood events, that this prescriptive approach does not yield acceptable building performance in many cases. Any approach that relies mostly on elevation for protection (including the elevation of flood protection devices) can experience catastrophic losses when just a little water goes over the top of the protection or wets the underside of the elevated building. The PBD approach addresses these shortcomings in the prescriptive approach. Uncertainties with flood loads are very high and using PBD provides an improved way to deal with the risks associated with these uncertainties.

In contrast, the PBD flood design approach would consist of the following steps:

- For any given building and use, determine how the structure and its associated operation must perform; in other words, establish performance objectives for the building and use. This is extremely important; the process requires that the design team collaborate with the owner and/or developer to determine what the performance objectives really are rather than rely on the owner’s design team leader to dictate such performance levels. For instance, a school used as a shelter might need to be open and accessible prior to a major flood up to the time the flood reaches the school; subsequently the school must provide protection during the flood, and finally must allow for safe departure when the flood recedes. A hospital emergency room might need to be open as long as possible before a flood, might be able to take a limited number of patients into the hospital at a temporary entrance that is on higher ground for triaging, then return to normal operation after the flood recedes.
- Create a matrix of performance vs. hazard levels in order to better define the expected performance under various flood hazard levels. This will help define the needed flood hazard return periods and thus the required design flood depths and the extent of the floodplain.
- Develop a design that addresses the expected performance levels.
- Verify the expected performance through analytical simulation, testing or some combination thereof.
- Have the design peer reviewed.

The primary differences in the prescriptive and performance approaches are how the hazard levels are determined and how the expected performance is addressed directly by the design, instead of expecting performance to be achieved simply because prescriptive parameters were used in the design. Using PBD also allows flood hazard uncertainty to be dealt with directly in the design by addressing the performance needs likely affected by that uncertainty.

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[1] There are two reports that provide guidance on flood-resistant design for buildings. This report is one; the other deals with flood-resistant design for existing buildings which is Report No. CRBCPI-Y5-R4 .

4.3 Scope Limitations and Building Code Requirements

It is recommended that PBD be used for those building and use types that are the most important to the community or those where strict adherence to elevation above the FCL is difficult or costly to achieve. The NBC identifies these buildings as Normal Importance (high-rise or multi-family residential), High Importance and Post-disaster Importance Categories. Some PBD solutions are likely to require some human intervention in order to be effective, and solutions that require human intervention seldom are successful when home owners supply the labor. Commercial, industrial, institutional, and government facilities usually have access to a pool of labor that could be deployed to create some flood solutions if necessary. It is also recommended that the PBD approach might be most beneficial to existing buildings. The approach can certainly be used for new facilities; however, new facilities are not constrained by existing ground elevations or placement within the boundaries of the floodplain. Existing buildings have the placement and elevation already established.

For purposes of this Chapter 4, it is assumed that the flood load provisions will embody the concepts and formulas suggested in Chapters 2 and 3.

4.4 Development of Performance-Based Design Requirements for Flood

This section describes how to establish performance objectives for buildings, how to determine hazard levels to use for design, and how to determine the acceptable damage level for the design hazard level that will ensure that the performance objectives are met.

4.4.1 Performance Objectives for Various Building Types Defined in the National Building Code of Canada

Performance objectives should define how the owner or designer expects the building or the systems within the building to perform before, during, and after the flood. Since a premise for the use of PBD methods is they would be most helpful in the design of Normal, High and Post-disaster category of buildings, the performance objectives should focus on these building and use types (Figure 4-1).

Irrespective of the hazard level or the acceptable damage states, performance objectives should be developed with the occupancy and use of the building in mind. Some possible performance objectives follow for the Normal Importance, High Importance and Post-disaster Importance uses listed in Table 4.1.2.1 of the NBC of Canada and shown in this chapter as Figure 4-1. The following sections provide examples of PBD objectives related specifically to flooding but are not intended to be all-inclusive or exhaustive.

4.4.1.1 Normal Importance Category

HIGH-RISE CONDOMINIUMS/APARTMENTS/HOTELS

- High-rise condominiums, apartments or hotels (as illustrated in Figure 4-3) are places where flood-impacted occupants could shelter-in-place if the upper floors of these buildings is above the design flood elevation. It must be recognized that, if occupants are sheltered in these buildings, it is possible that they will need to be housed for an extensive period of time until flood waters recede and normal access into and out of the buildings is allowed. If access might be required for emergency vehicles for life-saving purposes, then this consideration must be considered in the design.

- Performance objectives in this use case must focus on keeping occupants safe with normal building services available as much as possible. If any occupants require extraordinary medical attention or access to outside-the-building services, then PBD should not be considered.
- The floors that are below the design flood elevation must be designed to allow flood water to inundate these spaces. This means that elevators and building mechanical systems must be able to accommodate flooding. The most straightforward accommodation for these services is to place building mechanical equipment either in a floodproofed enclosure or place them on an elevated floor of the building. Elevator equipment must be placed in a floodproofed enclosure if the elevation mechanical equipment is in a pit in the lowest floor. If the elevation mechanical system is driven from the top, then the elevator system must be automatically prevented from descending into flood water or the flooded space until the flood has receded and the space cleaned.
- The potable water and wastewater systems must be designed to not be compromised by flood water, and these systems must be continuously available to building occupants for the duration of the flood.
- The power and auxiliary electrical equipment such as fire alarms, security, and internet must be designed to be continuously available to building occupants. This will probably require the installation of an emergency generator to supply power. The generator must be placed in a location that is not subject to flood damage, and the fuel supply system must not be affected by flood water.
- Building owners and local governments must consider issues of liability if existing high-rise buildings are used as shelters during and after a flood.

Figure 4-3: The Bow Skyscraper Centre in Calgary, AB.

4.4.1.2 High Importance Category

SCHOOLS:

- Schools used for shelters should be accessible by shelter occupants up to one hour before shelter closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain shelter access.
- A floor of the school must be higher than the design flood elevation in order to keep shelter occupants safe, healthy and dry. Meeting this objective means there must be sufficient space on the upper floor(s) to safely house the shelter occupants. There must be means for disabled or infirmed shelter occupants to gain access to these upper floors. Figure 4-4 illustrates an elevated school building.
- School mechanical systems must be elevated such that flood damage will not disable these systems during the time the shelter is occupied. Meeting this objective means there will be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the shelter for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for the entire shelter population during the entire sheltering period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and

washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.

- Mechanical, plumbing or electrical systems will not leak, overflow, or fail because of flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or failure will not affect the shelter occupants in any way that jeopardizes their safety during the time period required for sheltering. Potable water must be available for drinking. Kitchens or food service must be available for long duration occupancies.

Figure 4-4: Elevated school building (source: FEMA 424)

COMMUNITY CENTRES**:**

- Community centres should be accessible by occupants up to one hour before closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain centre access.
- A floor of the centre must be higher than the design flood elevation in order to keep occupants safe and dry. Meeting this objective means there must be sufficient space on the upper floor(s) to safely house the occupants. There must be means for disabled or infirmed occupants to gain access to these upper floors.
- Centre mechanical systems must be elevated such that flood damage will not shut down these systems during the time the centre is occupied. Meeting this objective means there will be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the centre for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for the entire centre population during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.

- Mechanical, plumbing or electrical systems will not leak, overflow, or fail because of flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or being energized will not affect the occupants in any way that jeopardizes their safety during the time period required for occupancy. Potable water must be available for drinking.
- Unless a community centre is used for a shelter, the community should consider not occupying a centre during a flood. Many community centres are one-story buildings and have the first and lowest floor at grade. Protecting the building interior, any occupants, and mechanical systems will require floodproofing the building to keep the water out (see Technical Report Guidelines for Improving Flood-Resistance for Existing Buildings). While it is feasible to reduce the impact of flooding on the building and equipment by floodproofing, it is not considered safe to allow occupancy of the centre during the flood. Any minimal overtopping of flood protection devices will allow the entire building to flood with nowhere for the occupants to go to avoid the flood water.
- Security systems for the centre should be maintained and not affected by flood water. If the security system could be affected by flooding, a backup security plan should be developed.

MANUFACTURING AND STORAGE OF TOXIC, EXPLOSIVE, OR HAZARDOUS SUBSTANCES:

- Store toxic, explosive or hazardous substances at an elevation that would not allow them to get wet from a design flood if the flood water would initiate a chemical reaction, a release of harmful emissions either into the water or air, or would degrade the substances such that their value would be reduced and create a significant product monetary loss. Figure 4-5 illustrates storage tanks located at grade.
- Store toxic, explosive or hazardous substances at an elevation or location such that access to them is still available, especially if these substances might have to be moved, quarantined or protected during or immediately after the flood.
- Store toxic, explosive or hazardous substances at an elevation or location that minimizes their exposure to high humidity or moisture if such humidity or moisture would render the substances not suitable for production or further use in the manufacturing process.
- Elevate or locate the handling equipment for the substances such that the operability of the equipment is maintained throughout the time of the flood.
- Site and cyber security systems should be maintained and not affected by flood water. If the security systems could be affected by flooding, a backup security plan should be developed.

Figure 4-5: Image of chemical storage facility located at grade near St. Clair River (source: file photo/The Observer)

4.4.1.3 Post-Disaster Buildings

HOSPITALS:

- Hospitals should be accessible by patients up to one hour before closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain hospital access. This objective could vary depending on which part of the hospital is being addressed. The emergency room operation might have more or less stringent requirements than the general admission areas.
- A lowest patient-accessible floor of the hospital must be higher than the design flood elevation in order to keep patients safe and dry. Alternatively, the lowest patient-accessible area must be floodproofed with patient access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of patient access. There must be means for disabled or infirmed patients to gain access to the lowest accessible floors. Figure 4-6 illustrates flooding during a hurricane event in the US (FEMA Hurricane Sandy MAT Report, 2013).
- Upper floors of the hospital may be used as shelters, even though some sheltered occupants will likely be patients in the hospital. The upper floors must be higher than the design flood elevation.
- Hospital mechanical or power generation systems must be elevated such that flood damage will not shut down these systems during the time the hospital is occupied. Meeting this objective means there will be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the centre for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for an established subset of the hospital population during the entire occupancy period. Meeting this objective means that no

restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.
- Mechanical, plumbing or electrical systems will not leak, overflow, or fail because of flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or failing will not affect the occupants in any way that jeopardizes their safety and health during the time period required for occupancy. Potable water must be available for drinking.
- Records and hospital computer systems must be either elevated above the design flood or be in areas that can be floodproofed. Computer systems must be backed up and stored off site away from and above any flood threat.
- Physical security in psychiatric units and other site and cyber security systems should be maintained and not affected by flood water. If the security systems could be affected by flooding, a backup security plan should be developed. An influx of patients caused by a threat or virus should not disrupt normal hospital operations.

Figure 4-6: Flooding in New Jersey Hospital – Hurricane Sandy (source: FEMA)

TELEPHONE EXCHANGES:

- Telephone exchange locations are likely to have substantial operating assets at grade and thus these assets will be susceptible to flood damage. The lowest operational floor should be either elevated above the design flood elevation or have the openings in the walls be protected with flood protection barriers.
- This building use type is most likely to not be occupied at all or be minimally occupied. The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways,

and elevator access openings protected from flooding while still allowing some form of occupant access.

- Upper floors of the facility (if any) may be used like shelters. The upper floors must be higher than the design flood elevation.
- Telephone exchange facilities must have operational systems either elevated or protected by floodwalls so flooding will not shut down these systems during the time the facility must be operational.
- Restrooms (if any) must be available and functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal. Potable water must be available for drinking.
- Communication systems must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases. These systems must be maintained in a secure condition during disruptive local events to preserve communication abilities.

POWER GENERATING STATIONS AND ELECTRICAL SUBSTATIONS:

- Power generating stations are likely to have substantial operating assets at grade and thus these assets will be susceptible to flood damage. These facilities are also likely to cover significant acreage. The power utility will have to decide if individual building or structure protection is more cost-effective than protecting the entire site with a flood protection barrier such as a floodwall.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Power generating facilities must have operational systems either elevated or protected by floodwalls so flooding will not shut down these systems during the time the facility must be operational.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal. Potable water must be available for drinking.
- Substations will likely fail if flood water reaches the substation location. Insuring continuous operation means that the equipment is elevated and located such that being inundated with flood water will not affect the operation in any way.
- Communication systems used by personnel at the power generating plants must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must

exist during the flood impacting the facility, and must operate during recovery and clean up phases.

CONTROL CENTRES FOR AIR, LAND, AND MARINE TRANSPORTATION:

- Control centres should be accessible by occupants up to one hour before closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain control of centre access.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access. There must be means for disabled occupants to gain access to the lowest accessible floors.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Control centre mechanical or power generation systems must be elevated such that flood damage will not shut down these systems during the time the facility must be operational. Meeting this objective means there will be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the facility for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.
- Mechanical, plumbing or electrical systems will not leak, overflow, or fail because of flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or failing will not occur during the time period required for occupancy. Potable water must be available for drinking.
- Communication systems used by personnel at the transportation control centres must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.

PUBLIC WATER TREATMENT PLANTS:

- Water treatment plants are likely to have substantial operating assets at grade and thus these assets will be susceptible to flood damage. These facilities are also likely to cover significant acreage. The water utility will have to decide if individual building or structure protection is more cost-effective than protecting the entire site with a flood protection barrier such as a floodwall.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways,

and elevator access openings protected from flooding while still allowing some form of occupant access.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Water treatment facilities must have operational systems either elevated or protected by floodwalls so flooding will not shut down these systems during the time the facility must be operational.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal. Potable water must be available for drinking.
- Communication systems used by personnel at the water treatment plants must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.

SEWAGE TREATMENT PLANTS:

- Sewage treatment plants are likely to have substantial operating assets at grade and thus these assets will be susceptible to flood damage. These facilities are also likely to cover significant acreage. The wastewater utility will have to decide if individual building or structure protection is more cost-effective than protecting the entire site with a flood protection barrier such as a floodwall or dike (see Figure 4-7).
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Sewage treatment facilities must have operational systems either elevated or protected by floodwalls, so flooding will not shut down these systems during the time the facility must be operational.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal. Potable water must be available for drinking.
- Communication systems used by personnel at the sewage treatment plants must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.
- Most sewage treatment facilities have an outflow for treated water to discharge into a body of water, usually a river or stream. When a flood occurs, this outflow will be choked off,

and flood water will usually overflow the separation tanks and thus the site. Flood mitigation planning should consider this likely event.

Figure 4-7 Flooded wastewater treatment plant in Calgary, AB (Source: City of Calgary https://maps.calgary.ca/riverflooding/)

EMERGENCY RESPONSE FACILITIES:

- Emergency response facilities should be accessible by occupants up to one hour before closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain facility access.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access. There must be means for disabled occupants to gain access to the lowest accessible floors.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Emergency response facility mechanical or power generation systems must be elevated such that flood damage will not shut down these systems during the time the facility must be operational. Meeting this objective means there must be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the facility for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.
- Mechanical, plumbing or electrical systems will not leak, overflow, or be energized by flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or being energized will

not affect the occupants in any way that jeopardizes their safety during the time period required for occupancy. Potable water must be available for drinking.

- Communication systems used by personnel at the emergency response facilities must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.

FIRE, RESCUE, POLICE STATIONS, AND VEHICLE HOUSING:

- Fire, rescue and police facilities should be accessible by occupants up to one hour before closing due to the impending flood. Meeting this objective would require an estimate of the time of maximum flooding compared to the elevation of the access routes that would be used to gain building access.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access. There must be means for disabled occupants to gain access to the lowest accessible floors.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Fire, rescue and police facility mechanical or power generation systems must be elevated such that flood damage will not shut down these systems during the time the facility must be operational. Meeting this objective means there will be no loss of power. Primary power supplies might be severed but emergency generators could be in place to service the facility for the required time frame. This also means that generator fuel supplies must be sufficient for the required time frame.
- Restrooms must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal.
- Mechanical, plumbing or electrical systems will not leak, overflow, or be energized by flood water should they experience some level of flooding. Meeting this objective means that the equipment is elevated and located such that leaking, overflowing, or being energized will not affect the occupants in any way that jeopardizes their safety during the time period required for occupancy. Potable water must be available for drinking.
- Communication systems used by personnel at the fire, rescue and police stations must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.
- Emergency vehicles must be housed in a flood-safe area – either ground that is higher than the design flood elevation or a parking garage with floors higher than the design flood. There also must be access from the facility to the vehicle storage location before the arrival of the flood, during the flood potentially and after the flood to help with rescue and recovery.

COMMUNICATIONS FACILITIES FOR RADIO AND TV:

- Communication locations are likely to have substantial operating assets at grade and thus these assets will be susceptible to flood damage. The lowest operational floor should be either elevated above the design flood elevation or have the openings in the walls be protected with flood protection barriers.
- The lowest occupant-accessible floor of the facility must be higher than the design flood elevation in order to keep occupants safe and dry. Alternatively, the lowest occupant-accessible area must be floodproofed with occupant access portals such as doors, driveways, and elevator access openings protected from flooding while still allowing some form of occupant access.
- Upper floors of the facility may be used as shelters. The upper floors must be higher than the design flood elevation.
- Communication facilities must have operational systems either elevated or protected by floodwalls so flooding will not shut down these systems during the time the facility must be operational.
- Restrooms (if any) must be available and remain functional for an established occupant load during the entire occupancy period. Meeting this objective means that no restroom facilities will overflow from sewage backup, that potable water will be available for flushing toilets and washing hands, and that waste products can be disposed of via the sanitary sewer system or a holding tank is available for waste disposal. Potable water must be available for drinking.
- Communication systems must be operational during the entire flood event. This operational metric means that communication capability must exist for some period of time before the flood arrives, must exist during the flood impacting the facility, and must operate during recovery and clean up phases.

4.4.2 Flood Hazard Levels

Flood hazard levels must be defined to establish the anticipated design event associated with the defined performance objectives. The generalized hazard levels can be defined as:

- Routine (which could relate to serviceability)
- Design (which would likely affect the building or the operation)
- Extreme (which would likely affect the community)

A routine flood hazard is one that affects the day-to-day use and operation of a facility. In many cases, this flood level could be the 1:100 AEP flood or a lesser flood level stipulated by the Authority Having Jurisdiction (AHJ). Given that the exceedance probability for a 1:100 AEP flood is 39% over the 50-year service life of a building (Table 2-6 in Chapter 2 is repeated here as Table 4-1 for ease of use), this high exceedance probability suggests a frequent occurrence. The recommended routine flood hazard event magnitude is the 1:100 AEP.

The design flood hazard is one that affects the use of the building or operational function in the building. Depending on the use or importance of the functions, the design flood hazard return period should be determined by the risk the owner or operator of the facility is willing to assume. In order to reduce the flood probability of exceedance to 10% for a service life of 50 years, the flood AEP must be 1:500 years. In order to reduce it further to 5%, the flood AEP must be 1:1000 years. For a 100-year service life, there is a 10% probability of exceedance for a 1:1000 AEP. See

Table 4-1 for the flood probabilities for event return periods (i.e. MRIs) vs. time periods (service lives). The probabilities listed in Table 4-1 table assume stationarity, and therefore do not include consideration of climate change. Thus the numbers in the table are un-conservative, i.e., the probability of exceeding a flood event with an AEP = 1,000 is probably more than 5% in 50 years.

Table 4-1 Flood Probability for Event MRI vs. Time Period (Service Life)

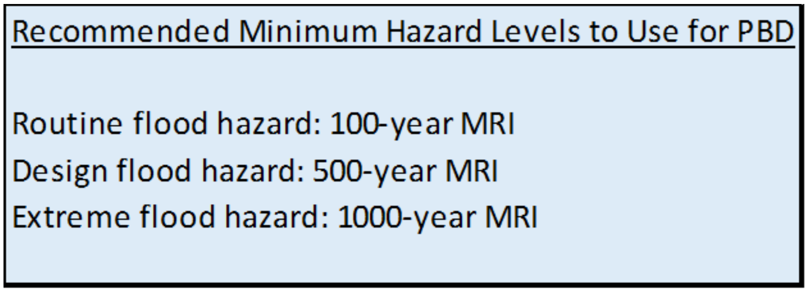
	Time Periods (Service Life)						
Event AEP	1 yr	10 yrs	20 yrs	30 yrs	50 yrs	70 yrs	100 yrs
1:10	0.10	0.65	0.88	0.96	0.99	1.00	1.00
1:20	0.05	0.40	0.64	0.79	0.92	0.97	0.99
1:50	0.02	0.18	0.33	0.45	0.64	0.76	0.87
1:100	0.01	0.10	0.18	0.26	0.39	0.51	0.63
1:200	0.005	0.05	0.10	0.14	0.22	0.30	0.39
1:500	0.002	0.02	0.04	0.06	0.10	0.13	0.18
1:750	0.0013	0.01	0.03	0.04	0.06	0.09	0.12
1:1000	0.001	0.01	0.02	0.03	0.05	0.07	0.10
1:2000	0.0005	0.005	0.01	0.01	0.02	0.03	0.05
1:3000	0.0003	0.003	0.007	0.01	0.02	0.02	0.03

The choice of design flood hazard level should consider that the flood design must protect the facility or the operation for a very long period of time without interruption and with very low effort to repair, replace, or time to bring the operation and the building use back to a fully operating condition after a flood event.

The extreme flood hazard level is one that will likely affect the entire community, with road closures and interruption of other transportation routes such as air, rail, or marine routes likely. If the facility of interest in designing for the flood hazard using PBD is a crucial facility to the community, the facility functionality should at a minimum, be no less than that of the community but the time required to bring functionality back to normal should be as soon as possible and much less time than the time required for the community as a whole to return to normal operations. The community flood design level should be on the order of at least 1:1000 AEP so that wide spread community functionality is not damaged or halted. This is an important decision for community leaders to make and to develop plans to achieve this objective. If the design flood hazard, however, is taken as a 1:1000 AEP, the extreme hazard must be greater than this design flood level, and thus should be in the range of 1500-2000 years. Figure 4-8 illustrates minimum recommended hazard design levels to use with PBD.

For example, a US community has a large area included in the flood hazard area. Two recent floods have cut off portions of the community from each other, interrupted operations for many small businesses, and affected churches and community centres. The flood-prone areas are partially protected by a levee, but the function of the levee is compromised by a highway overpass, and a railroad track. The flood study that was used to support the flood maps for the area included the following statement: *"Therefore, at this time, it must be assumed that the bridge opening cannot be adequately blocked to prevent flood flow from the Lumber River into the levee-protected area."*

(NIST, 2018).). The flood study recognized that the levee is not sufficient protection but does not offer a mitigation measure to complete the protection. It would be prudent for any business or community service located in this area to assume an extreme flood might occur and develop their own protection scheme for an extreme flood.



Recommended Minimum Hazard Levels to Use for PBD

Routine flood hazard: 100-year MRI
Design flood hazard: 500-year MRI
Extreme flood hazard: 1000-year MRI

Figure 4-8 Minimum Recommended PBD Flood Hazard Levels

4.4.3 Damage Levels

4.4.3.1 Defined Damage Levels

For the purposes of this chapter on PBD, the recommended damage levels follow those suggested in the ICC Performance Code (2018). Those damage levels and their associated descriptions are:

MILD:

- Minimal damage from flood water.
- No mold growth due to water infiltration.
- Damage is easy to clean up, and water is easy and quick to dispose.
- Occupancy overnight might be affected for a few days, but daytime occupancy should not be affected.
- No damage to the building structural system and no damage to interior partitions or ground-mounted equipment.
- No physical damage to components on the building envelope except for minor flood debris impacts.

MODERATE:

- Some damage from flood water that causes some interior material to be replaced (i.e. drywall, insulation, electrical components that are inundated).
- Any observed mold can be easily and quickly remediated such that downtime is less than one week.
- Water stains are easy to clean.
- Overnight occupancy could be affected for a few weeks, but daytime occupancy is not affected more than a few days.
- No damage to the building structural system but some interior damage to partitions and ground-mounted equipment will occur.
- Some physical damage to components on the building envelope that might compromise water infiltration until repairs are complete.

SEVERE:

- Extensive damage from flood water that causes partial or complete collapse of the structure. Progressive collapse scenarios should be evaluated for their impact to operations and recovery.
- Occupancy is delayed until damaged structure can be repaired or replaced. This downtime could be months to years depending on how rebuilding is financed.
- Occupancy is delayed until mold or other water damage can be mitigated, if necessary.
- Little residual strength is remaining in the structure.
- Operations are halted until structural and flood damage repairs can be completed.

The ICC Performance Code suggests a damage-hazard intensity matrix as shown here in Figure 4-9 (copied from Table 303.3 in ICC Performance Code). This chapter suggests a modification of this matrix to the one shown in Figure 4-10 to use for Performance-based Design for Flooding.

		Increasing Level of Performance → Performance Groups			
		Performance Group I	Performance Group II	Performance Group III	Performance Group IV
Magnitude of Design Event ↑	VERY LARGE (Very Rare)	SEVERE	SEVERE	HIGH	MODERATE
	LARGE (Rare)	SEVERE	HIGH	MODERATE	MILD
	MEDIUM (Less Frequent)	HIGH	MODERATE	MILD	MILD
	SMALL (Frequent)	MODERATE	MILD	MILD	MILD

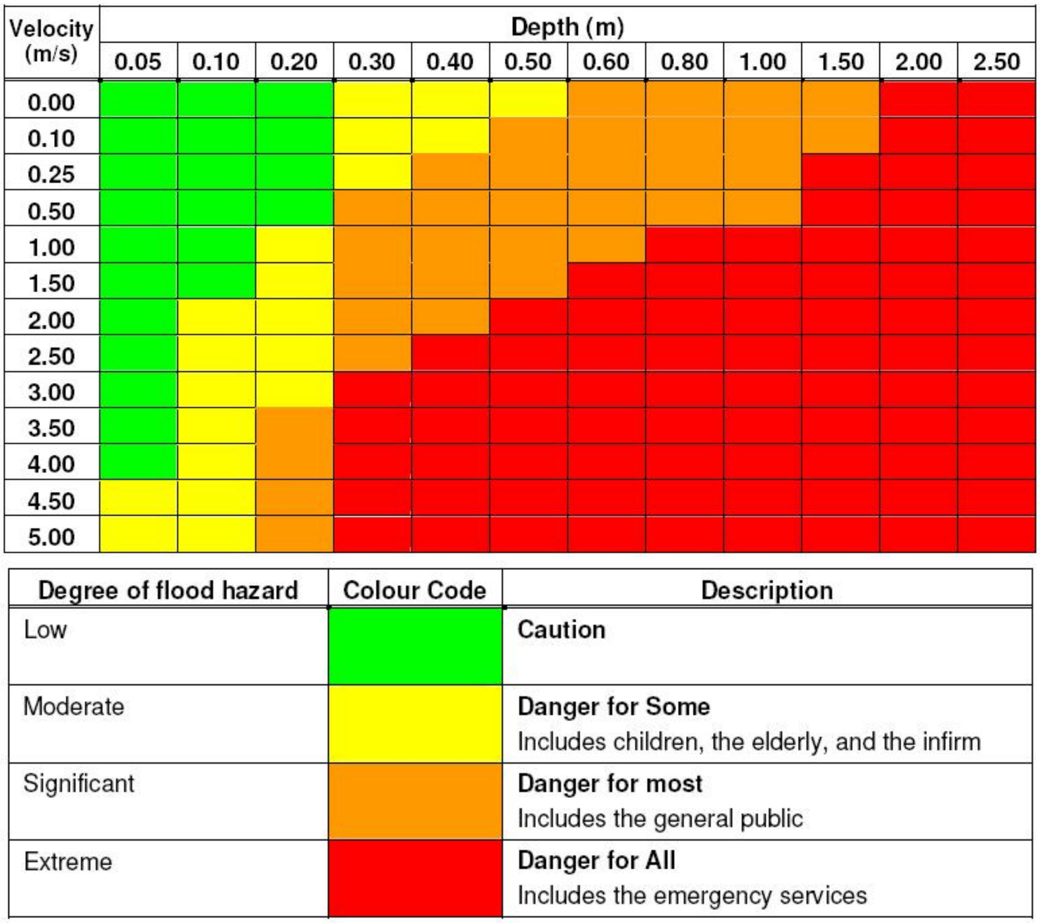
Figure 4-9 Damage-hazard intensity matrix (source: ICC Performance Code, 2018)

	Importance Categories for Buildings			
Event Magnitude	**Low**	**Normal.**	**High**	**Post-disaster**
Extreme (1:1000 years)	SEVERE	SEVERE	MODERATE	MILD
Design (1:500 years)	SEVERE	MODERATE	MILD	MILD
Routine (1:100 years)	MODERATE	MILD	MILD	MILD

Figure 4-10 Damage-hazard intensity matrix recommended for NRC PBD Guidance

The relationship of expected damage levels, hazard intensity and building use is extremely important in PBD. For example, using the matrix in Figure 4-10, for post-disaster buildings, an extreme event would be expected to create mild damage to the facility and operations within the facility. This means that no mold would be expected, and while overnight occupancy might be reduced for a few days, there is no loss of occupancy during the day, and ground-mounted equipment would not be affected by flood water. It also means that no structural damage would be expected and that only minor damage to the building envelope might occur due to impacts from flood-borne debris.

The flood hazard has been defined for Newfoundland using a flood hazard matrix developed by Royal Haskoning in the UK (RFP, 2012). This matrix is illustrated in Figure 4-11 and categorizes the flood hazard as low, moderate, significant, and extreme. The hazard description focuses on hazards to people and is based on flood parameters of depth and velocity. As suggested by the figure, any depth greater than 0.5 m creates a hazard defined as significant, and any velocity greater than 3 m/s and a depth of 0.2 m creates a hazard defined as significant. It is important to note that the flood hazard matrix is only focusing on hazards to people and not communities.



Velocity (m/s)	Depth (m)											
	0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.80	1.00	1.50	2.00	2.50
0.00	🟩	🟩	🟩	🟨	🟨	🟨	🟧	🟧	🟧	🟧	🟥	🟥
0.10	🟩	🟩	🟩	🟨	🟨	🟧	🟧	🟧	🟧	🟧	🟥	🟥
0.25	🟩	🟩	🟩	🟨	🟧	🟧	🟧	🟧	🟧	🟥	🟥	🟥
0.50	🟩	🟩	🟩	🟧	🟧	🟧	🟧	🟧	🟧	🟥	🟥	🟥
1.00	🟩	🟩	🟨	🟧	🟧	🟧	🟧	🟥	🟥	🟥	🟥	🟥
1.50	🟩	🟩	🟨	🟧	🟧	🟧	🟥	🟥	🟥	🟥	🟥	🟥
2.00	🟩	🟨	🟨	🟧	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥
2.50	🟩	🟨	🟨	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥
3.00	🟩	🟨	🟨	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥
3.50	🟩	🟨	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥
4.00	🟩	🟨	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥
4.50	🟨	🟨	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥
5.00	🟨	🟨	🟧	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥	🟥

Degree of flood hazard	Colour Code	Description
Low	🟩	**Caution**
Moderate	🟨	**Danger for Some** Includes children, the elderly, and the infirm
Significant	🟧	**Danger for most** Includes the general public
Extreme	🟥	**Danger for All** Includes the emergency services

Figure 4-11 Flood hazard index used in Newfoundland flood mapping

The PBD design process looks carefully at the expected damage levels and the performance objectives to determine what must be done to meet the performance objectives. If the building is a hospital, the performance objectives listed in Section 4.4.1.3 must be compared to the expected damage levels to determine how the hospital and its operations must be built to achieve its performance objectives. In addition, the hazard level must be reviewed to determine how robust the design must be for the performance levels to be achieved. It is possible that the service life of the hospital is 50 years, and the owners do not want more than a 2% chance that the hospital would be damaged during the flood event. Using Table 4-1 as a guide, this means the flood return period (AEP) should be 1:2000, and thus 1:2000 AEP is the design condition. The design condition should not permit any more than mild damage to a post-disaster building thus, the design should be developed such that only mild damage will occur. Risk reduction does come at a cost, and increasing return periods, and thus increasing flood protection levels, costs money. Facility owners/managers must decide how much they are willing to spend to reduce the risk of damage and loss of operation of their facility.

The Ontario Technical Guide on River and Stream Systems: Flooding Hazard Limit has a table of risk that specifies the required return period (AEP) for a project life of 50 and 100 years to achieve a specified risk level. The table (shown as Figure B-1 in the guide) is provided here as Table 4-2; risk percentages of 2% and 5% have been added using the formula provided in the guide. This table identifies the risk explicitly; Table 4-1 in this chapter illustrates risk as a function of AEP and project life.

Table 4-2 Return periods in years for various risk levels and project lives

Risk R (%)	Return Period, years	
	Project Life = 50 years	Project Life = 100 years
1	4977	9953
2	2488	4976
5	950	1900
10	475	950
25	174	348
39	100	200
50	73	145
64	50	100

Depth-damage functions may be useful in determining levels of damage for certain building types and uses vs. flood depths. In the US, the USACE and FEMA have published depth-damage functions. These have been developed for a specific storm event and developed nationally using many storm events that have occurred throughout the nation.

A comparison of hospital damage and performance for extreme events immediately suggests at least the following design parameters as listed in Table 4-3 for a damage condition not to exceed Mild.

Table 4-3 Comparison of expected damage and performance objectives for a Mild damage condition of a hospital

Specific damage	Performance level
No structural damage	Water does not damage the structure
Minor damage to building envelope caused by flood-borne debris	Building envelope may be damaged but water will not inundate the interior and cause a loss of operation
No loss of power	Equipment must be elevated or floodproofed to XX m elevation
Upper floors used as shelter	Elevation of upper floor must be above XX m elevation
Lowest patient-accessible floor remains accessible	Elevation of patient-accessible floor must be above XX m elevation
Sewage systems and potable water are not compromised by flood water	Elevation of systems must be above XX m elevation or be floodproofed to that elevation
Record storage and computer systems are not damaged by flood water	Elevation of floor must be above XX m elevation or be floodproofed to that elevation

The required design condition is then determined using the flood design guidance information described in Chapters 2 and 3. The steps in this process are:

- Locate the facility on the appropriate floodplain boundary map showing current conditions.
- Determine the flood depths and velocity from the flood maps (see Chapters 2 and No. 3 for guidance on how to determine these flood parameters).
- Determine (or decide) what flood hazard level is appropriate for the facility under consideration.
- Select a design MRI for the flood hazard level determined for the facility under consideration.
- Determine the flood elevation for the selected MRI using the method described in Chapters 2 and No. 3 that uses the current flood elevations shown on the current maps to extrapolate to a longer return period.
- Determine if climate change is to be incorporated into the flood design. If so, use maps or studies developed for the province and location of interest to determine projected flood elevations for a climate change scenario (e.g. 50 years, 100 years, 2050, 2100, or a similar time frame scenario).
- Using the flood depths and velocities for the design condition that matches the performance objective for the facility under consideration, determine the flood loads and velocities that the facility must be designed to resist to meet the objectives.
- Summarize the FCL required for all of the performance levels. Completing the step above will also establish the design flood elevation or FCL that the facility floors, equipment, and utilities must be built to in order to avoid flood water for the design condition required to meet the performance objective.

4.4.4 Acceptance Criteria

There needs to be a method for determining how to evaluate the effects of a flood in accordance with the performance objectives. The most likely effects of a flood on robust structural systems used for normal, high, and post-disaster use buildings will involve water inundation into the building. The rate of inundation, the water levels anticipated, and the effect of the water inside the building must be considered in determining whether the performance objectives can be met with the proposed flood design elements.

The criteria for the acceptance of the three categories of performance objectives take the form of (these follow the form used in the Prestandard for Performance-Based Design for Wind):

- Occupant comfort – used for expected mild damage levels which includes some water infiltration into the interior and minimal mold growth acceptable in some occupant levels
- Operational – used for mild or moderate damage levels but requires critical equipment and occupant spaces to not be inundated with water and mold growth that does not affect operations
- Continued occupancy – used for any damage level and requires critical equipment and occupant spaces to not be inundated with water and water does not create associated problems with increased humidity or mold in any critical equipment or occupant spaces

The acceptance criteria may use expected water infiltration rates as a measure of the predicted water levels over some time period. The water levels can potentially be controlled with sump pumps or other water collection and disbursement systems. Humidity and mold growth may be controlled by mechanical air movement or air drying as needed to meet performance objectives.

4.5 Provincial/City Examples

This section illustrates how PBD might be used for an imaginary hospital located in Newfoundland-Labrador. The methodology follows the suggested method in Chapter 2 using flood maps and flood hazard data for current and future (2100) conditions. It uses the method for establishing flood design conditions provided in Chapter 2, and the flood load formulas provided in Chapter 3.

4.5.1 Newfoundland-Labrador Case Study

The process of developing Performance-based Design parameters for flooding of a hospital located along the Waterford River will be described in this example. The hospital and its proposed location are fictitious but are used to illustrate how this process could be used for a performance-based flood design. A sketch of a hospital campus is shown as Figure 4-12. There are varying lower level floor elevations (LLFE) in the various buildings while the FCL is the same throughout the campus.

Another example for Alberta follows in Section 4.5.2. The process will generally follow the steps described above.

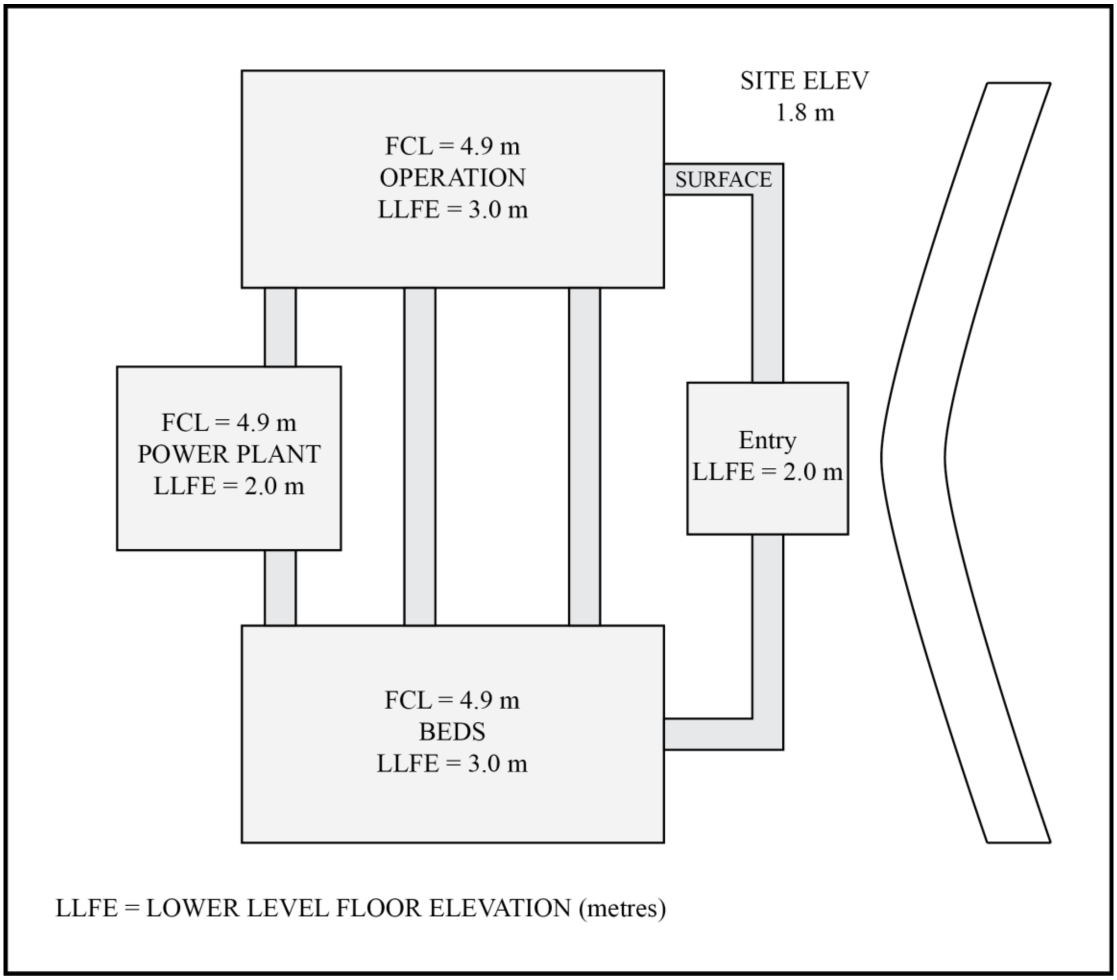


Figure 4-12 Proposed Hospital Campus Layout

Steps 1 and 2: Current Flood Design Requirements.

The current flood mapping parameters used for Newfoundland include mapping for the 1:20 AEP and 1:100 AEP events. The 1:20 AEP event defines the boundaries of the floodway where flood depths are > 1 m and velocities are > 1 m/s. The 1:100 AEP event defines the boundary of the flood fringe (the furthest extent of the flood plain) where flood depths are expected to be < 1 m and velocities are expected to be < 1 m/s.

Figure 4-13 illustrates a portion of the Waterford River near St. John's Harbor. One transect is highlighted (Transect WR 1-10) with a red arrow for study and the development of the required design information. The water surface elevation of the current flood map for the 1:20 AEP is 3.3 m. The water surface elevation for the 1:100 AEP is 3.8 m. The elevation of the ground is not clear on the flood map but topographic contours are shown and it is clear that the ground elevation is less than 5 m (contour indicated on Figure 4-13). The hospital location is shown as a red asterisk (*).

Figure 4-13 Portion of Waterford River Current Floodplain Boundary; Red arrow identifies Transect WR 1-10; Hospital shown as *

The current flood velocity information is shown in Figure 4-14. Yellow depicts a velocity of 0-1 m/s. The lighter colour orange just outside the floodway depicts a velocity of 1-2 m/s and the orange colour along the floodway depicts a velocity of 2-3 m/s.

Figure 4-14 Portion of Waterford River Velocity at 1:100 MRI Current Floodplain Conditions; Red arrow identifies Transect WR 1-10; Hospital shown as *

Steps 3, 4 and 5: Determine the Flood Hazard Level

Using Figure 4-10 (Damage-hazard Intensity Matrix), the recommended event period must be selected. This example is for a hospital which is determined to be a post-disaster building/occupancy type. Since the event is for design, the expected performance damage state after the event is classified as MILD.

In order to achieve a MILD damage state for a post-disaster hospital that has a probable design life of 50 years, it is recommended that the expected flood probability be very low. A 2% probability of an event occurring is selected and this probability level equates to a 1:2000 AEP. Using the technique for extrapolating to a larger return period from the flood elevation data provided (illustrated in Chapter 2, Section 2.14.2), the extrapolated flood elevation for a 1:2000 AEP event is 4.73 m (rounded down to 4.7 m) elevation. Figure 4-15 is a log graph illustrating the plotted flood elevations for 1:20 AEP and 1:100 AEP. The 1:2000 flood elevation is calculated using the equation shown on the graph of y = 0.3107xln(x) + 2.3693 and the result is 4.73 m.

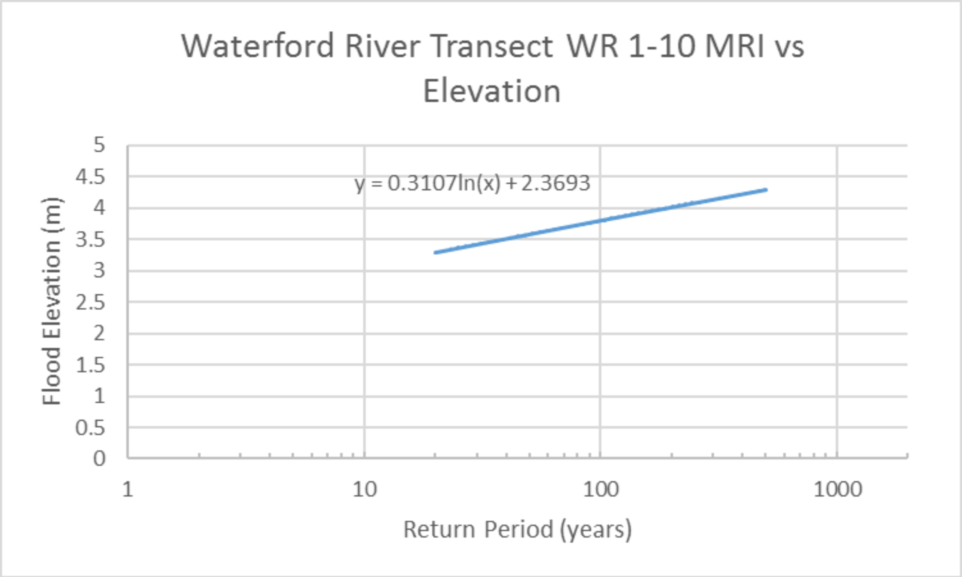


Figure 4-15 Waterford River Transect WR 1-10 Projected MRI

Without ground elevations, the flood depth cannot be determined for the 1:20 and 1:100 AEP events. However, the differences in flood loads between the 1:100 and 1:2000 AEP events can be determined so the user has some order of magnitude of the load increase that would need to be considered. The difference in water surface elevations for the 1:2000 and 1:100 AEP events is 4.7 m – 3.8 m = 0.9 m.

Step 6: Determine the Impact of Climate Change

For the Waterford River, maps have been prepared illustrating the impact of climate change by the year 2100. The projected water surface elevations for 1:20 AEP event is 3.7 m and the projected elevation for the 1:100 AEP event is 4.2 m, and these elevations are shown on Figure 4-16. Figure 4-17 illustrates the water velocity in 2100 and is depicted as 0-1 m/s in the yellow coloured area and 2-3 m/s in the orange coloured area. This mapped velocity is unchanged from the current conditions while the expected flood elevation is expected to increase by 2100. The velocity is expected to be unchanged.

These projected water surface elevations for the year 2100 were developed by an engineering consultant working on behalf of the Newfoundland and Labrador Department of Municipal Affairs and Environment (NRC, 2019). For this climate change study, the consultant used precipitation intensity-duration-frequency (IDF) values corresponding to 20- and 100-year return periods from the Ruby Line Pump House close to the watershed for the 2100-year time frame. The precipitation IDF values were converted to hyetographs and then integrated with the HEC-HMS (Hydrologic Modeling System) model to simulate hydrographs, from which the future water surface elevations were derived using HEC-RAS (River Analysis System).

There is also a flood hazard map produced for the year 2100 illustrating the flood hazard in accordance with the Royal Haskoning hazard scale shown as Figure 4-11. The flood hazard map illustrating the effects of climate change in 2100 is shown here as Figure 4-16.

Figure 4-16 Portion of Waterford River Climate Change Floodplain Boundary; Red arrow identifies Transect WR 1-10; Hospital shown as *

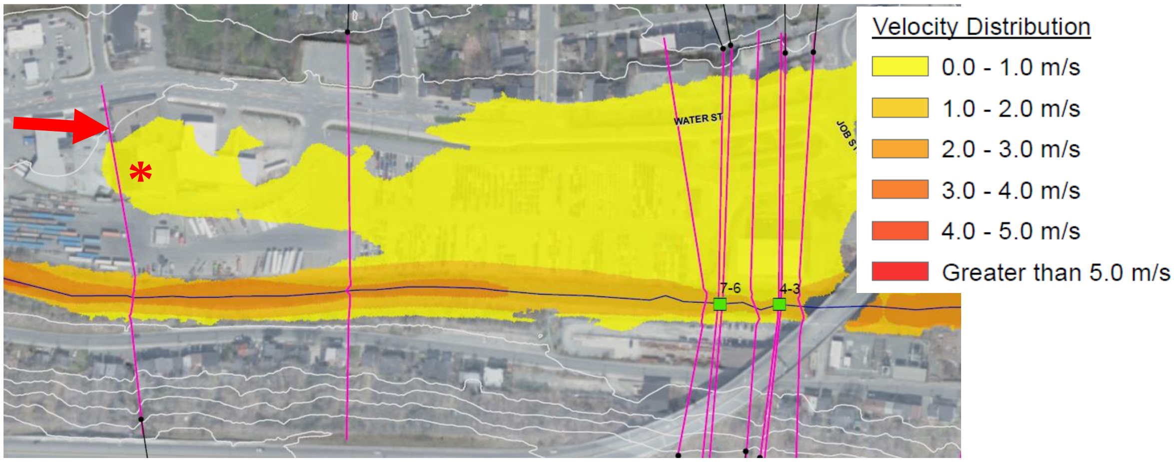


Figure 4-17 Portion of Waterford River Climate Change Velocity Distribution; Red arrow identifies Transect WR 1-10; Hospital shown as *

Using the methodology illustrated in Steps 3, 4, and 5 to determine the 1:2000 AEP water surface elevation including climate change using the elevations at the 1:20 and 1:100 AEP events, the 1:2000 AEP elevation in 2100 is expected to be 5.1 m (using the equation shown in Figure 4-19).

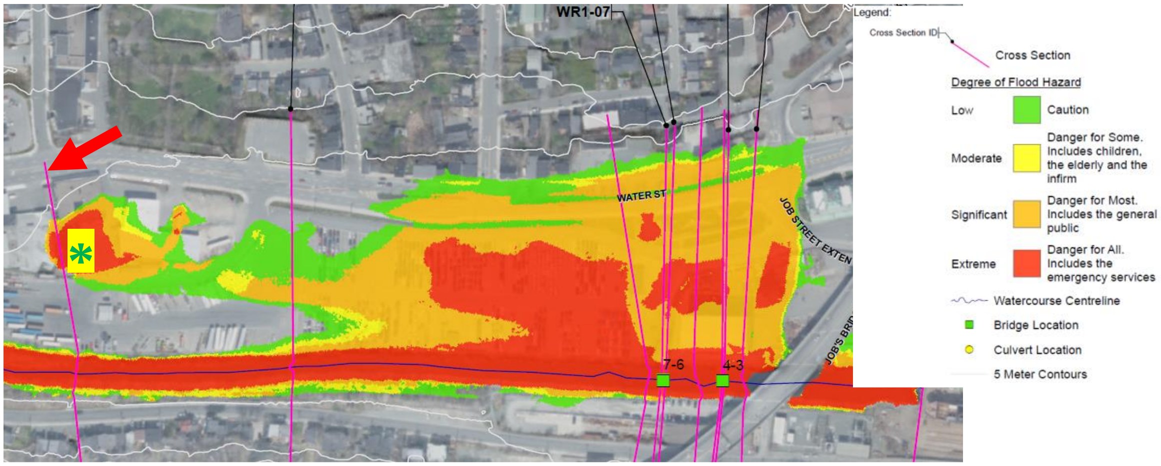


Figure 4-18 Portion of Waterford River Climate Change Flood Hazard Map; Red arrow identifies Transect WR 1-10; Hospital shown as *

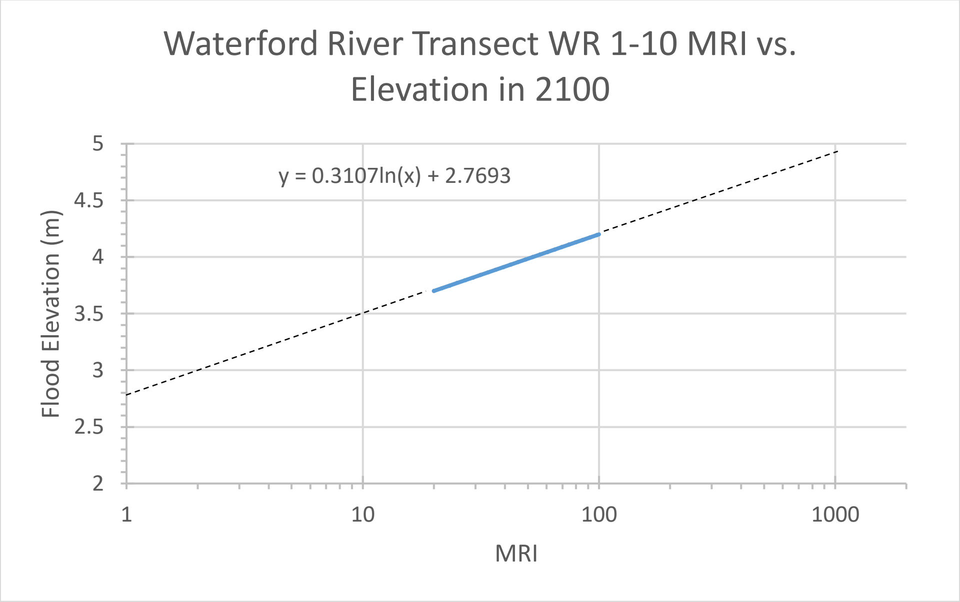


Figure 4-19 Waterford River Transect WR 1-10 Projected MRI in 2100 (Dashed lines show extension of flood elevations from MRI = 1 to MRI = 1000)

<u>Step 7: Determine Flood Loads and Velocities for Design Conditions</u>

Since we do not know the flood depth (the ground surface elevation is unknown), the difference in elevations between the performance-based design condition and the current condition will be used to determine the increase in flood loads. Using the flood load equations from Chapter 3, the hydrostatic load increase (force of standing water) is: $f = 1/2\gamma d^2$ where f is the force per unit length of wall (N/m), γ is the unit weight of water (1 g/m^3), and d is the depth of water (m).

There are several possible design conditions for this case study. The possible design conditions are organized by possible AEP flood elevations. A FCL is selected for the flood design; future conditions (FC) are added as appropriate. The possible design conditions are listed below and summarized in Table 4-5.

- FCL = 1:500 MRI (design condition per Figure 3-8)
- FCL = 1:1000 MRI + FC (future conditions for climate change)
- FCL = 1:2000 MRI + FC (future conditions for climate change)

Table 4-4 Summary of FCL for PBD Hospital Design Example

Flood Event	FCL	FC	Design FCL
1:500	4.3 m		4.3 m
1:1000	4.5 m	0.4 m	4.9 m
1:2000	4.7 m	0.4 m	5.1 m

As shown in Table 4-5, the resulting FCL for a 1:1000 event plus FC is somewhat higher than a 1:500 AEP event but only slightly less than the more restrictive 1:2000 AEP flood event. The proposed FCL for the design flood event of 1:2000 AEP plus future conditions is the recommended FCL for this design condition given the case study facility is a hospital; the design FCL is 5.1 m.

FLOOD LOADS:

- Hydrostatic:

The increase in hydrostatic load due to a 1:2000 AEP event in year 2100 is

$$f = 1/2\gamma d^2 = (0.5)x(1\ g/m^3)x(5.1^2m-3.8^2m) = 5.8 \text{ N/m} \qquad \text{(Eq. 4-1)}$$

- Hydrodynamic:

The increase in hydrodynamic loads is due to an increase in depth caused by the stagnation pressure on the upflow side of the hospital as described in Chapter 3, Section 6.2. Equation (9) provides a method for accounting for velocity less than approximately 3 m/s. The estimated flow velocity at the hospital site is in the range of 0-1m/s. The upper end of the range at 1 m/s will be used for purposes of this example.

$$\Delta D_f = aV^2/2g \qquad \text{(Eq. 4-2)}$$

For the current condition, where $V = 1$ m/s and $\alpha = 1.25$, $\Delta D_f = 1.25x12/2x9.81 = 0.064$ m, which is added to the current flood depth. Again, the flood depth is not known but this increase in depth to account for a low velocity is added to the current flood elevation so the combined velocity and flood elevation = 3.8 m + 0.064 m = 3.864 m.

For the climate change condition, $V = 1$ m/s and thus the added depth due to the low velocity flow is = 0.064 m. The flood elevation for the climate change condition at 1:2000 AEP design event is 5.1 m + 0.064 m = 5.164 m.

The increase in the combined hydrostatic and hydrodynamic load for the 1:2000 AEP event in year 2100 is

$$f = \tfrac{1}{2}x\gamma(d+d_f)^2 = (0.5)x(1g/m^3)x(5.164^2-3.864^2) = 5.87\ N/m \qquad \text{(Eq. 4-3)}$$

which is a slight increase over the load created by the increase in flood elevation only without the influence of velocity.

Step 8: Summarize the Design Conditions

The design conditions for the required performance levels for the year 2100 for this hospital are summarized in Table 4-5. It is not possible to establish the design condition for the structural frame with more precision without knowing the hospital layout, the anticipated elevation of the various hospital entrances, and the type of structural frame that will be used. If the hospital already exists, then the flood design will need retrofitting to ensure flood-resistant improvements to the structure. The most difficult design issue is stopping water from leaking into areas of the building where the building envelope has been damaged or where any crack or crevice allows water to migrate into the interior. The design solution should incorporate some method of collecting and disposing of the water that does find its way into the interior (see Technical Report on Guidelines for Improving Flood-Resistance of Existing Buildings).

Table 4-5 Comparison of expected damage, performance objectives and design requirements for MILD Damage Condition of a Hospital for Year 2100

Specific damage	Performance Objective	Design Requirement for 2100
No structural damage	Water does not damage the structure	Structural frame shall either resist flood loads for flood elevation of 5.1 m or frame is above this elevation. The frame must be able to resist an increase of 5.87 N/m wherever the highest flood elevations are expected.
Minor damage to building envelope caused by flood-borne debris	Building envelope may be damaged but water will not inundate the interior and cause a loss of operation	Water leakage into interior shall be minimized to not cause a loss of operation. Interior will be protected with a water collection and disposal system for any interior leakage. Pumps shall be sized to pump out any leaking water and keep the water level on the interior to a maximum of 25 mm (an arbitrary limit established for this example)
No loss of power	Equipment must be elevated or floodproofed to 5.1 m elevation	FCL = 5.1 m
Upper floors used as shelter	Elevation of floor must be above 5.1 m elevation	FCL = 5.1 m
Lowest patient-accessible floor remains accessible	Elevation of floor must be above 5.1 m elevation	FCL = 5.1 m
Sewage systems and potable water are not compromised by flood water	Elevation of systems must be above 5.1 m elevation or be floodproofed to that elevation	FCL = 5.1 m
Record storage and computer systems are not damaged by flood water	Elevation of floor must be above 5.1 m elevation or be floodproofed to that elevation	FCL = 5.1 m

4.5.2 Evaluating Performance Objectives

It is difficult to evaluate the expected performance of a facility affected by flooding given the nature of damage caused by flooding. Flooding causes damage principally by:

- Inundation covering a portion of the facility or equipment with flood water and rendering the facility, its contents or the equipment inoperable or wetted and damaged.

- Increasing moisture levels in the atmosphere thus potentially damaging equipment that cannot tolerate the higher moisture levels or in the absence of de-humidification equipment, higher moisture levels can create mold and mildew.

Design flood elevations are established such that no flood damage is expected when flood levels do not exceed the established elevation and it is well known that damage can be significant when those flood levels are exceeded. So even though the established design flood elevation (FCL) has included greater return periods (1:2000 AEP) and the effects of climate change in 2100, there is no guarantee that this higher flood level will not be exceeded, and thus no guarantee that no damage will occur.

Based on the table of probabilities, there is a 2% probability that the established flood elevation will be equaled or exceeded in the 50-year service life. If the probability is taken as a proxy for the possible level of damage, then the hospital would experience some level of damage or loss if the FCL is exceeded. Using this estimated probability of flood elevation being exceeded, a potential loss level can be estimated, and hospital administrators and the hospital design team can evaluate one system or operation at a time to determine if this loss level is acceptable or not, and if a loss is experienced, would the stated performance objective still be met.

There are many issues to consider in evaluating if the performance objectives can be met. For this example, in comparing the existing flood elevations and the recommended design FCL, at a minimum the following issues would require study:

- Where is the area of the campus that people can shelter (if there is one) and what is the required elevation?
- Where is the equipment located that meets the performance objectives and what is the elevation of that equipment?
- Where is access required to the facility and what is needed for access (i.e. ramps, elevator, level ground from surface drives and walkways to buildings)?
- What type of operations are included in the “Operations” Building and are they required to meet the performance objectives?
- Is there an emergency entrance, and if so, where is it?
- Where are the outdoor tanks and equipment, and how are these protected to meet the performance objectives?

The answers to these questions help define the evaluation of the performance objectives because they provide a systematic approach to addressing some portion of the stated performance objectives and requirements.

The normal flood design approach for a hospital would be to require the entire hospital site to be at or above whatever the FCL is for the desired MRI. In this case, that would mean that the entire site would need to be at or above 5.1 m. It is clear from Figure 3-11 that the site is below elevation 5 m. Creating an entire site at elevation 4 m or above would require large amounts of fill. In the US, an alternative flood protection scheme would be to either build a floodwall around the entire site, so the top is at elevation 5.1 m, or to floodproof the buildings to elevation 5.1 m. Any of these flood protection schemes are expensive. Creating a PBD solution allows for the flood protection of the portions of the hospital campus that specifically address the performance objectives while accepting the possibility that those portions of the campus that are not critical to operational performance will get wet.

Peer review is a requirement for PBD solutions. The peer review shall consist of one or more persons having the necessary knowledge to evaluate compliance of the design with performance objectives, including knowledge of expected performance, the spectrum of hazards, structural behaviour and materials of construction to determine structural resistance and performance characteristics. It must be clear to the peer reviewer how the PBD objectives are achieved and to be able to state that the design meets those objectives. It is not the responsibility of the peer reviewer(s) to redesign the protection systems or to suggest alternative design approaches; only to determine whether the performance objectives are likely to be achieved with the design, as submitted. Sometimes the design flood protection involves the installation of flood barriers or shields across openings in buildings or the protection may involve the use of sealants for closing off holes in the building envelope. These products must have some test history and certifications in order for the reviewer to know that the products will protect to the required FCL (see the Technical Report on Guidelines for Improving Flood-Resistance for Existing Buildings).

4.5.3 Alberta Case Study

The process of developing Performance-based Design parameters for flooding of a school located along the Rosebud River will be described in this example. The school and its proposed location are fictitious but are used to illustrate how this process could be used for a performance-based flood design. A sketch of a school campus is shown as Figure 4-20. A portion of the school also houses a community shelter thus may need to be constructed to a higher return period. There are varying lower level floor elevations (LLFE) in the various buildings.

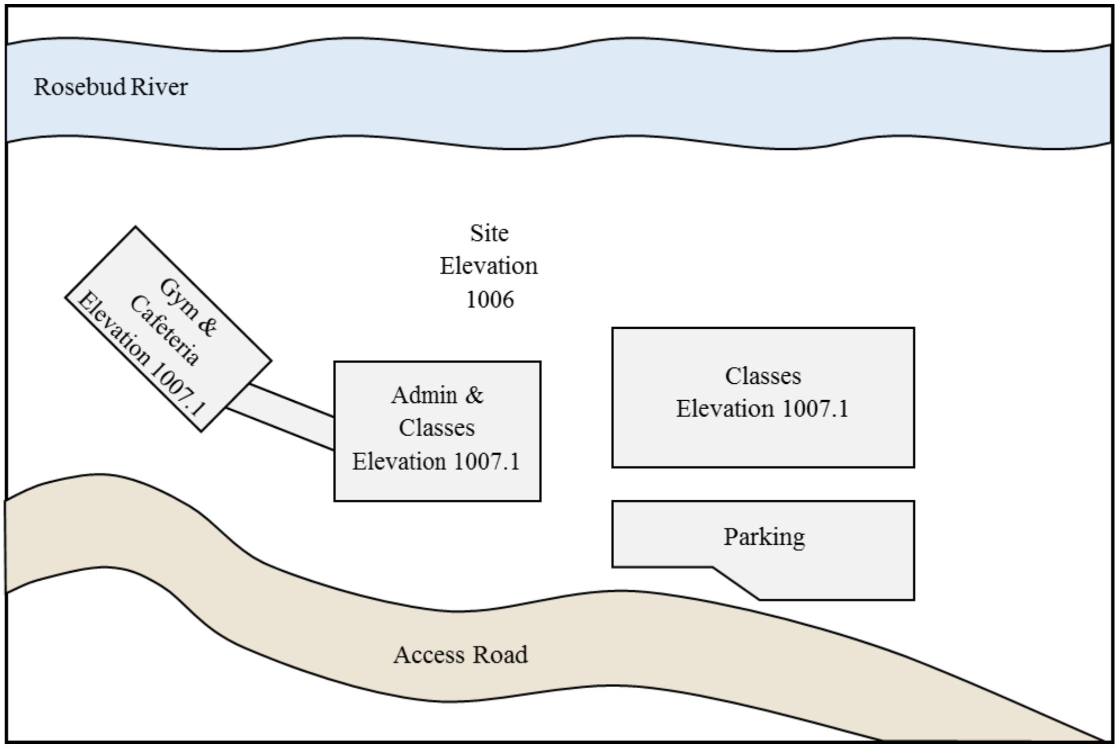


Figure 4-20 School Campus along Rosebud River

Steps 1 and 2: Current Flood Design Requirements.

The current flood study parameters used for the Rosebud River in Alberta include flood elevations for the 1:10 AEP and 1:100 AEP events. The floodway boundaries where flood depths are > 1 m and velocities are > 1 m/s are shown on flood maps included in the flood study. The 1:100 AEP event defines the boundary of the flood fringe (the furthest extent of the flood plain) where flood depths are expected to be < 1 m and velocities are expected to be < 1 m/s.

Figure 4-21 is a Google Earth image showing the river section and site location for the school. Figure 4-22 shows the transects indicated as XS 20 and XS 21. The water surface elevation of the 1:100 AEP at XS 20 is 1006.27 m. The water surface elevation for the 1:100 AEP at XS 21 is 1006.56 m. The elevation of the ground at the site is approximately 1006.0 m. The school location is shown as a black asterisk (*).

Figure 4-21 Google Earth Image of Rosebud River and School Site Location (shown as yellow star)

The width of the floodplain is approximately 76 m at XS 20. There is approximately 100 m of flat land at the site for the school. Since the site is between two cross sections, it is permitted to interpolate between these cross sections to determine the 1:100 AEP at the site. The distance between cross sections is 424 m (found in the flood study) (Didsbury, 2006). The distance from XS 20 to the site is approximately 114 m (scaled from the flood map in the study).

The interpolated design flood elevation for a 1:100 AEP event at the site would be 1006.35 m. The difference in water surface elevations between the two cross sections is 0.29 m over the 424 m cross section distance. The ratio of the site distance of 114 m from XS 20 to the difference in elevations over the cross-section distance, increases the water surface elevation at the site by 0.08 m to 1006.35 m. These differences are very small and are unlikely to influence the design in any meaningful way for this case study.

There is no information in the flood study about potential river velocities. There is, however, sufficient information in the flood study and the scaling of distances on the flood map, that an approximate cross section can be constructed. With this created cross section, the velocity determination methodology described in Chapter 2 and Appendix A can be used to estimate the river velocity. The cross section at XS 20 is shown in Figure 4-22.

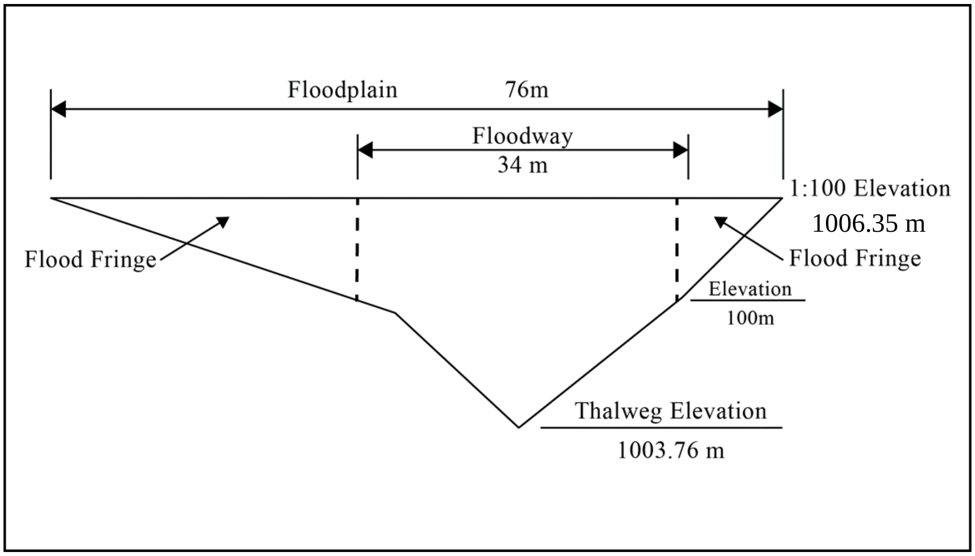


Figure 4-22 Rosebud River Profile Created for XS 20

Using the methodology in Chapter 2 and Appendix A, the velocity at XS 20 for 1:100 AEP event is very low – 0.67 m/s in floodway, 0.39 m/s in flood fringe area 1 (left side of floodway), and 0.41 m/s in flood fringe area 2 (right side of floodway). The average velocity is 0.59 m/s. The velocities are very approximate given the method of development. The distances that must be used for the river cross-section are shown on an enlarged section of the flood map showing the two cross sections used in this case study (Figure 4-23).

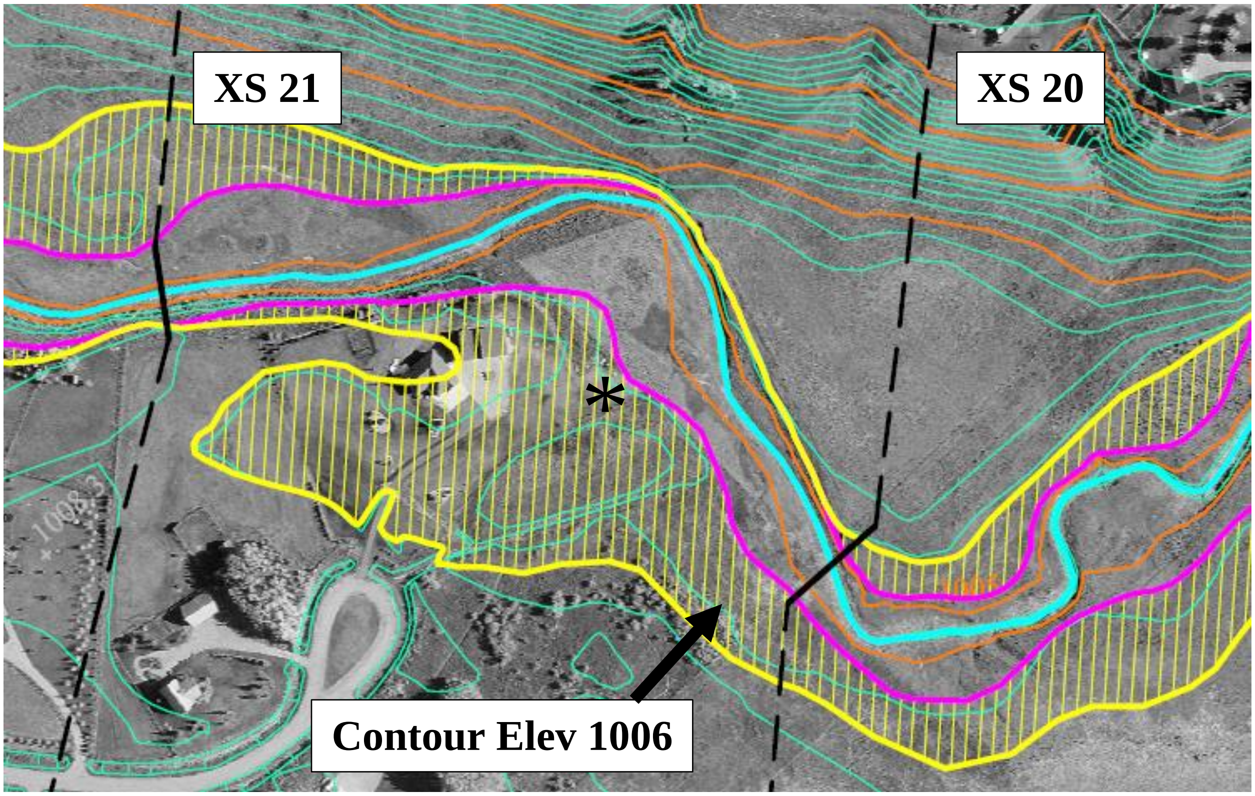


Figure 4-23 Enlarged Portion of Flood Map Showing XS 20 and 21 (school shown as *)

Using the same methodology to determine the velocities at XS 21, the velocity in floodway = 0.71 m/s, and is 0.43 m/s in the flood fringe. The average velocity is 0.65 m/s. The interpolated velocity at the site is 0.61 m/s.

<u>Steps 3, 4 and 5: Determine the Flood Hazard Level</u>
Using Figure 4-10 (Damage-hazard Intensity Matrix), the recommended event period must be selected. This example is for a school which is determined to be a high building/occupancy type. Since the event is for design, the expected performance damage state after the event is classified as MILD.

In order to achieve a MILD damage state for a high occupancy school that has a probable design life of 50 years, it is recommended that the expected flood probability be very low. A 6% probability of an event occurring is selected and this probability level equates to a 1:750 AEP, which is the recommended AEP event for a High occupancy building. Using the technique for extrapolating to a larger return period from the flood elevation data provided (illustrated in Chapter 2, Section 2.14.2), the extrapolated flood elevation for a 1:750 AEP event is 1006.63 m elevation at XS 20 and 1006.85 m at XS 21. The design flood elevation at the site for a 1:750 AEP is interpolated to be 1006.69 m. The ground elevation at the site is approximately 1006 m, so the design flood depth is 0.69 m.

Step 6: Determine the Impact of Climate Change

There is no information about the impact of climate change in this area of Alberta. However, climate change should be assumed to affect the site over time. An additional 0.4 m will be added to the FCL to account for future conditions.

Step 7: Determine Flood Loads and Velocities for Design Conditions

Using the flood load equations from Chapter 3, the hydrostatic load at the site (force of standing water) is: $f = 1/2\gamma d^2$ where f is the force per unit length of wall (N/m), γ is the unit weight of water (1 g/m^3), and d is the depth of water (m).

The recommended design condition for this case study is for a 1:750 AEP. A FCL is selected for the flood design; future conditions (FC) are added as appropriate. The possible design conditions are listed below and summarized in Table 4-6.

- FCL = 1:750 MRI (design condition per Figure 3-8)
- FCL = 1:750 MRI + FC (future conditions for climate change)

Table 4-6 Summary of FCL for PBD School Design Example

Flood Event	FCL	FC	Design FCL
1:750	1006.69 m		1006.69 m
1:750	1006.69 m	0.4 m	1007.09 m

As shown in Table 4-6, the resulting FCL for a 1:750 event without FC does not meet the definition of design flood depth recommended in Chapter 2 (Section 2.18). The proposed FCL for the design flood event of 1:750 AEP plus future conditions is the recommended FCL for this design condition. The recommended design FCL is 1007.09 m, or rounded to 1007.1 m. The design flood depth is therefore, 0.69 + 0.4 = 1.09 m, rounded up to 1.1 m.

FLOOD LOADS:

• Hydrostatic: The hydrostatic load due to a 1:750 AEP event that includes future conditions is

$$f = 1/2\gamma d^2 = (0.5)x(1\ g/m3)x(1.12m)^2 = 0.6\ N/m \qquad \text{(Eq. 4-4)}$$

- Hydrodynamic:

The increase in hydrodynamic loads is due to an increase in depth caused by the stagnation pressure on the upflow side of the school as described in Chapter 3, Section 6.2. Equation (3-9) provides a method for accounting for velocity less than approximately 3 m/s. The estimated flow velocity at the school site is estimated to be 0.61 m/s.

$$\Delta D_f = aV^2/2g \qquad \text{(Eq. 4-5)}$$

where:

a is a drag coefficient
V = velocity (m/s)

g = gravitational constant 9.81 m/s^2

For the design condition, where V = 0.61 m/s and α = 1.25, ΔD_f = 1.25x0.612/2*9.81 = 0.02 m, which is added to the design flood depth. The depth that accounts for a low velocity is added to the flood depth so the combined velocity and flood elevation = 1.1 m + .02 m = 1.12 m.

The revised hydrostatic load that includes the hydrodynamic effect is

$$f = 1/2\gamma d^2 = (0.5)x(1\ g/m^3)x(1.12m)^2 = 0.63\ N/m \qquad \text{(Eq. 4-6)}$$

Step 8: Summarize the Design Conditions

The design conditions for required performance levels for future conditions for this school are summarized in Table 4-7. It is not possible to establish the design condition for the structural frame with more precision without knowing the school layout, the anticipated elevation of the various entrances, and the type of structural frame that will be used. If the school already exists, then the flood design will need retrofitting to ensure flood-resistant improvements to the structure. The most difficult design issue is stopping water from leaking into areas of the building where the building envelope has been damaged or where any crack or crevice allows water to migrate into the interior. The design solution should incorporate some method of collecting and disposing of the water that does find its way into the interior (see Technical Report on Guidelines Improving Flood-Resistance for Existing Buildings).

Table 4-7 Comparison of expected damage, performance objectives and design requirements for MILD Damage Condition of a School for Future Years

Specific damage	Performance Objectives	Design Requirements for Future
No structural damage	Water does not damage the structure	Structural frame shall either resist flood loads for flood elevation of 1007.1 m or frame is above this elevation. The frame must be able to resist 0.63 N/m wherever the highest flood elevations are expected.
Minor damage to building envelope caused by flood-borne debris	Building envelope may be damaged but water will not inundate the interior and cause a loss of operation	Water leakage into interior shall be minimized to not cause a loss of operation. Interior will be protected with a water collection and disposal system for any interior leakage. Pumps shall be sized to pump out any leaking water and keep the water level on the interior to a maximum of 25 mm (an arbitrary limit established for this example)
No loss of power	Equipment must be elevated or floodproofed to 1007.1 m elevation	FCL = 1007.1 m
Upper floors used as shelter	Elevation of floor must be above 1007.1 m elevation	FCL = 1007.1 m
Lowest accessible floor remains accessible	Elevation of floor must be above 1007.1 m elevation	FCL = 1007.1 m
Mechanical, electrical, plumbing, and restrooms systems must remain operational	Elevation of floor must be above 1007.1 m elevation	FCL = 1007.1 m
Sewage systems and potable water are not compromised by flood water	Elevation of systems must be above 1007.1 m elevation or be floodproofed to that elevation	FCL = 1007.1 m

4.5.4 Evaluating Performance Objectives

It is difficult to evaluate the expected performance of a facility affected by flooding given the nature of damage caused by flooding. Flooding causes damage principally by:

- Inundation covering a portion of the facility or equipment with flood water and rendering the facility, its contents or the equipment inoperable or wetted and damaged.
- Increasing moisture levels in the atmosphere thus potentially damaging equipment that cannot tolerate the higher moisture levels or in the absence of de-humidification equipment, higher moisture levels can create mold and mildew.

Design flood elevations are established such that no flood damage is expected when flood levels do not exceed the established elevation and it is well known that damage can be significant when those flood levels are exceeded. So even though the established design flood elevation (FCL) has included greater return periods (1:750 AEP) and the effects of climate change have been considered, there is no guarantee that this higher flood level will not be exceeded, and thus no guarantee that no damage will occur.

Based on the table of probabilities, there is a 6% probability that the established flood elevation will be equaled or exceeded in the 50-year service life. If the probability is taken as a proxy for the possible level of damage, then the school would experience some level of damage or loss if the FCL is exceeded. Using this estimated probability of flood elevation being exceeded, a potential loss level can be estimated and school administrators and the school design team can evaluate one system or operation at a time to determine if this loss level is acceptable or not, and if a loss is experienced, would the stated performance objective still be met.

There are many issues to consider in evaluating if the performance objectives can be met. For this example, in comparing the existing flood elevations and the recommended design FCL, at a minimum the following issues would require study:

- Where is the area of the campus that people can shelter (if there is one) and what is the required elevation?
- Where is the equipment located that meets the performance objectives and what is the elevation of that equipment?
- Where is access required to the facility and what is needed for access (i.e. ramps, elevator, level ground from surface drives and walkways to buildings)?

The answers to these questions help define the evaluation of the performance objectives because they provide a systematic approach to addressing some portion of the stated performance objectives and requirements.

The normal flood design approach for a school would be to require the entire site to be at or above whatever the FCL is for the desired MRI. In this case, that would mean that the entire site would need to be at or above 1007.1 m. It is clear from Figure 4-23 that the site is approximately at 1006 m, thus the site would need to be built up to 1007.1 m, an increase of 1.1 m. Creating an entire site at elevation 1007.1 m or above would require large amounts of fill. In the US, alternative flood protection schemes would be to either build a floodwall around the entire site so the top is at elevation 1007.1 m, or to floodproof the buildings to elevation 1007.1 m. Any of these flood protection schemes is expensive. Creating a PBD solution allows for the flood protection of the portions of the school campus that specifically address the performance objectives while accepting the possibility that those portions of the campus not critical to operational performance will get wet.

4.5.5 Gaining Acceptance for PBD

PBD should be an alternative method that could be used for design. If the PBD performance objectives are evaluated and found to be met by the flood design, and if the performance of the flood design can be shown to be comparable to a prescriptive flood solution, this should gain acceptance/compliance by the Authority Having Jurisdiction (AHJ). A peer review shall be submitted to the AHJ along with the design documents.

In many ways, flood PBD exceeds the prescriptive approach as the design flood levels will exceed the required minimums even though not all parts of a facility may be protected to the same level.

4.6 PBD Design Compared to Prescriptive Flood Design

A comparison of the PBD flood design discussed above for a school in Alberta along the Rosebud River will be made with a prescriptive approach for meeting the flood regulations for the case study school.

4.6.1 Prescriptive Approach to Alberta Case Study

There was no information available on flood ordinances or flood design requirements for any community along the Rosebud River that could be used for a prescriptive comparison, so ASCE 24 is used to provide a comparative basis for design.

ASCE 24 (see Figure 2-27 in this report) uses Flood Design Class instead of occupancy or use to classify buildings with respect to the flood hazard. Flood Design Class 3 is similar to the NBC High Importance Category used for this school. ASCE requires that the minimum elevation of the top of the lowest floor be set at the Base Flood Elevation (BFE) plus 1 ft (0.3 m). The BFE in the US is the 1:100 AEP flood elevation.

The interpolated 1:100 AEP at the school site using the flood map is estimated to be 1006.35 m. This represents the BFE as defined by ASCE 24. The flood design elevation for all of the lowest floors for the entire school would be 1006.35 + 0.3 = 1006.65 m. The ground elevation is 1006 m therefore either fill must be placed in the floodplain to build up the site by 0.65 m or the floor systems must be placed on crawl space foundations that elevate the top of the lowest floor to elevation 1006.65 m.

By contrast with PBD, the design is based on an evaluated risk of damage of 6% which equates to a 1:750 AEP event for this school. The flood design level is thus 1007.1 m, which is 0.45 m higher than the prescriptive design level. The PBD approach is to allow the floors to be at ground level while elevating the important mechanical systems and crucial shelter spaces to the 1007.1 m level. This allows students and staff to be able to enter/exit the lowest floors without stairs while protecting the building services from damage. If the prescriptive flood design requirement was to elevate to 1007.1 m, then either fill or a building foundation would be required to bring the floor of the buildings up 1.1 m to the design flood elevation at a substantial increase in cost over the PBD design method.

5 Summary and Recommendations

The guidance included in this report is written to provide a national standard for flood design so all designers across the country can use a standardized approach to determining flood loads based on a consistent risk in accordance with the Importance Categories for Buildings table in the NBC. There is much diversity in how provinces treat the flood hazard in terms of both the annualized exceedance probability (AEP) used in design and in how the flood hazard information is displayed to the public. This guidance provides a method to convert existing AEP flood elevations to a higher (and recommended design level) AEP flood elevation.

The recommended flood design levels, as described above, are linked to the Importance Categories for Buildings Table in the NBC. The recommended design levels for those buildings covered by the NBC are:

Importance Category	Recommended Flood Design AEP
Low	1:100
Normal	1:500
High	1:750
Post-disaster	1:1000

There is significant coverage of probabilities of flood events and the associated risk of those events over various time frames in this report. The selected design AEPs are based on reducing the risk of flood damage to the various NBC building importance categories, as shown in **Table 2-6 Flood Probability for Event AEP vs. Time Period.** For example, the exceedance probability in 50 years for a 1:100 AEP event is 39%, for a 1:500 AEP is 10%, for a 1:750 AEP is 6%, and for a 1:1000 AEP is 5%. Decisions about flood protection levels should be made based on risk and cost. Recommending the design levels above will help ensure that protection from the flood hazard will be risk consistent across the country.

The primary determinant of flood loads is flood depth and flood velocity yet neither of these parameters is easily found on Canadian flood maps. There has been methodology created to address both these issues and a method for determining these parameters is recommended. Examples have also been included illustrating how to use these methodologies. Formulas and provisions have been included for determining flood loads in riverine, coastal, and Great Lakes locations. It must be stated that methods and formulas for determining flood loads are only as good as the information available to use for development of the design information. Old flood maps, out of date hydraulic methods, lack of recent topographical information, or the use of old datums will all contribute to some disparity in the results suggested in this guidance.

Guidance is provided on how to determine the flood hazard that includes:

- Finding the flood depth
- Finding or determining the flood velocity
- Determining the likely flood duration
- Determining the potential for flood debris
- Consideration of erosion and scour at flood sites
- Finding the critical wave height in coastal locations

Flood load formulas are provided for the following load conditions:

- Hydrostatic forces on vertical walls due to standing water
- Hydrostatic forces on horizontal surfaces due to submergence (buoyancy)
- Hydrodynamic forces due to moving water
- Wave forces due to waves impacting building foundation elements (piles, columns, walls)
- Wave run up on steep sloped coastlines
- Debris impact loads for various types of flood debris including ice

The recommendations for determining the design Flood Construction Level (FCL) for riverine and coastal flood plains are:

The FCL for riverine conditions should include:

Design flood elevation for a specified MRI (or AEP) + Future conditions that increase the flood elevation such as sea level rise or increased development that increases surface run off

The FCL for coastal conditions should include:

Design flood elevation for a specified MRI (or AEP) + Wave effects above the design stillwater elevation + Future conditions that increase the flood elevation such as sea level rise or increased development that blocks more flood flow ashore

This guidance also includes a chapter on performance-based design for flood which attempts to bring in an alternative method for improving flood-resistance without necessarily adhering to the prescriptive recommendations offered here. This PBD guidance focuses on how flood resistance can be achieved by focusing on building or operational performance. This method is supported by two case studies in two different Canadian locations.

Appendix D is a summary of recommended design conditions and flood load formulas to include in the Guide. While not written out in full at this time, the recommended commentary language would include additional information about the flood condition or formula that would further explain how to use the provision and under what circumstances they would apply.

The flood hazard determination methods and the flood load formulas have also been used in the report on Guidelines for Improving Flood-Resistance for Existing Buildings.

There are many possible approaches to some aspects of flood design; there are many different equations depending on the flood conditions, and there are many variations on which parameters to use. Throughout this guide, the recommended methods, formulas, or approaches are those that are considered best practices and those that are more easily applied by practitioners.

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APPENDICES

Appendix A MRI Scaling Method

In this Appendix, methods are presented for scaling the flood elevation from a known MRI basis, for example at a100-year level, to a design basis, for example at the 1000-year level. The goal is to present a methodology that can potentially be used in any Province to adjust from a known water surface elevation to a desired water surface elevation, using only basic information contained in flood maps or prior flood studies.

The method of MRI extrapolation using scaling factors is being recommended for use across Canada for two primary reasons:

1. Actual water surface elevations for rivers, lakes, and coastlines for higher MRIs across the country either do not exist or are difficult to find and would burden any user if such a search of elevations for each MRI of interest was required, and
2. This report is recommending the use of higher MRI's of 100, 500, 750, and 1000-years which in many cases are higher than those currently used for flood mapping purposes in many provinces.

The scaling methods discussed here should not be used if a Province or Approving Authority conducts a flood hazard study to obtain water surface elevations for these higher return levels based on either an empirical analysis of historical water level data from gauge records or a numerical analysis of water levels. The methods outlined here would only be used in cases with limited data where approximate values of 500, 750, and 1000 yr water levels are to be obtained using flood hazard maps of flood studies that may only list water levels at much lower recurrence intervals.

For this report, flood studies were collected for coastal, Great Lakes and riverine locations across Canada to obtain water surface elevation for flood events covering a range of MRI's. But the number of flood studies that could be located with flood elevations over a range of return periods is limited; often studies listed flood elevations for only one or two MRI's and not a wider range.

Given the difficulty in finding information on flood levels in Canada, the limited Canadian studies were supplemented with additional data for regions of the U.S. adjacent to Canada. This data from U.S. flood studies is not intended to replace Canadian data but is used here to as an interim way to establish the methodology until more data becomes available in Canada. Even so, the results from U.S. sites are compared to the more limited sites found in Canada, and results are broadly consistent.

A.1 Background of MRI Scaling Method

Return periods of flood events are often plotted in a format with flood level, S, as a function of mean recurrence interval, MRI, using a semi-log format with S plotted as a function of the natural logarithm of MRI, or ln(MRI). An example from a riverine site in Alberta (Didsbury, 2006) is given in Figure A-1 to illustrate, showing results at two different river transects.

Data points used in this type of analysis usually come from two sources. First, if data is available from stream or river gages, coastal tide gages, or lake level gages, the measured values can be plotted based on the plotting position method to reflect historical occurrences of extreme flood events. This is especially useful if long term gage records are available, though very few locations contain long term records. A second method is to conduct a numerical analysis in which expected extreme water levels are simulated from rainfall intensity-runoff modeling, or from coastal (or lake) storm surge modeling.

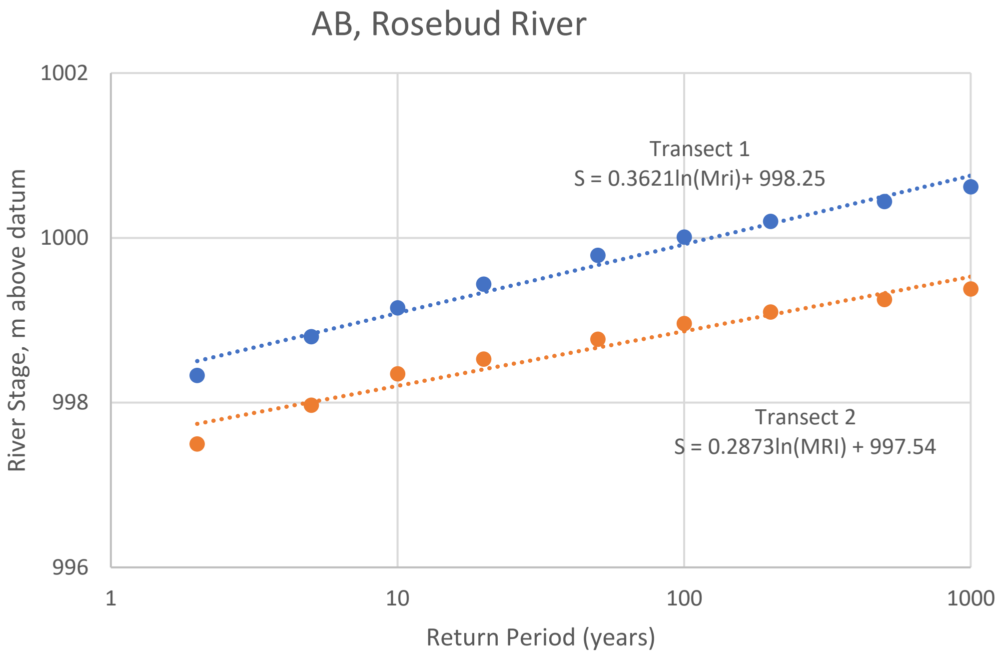


Figure A-1 Graph of Flood Event Return Periods for Alberta, Rosebud River (NHC, 2006)

A common problem with either type of analysis is that flood events may not be determined for return periods appropriate for flood hazard reduction. For example, it is common to extrapolate data to the 100-year level, or to conduct model studies to establish the 100-year level, but to not extrapolate to higher MRI events. As noted in Chapter 2, it is recommended that design flood events be based on MRI's of 100, 500, 750 and 1000 years for different classes of buildings.

When extreme value data are plotted in the format shown in Figure A-1, the data often plot as nearly a straight line with the water level at any MRI given as

$$S = a + b \ln(MRI) \qquad \text{(Eq. A-1)}$$

where:

S = water surface elevation, in m (ft) from a defined datum.
MRI = mean recurrence interval, in years (inverse of AEP)
a = water level intercept when MRI=1 yr
b = slope

The degree to which data follow a straight line at higher MRI values depends on the low-probability tail of the extreme value probability distribution. If sufficient data exists, or if sufficient information is available from a numerical study, then standard extreme value distributions may be fitted to the data. Examples would include a Generalized Extreme Value distribution, or commonly used distributions such as Weibull, Gumbel, log-normal, etc. With an extreme value distribution that best fits the data, results can then be extrapolated to higher MRI values. Confidence limits may also be included. This would be the preferred method of establishing design flood elevations for the 500 to 1000 year values when data or numerical results exist for this analysis.

But in many instances, a straight-line fit of the data is reasonable over a wide range of MRI values. Two examples are shown below. The first shows levels in Lake Superior at Duluth, MN (Figure A-2) showing that the straight-line fit may reasonably describe conditions in the Great Lakes (Kriebel, personal communication). The second example, for flood levels for the North Saskatchewan River, AB (Figure A-3), shows that the same approach applies reasonably well to river water surface elevations. The small return period, i.e. 2-year, water surface elevations tend to not follow the straight line quite as well as other return period data points.

In instances where data or simulation results are more limited, or when water level values are only known from a flood hazard map, it is likely that a user may only know two points on the S-vs-ln(MRI) plot. In many Provinces, a user would be able to obtain the 20 and 100 year MRI flood elevations, as these are commonly reported. In other Provinces, values may be known at the 2 yr, 10 yr, 20 yr, 50 yr, 200 yr or 500 yr levels from flood maps or older flood hazard studies. For any of these, a straight line of the form given in equation A-1 can be fit through the two points to define a and b, and the equation can then be used to extrapolate to other MRI values.

In cases where water level values are known for two MRI values, equation A-1 can be re-written in an alternate form in terms of the two known values as

$$S_{MRI} = S_{R1} + \alpha\,(S_{R2} - S_{R1}) \qquad \text{(Eq. A-2)}$$

where:

S_{MRI}= water level at return period of interest
S_{R1} = water level at lowest reference return period known, MRI_1
S_{R2} = water level at highest reference return period known, MRI_2
α = MRI scaling factor based on the two reference return periods

The scaling factor α can then be tabulated for different combinations of the two reference MRI values. In practice, the scaling parameter can be found in one of two ways as outlined below: (1) using theoretical relationships or (2) using empirical relationships.

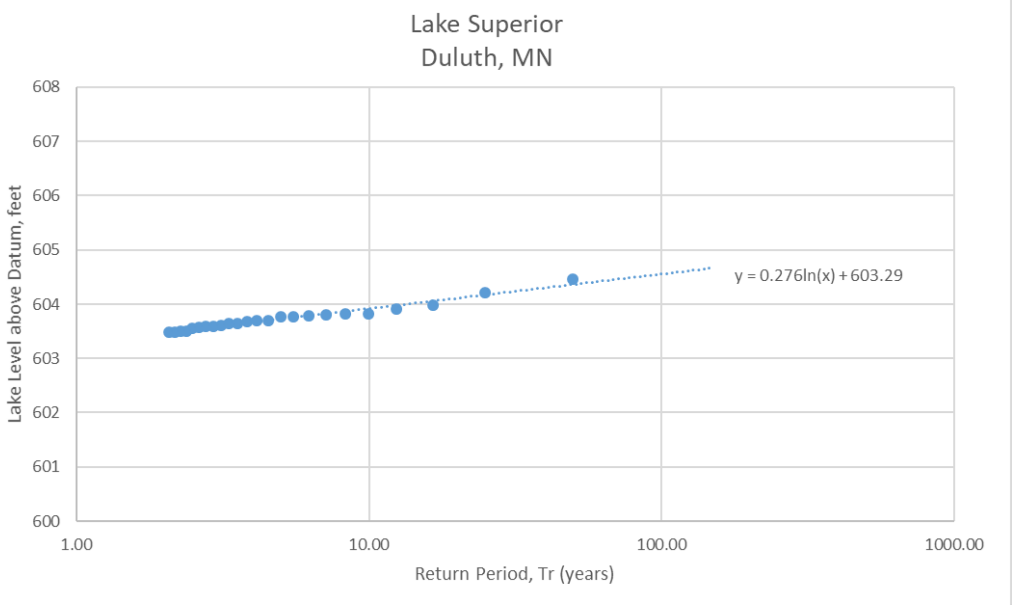


Figure A-2 Graph of Lake Superior levels from NOAA gage 9099064 Duluth, MN

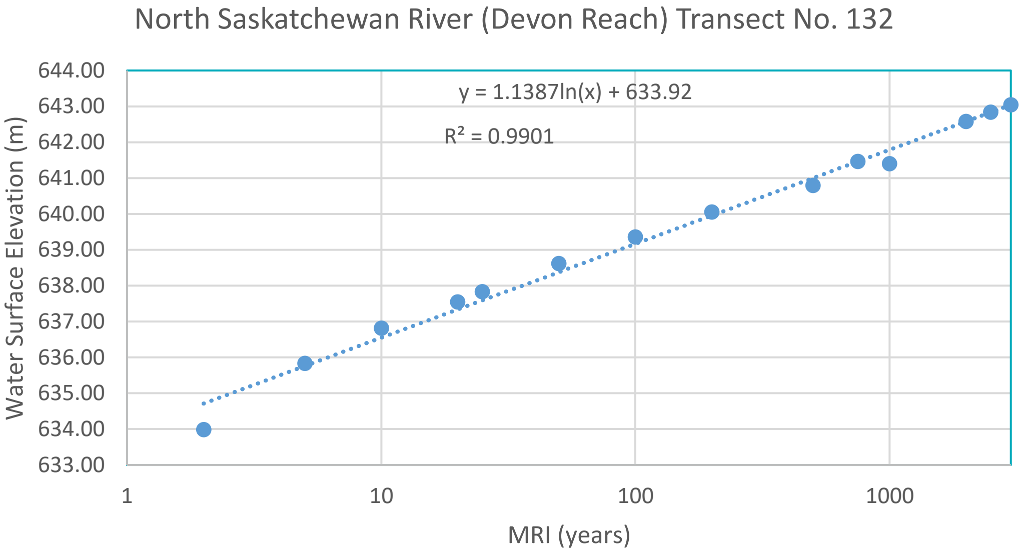


Figure A-3 Water Surface Elevations for North Saskatchewan River (Devon Reach), AB (North Saskatchewan River Flood Risk Mapping Study, 2007)

A.2 Theoretical Scaling Parameters

For the straight line in the S-vs-ln(MRI) format, the scaling factor is given theoretically as a function of two known and one desired MRI values as

$$\alpha \ = (\ln(MRI) - \ln(MRI_1)) \ / \ (\ln(MRI_2) - \ln(MRI_1)) \qquad \text{(Eq. A-3)}$$

For example, if water level values are known for 20 and 100 year MRI levels, and if the 500-year value is desired, then the theoretical scaling value is a = (ln(500)-ln(20))/(ln(100)-ln(20)) = 2.0.

Theoretical values of the scaling parameter are given in Table A-1. To develop this table, the higher reference elevation SR2, is adopted for a 100-year MRI, so that MRI2 = 100. This is used here because the 100-yr value is the most commonly identified flood level across the Provinces. The table then gives values of the scaling parameter a for different lower reference levels SR1 with return periods of 2, 5, 10, or 20 years.

Table A-1 Theoretical values of MRI scaling parameters

	Reference Level			
MRI	**2 year**	**5 year**	**10 year**	**20 year**
2	**0.00**			
5	**0.23**	**0.00**		
10	**0.41**	**0.23**	**0.00**	
20	**0.59**	**0.46**	**0.30**	**0.00**
50	**0.82**	**0.77**	**0.70**	**0.57**
100	**1.00**	**1.00**	**1.00**	**1.00**
200	**1.18**	**1.23**	**1.30**	**1.43**
500	**1.41**	**1.54**	**1.70**	**2.00**
750	**1.52**	**1.67**	**1.88**	**2.25**
1000	**1.59**	**1.77**	**2.00**	**2.43**
2000	**1.77**	**2.00**	**2.30**	**2.86**

It is noted that the value of α is 0.0 when evaluated at the lowest reference water level, 1.0 when evaluated at the higher reference water levels, and greater than 1.0 when evaluated at MRI values above the higher reference level. The scaling also is strongly dependent on the lower reference level.

As an example of how to apply these values, assume that for a river site a flood map indicates that the 20-year level is S_{20}=106.4 m while the 100-year level is S_{100}=107.2 m. For the 20-year lower reference level, from the last column in Table A-1, the 500-year flood level would be estimated with α = 2.00 so that as $S_{500}=106.4 + 2.00\,(107.2-106.4) = 108.0$ *m*. The 1000-year flood level would be estimated with α = 2.43 so that as $S_{1000}=106.4 + 2.43\,(107.2-106.4) = 108.3$*m.*

A.3 Empirical Scaling Parameters

A second way of defining the scaling parameter is to use data, either measured values or numerically generated values, to define empirical values of α. These would implicitly account for any curvature in the S-vs-ln(MRI) plot as the empirical values of α may differ from the theoretical values based on a straight line fit. From equation A-2, the empirical parameters based on known water levels are given by

$$\alpha = (S_{MRI} - S_{R1}) \,/\, (S_{R2} - S_{R1}) \qquad \text{(Eq. A-4)}$$

Locations for riverine, coastal and Great Lake conditions were studied in order to determine if one set of empirical scaling factors could be used in any of these three general locations. It was found that while many values are similar to the theoretical values in Table A-1, some differences exist so that separate empirical values are given here for Great Lakes, coastal regions, and for riverine conditions.

In the following, the empirical scaling values again adopt a 100-yr upper reference, consistent with the fact that the 100-year level is the most commonly used reference level in the flood studies and flood maps reviewed. The lower reference frequently differs from location to location or from study to study (or map to map). Results are therefore given for a range of possible lower reference values.

In addition, the empirical values can be analyzed over several data sets to establish uncertainty bands or confidence limits. In this study, data were collected across Canada and the US regions adjacent to Canada, in order to develop the mean and standard deviation of the empirical scaling factors. The recommended factors have been then increased based on the standard error to represent a 95% upper confidence level.

A.3.1 Great Lakes Locations

Water level elevations at a range of MRI values for the Great Lakes are difficult in locate in Canada. The scaling factor for the Great Lakes therefore uses US Flood Insurance Study (FIS) data for 53 sites along the US side of the Great Lakes. There was little difference in behaviour from lake to lake, or from site to site on each lake, and it was therefore assumed that water level conditions on the Canadian side of each lake would behave similarly to those on the U.S. side. The FIS reports for all 53 sites included still water elevations without freeboard for four return periods 10-year, 50-year, 100-year, and 500-year, all based on numerical modeling studies. All values were referenced as water level above the chart datum of each lake, defined in Table A-2.

Table A-2 Great Lakes chart datums used for reference

Lake	Chart Datum (m) IGLD85	Chart Datum (ft) IGLD85
Superior	183.2	601.1
Michigan	176.0	577.5
Huron	176.0	577.5
St. Clair	174.4	572.3
Erie	173.5	569.2
Ontario	74.2	243.3

Empirical scaling factors are determined using the 100-year levels as the upper reference value, and with two possibilities the lower reference level. First, scaling values were based on the lake chart datum (CD) and equation A-4 was solved directly for the scaling parameter α. In this case, flood levels can be redefined as elevation above the zero chart datum so that the lower reference level is $S_{R1}=0$. From equation A-4, α is simply given as the ratio of the water level at any return period divided by the 100-year value, as $\alpha = S/S_{100}$. Second, scaling parameters were developed from equation A-4 using the lowest reference level used in the FIS, at the 10-year level.

For each of the 53 sites, the empirical scaling parameters were first found for the 10, 50, 100, and 500 year MRI values. These were then used in a regression analysis to interpolate or extrapolate to other MRI values not in the data set. The mean and standard deviation of the scaling parameters were then found from the results of the 53 sites. Figures A-4 and A-5 illustrate the procedure, showing the mean values of the scaling parameter from the 53 sites and showing the mean regression equation.

Results of this analysis are given in Tables A-3 and A-4. Results are then tabulated for the two methods of analysis: (1) using the lake chart datum in Table A-3 and (2) using the 10 year water level as reference in Table A-4. The tables show the mean value of α as well as the standard deviation in the values of α from the 53 sites. The last column gives recommended values of α (in bold text) based on a 95 percent upper confidence limit.

Results indicate that if the CD of each lake is used as a zero reference, the 500-yr water level for the 95% confidence limit is 1.14 times the 100-year level. The 750 and 1000 year levels would then be 1.17 and 1.20 times the 100-year value respectively. If the 10-year water level is used as reference, then from equation A-2 the 500 year level would be given as $S_{500} = S_{10} + 1.67*(S_{100}-S_{10})$, with scaling values of 1.84 and 1.96 for the 750 and 1000-year values respectively.

A comparison of these empirical values using the 10-year reference, to the theoretical values for the same reference, repeated from Table A-1, shows that for higher return periods, the empirical values are somewhat smaller than the theoretical values. The theoretical values would then give a somewhat conservative overestimate of the design water levels for 500, 750, and 1000 yr return levels for the Great Lakes. For the 53 Great Lakes sites, the standard deviation is low at all MRI levels so the upper confidence limit is close to the computed mean value of α.

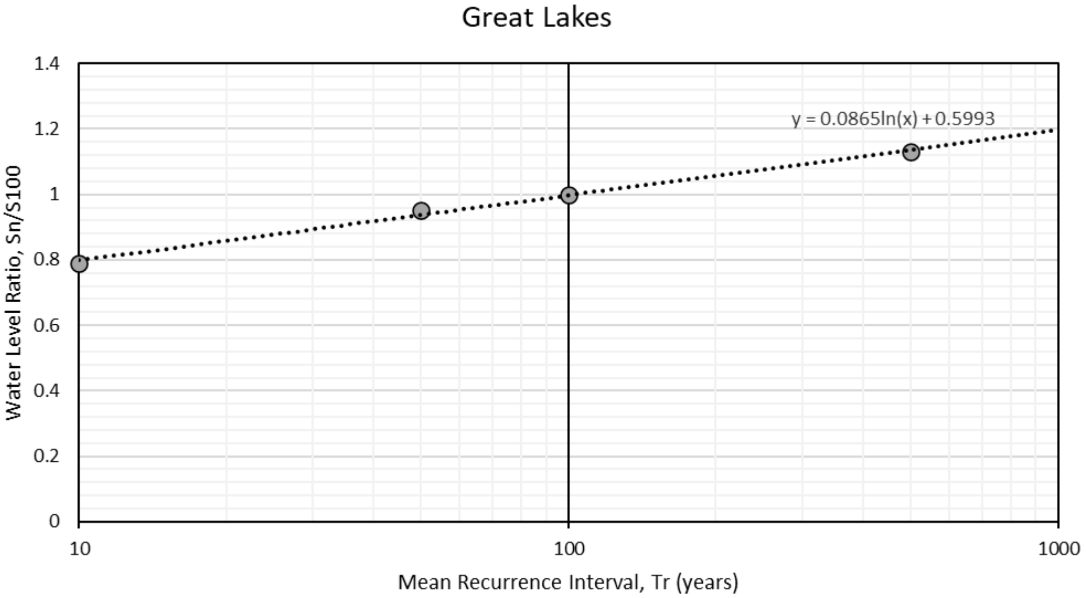


Figure A-4 Mean scale ratios using lake chart datum reference

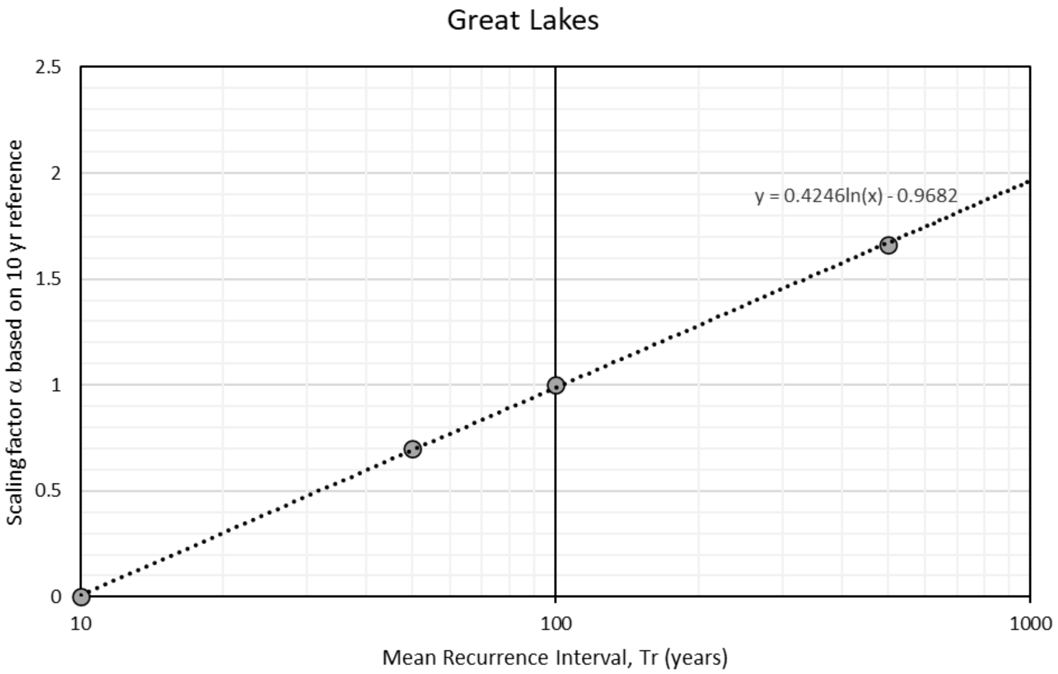


Figure A-5 Mean scaling factors using 10-year water level reference

Table A-3 Empirical scaling factors for Great Lakes using chart datum reference

MRI (years)	mean Using CD as reference	std dev Using CD as reference	α **95% Confidence Limit Using CD as reference**
		53 sites	
10	0.81	0.03	**0.81**
20	0.86	0.02	**0.87**
50	0.94	0.01	**0.94**
100	1.00	0.00	**1.00**
200	1.06	0.01	**1.06**
500	1.13	0.02	**1.14**
750	1.16	0.02	**1.17**
1000	1.19	0.03	**1.20**
2000	1.25	0.04	**1.26**

Table A-4 Empirical scaling factors for Great Lakes using 10-year water level reference

MRI (years)	Theory	mean Using 10-yr level as reference	std dev Using 10-yr level as reference	α **95% Confidence Limit Using CD as reference**
			53 sites	
10	**0.00**	0.00	0.00	**0.00**
20	**0.30**	0.32	0.01	**0.32**
50	**0.70**	0.70	0.02	**0.70**
100	**1.00**	0.99	0.03	**0.99**
200	**1.30**	1.27	0.04	**1.29**
500	**1.70**	1.66	0.05	**1.67**
750	**1.88**	1.82	0.06	**1.84**
1000	**2.00**	1.94	0.06	**1.96**
2000	**2.30**	2.23	0.07	**2.25**

A.3.2 Coastal Locations

A similar analysis was carried out on coastal locations. Once again, data from Canada is difficult to find as water levels at different return periods are only available in a few flood studies conducted for selected municipalities, and often at only one or two return periods rather than across a wider range. Results are again more accessible for U.S. coastal locations and the analysis below is based on FEMA FIS results for 55 coastal sites in the U.S., 36 in the U.S. northeast Atlantic coast for states bordering the Gulf of Maine and 19 sites in states adjacent to British Columbia. The data excludes any wave heights, wave runup or wave setup when these influences were identified in the FIS.

There were six Canadian coastal locations studied to determine scaling factors to use for comparison to those from the larger U.S. data set. Three of these sites are in Newfoundland-Labrador, one is in Nova Scotia, and two in British Columbia. These studies reported still water elevations based HHWLT levels plus storm surge. Waves and freeboard are not included in the elevations.

Scaling factors for coastal regions are again determined using the 100-year level as the upper reference level. Values may then be determined starting with a lower reference basis as used in equation A-4, and three methods of analysis were included. First, scaling values were based on the chart datum (CD) as also determined for the Great Lakes, with values of the scaling ratio given as $\alpha = S/S_{100}$. Second, scaling parameters were developed from equation A-4 using the lowest reference level used in the FIS, at the 10-year level. Third, scaling parameters were developed using the lowest reference level at the 20-year level, as some Provinces map the 20-year water level.

The analysis of coastal data followed the same process as for Great Lakes data. Empirical scaling parameters were first found at all 55 U.S. sites using the four MRI values tabulated in the FEMA FIS: 10, 50, 100, and 500 years. Regression analysis was then used at each site to interpolate and extrapolate to other MRI values. Results of the 55 sites were then compiled to determine the mean and standard deviation of the scaling parameters.

Figure A-6, A-7, and A-8 show the mean relationship for the four MRI values in the original data set. Values are then tabulated in Tables A-5, A-6, and A-7 using the chart datum, the 10-yr water level, and the 20-yr water level as reference values respectively.

Initial analysis showed that the two U.S. coasts behave very similarly in this scaling format, so that results of all 55 U.S. sites are combined. One factor likely influencing this result is that most of the U.S. sites considered have similar high tide ranges. The limited sites in Canada also behave very similarly to the U.S. sites. The standard deviation of the Canadian sites is smaller which may be a real physical effect or may be due to the small sample size.

Tables A-6 and A-7 also give the theoretical value of the scaling parameter, repeated from Table A-1. Unlike the Great Lakes data, the theoretical values tend to agree quite well with the empirical values.

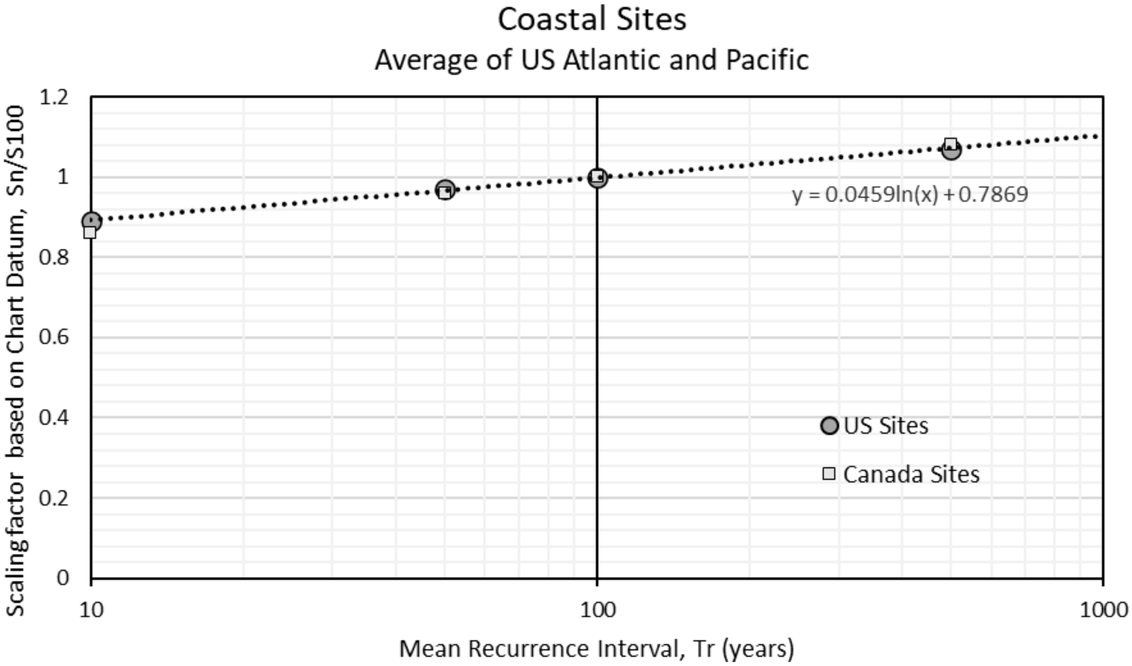


Figure A-6 Scaling factors for coastal sites using chart datum reference

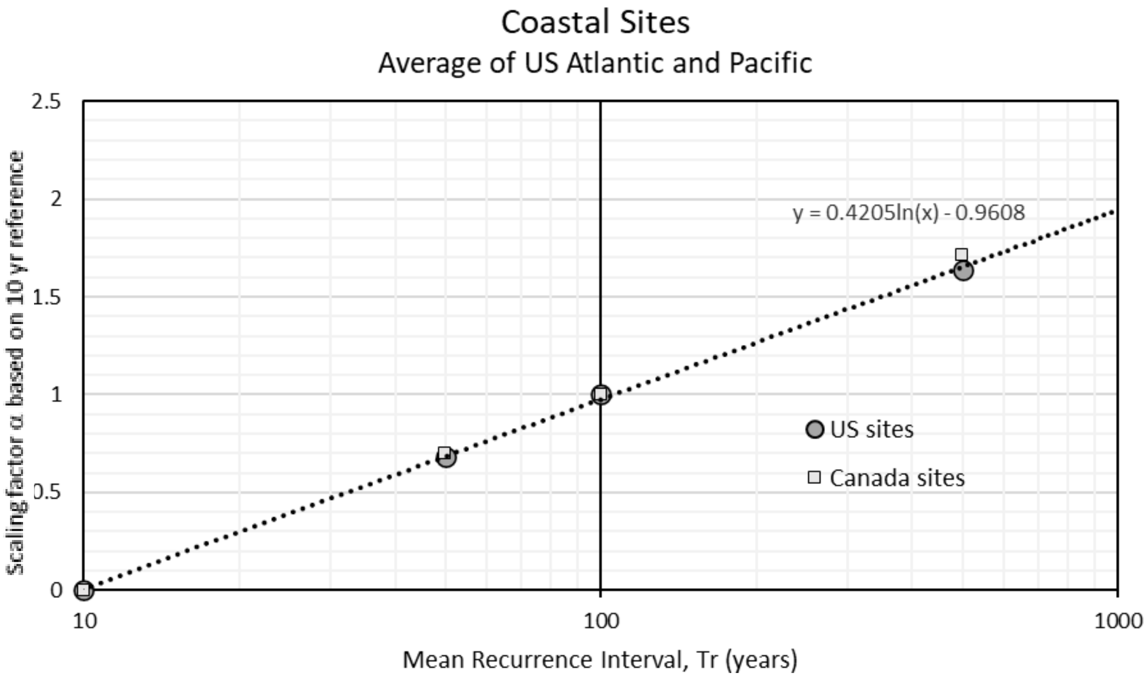


Figure A-7 Scaling factors for coastal sites using 10-year water level reference

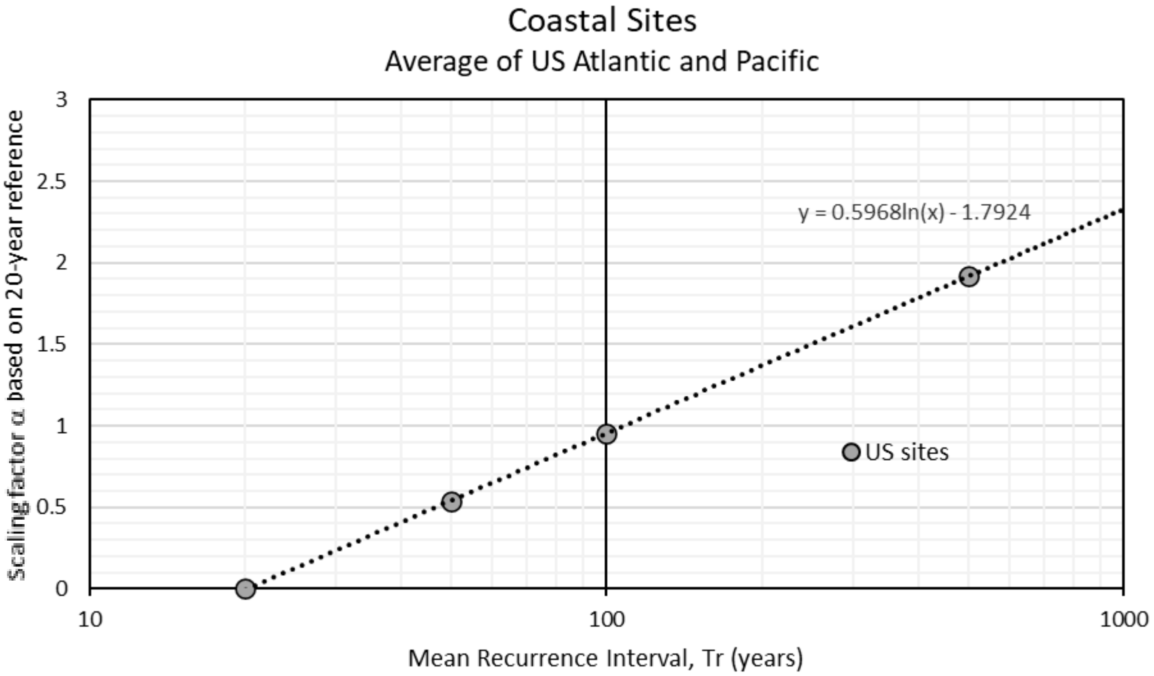


Figure A-8 Scaling factors for coastal sites using 20-year water level reference

Table A-5 Coastal scaling factors using Chart Datum as reference

MRI (years)	mean α U.S. Coastal Sites using CD reference	std dev	α 95% Confidence Limit Using CD reference
		55 sites	
10	0.89	0.08	**0.91**
20	0.92	0.05	**0.93**
50	0.96	0.02	**0.97**
100	1.00	0.01	**1.00**
200	1.03	0.03	**1.04**
500	1.08	0.06	**1.09**
750	1.10	0.08	**1.12**
1000	1.11	0.09	**1.13**
2000	1.14	0.11	**1.17**

Canada Scaling Factors using CD reference	std dev
6 sites	
0.88	0.04
0.92	0.03
0.97	0.01
1.00	0.00
1.04	0.01
1.09	0.03
1.11	0.04
1.12	0.04
1.16	0.05

Table A-6 Coastal scaling factors using 10-year reference

MRI (years)	Theory	mean α Using 10-yr reference	std dev Using 10-yr reference	α 95% Confidence Limit Using 10-yr reference
			55 sites	
10	**0.00**	0.00	0.05	**0.00**
20	**0.30**	0.30	0.02	**0.30**
50	**0.70**	0.68	0.04	**0.69**
100	**1.00**	1.00	0.00	**1.00**
200	**1.30**	1.26	0.12	**1.29**
500	**1.70**	1.64	0.17	**1.69**
750	**1.88**	1.81	0.20	**1.86**
1000	**2.00**	1.93	0.21	**1.99**
2000	**2.30**	2.22	0.25	**2.28**

Canada Scaling Factors using 10-yr reference	std dev
6 sites	
0.00	0.00
0.30	0.01
0.70	0.01
1.00	0.01
1.30	0.02
1.71	0.04
1.88	0.04
2.01	0.05
2.31	0.06

Table A-7 Coastal Scaling factors using 20-year reference

MRI (years)	Theory	mean a Using 20-yr reference	std dev Using 20-yr reference	α 95% Confidence Limit Using 20-yr reference	Canada Scaling Factors using 20-yr reference	std dev
			55 sites		6 sites	
20	**0.00**	0.00	0.00	**0.00**	**0.00**	0.00
50	**0.57**	0.54	0.06	**0.56**	**0.57**	0.02
100	**1.00**	0.95	0.12	**0.98**	**0.99**	0.01
200	**1.43**	1.37	0.17	**1.41**	**1.43**	0.04
500	**2.00**	1.92	0.25	**1.98**	**1.99**	0.08
750	**2.25**	2.16	0.28	**2.23**	**2.24**	0.09
1000	**2.43**	2.33	0.30	**2.41**	**2.42**	0.10
2000	**2.86**	2.74	0.36	**2.84**	**2.85**	0.13

A.3.3 Riverine Locations

Development of riverine scaling factors is somewhat different than lake or coastal locations for two reasons. First, river elevation above a common chart datum varies greatly, so that no consistent analysis is possible using the zero-elevation chart datum. Second, the water surface elevation also varies continuously along the length of a river from transect to transect, so that there is no common baseline reference available from transect to transect.

Unlike the lake and coastal sites, there is no readily available data set in the U.S. that can be used to evaluate empirical scaling parameters. As a result, all values used in this study are based on analysis of Canadian river systems. Five flood studies were located in which water levels were reported across a wide range of MRI values, these include one in British Columbia, three in Alberta, and one in Newfoundland-Labrador. From these, 20 river transects were selected for use. In addition, water levels were obtained for 10 transects from each of four riverine sites, two in Alberta and one each in Manitoba and Saskatchewan, based on model studies sponsored by the NRC as a related part of this study.

As a result, a data set of 60 transects was used. These differed somewhat in terms of the actual MRI values reported in the studies. Some modeled riverine data started with 2-year MRI water surface elevations, some started with 10-year MRI, and some data began with 20-year MRI. The NRC-sponsored studies extended to a 2500-yr MRI; most other studies extended to a 1000-yr MRI.

With the disparate data sets, analysis proceeded similarly to the lake and coastal analysis, first determining empirical scaling parameters foreach of the 60 sites, then fitting a regression line through the scaling parameter values at each site. With this, values can then be interpolated or extrapolated to other MRI values, and the mean and standard deviation from the 60 sites can be determined.

Results for the riverine sites are given in Figures A-9 and A-10. Table A-8 and A-9. Empirical results for the riverine scale factors show that values are a few percent higher than theoretical values for the higher MRI levels.

As an example of how to use these scaling factors, consider a river location in which the 20-year water level is 122.0 m while the 100-year level is 123.2 m, both above the CGVD2013 datum. The design requires a water surface elevation for the 1000-year event. Using Table A-9 due to the 20-year reference level, the scaling factor for the 95% upper confidence limit is $\alpha = 2.54$ for the 1000-year event is 2.44. The 1000-yr level is then estimated as

$$S_{1000} = S_{20} + 2.54\times(S_{100}-S_{20})$$

Substituting values gives:

$$S_{500} = 122.0 + 2.54\times(123.2-122.0) = 125.0\ m$$

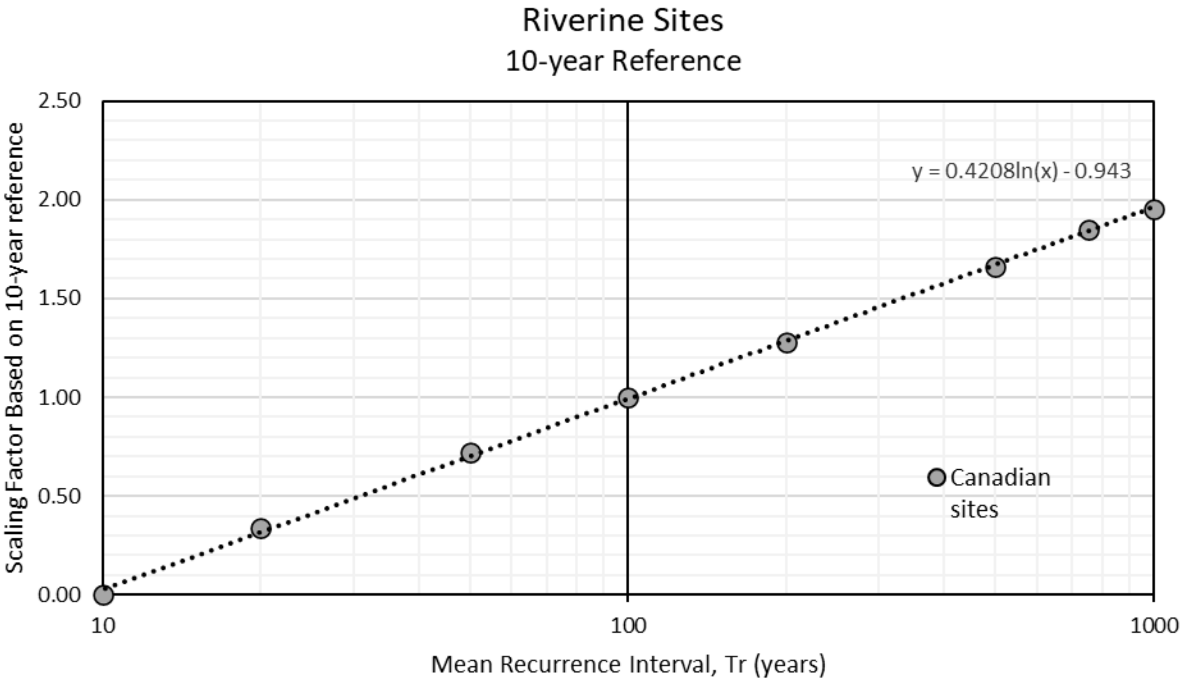


Figure A-9 Riverine scaling factors using 10-year water level reference

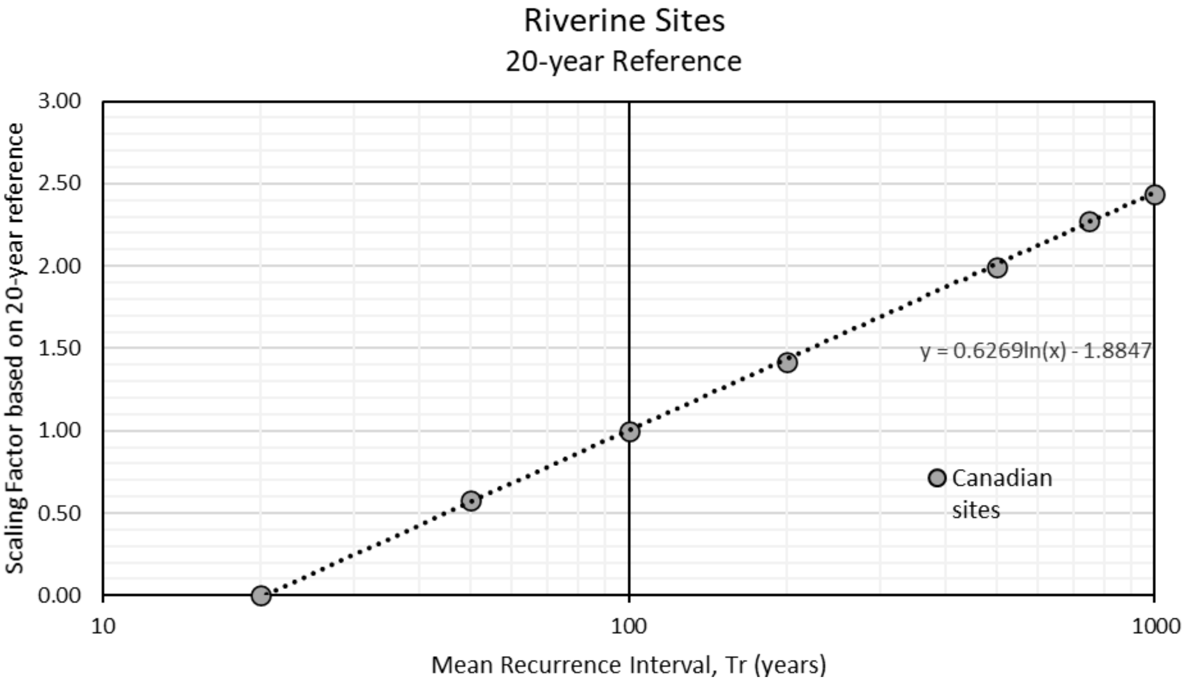


Figure A-10 Riverine scaling factors using 20-year water level reference

Table A-8 Riverine scaling factors using 10-year water level reference

MRI (years)	Theory	Mean Scaling Factors 10-yr reference	std dev	α 95% Confidence Limit Using 10-yr reference
			60 sites	
10	**0.00**	0.00	0.00	**0.00**
20	**0.30**	0.34	0.12	**0.37**
50	**0.70**	0.72	0.06	**0.74**
100	**1.00**	1.00	0.00	**1.00**
200	**1.30**	1.28	0.08	**1.30**
500	**1.70**	1.66	0.20	**1.71**
750	**1.88**	1.85	0.25	**1.91**
1000	**2.00**	1.95	0.32	**2.04**
2000	**2.30**	2.26	0.34	**2.35**

Table A-9 Riverine scaling factors using 20-year water level reference

MRI (years)	Theory	Mean Scaling Factor 20-yr reference	std dev	α 95% Confidence Limit Using 20-yr reference
			60 sites	
20	**0.00**	0.00	0.00	**0.00**
50	**0.57**	0.58	0.04	**0.59**
100	**1.00**	1.00	0.00	**1.00**
200	**1.43**	1.42	0.08	**1.44**
500	**2.00**	2.00	0.22	**2.05**
750	**2.25**	2.27	0.28	**2.34**
1000	**2.43**	2.44	0.40	**2.54**
2000	**2.86**	2.90	0.36	**2.99**

A.4 Alternate Reference Levels

For Provincial locations where two water levels are known, but where the 10, 20, and 100-year values used in previous table may not be known, theoretical scaling factors would change and can be derived for any other two reference levels using equation A-3. For example, British Columbia and Manitoba map the 200- and 500-yr levels respectively instead of a 100-yr water level and scaling factors would have to be re-derived for application in those locations. As noted in the previous analysis, most empirical factors are within a few percent of the theoretical values. Table A-10 therefore gives theoretical scaling values for a range of upper and lower reference levels that may be useful.

Table A-10 Theoretical scaling factors for range of upper and lower reference levels

	Upper reference: 100 yrs				**Upper reference: 200 yrs**				**Upper reference: 500 yrs**			
MRI	**2 year**	**5 year**	**10 year**	**20 year**	**2 year**	**5 year**	**10 year**	**20 year**	**2 year**	**5 year**	**10 year**	**20 year**
2	0.00				0.00				0.00			
5	0.23	0.00			0.20	0.00			0.17	0.00		
10	0.41	0.23	0.00		0.35	0.19	0.00		0.29	0.15	0.00	
20	0.59	0.46	0.30	0.00	0.50	0.38	0.23	0.00	0.42	0.30	0.18	0.00
50	0.82	0.77	0.70	0.57	0.70	0.62	0.54	0.40	0.58	0.50	0.41	0.28
100	1.00	1.00	1.00	1.00	0.85	0.81	0.77	0.70	0.71	0.65	0.59	0.50
200	1.18	1.23	1.30	1.43	1.00	1.00	1.00	1.00	0.83	0.80	0.77	0.72
500	1.41	1.54	1.70	2.00	1.20	1.25	1.31	1.40	1.00	1.00	1.00	1.00
750	1.52	1.67	1.88	2.25	1.29	1.36	1.44	1.57	1.07	1.09	1.10	1.13
1000	1.59	1.77	2.00	2.43	1.35	1.44	1.54	1.70	1.13	1.15	1.18	1.22
2000	1.77	2.00	2.30	2.86	1.50	1.62	1.77	2.00	1.25	1.30	1.35	1.43

Appendix B Prescriptive Approach for Estimating Riverine Velocity

This Appendix contains a simple prescriptive method that can be applied in many riverine situations to estimate flow velocity based on mapped flood elevations. Flow velocity is required to estimate flood loads, but at present very few flood maps or flood studies indicate velocities across the flood way or flood fringe. While it is often assumed that velocities in the flood fringe are less than 1 m/s, this is not sufficient resolution for an accurate estimate of hydrodynamic flood loads. As a result, a method is required to allow a user to generate more accurate estimates of flow velocity.

The proposed method, based on established principles of open channel flow in rivers, allows for a rather simple estimate of flow velocity based on (1) water surface slopes obtained from flood maps, (2) a cross sectional area of flow obtained from GIS mapping or traditional land surveying, and (3) a description of ground roughness through selection of Manning's n values. The method appears suitable for a prescriptive building code, as it can be summarized in just a few equations, all based on well-known open channel flow expressions.

B.1 Background

Design standards for riverine flood loads require a user to identify both a flood elevation and a flow velocity. At present, flood elevations are available from flood maps, but flow velocities are generally not mapped and thus a user has no way to develop a rational estimate of the flow velocity.

ASCE 7-16 does not give a prescriptive method for estimating riverine flow velocities in the main code. The ASCE 7-16 Commentary provides interim guidance that the flow velocity is likely in a range between

$$V = d_f \,/\, (1\text{ sec}) \qquad \text{to} \qquad V = (g\, d_f)^{1/2} \qquad \text{(Eq. B-1)}$$

where:

V = flow velocity (m/s or ft/s)
d_f = flood depth at site (m or ft)
g = gravity (9.81 m/s^2 or 32.2 ft/s^2)

However, the reference cited in ASCE 7-16, the FEMA Coastal Construction Manual (2000), implies that the relationships may have been developed for coastal regions and not for riverine situations.

The second expression above, based on the speed of critical flow with a Froude number of unity, would likely apply as an upper bound for riverine flows where the flows remain sub-critical. However, it is not known if the first expression has any physical basis or if it applies to riverine flows at all. As is known, some overbank regions of riverine flood plains have standing water and provide for flood storage but may not contribute to flow and thus may have velocities near zero as a lower bound.

B.2 Overview of Proposed Method

The method proposed below would require use of Provincial flood maps to obtain flood elevations for a riverine transect drawn perpendicular to the flow through the site of interest, and for two additional transect locations some distance up- and down-stream from the site. The flood elevations at the up- and down-stream transects, and the separation distance between the up-and down-stream locations, would then define the water surface slope for the site of interest.

For the site of interest, GIS mapping information, or a detailed local topographic survey, would then be used to obtain ground elevations for the transect below the mapped flood elevation. Using the depths across the transect, and the water surface slope from up- and down-stream transects, application of the well-known Manning equation then allows an estimate of mean or average velocity for the transect.

Additionally, the transect at the site may be subdivided into distinct flow sections, including the floodway or main flow channel, as well as left and right overbank sections across the flood fringe or flood way. Application of the Manning equation to each subdivision of the transect then allows an estimate of the mean velocity in each sub-region, accounting for any change in roughness values based on changing Manning's n values in each sub region. Because a centre channel (floodway) will normally have deeper depths and lower Manning's *n* values, while a building site in the flood fringe will likely have shallow depths and large values of the Manning's *n* roughness due to buildings, vegetation, etc, the method naturally indicates much higher velocities in the floodway and lower velocities in the flood fringe.

B.3 Proposed Method

In open-channel riverine flows, if a transect is drawn across the flow channel to define the cross-sectional area of the flow, then the flow rate through the cross section can be given by the continuity equation as

$$Q = A\,V \qquad \text{(Eq. B-2)}$$

where:

Q = Flow rate or river discharge (m^3/s or ft^3/s)
A = Cross sectional area of the flow channel, perpendicular to the flow (m^2 or ft^2)
V = Flow velocity, averaged over the cross section (m/s or ft/s)

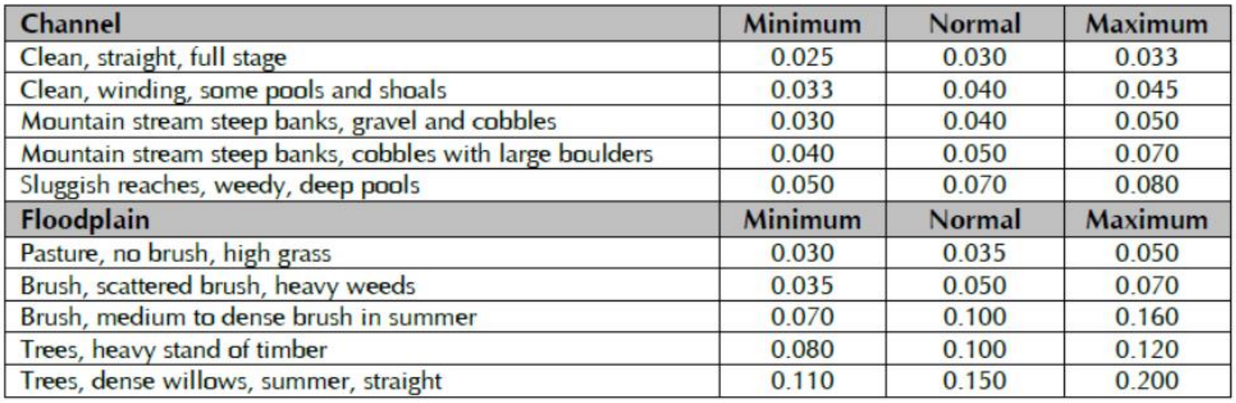
The mean velocity for the cross section, V, is typically given by the Manning equation, which empirically relates the flow velocity to the slope of the energy grade line as (reference HEC-RAS manual and other open channel flow sources such as text by Chow)

$$V = \frac{k}{n} R_h^{2/3} S_f^{1/2} \qquad \text{(Eq. B-3)}$$

where:

S_f = slope of energy grade line (m/m or ft/ft), or energy gradient, given by head loss from upstream to downstream
R_h = Hydraulic radius (m or ft), given by $R_h = A/P$ or approximately by $R_h \sim A/W$
P = Wetted perimeter of the cross-sectional area (m or ft)
W = Width of the cross section (m or ft)
k = Coefficient associated with units (k=1 in SI units, k=1.486 in US Customary units)
n = Manning's roughness coefficient, related to ground cover, flow turbulence, flow meandering, and flow obstructions.

Figure B-1 provides some typical Manning roughness coefficients.



Channel	**Minimum**	**Normal**	**Maximum**
Clean, straight, full stage	0.025	0.030	0.033
Clean, winding, some pools and shoals	0.033	0.040	0.045
Mountain stream steep banks, gravel and cobbles	0.030	0.040	0.050
Mountain stream steep banks, cobbles with large boulders	0.040	0.050	0.070
Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
Floodplain	**Minimum**	**Normal**	**Maximum**
Pasture, no brush, high grass	0.030	0.035	0.050
Brush, scattered brush, heavy weeds	0.035	0.050	0.070
Brush, medium to dense brush in summer	0.070	0.100	0.160
Trees, heavy stand of timber	0.080	0.100	0.120
Trees, dense willows, summer, straight	0.110	0.150	0.200

Figure B-1 Typical Manning Roughness Coefficients (source: Hydrotechnical Study of Stephenville Crossing/Black Duck Siding, 2012)

The Manning equation can be combined with the continuity equation to give

$$Q = A \frac{k}{n} R_h^{2/3} S_f^{1/2} \qquad \text{(Eq. B-4)}$$

This expression may be applied if the frictional head losses from upstream to downstream are known so that the slope of the energy grade line is known, as illustrated in Figure B-2. In practice, this is difficult to determine unless the velocities at up and downstream cross section are also known, and velocities are generally not available in the typical Provincial flood hazard mapping programs.

As an approximation, it is known that if the flow does not vary dramatically over some short distance from up- to down-stream, then the principles of uniform flow may be used to approximate the slope of the energy grade line with the slope of the water surface. This is useful as the water surface elevations are mapped and surface slopes may therefore be estimated from the flood maps.

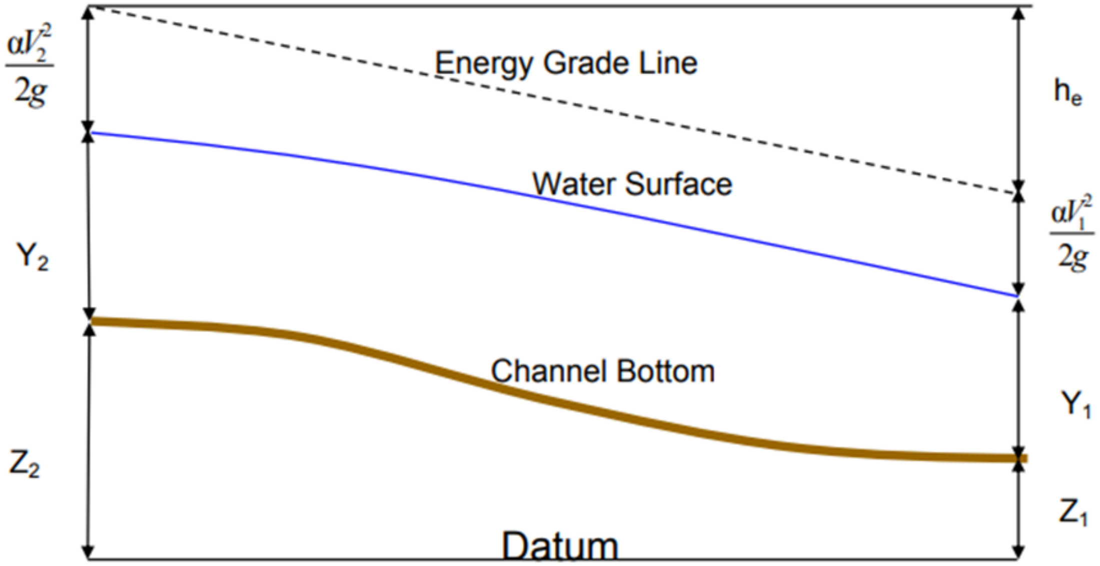


Figure B-2 Energy grade line illustration (source: HEC-RAS Manual)

The approximation of flows as uniform are discussed by Chow (1959) and may be adopted as a simplified approximation suitable for prescriptive flood loading guidelines. Guidance language would direct a user to conduct a site-specific analysis using a detailed numerical model if flow velocities became too high or if the site conditions indicate rapidly-varying flows due to strong flow constrictions, steep bed slopes, or flow-control structures such as dams or weirs. Guidance language would also allow a user to conduct a site-specific analysis in any situation as a means of refining the estimates of flow velocity.

Under the assumption of uniform flow, the slope of the energy grade line, S_f, may be approximated by the slope of the water surface, S, as obtained from consideration of locations up and down stream of the site of interest on the flood maps. The flow rate from equation B-4 may then be written in simpler form by defining the conveyance of the river channel, K, as

$$Q = K\ S_S^{1/2} \qquad \text{(Eq. B-5)}$$

where:

S_S = slope of water surface (m/m or ft/ft), from upstream to downstream water surface elevations
K = conveyance or discharge capacity of cross section discharge (m^3/s or ft^3/s)

From the Manning equation, the conveyance is given by

$$K = A\,\frac{k}{n}\ R_h^{2/3} \qquad \text{(Eq. B-6)}$$

The conveyance has the same units as the flow (m^3/s or ft^3/s) and is useful because it is a property of the flow cross section geometry and roughness, and it indicates the carrying or discharge capacity of the cross section. From the continuity equation (B-1), the average velocity over the cross section is then given by

$$V = \left(\frac{K}{A}\right) S_S^{1/2} \tag{Eq. B-7}$$

The methods outlined above are used in the Slope-Conveyance Method for 1D river flows and are a simple first-order method for relating river stage (through the effect of water elevation on cross sectional area and conveyance) to discharge. The method is usually applied iteratively to stream cross sections when the discharge is known, say for some specific flow event, but water surface and velocity are not known. In that case, the method is applied by iteratively adjusting the flood stage (and area and conveyance) until the flow from equation B-3 matches the known discharge.

For application based on existing flood maps, the method may be applied somewhat differently and more simply since the flood elevations have already been determined and mapped. As a result, the water surface elevations may then be used directly to compute the slope S_S. The conveyance, K, may then be estimated using topographic data for a cross section, computing the area and hydraulic radius for the cross section below the known flood elevation. With this information, the mean or average velocity of the cross section can be obtained directly from equation B-7.

As discussed in the HEC-RAS Manual and other references on open channel flows (Chow, 1959), the velocity computed above is the mean or average for the entire cross section of the flow. But it is known that actual velocities will vary with location in the river channel as depicted below in Figure B-3 (Chow, 1959). Velocities are generally higher than the mean velocity in the centre of the channel or floodway, but lower than the mean velocity in the shallower (and possibly rougher) overbank area and flood fringes.

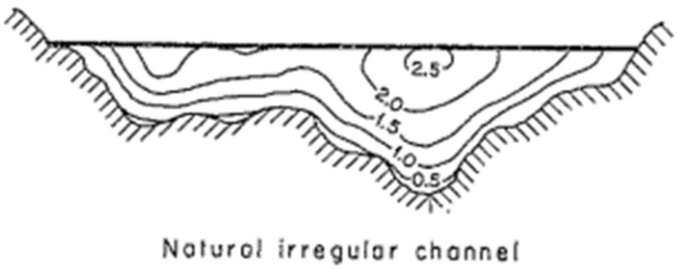


Figure B-3 Velocity contours for a typical river cross section (Chow, 1959)

To improve the resolution of the velocity field, the HEC-RAS manual suggests that the flow cross section and conveyance may be divided into distinct sub regions, each with a different geometry and roughness or Manning's n value. As shown in Figure B-4 (HEC-RAS) the subdivision would normally account for flow in the main channel or floodway, and for flow in the left and right overbank areas or flood fringes. Each flood fringe may be further subdivided if, for example, the flood depth changes dramatically, or the roughness differs from one to another, i.e. one may be open grass land, another urban development, etc.

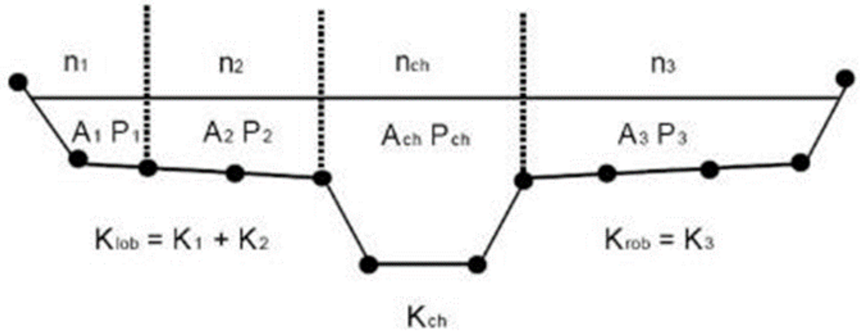


Figure B-4 Subdivision of cross-sectional area to define main river channel (floodway) along with left and right overbank sections (flood fringe) (source: HEC-RAS Manual, 2016)

If the cross section is subdivided, then the total cross-sectional area, conveyance, and flow rate will be given by the sum of values defined in each sub-region (using subscripts to denote each sub-region) as

$$A = A_1 + A_2 + A_3 +_{\dots} = \sum A_i \quad \text{(Eq. B-8)}$$

$$K = K_1 + K_2 + K_3 +_{\dots} = \sum K_i \quad \text{(Eq. B-9)}$$

$$Q = Q_1 + Q_2 + Q_3 +_{\dots} = \sum Q_i \quad \text{(Eq. B-10)}$$

where the Manning equation is applied in each sub-section as

$$K_i = A_i \frac{k}{n_i} R_{h_i}^{2/3} \quad \text{(Eq. B-11)}$$

$$Q_i = K_i S_S^{1/2} \quad \text{(Eq. B-12)}$$

$$V_i = \frac{K_i}{A_i} S_S^{1/2} = \frac{Q_i}{A_i} \quad \text{(Eq. B-13)}$$

With this approach, the floodway or main channel normally has by far the largest portion of the flow and conveyance, often with low values of Manning’s n, giving the highest velocities. Shallow overbank areas in the flood fringe will often have smaller values of conveyance but larger values of roughness, resulting in smaller velocities.

Equations B-8 through B-13 form a simple recommended method that can be used to estimate flow velocity given mapped water surface elevations. A user would need to define the slope of the water surface using mapped flood elevations up and down-stream from a proposed building site. The user would then need to use topographic survey data to determine the conveyance of the cross section, and would then need to subdivide the area (and conveyance) into the main floodway and two or more flood fringe sections. Finally, the user would need to define Manning’s n values for each sub-section of the flow. With this information, equations B-8 through B-13 can be implemented in a simple spreadsheet to determine the mean velocity in each sub-section of the cross section, including at the proposed building site.

B.4 Step-by-Step Application of Method

1. Obtain the flood map for a given area and identify water levels for three transects.

An ideal example is shown in Figure B-5a as it indicates both elevation and distance along the river channel. In this case, a transect would be drawn for the location of interest and adjacent transects up and down stream would be identified. The up and downstream water level elevations would be defined from a reference datum as elevations E_1 and E_2, and the water level at the site of interest would be interpolated between E_1 and E_2, and defined as elevation E. The distance along the channel from up to downstream transects would also be determined, as length L.

A less-than-ideal example of a flood map is shown in Figure B-5b which shows that extent to the flood hazard zone but gives no specific indication of flood elevation or distance along the river channel. In this case, three transects would again be defined. But to define elevations, these transects and the mapped boundaries of the flood hazard zones would be defined either from on-site topographic surveying or from a GIS map also showing ground topography. The topographic elevation contours corresponding to the limits of the mapped flood hazard zones would be used to establish the elevations E_1 and E_2 of up and downstream water levels as well as elevation E for the transect of interest. Distance from up to downstream transects would again be obtained.

For building code development, guidelines will need to be developed for minimum and maximum acceptable spacing of the transects. In general, the up and down stream transects must be far enough apart to produce a measurable change in water surface elevation from E_1 to E_2, but must be close enough so that flow characteristics are similar and approximately uniform from upstream to downstream without any unusual change in the flow conditions (such as a dam or spillway, bridge constriction, levee or other similar flood obstruction).

2. Determine the average water surface slope from up to downstream transects.

Mapping or survey data would be used to define the spacing of up and downstream transects along the river channel and they would be defined as channel length L. The average water surface slope is then given by

$$S_S = (E_1 - E_2)/L \qquad \text{(Eq. B-14)}$$

For inclusion in a building code, guidelines will need to be developed for minimum and maximum acceptable slopes. If slopes are too high, then rapidly varying unsteady flow will occur and a site-specific analysis would be required. If slopes are very small, then the velocities will be very small and the code could prescribe a default velocity of V=0 to V=1 m/s.

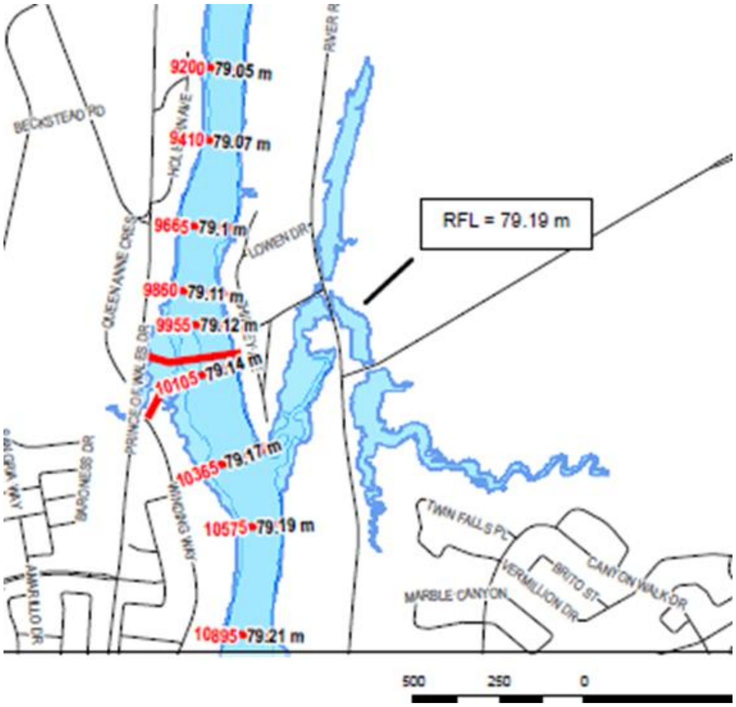


Figure B-5a Example of flood map showing flood elevations (and distance along channel) for distinct river transects (source: Rideau Valley Conservation Authority)

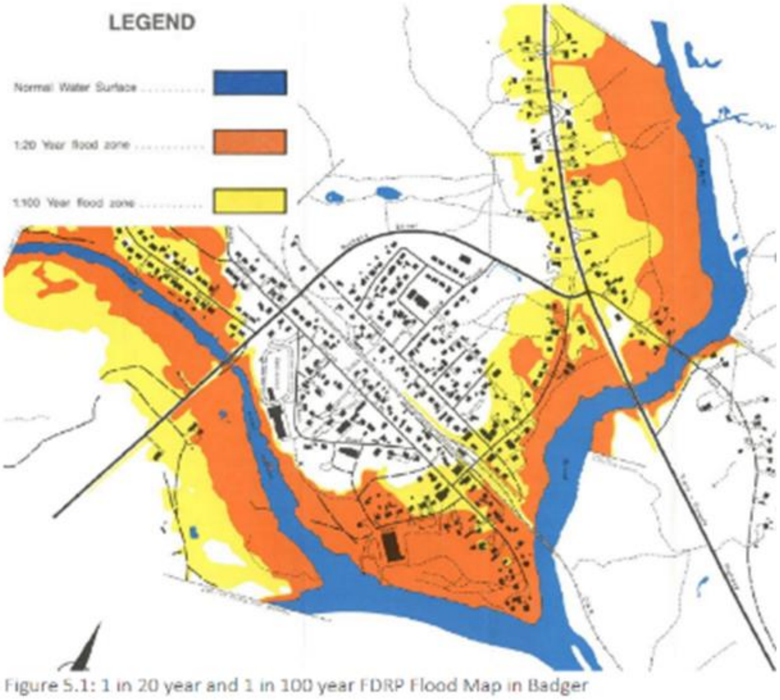


Figure B-5b Example of flood map with no elevations or length scale shown (source: Flood Information Map for Badger, Newfoundland)

3. **Determine ground elevations across the transect of interest, sufficient to determine the cross-sectional area of the transect below the mapped flood elevation.**

Ground elevations across the transect could be obtained from Lidar-based digital elevation data, using GIS software to facilitate extraction of water depths across the transect below the mapped flood elevation. Alternatively, a user could conduct a traditional topographic survey on site. Either way, ground elevations, G_i, would be defined at various points or locations, i, across each transect. From the mapping or survey data, the ground elevations G_i and mapped flood level E would be used to define the flood depths, d_i, at points across the transect, as $d_i = E - G_i$.

The cross-sectional area of the transect, A, can be determined from the flooded depths across the transect. In addition, the wetted perimeter of the transect, P, can be determined, or as an approximation, the width of the flooded transect across the river channel (from left flooded bank to right flooded bank), W, can be determined. These can similarly be defined for subsections of the flow channel.

It is noted that the cross-sectional area and wetted perimeter (or width) of the flooded transect should only include sections that contribute to or convey the net flow. It is common in some areas with very wide flood plains for some overbank areas to act as flood storage, with standing flood water but without any substantial net flow or discharge. These areas are termed "ineffective areas" (HEC-RAS). As shown in Figure B-6, these areas may be identified by estimating the lateral extent of the flow channel and eliminating areas of stagnant conditions where a strong downstream flow could not occur.

In a flood code, this will likely be something the user must estimate, and guidelines will be needed for how to draw these flow boundaries to define the ineffective areas. Once identified, the ineffective areas would have standing flood water but no flow so that $V = 0$. These areas would then be excluded from further analysis.

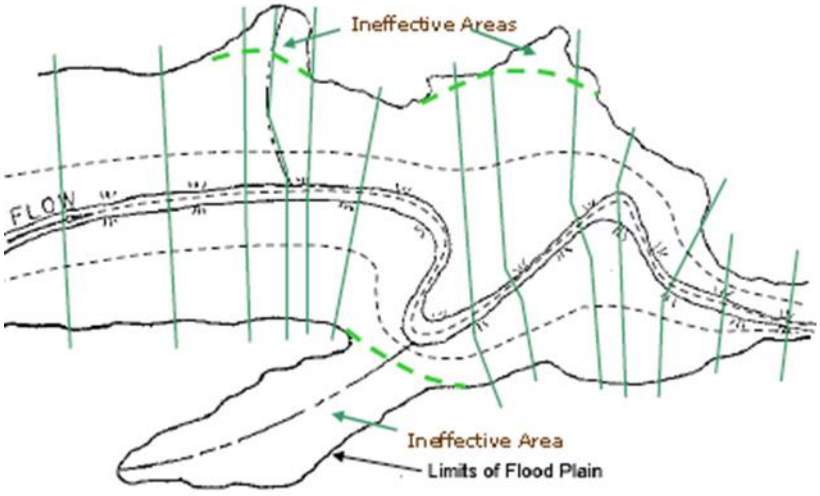


Figure 3-4 Example Cross Section Layout

Figure B-6 Illustration of an area of ineffective flow along a stream section

4. Sub-divide the cross section into the main flow channel (floodway) and two or more overbank or flood fringe sections.

A sample cross section is shown in Figure B-7, which, for simplicity shows just three distinct subsections. In this case, the limits of the main floodway in Zone 1 might correspond to the 20-year MRI flood event while the limits of the flood fringe may correspond to the 100-year MRI flood event. A hypothetical site of a proposed building might be in Zone 3, in the flood fringe which might also include other buildings in a dense development. Zone 2 of the flood fringe might have different characteristics (perhaps trees, or roadway) and is represented as a separate zone.

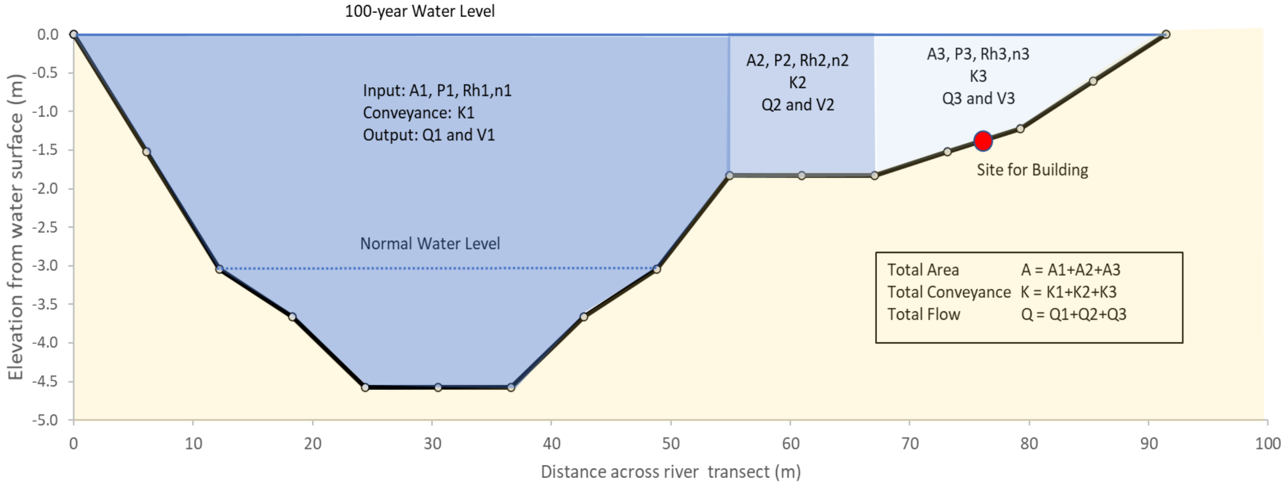


Figure B-7 Sample stream cross section showing subsections

5. Estimate the Manning n value(s) representing roughness for the flow in each subsection.

A unique Manning n value then must be identified for each flow sub-section. A sample table of Manning's n values is shown in Figure B-1 and a more complete version should be included in the code. Overbank regions can reflect vegetation, roadways, effects of buildings, etc. Values appropriate for Canada should be included, based on additional research. The building code commentary should have considerable discussion of Manning's n values with a wide range of references.

6. Estimate the flow conveyance, flow rate, and velocity in each portion of the cross section.

Sample calculation s for the hypothetical cross section area are shown below in Table B-1 to illustrate the procedure. With the geometric properties of the cross section, and with Manning n values, the conveyance of each sub-section of the transect, K_i, can be computed using equation B-11. The flow rate in each portion of the cross section, Q_i, is then computed using equation B-12. Summation of the flow rates in each section could be used as a check to ensure that the overall discharge, if known from a flood study, is preserved. Finally, application of equation B-13 gives the flow velocity in each section as $V_i=Q_i/A_i$.

Note that the geometric properties of the section will usually give the largest sub-area, A_i, in the main floodway, with lower values in the flood fringe sections. However, extremely wide flood plains may give large areas in the flood fringe. In the example below, the floodway in zone 1 occupies 79% of the total cross-sectional area while 10% of the area is in flood fringe zone 2 and 11% is in flood fringe zone 3.

The hydraulic radius for river channels is usually taken as area divided by width and is therefore a measure of average depth in each section. The flood fringe sections will therefore almost always have shallower depths than the floodway. The flood fringe zones will likely also have larger Manning's n values due to significant roughness compared to the main floodway, although open ground, large paved areas, and other factors may also give small values of Manning's n in portions of the flood fringe.

The conveyance term in equation B-11 accounts for all these factors and indicates the ability of each section to convey flow. In the example below, the floodway in zone 1 has 92% of the total conveyance, while the flood fringe zone 2 has 6% and the shallowest flood fringe zone 3 has just 2% of the conveyance.

Table B-1 Flow velocities calculated for sample cross section

	Floodway Zone 1	Flood Fringe Zone 2	Flood Fringe Zone 3	Sums for Cross Section	Comments
Water Surface Slopes between Transects					
S_S	0.007	0.007	0.007		slope between transects
Geometric Properties from Analysis of Cross Section					
A (m^2)	180.2	22.3	26	228.5	sum for total cross-section area
W~P (m)	54.9	12.2	24.4		wetted perimeter approx. as width
Rh (m)	3.282	1.828	1.066		hydraulic radius approx. as A/w
Manning's n for each section and composite value for river					
n	0.03	0.04	0.08		
Conveyance for each section and for river cross section					
K	13266	833	339	14439	
Flow rate for each section and for river cross section					
Q (m^3/s)	10352	650	265	11267	
Velocity in each section					
V (m/sec)	6.16	3.13	1.09		

When the conveyance is coupled with the water surface slope term in equation B-12, the method then accounts for the hydraulic gradient driving the flow to give the flow rate in each section. Note that areas with very flat gradients (like the Rideau River example in Figure B-5a) will give generally small flow rates and thus low flow velocities. Areas with steeper gradients will have the potential for greater flow rates.

Finally, from the continuity equation B-1, the velocity in each section is given by $V_i=Q_i/A_i$. In the example above, the floodway in zone 1 has a mean velocity of 6.16 m/sec owing to the rather steep slope assumed (water surface dropping 7 m per 1000 m) and due to the high conveyance. The velocity drops to 3.13 m/s in flood fringe zone 2, and then drops to just 1.09 m/s in the shallowest flood fringe zone 3. Note that for this example, the prescriptive method gives higher velocities in the flood fringe than is commonly assumed in many Canadian flood maps, where flood fringe velocities are assumed to be less than 1 m/s.

B.5 Example Application

The following example is developed from river cross section profiles for Harry's River in Newfoundland-Labrador. There are no comparative velocities in any hydrologic study for the river; the flood maps and profiles do provide water surface elevation for both the 1:20 AEP and 1:100 AEP events. Figure B-8 illustrates a small section of the flood plain of Harry's River and shows the transects used for the example.

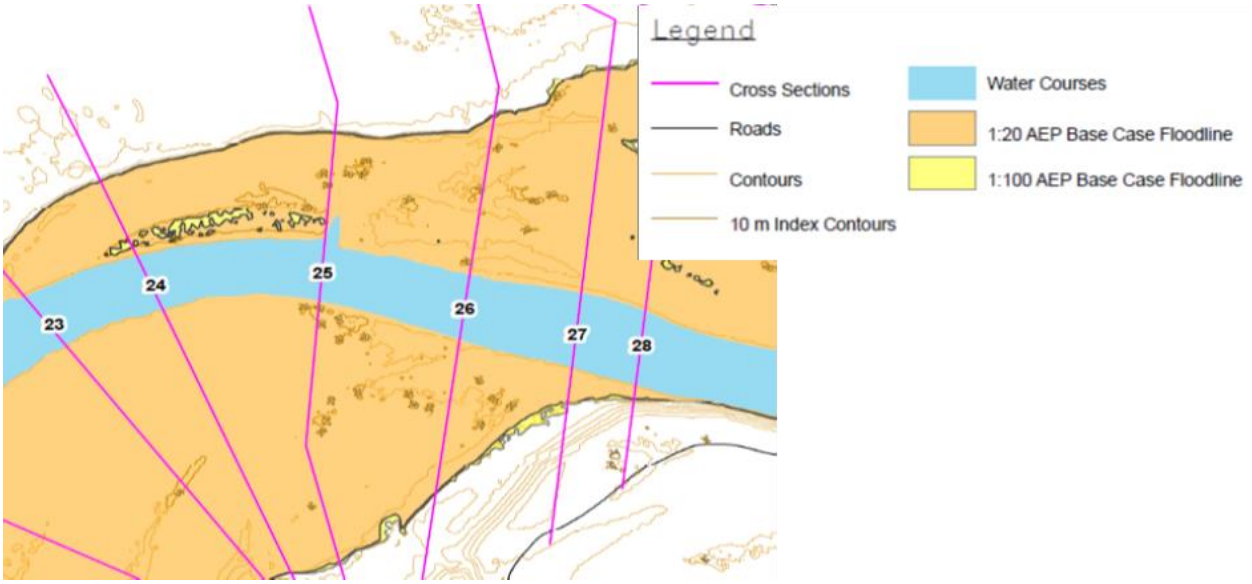


Figure B-8 Portion of Harry's River used for velocity example

The two transect profiles are shown in Figure B-9. The profiles were used to develop the cross-sectional areas and the wetted perimeters. The Manning roughness numbers were provided in the hydrologic study. The slope of the river surface was provided by the hydrologic study by using the water level for the transects on each side of the transects of interest, and the transect distances were provided by the study.

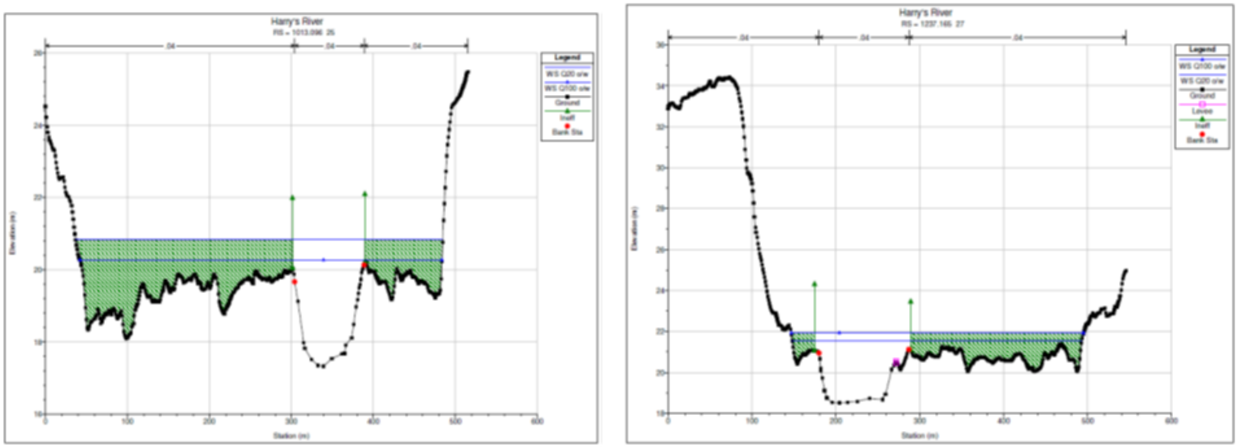


Figure B-9 River cross sections for Transect 25 (left) and Transect 27 (right)

Using the formulas and methods described above, the flow and velocity results are listed in Tables B-2 and B-3.

Table B-2 Flow velocities calculated for Harry's River Transect 25

	Floodway Zone 1	Flood Fringe Zone 2	Flood Fringe Zone 3	Sums for Cross Section	Comments
Water Surface Slopes between Transects					
S_S	0.0049	0.0049	0.0049		slope between transects
Geometric Properties from Analysis of Cross Section					
A (m^2)	214.4	370.5	107.3	228.5	sum for total cross-section area
w~P (m)	88	268	95		wetted perimeter approx. as width
Rh (m)	2.436	1.382	1.129		hydraulic radius approx. as A/w
Manning's n for each section and composite value for river					
n	0.035	0.07	0.07		
Conveyance for each section and for river cross section					
K	11091	6568	1662	19332	
Flow rate for each section and for river cross section					
Q (m^3/s)	8499	5033	1274	14806	
Velocity in each section					
V (m/sec)	3.61	1.24	1.08		
V (lower bound-depth/1 sec)	3.0	1.8	1.5		maximum depth in section used
V (upper bound-(g*depth)^0.5)	5.42	4.20	3.84		

Table B-3 Flow velocities calculated for Harry’s River Transect 27

	Floodway Zone 1	Flood Fringe Zone 2	Flood Fringe Zone 3	Sums for Cross Section	Comments
Water Surface Slopes between Transects					
S_S	0.0024	0.0024	0.0024		slope between transects
Geometric Properties from Analysis of Cross Section					
A (m^2)	296.3	300.0	27.1	623.4	sum for total cross-section area
w_P (m)	115	209	27		wetted perimeter approx. as width
Rh (m)	2.577	1.435	1.004		hydraulic radius approx. as A/w
Manning’s n for each section and composite value for river					
N	0.035	0.07	0.07		
Conveyance for each section and for river cross section					
K	15911	5453	388	21752	
Flow rate for each section and for river cross section					
Q (m^3/s)	11763	4032	287	16082	
Velocity in each section					
V (m/sec)	2.62	0.89	0.70		
V (lower bound-depth/1 sec)	3.09	1.6	1.8		maximum depth in section used
V (upper bound-(g*depth)^0.5)	5.51	3.96	4.20		

In these examples, the difference in water surface slope from the flood map produced notable difference in velocities at the two transects. The velocities are then compared to the prescriptive lower and upper bound velocities described in Section B.1 of this appendix. There is not good agreement between the calculated velocities and the prescriptive velocities for this example. The computed velocities are sometimes larger than the lower bound suggested by equation B-1, but they rarely reach the upper bound suggested by equation B-1. The conclusion from these three examples is that the prescriptive lower and upper bound velocities should not be used or suggested in the NBC.

Also note that for these two Harry’s River examples, the velocities in the flood fringe would provide more rational way to estimate hydrodynamic flood loads compared to the adoption of the definition used in many Canadian flood maps of the flood fringe simply having velocities less than 1 m/s. The velocity in the flood fringe Zone 2 area for Transect 25 is greater than 1 m/s by 24%. The velocity in Zone 3 is slightly higher than the floodway velocity cutoff. Other values are less than 1 m/s but are quantified to allow a more accurate estimate of flood loads than simply assuming either 0 or 1 m/s in the flood fringe.

B.6 Preliminary Evaluation of Prescriptive Method

During this study, a parallel study effort funded by the NRC developed numerical modeling data sets for selected locations around Canada. Two studies by Northwest Hydraulic Consultants Ltd[2] were made available in which the HEC-RAS numerical model as used to simulate flood discharge, elevation, and velocity for flood events with mean recurrence intervals of 10, 20, 50, 100, 200, 500, 1000, and 2500 years.

For each study site, both in Alberta, results were available for 10 river transects. Each transect was subdivided into 20 m wide panels across the entire transect from the left bank to the right bank, spanning the flood fringe and floodway. For each of the two locations, the 10 river transects allowed evaluation of the prescriptive velocity method for 8 transects using, for example transects 1 and 3 to estimate the surface slope at transect 2, transects 2 and 4 to estimate the surface slope at transect 3, and so forth. At each transect, and in each 20 m wide panel, the Manning's n value used in HEC-RAS was tabulated. As a result, the prescriptive method could be applied to predict the man flow velocity in each 20-m wide panel, and results could be compared directly to the mean velocity in the panel predicted by the HEC-RAS model.

Results of this analysis are given in Figures B-10 and B-11. In each figure, velocities predicted by the prescriptive method are compared to those from HEC-RAS with a line of perfect agreement indicated. Results are shown for the 50, 100, 500, and 1000 year flood events. Broad conclusions are that most values from the prescriptive method either agree quite well with the HEC-RAS results or a somewhat conservative and overestimate the velocity.

In cases where data points form a diagonal line, these indicate results for a particular transect. Most of the error between predicted and modeled velocities can be attributed to the estimated water surface slope. In some cases, the up- and down-stream transects used to estimate water surface slopes were spaced far apart, as much as 2,000 m, and this resulted in an overestimate of the local water surface slope at the transect of interest. More refinement in transect locations, closer to the transect of interest, would have produced smaller water surface slopes and velocities in better agreement with the HEC-RAS results. The best results appeared to be achieved when up and downstream transects were less than 1,00 m apart, or less than 500 m either side of the transect of interest.

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[2] Northwest Hydraulic Consultant Ltd., "Generating Flood Data from Floodplain Mapping Studies for Deriving Flood Loads to Support Building Design for Canadian Codes and Standards. Riverine Case Study 1 – North Saskatchewan River at Edmonton." Draft Report, 14 Aug 2020

Northwest Hydraulic Consultant Ltd., "Generating Flood Data from Floodplain Mapping Studies for Deriving Flood Loads to Support Building Design for Canadian Codes and Standards. Riverine Case Study 2 – Peace River at Peace River." Draft Report, 30 Sep 2020

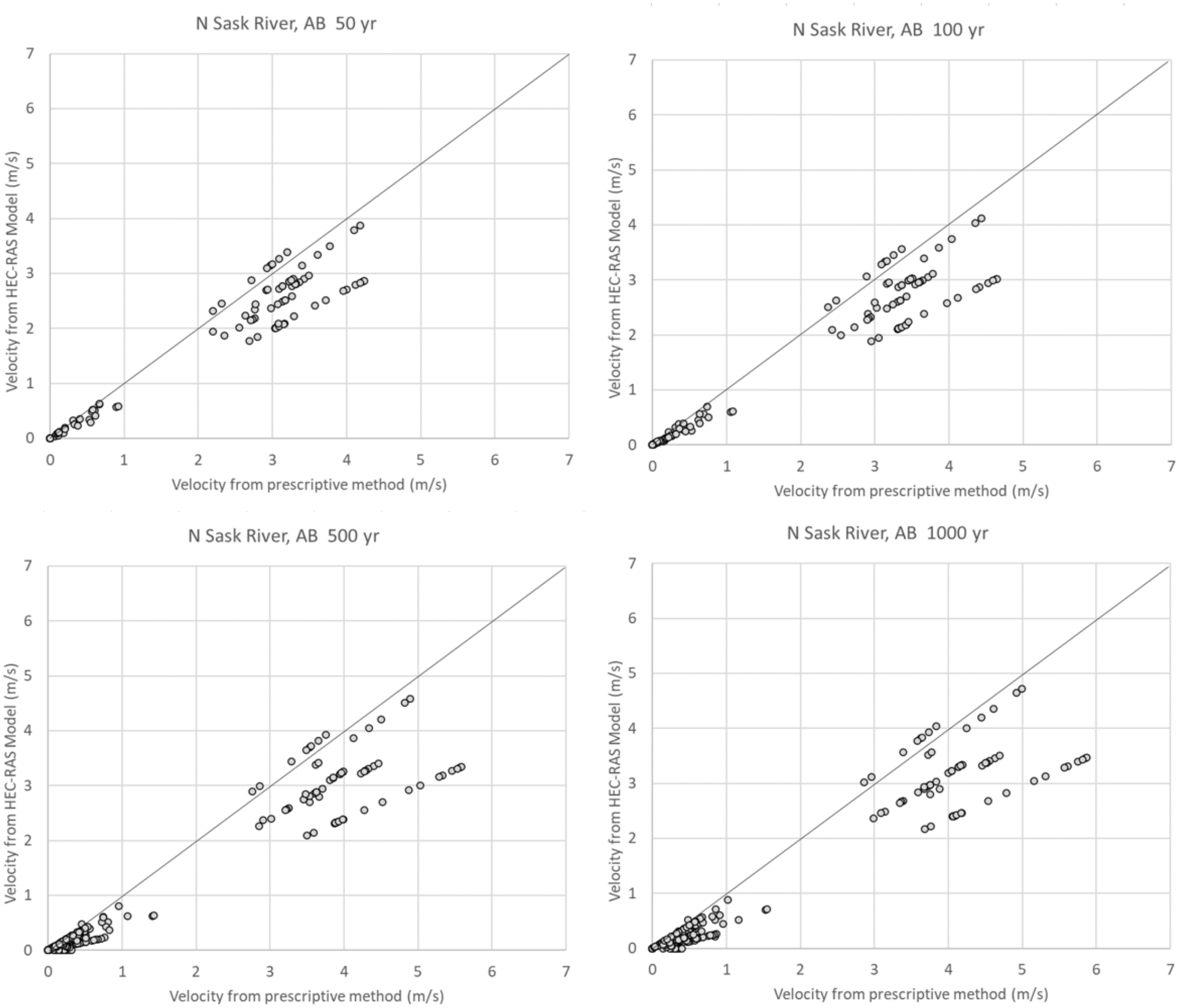


Figure B-10 Comparison of predicted velocities to velocities modeled in HEC-RAS, from NHC Ltd

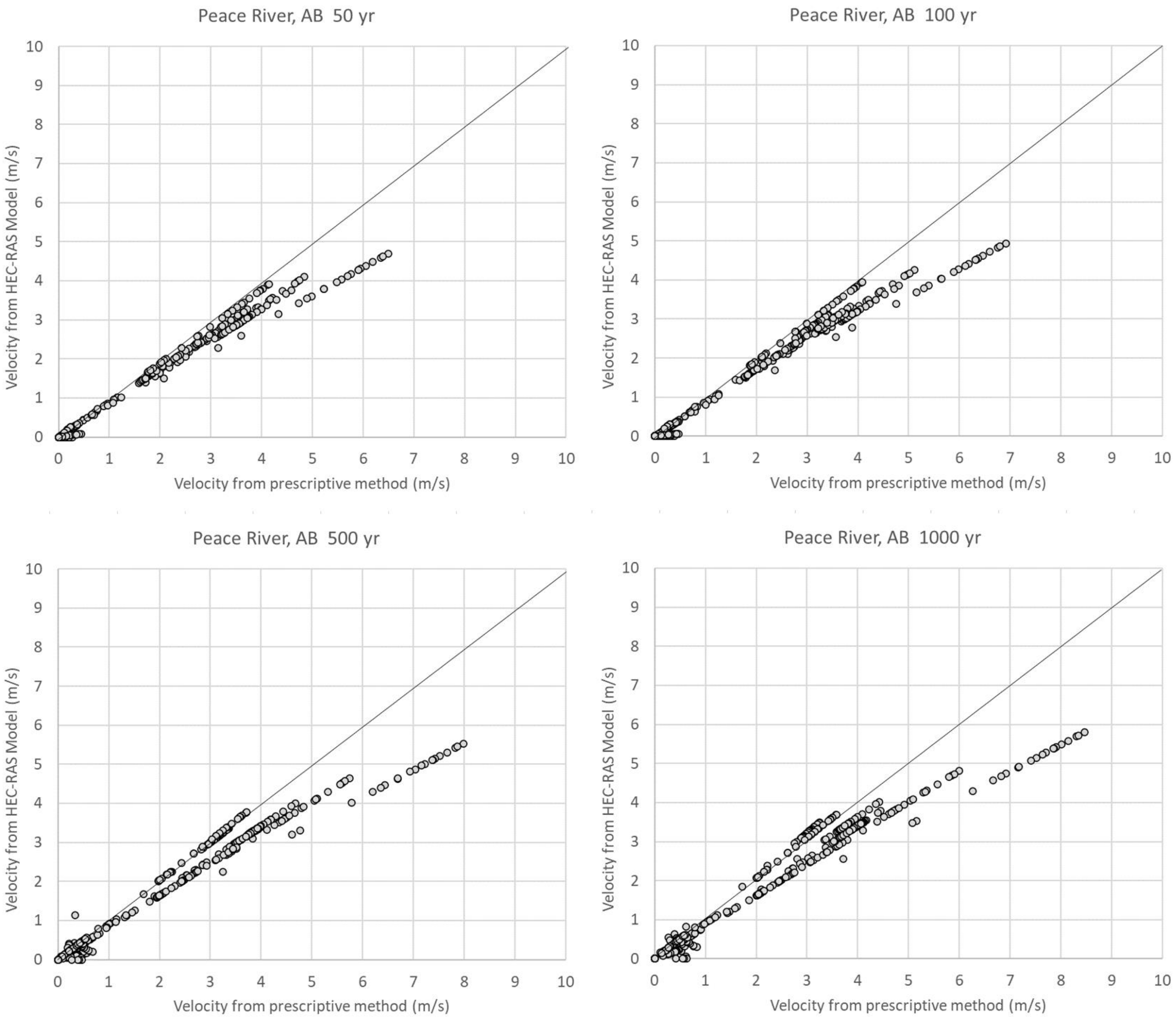


Figure B-11 Comparison of predicted velocities to velocities modeled in HEC-RAS, from NHC Ltd

Appendix C Climate Change Studies

Appendix C includes two case studies from British Columbia and one study from Newfoundland and Labrador. Each study approaches the inclusion of climate change slightly differently and may provide a useful approach to inclusion of climate change in the design requirements for flood-resistant buildings.

The information that follows is a very short summary of the technical reports published for these studies. For complete details and information about how the studies were conducted as well as a discussion regarding the results, the referenced technical reports should be reviewed. The technical reports are included in the reference section. Return periods provided in this appendix are stated in terms of Annual Exceedance Probabilities (AEP) or the chance that an event will be equaled or exceeded in any given year. The AEPs are shown as a ratio or as a corresponding percentage (e.g. 1:100 or 1% per year) and is equivalent to the term "return period", in which AEP = 1/RP

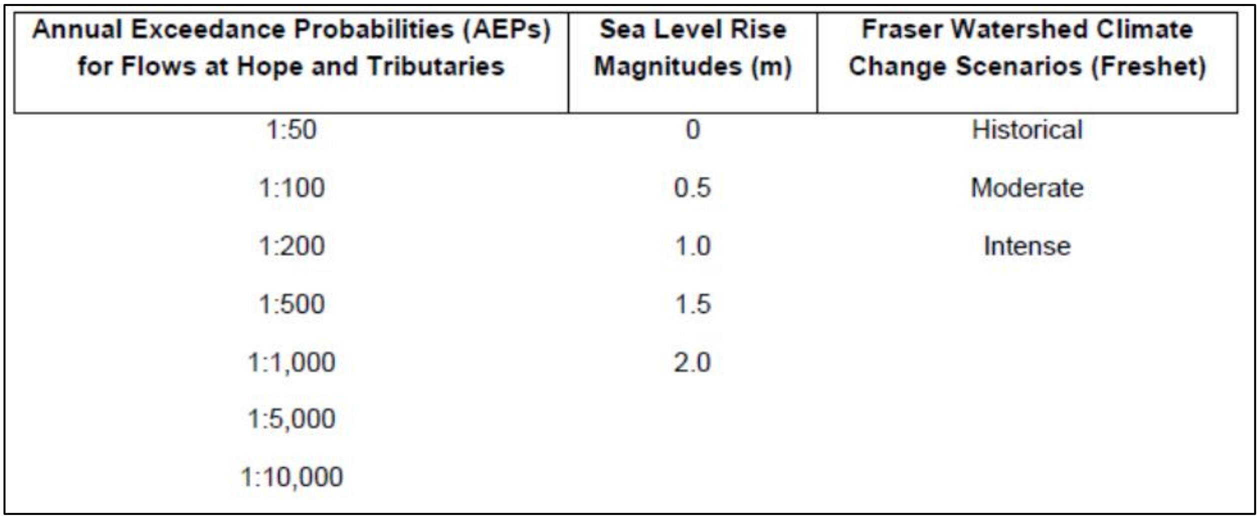
C.1 Fraser River, BC

The Fraser River study was published in 2014 (Fraser River Flood Scenarios 2014). The purpose of the study was to create a planning tool for the community using a re-examination of the Lower Fraser River flood profiles that includes the anticipated effects of sea level rise, potential climate changes on flood flows, and that incorporates updated flood protection standards. The study does not consider the effects of subsidence.

The consultant provided the following information based on their scope of work:

1. Annual maximum daily discharges for a range of AEP from 1:50 to 1:10,000 for the Fraser River at Hope and its tributaries.
2. Annual maximum daily discharges for a range of AEPs from 1:50 to 1;10,000 considering climate change scenarios described as "Moderate" and "Intense".
3. Winter ocean levels for a range of AEPs from 1:50 to 1:10,000 using a joint probability approach of high tide plus storm surge. Sea level rise scenarios of 0, 0.5, 1.0, 1.5, and 2.0 were then applied to these levels.

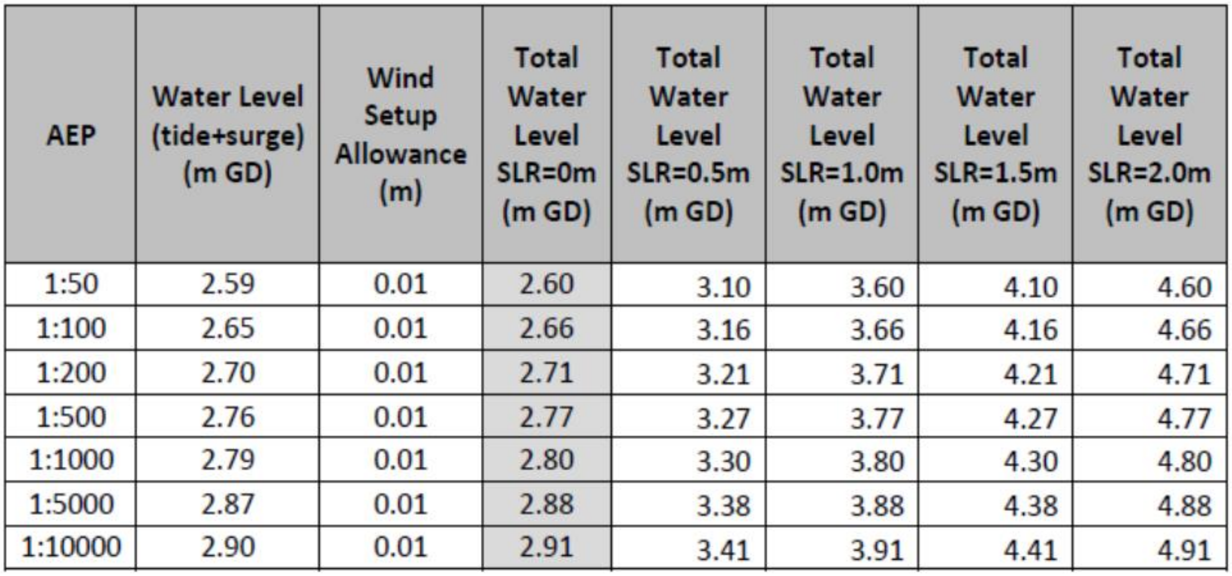
The flood scenarios considered in the consultant's report are shown in Figure C-1 and were provided in the study report as Table 2.



Annual Exceedance Probabilities (AEPs) for Flows at Hope and Tributaries	Sea Level Rise Magnitudes (m)	Fraser Watershed Climate Change Scenarios (Freshet)
1:50	0	Historical
1:100	0.5	Moderate
1:200	1.0	Intense
1:500	1.5	
1:1,000	2.0	
1:5,000		
1:10,000		

Figure C-1 Flood Scenarios Included in Flood Study Report (source: Fraser River Flood Scenarios 2014)

The resulting ocean levels for the various AEPs and various sea level rise scenarios at the Fraser River outlets to the ocean are shown in Figure C-2 and were provided in the report as Table 6. The sea level rise estimates are associated with predicted increases above the current levels in 2010 for years 2050, 2100, 2150, and 2200. Therefore the 'intense' increase of 1.0 m represents the anticipated sea level rise in year 2100.



AEP	Water Level (tide+surge) (m GD)	Wind Setup Allowance (m)	Total Water Level SLR=0m (m GD)	Total Water Level SLR=0.5m (m GD)	Total Water Level SLR=1.0m (m GD)	Total Water Level SLR=1.5m (m GD)	Total Water Level SLR=2.0m (m GD)
1:50	2.59	0.01	2.60	3.10	3.60	4.10	4.60
1:100	2.65	0.01	2.66	3.16	3.66	4.16	4.66
1:200	2.70	0.01	2.71	3.21	3.71	4.21	4.71
1:500	2.76	0.01	2.77	3.27	3.77	4.27	4.77
1:1000	2.79	0.01	2.80	3.30	3.80	4.30	4.80
1:5000	2.87	0.01	2.88	3.38	3.88	4.38	4.88
1:10000	2.90	0.01	2.91	3.41	3.91	4.41	4.91

Figure C-2 Ocean Levels at Fraser River Outlets with Various SLR Scenarios (Source: Fraser River Flood Scenarios 2014)

River flows were also studied for 'Moderate' and 'Intense' climate change scenarios. The climate change scenarios included variations on snowpack and melting times, and temperatures across the floodplain. Dam breaches and floods caused by landslides were also considered in the study. The study makes a strong case for considering the level of uncertainty in climate change predictions and on the resulting possible changes in flood flows and water surface elevations. Projections were frequently provided with 5% and 95% confidence levels. The study indicates that the future 50-year flood is approximately equal to current return periods of 200 and 500 years, illustrating the non-stationarity of flood events projected into the future.

C.2 City of Surrey, BC

The City of Surrey study was published in 2012 (Serpentine 2012). The purpose of the study was to examine the combined impacts of sea level rise and subsidence by the year 2100 on water elevations and the adequacy of the existing dike systems. Some of the project goals were:

1. Determine future projected sea levels in Mud and Semiahmoo Bays consistent with guidelines published by the Ministry of Environment
2. Determine future projected sea levels through statistical analysis of various duration tidal patterns, storm surge and wind/wave setup

The 1:200 AEP event was the only return period developed for this study. The study covered the Serpetine, Nicomekl and Campbell Rivers. The study assumed 0.97 m sea level rise and subsidence of 0.225 m by 2100. The total net increase in water levels due to sea level rise and subsidence used in the study was 1.195 m. Future increases in storm frequency and intensity were not considered in this study.

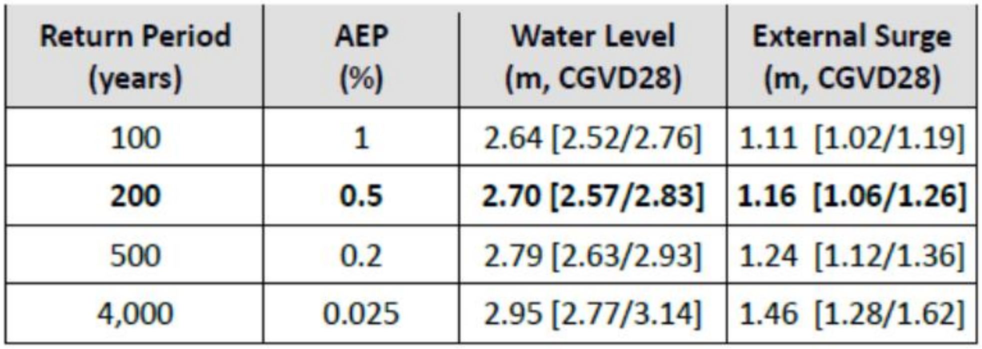
Equations for determining the Designated Flood Level and the FCL used in the study are:

Designated Flood Level (DFL) = Relative Sea Level Rise (RSLR) + HHWLT + 200yr Storm Surge + Max Wind Setup

Flood Construction Level (FCL) = DFL + 0.5 x 2% Runup + Freeboard (0.6 m)

Dike Crest Elevation (DCE) = DFL + 2% Runup + Freeboard (0.6 m)

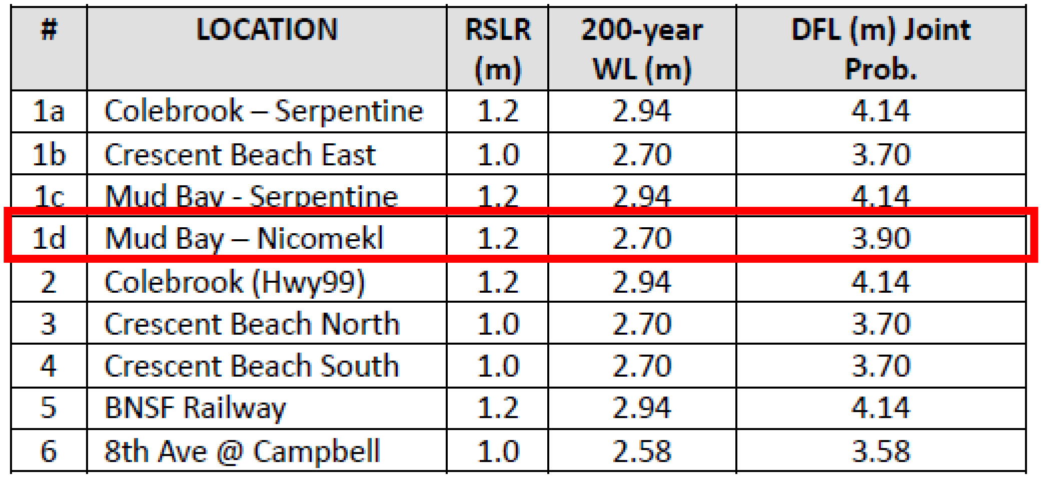
The study found that most dikes were overtopped by the 1:200 AEP event in 2100 and thus runup was added to the FCL and Dike Crest Elevation. The Dike Crest Elevation was arbitrarily increased by 1.5 m to avoid being overtopped in the study. At one location at the Nicomekl River and current conditions in 2010, the water levels for various return periods were determined by the joint probability method to be as listed in Figure C-3.



Return Period (years)	AEP (%)	Water Level (m, CGVD28)	External Surge (m, CGVD28)
100	1	2.64 [2.52/2.76]	1.11 [1.02/1.19]
200	**0.5**	**2.70 [2.57/2.83]**	**1.16 [1.06/1.26]**
500	0.2	2.79 [2.63/2.93]	1.24 [1.12/1.36]
4,000	0.025	2.95 [2.77/3.14]	1.46 [1.28/1.62]

Figure C-3 Water Elevations (m) at Nicomekl River in 2010

A comparison of years 2010 and 2100 water surface elevations is shown in Figure C-4. A sea level rise of 1.2 m net increase (seal level rise + subsidence) is added to the 2010 water elevations and is shown in the column titled DFL Joint Probability. The Nicomekly River location is highlighted in red.



#	LOCATION	RSLR (m)	200-year WL (m)	DFL (m) Joint Prob.
1a	Colebrook – Serpentine	1.2	2.94	4.14
1b	Crescent Beach East	1.0	2.70	3.70
1c	Mud Bay - Serpentine	1.2	2.94	4.14
1d	Mud Bay – Nicomekl	1.2	2.70	3.90
2	Colebrook (Hwy99)	1.2	2.94	4.14
3	Crescent Beach North	1.0	2.70	3.70
4	Crescent Beach South	1.0	2.70	3.70
5	BNSF Railway	1.2	2.94	4.14
6	8th Ave @ Campbell	1.0	2.58	3.58

Figure C-4 Comparison of Water Surface Elevations (m) (2010 and 2100); Nicomekl highlighted in red

The DFL for year 2100 includes sea level rise but does not include wave runup, so the design flood elevation needs to be increased an additional 0.23 m to include the wave effect.

Figure C-5 illustrates the location of dikes in the flood study area and shows the comparison of existing dike crest elevations (DCE), the required DCE for current conditions in 2010, and the required DCE for 2100 flood conditions. Note that there are several locations where the required DCCE for 2100 is more than 2-3 m higher than the existing DCE indicating substantial work is required to raise the DCE in order to not have the dikes overtopped by flooding in 2100. Dike overtopping renders the dikes ineffective at stopping or reducing flooding. There was no information in the study about what the consequences of such overtopping would be. Of course, the floodplain boundaries would be substantially larger, and many structures would be potentially flooded if such overtopping were to occur.

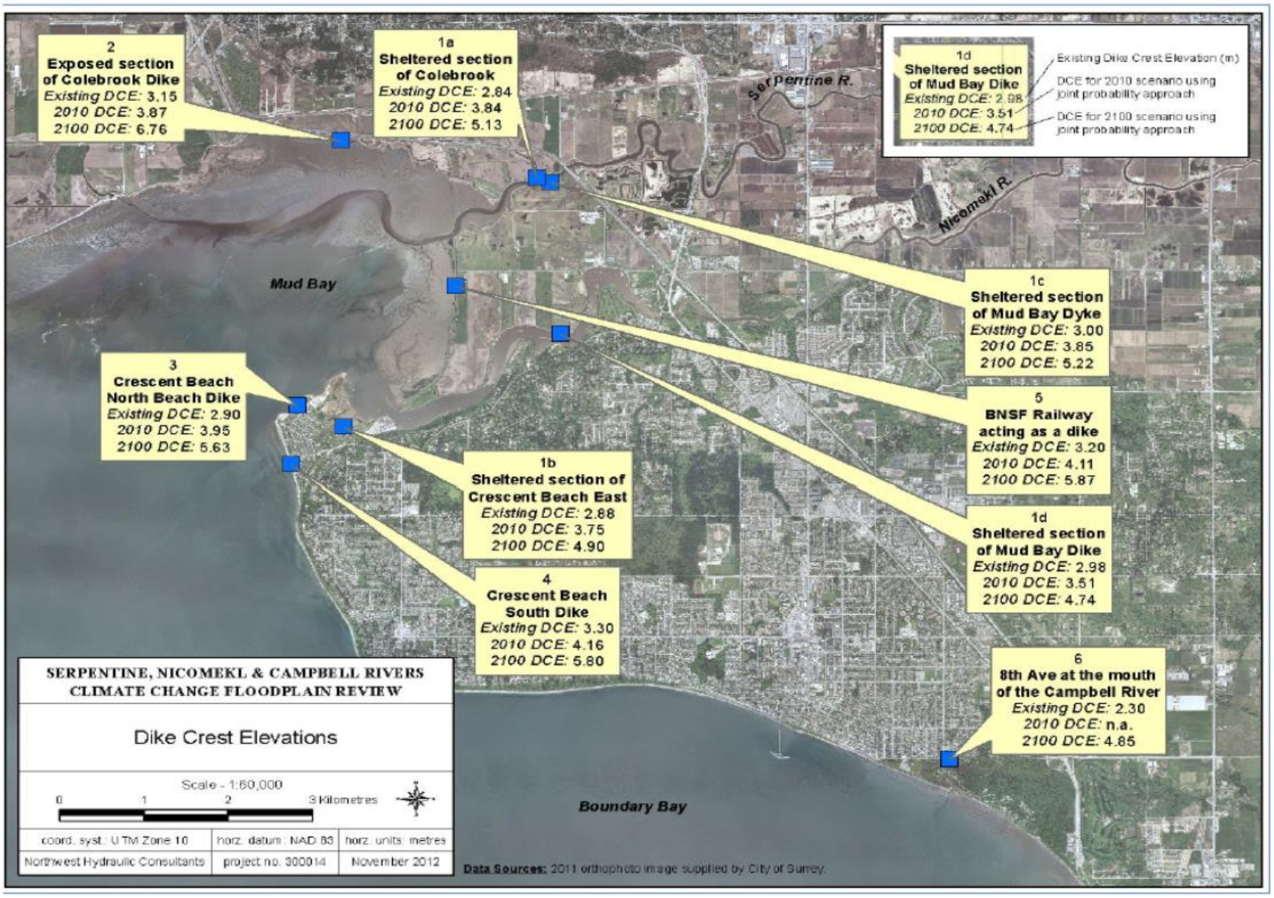


Figure C-5 Dike Crest Elevations (m) (Existing, 2010, and 2100)

C.3 Waterford River, Newfoundland and Labrador

The Waterford River study was awarded to the consultant in late 2015 and was published in 2018 (Waterford River 2018). The previous flood study was published in 1998, so the study was 20 years old when this revised study was published. The hydrologic studies for both current and future conditions were based on 1:20 and 1:100 AEP events.

The climate change projections were made for the year 2100 and include the effects of both increased precipitation and sea level rise. The precipitation levels were estimated for the future using intensity-duration-frequency curves, land cover types, and rainwater detention rates. The estimated future conditions at the coast were based on sea level rise projections for the year 2100 and consider the joint probabilities of the sea level rise and water surface elevation increases caused by precipitation.

Information (mostly in the form of maps) was developed for the following flood parameters: water surface elevation (m), depth (m), flow (m^3/m), velocity (m/s), and inundation boundaries. The velocity and depth information were used to develop flood hazard maps based on the Royal Haskoning flood hazard matrix. Cross sections were cut across the floodplain where flood elevations and velocities were developed. The maps shown below as Figures C-6 thru C-7 show the cross sections. The flood elevations are shown at the cross section with the first number being the 1:20 AEP and the second number being the 1:100 AEP. The velocities are shown as a range by colour coding the map. Ground elevation contours are shown on these maps, but the scale illustrated here make those contours difficult to read, Contours on larger maps would be easier to read.

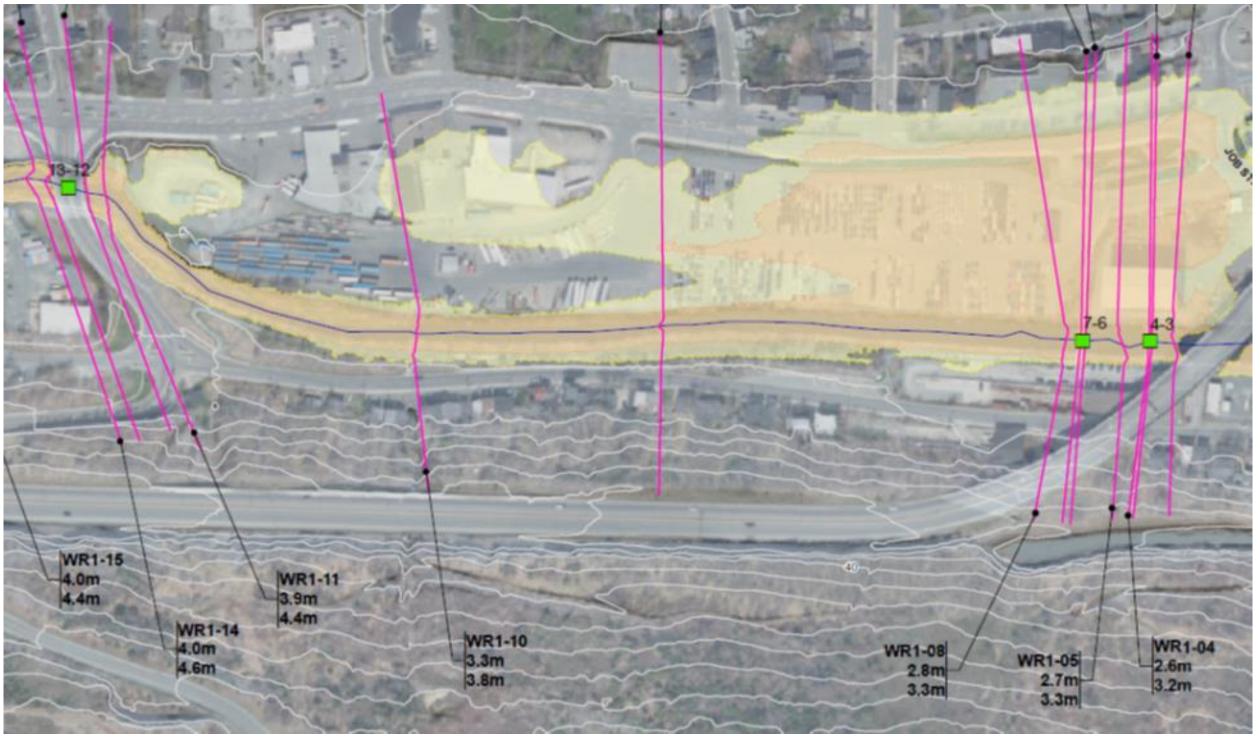


Figure C-6 1:20 and 1:100 AEP Floodlines Waterford River, NL Current Conditions

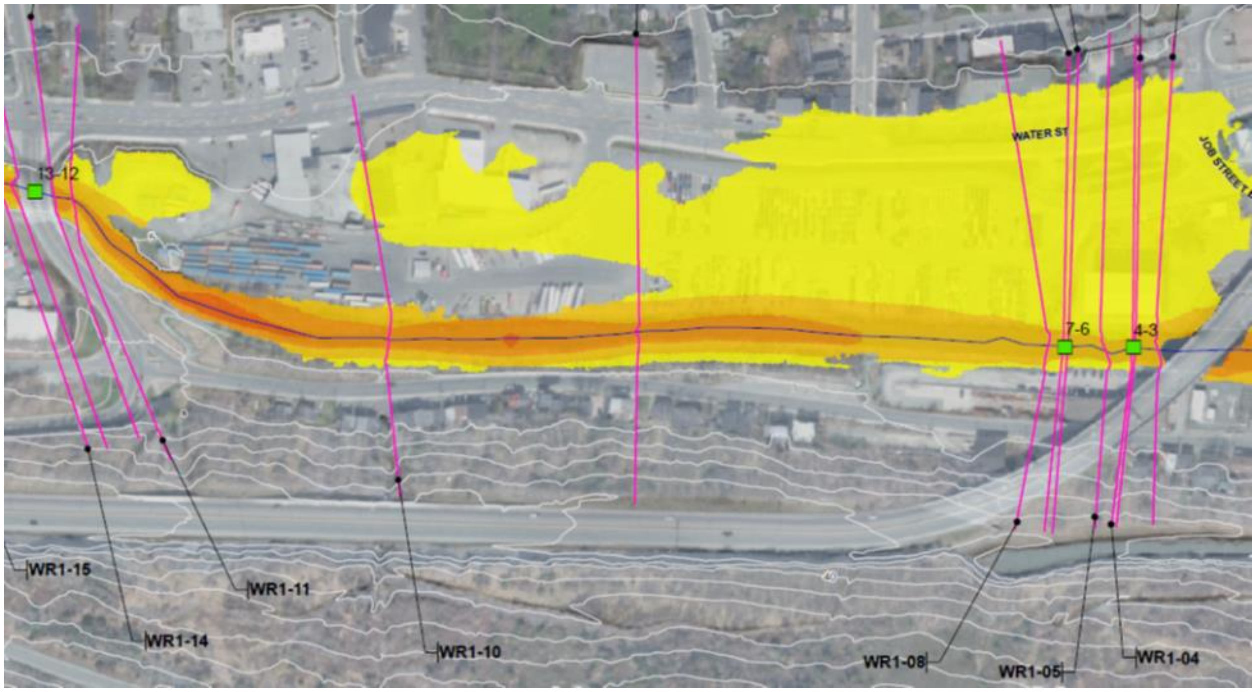


Figure C-7 1:100 AEP Velocity Waterford River, NL Current Conditions

Figure C-8 1:20 and 1:100 AEP Floodlines Waterford River, NL Future Conditions

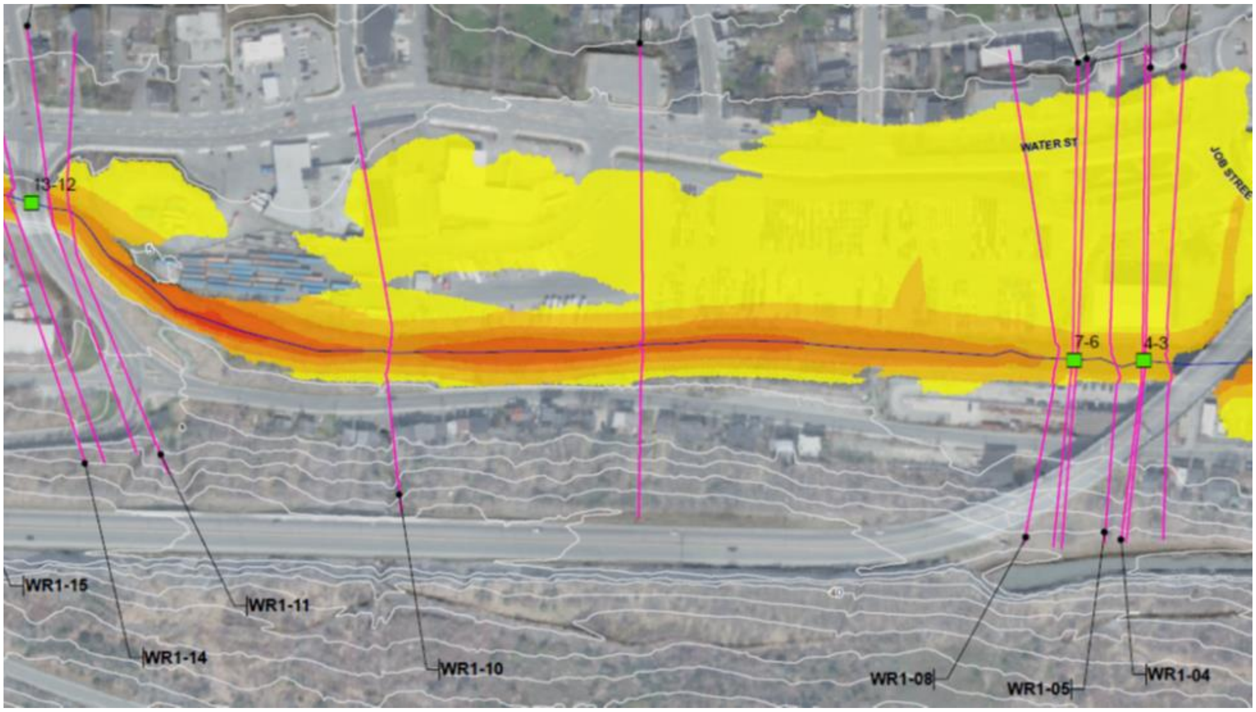


Figure C-9 1:100 AEP Velocity Waterford River, NL Future Conditions

If the 1:20 and 1:100 AEP events are extended to other return periods using the method in Chapter 2, and the return periods for the current conditions and future conditions are compared, the resulting flood elevations for Cross Section WR1-11 are illustrated in Table C-1.

Table C-1 Comparative flood elevations for current and future conditions at cross section WR1-11 for varying return periods

	AEP Return Periods and Associated Flood Elevations (m)					
Design Flood Conditions	1:20	1:50	1:100	1:200	1:500	1:1000
Current	3.9	4.2	4.4	4.6	4.9	5.1
Future	4.3	4.7	5.0	5.3	5.7	6.0

The maps illustrating velocity are based on a range of velocities in metres/second. These maps are not as definitive as the water surface elevation maps since the colours on the map must be interpreted to establish the velocity range.

References

"Serpentine, Nicomekl & Campbell Rivers Climate Change Floodplain Review", Final Report, City of Surrey, Northwest Hydraulic Consultants, 2012

"Simulating the Effects of Sea Level Rise and Climate Change on Fraser River Flood Scenarios", Final Report, BC Ministry of Forests, Lands and Natural Resource Operations, Northwest Hydraulic Consultants, 2014

"Waterford River Area Flood Risk Mapping Study", Final Report, Department of Municipal Affairs and Environment, CBCL Limited Consulting Engineers, 2018

Appendix D Summary of Recommendations for Flood-Design of Buildings

This appendix is a summary of design flood conditions and flood load formulas discussed in Chapters 2 and 3 of this guide. The purpose of this summary is to provide a user with a synopsis of the design flood parameters covered in this guide. This synopsis could be used by developers of continuing education, by building code authorities, or by provincial governments to enhance their current flood design guidance.

The scope of this guidance is limited to those conditions that are developed into a flood hazard map and are thus available to the public for use in flood design. Present and future flood conditions are considered. Flood sources to be considered include overland flood due to high river stages including ice jam effects, elevated coastal (and Great Lakes) water levels, groundwater effects of saturated soils, and above-ground water that flows outside defined channels as long as flooding from these sources is included in flood hazard maps. Flood conditions apply to new and existing buildings.

There is no limitation on the use of flood design conditions and formulas covered in this summary as long as there is an adequate method for determining the flood hazard that is accepted by the Authority Having Jurisdiction. The flood hazard includes flood depth, flood velocity, wave effects, scour and erosion caused by flooding, and the inclusion of future conditions at any site for which a flood design is required. The flood hazard may be determined by a flood map developed at the community, city or province level or may be developed by a site-specific flood hazard study.

D.1 Flood Design Conditions and Considerations

The design flood is established by selecting the Mean Recurrence Interval (MRI) (or related AEP) for the importance category of the building based on Table 4.1.2.1 of the NBC used to define use and occupancy classes (and shown here as Table D-1.

D.1.1 Flood Depth

D.1.1.1 Design Flood Level

The design flood level or DFL should be established for the following mean recurrence intervals:

Table D-1 MRI recommendation for NBC Importance Categories for Buildings

Importance Category	Recommended Design Flood Elevation AEP
Low	1:100
Normal	1:500
High	1:750
Post-disaster	1:1000

Where data are not available, $SWEL_{MRI}$ shall be determined according to Equation D.3.

$$S_n = S_{R1} + \alpha * (S_{R2} - S_{R1}) \qquad \text{(Eq. D-1)}$$

Where α is a scaling factor for AEP other than 1:100 and varies depending on the riverine, coastal or Great Lakes condition. The scaling factor α to use for Eq. D.3 are provided in Table D.2 where SR1 is the water surface elevation for a 1:10 AEP and SR2 is the water surface elevation for a 1:100 AEP.

Table D-2 Scaling factor α for use with MRIs

MRI or AEP	Riverine Locations	Coastal Locations	Great Lakes Locations
1:100	1.0	1.0	1.0
1:500	1.71	171	1.67
1:750	1.91	1.88	1.84
1:1000	2.04	2.01	1.96

The scaling factors include an increase to account for uncertainty and to provide a 95% confidence level.

D.1.1.2 Riverine Design Flood Depth

The design flood depth is given by

$$d_f(\text{m}) = \text{DFL} - \text{ground elevation} + \text{future conditions} \qquad \text{(Eq. D-2)}$$

where:

Design flood elevation = elevation of the design flood determined for the building use and occupancy class at the MRI selected from Table D.1

Ground elevation = lowest ground surface adjacent to the building of interest inclusive of erosion and subsidence.

Future conditions (m) = relative sea level change, subsidence, erosion, hydrologic changes, land use change, and other factors expected to occur within the service life.

The Flood Construction Level (FCL) must be equal to or greater than the d_f.

D.1.1.3 Coastal Design Flood Depth

The coastal design flood depth

$$\boldsymbol{d_f}(\text{m}) = \text{DFL} - \text{ground elevation} + \text{wave setup} + \text{future conditions} \qquad \text{(Eq. D-3)}$$

where:

DFL = elevation of the design flood determined for the building use and occupancy class at the MRI selected from Table D.1 For coastal locations, this is sum of the HHWLT elevation + storm surge

Ground elevation = lowest ground surface adjacent to the building of interest inclusive of erosion and subsidence.

Future conditions (m) = relative sea level change, subsidence, erosion, hydrologic changes, land use change, and other factors expected to occur within the service life.

D.1.2 Flood Velocity

Flood velocity at the site of interest shall be determined using an acceptable method available to the industry. If specific velocity information is not provided in flood hazard information or in flood studies, approximate methods may be used.

An acceptable approximate method for riverine conditions uses the common hydraulic modeling tool from HEC-RAS (from US Army Corps of Engineers) to develop site-specific flood velocities. Equation 5.4 includes terms that must be developed to find velocity at a point in a stream or river.

$$V = \frac{k}{n} R_h^{2/3} S_f^{1/2} \qquad \text{(Eq. D-4)}$$

where:

S_f = slope of energy grade line (m/m or ft/ft), or energy gradient, given by head loss from upstream to downstream. In this approximate method, S is the slope of the stream or river taken from the water surface elevations of 2 transects on either side of the site of interest.

R_h = Hydraulic radius (m or ft), given by $R_h = A/P$ or approximately by $R_h \sim A/W$

A = area of the flooded cross section (m^2 or ft^2) illustrated in Figure D-1

P = Wetted perimeter of the cross-sectional area (m or ft). In this approximate method, P is the width of the stream or river taken at the water surface elevation across the width of the flood plain W.

W = Width of the cross section (m or ft)

k = Coefficient associated with units (k=1 in SI units, k=1.486 in U.S. Customary units)

n = Manning's roughness coefficient, related to ground cover, flow turbulence, flow meandering, and flow obstructions (typical Manning roughness coefficients can be found in flood studies or hydraulic engineering textbooks)

The velocity shall be determined separately for areas of the flood fringes and the floodway.

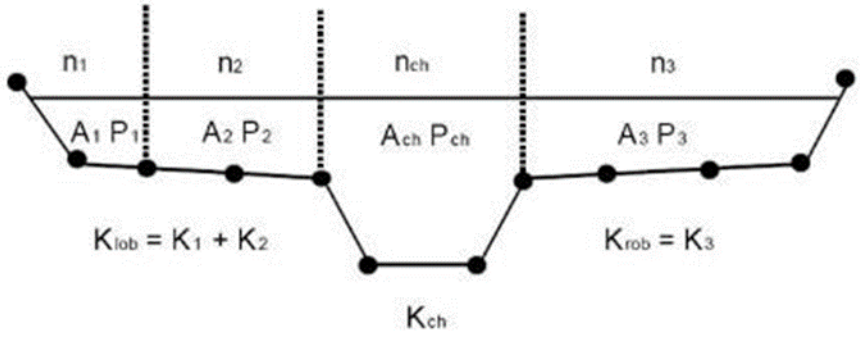


Figure D-1 Subdivision of cross-sectional area to define main river channel (floodway) along with left and right overbank sections (flood fringe) (source: HEC-RAS Manual, 2016)

Coastal flood velocity shall be considered using the formula in Section D.2.1 to determine coastal hydrodynamic loads. Coastal flood velocity shall be considered for both toward the inland direction and back toward the sea direction of flow.

D.1.3 Flood Duration

The duration of flooding shall be considered. Long duration floods that inundate buildings will create more damage to both buildings and contents. Mitigation efforts shall be attempted to reduce potential damage caused by long duration flooding.

D.1.4 Flood-borne Debris

Debris and their effects shall be considered in accordance with this section. Where the design stillwater flood depth at the site is less than 1 m (3 ft) the effects of debris may be neglected.

Design for debris impact loads shall account for the most severe effect of the impact loads over the required depth defined in section D.2 and shall consider strikes from all possible directions. Impact loads need not be considered on multiple structural elements simultaneously.

EXCEPTION: Where a site-specific study shows flow directions, debris impacts need only be considered from the directions shown in the site-specific study.

For riverine sites, debris strikes need only be considered from the upstream direction.

D.1.4.1 Debris Impact Objects Considered

All buildings and structures as required in D.3.8 shall be designed for impact loads from floating debris. Debris impacts to a building façade shall be considered when flood water inundation inside the building could create a significant loss to the building, its occupants, or the operation housed in the building. Debris considered shall be determined per Table D.3 based on building use and occupancy, flow depth, and structure type. A site hazard assessment shall be used to determine the applicability of debris strikes from shipping containers, ships/barges, and extraordinary debris. Debris loading shall be determined per Section D.2.1.

Table D-3 Debris type applicability

Debris Type	Applicable Use Categories	Threshold Depth (ft)	Impact to Primary Structure	Impact to Façade
Ice	Open, High, Post-disaster	1 m (3 ft.)	Yes	Yes
Wood Poles	Open, High, Post-disaster	1 m (3 ft.)	Yes	Yes
Passenger Vehicles	Open, High, Post-disaster	1 m (3 ft)	Yes	Yes
Small Vessels	Open, High, Post-disaster	1 m (3 ft)	Yes	Yes
Shipping Containers	Open, High, Post-disaster	1 m (3 ft)	Yes[1]	n/a
Ships/barges	High, Post-disaster	2 m (6 ft)	Yes[1]	n/a
Extraordinary Debris	Post-disaster	4 m (12 ft	Yes[1]	n/a

D.1.5 Erosion

The effects of erosion shall be included in the calculation of ground elevation used to establish flood depths and flood loads. The recession of the shoreline or local soils due to erosion shall be considered over the service life of the structure. A minimum service life of 50 years shall be used for this quantification.

EXCEPTION: Analysis of erosion is not required if soils adjacent to structure foundations are non-erodible or protected against erosion by elements or structures designed for the anticipated flood loads. Erosion need only be considered to the top of non-erodible strata.

D.1.6 Future Flood Conditions

The increased coastal and inland flood depths resulting from anticipated future conditions shall be quantified, including the effects of relative sea level change, subsidence, erosion, hydrologic changes, land use change, and other factors expected to occur within the service life. A project service life of not less than 50 years shall be used for this quantification. The minimum rate of relative sea level change shall be the historically recorded rate of relative sea level change. Where required by the Authorizing Authority, other future conditions shall be considered.

D.2 Flood Load Formulas and Provisions

This section includes the following subsections related to various types and elements of flood loads.

D.2.1 Hydrostatic Pressure

Hydrostatic pressure originates from the effective weight of the fluid acting on a submerged surface.

$$P = \rho g z = \gamma z \qquad \text{(Eq. D-5)}$$

where:

z = depth of submergence, from SWL to point of interest (including in submerged soils)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
ρ= water density, 1,000 kg/m^3 fresh, 1,025 kg/m^3 salt (1.94 slug/ft^3 fresh, 1.99 slug/ft^3 salt)
γ= water unit weight, 9,810 N/m^3 fresh or 10,055 N/m^3 salt (62.4 lb/ft^3 fresh, 64.0 lb/ft^3 salt)

D.2.2 Buoyancy

Vertical buoyant forces shall be computed for all portions of a structure below the design flood level as

$$F_B = \rho g \cancel{V} = \gamma \cancel{V} \qquad \text{(Eq. D-6)}$$

where:

F_B = vertical buoyant force due to hydrostatic pressures, in N (or lb)
$\cancel{V}$ = volume of water displaced by the structural element, in m^3 (or ft^3)

The buoyancy shall be applied over the surface of the submerged horizontal area and may be treated as a force per Eq. D.6 or applied over an area as N/m^2.

D.2.3 Vertical Hydrostatic Pressure

Vertical hydrostatic pressure may be computed when Archimedes Principle shown as Eq. D.6 cannot be readily used as illustrated in Figure D.2.

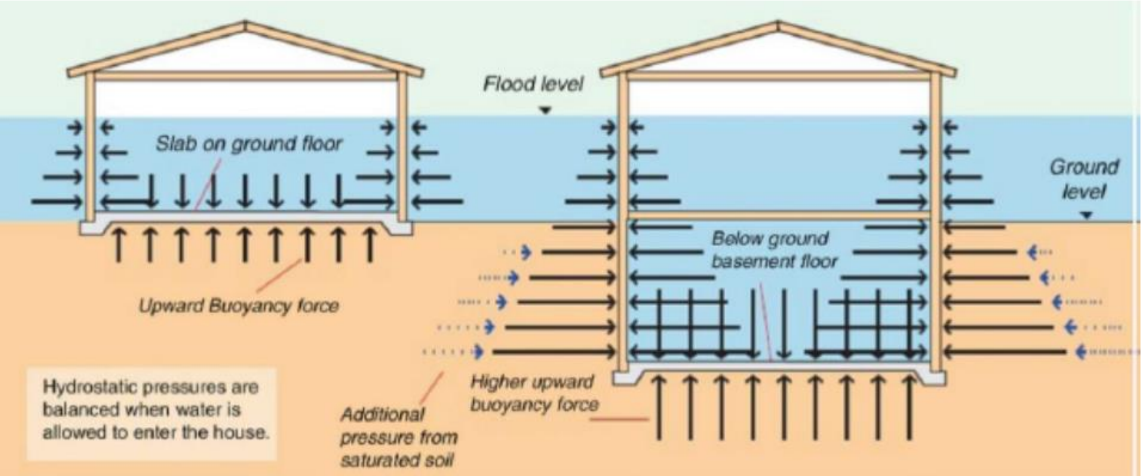


Figure D-2 Distribution of hydrostatic pressure when building retains fluid

The formula for determining vertical hydrostatic force is

$$F_v \;=\; \left(P_{bot} - P_{top}\right)A_h \;=\; \left(\rho\, g\, Z_{bot} - \rho\, g\, Z_{top}\right)A_h \tag{Eq. D-7}$$

where:

F_v = vertical hydrostatic force, in N (lb)
P_{bot} = hydrostatic pressure on bottom of structure or element, based on Z_{bot}
P_{top} = hydrostatic pressure on top of structure or element, based on submerged depth to top Z_{top}
A_h = contact area over which pressures act, in horizontal plane
Z_{bot} = submerged depth to bottom of structure or element, from free water surface
Z_{top} = submerged depth to top of structure or element, from free water surface

D.2.4 Lateral Hydrostatic Pressure

Lateral hydrostatic pressure acts over a submerged vertical surface. Pressures increase linearly with depth. Depth shall be taken as the depth of the FCL that is based on the design flood depth (Eq. D.1 or D.2). The pressure per unit wall length is

$$f_H \;=\; ½\, \rho\, g\, d_f^2 \tag{Eq. D-8}$$

where:

f_H = hydrostatic force per unit width along the wall, in N/m (or lb/ft)
d_f = design flood depth, based on design flood level minus ground elevation, including any added depth due to erosion or scour, as defined in Chapter 2.

The hydrostatic force for a wall length w is

$$F_H \;=\; w\, f_h = w\; ½\, \rho\, g\, d_f^2 \tag{Eq. D-9}$$

where:

F_H = hydrostatic force on the wall, in N or lbs
w = width of the wall perpendicular to the flow, in m (ft)

The hydrostatic force shall be applied at the 2/3 depth of the fluid depth.

For walls that are elevated above ground level, when a building is elevated on piles or column foundations, the hydrostatic force would then be given by

$$F_H \;=\; w\, f_H \;=\; w\; ½\, \rho\, g\, d_b^2 \tag{Eq. D-10}$$

where:

d_{b} = submerged height of wall from design water level to bottom of wall.

For a wall that extends below grade, the total hydrostatic force is given by

$$F_H \;=\; w\, f_H \;=\; w\, ½\, \rho\, g\, (d_f^2 + d_g^2) \qquad \text{(Eq. D-11)}$$

where:

d_g = submerged depth below grade

The point of application would then be at 2/3 (d_f + d_g) below the water surface.

This represents only the water (flood) load. The geotechnical loads due to soil effective stress would then be added, but this is not considered part of the flood load.

D.2.5 Hydrodynamic Loads

Hydrodynamic loads are caused by moving water impacting a structural element or building. Hydrodynamic flow creates drag on the element or building impacted by the flow. When flow velocity is lower than 3 m/s (10 ft/sec), the load may be converted to an additional flood depth which is added to the depth used for calculating hydrostatic loads. The additional depth surcharge is determined by

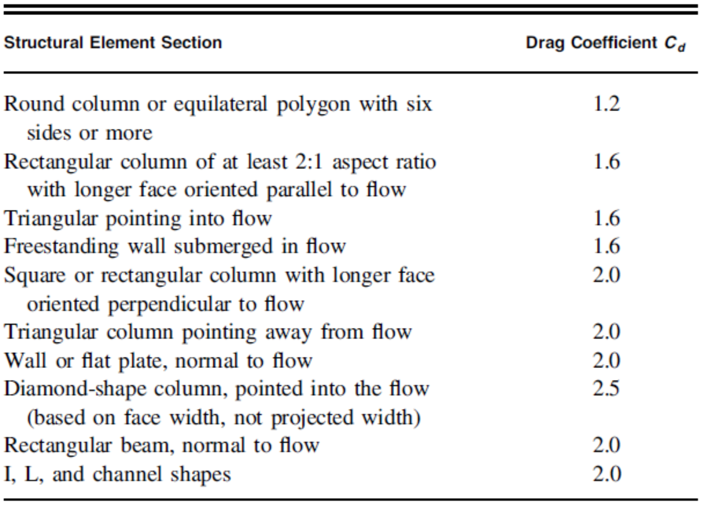
$$F_D \;=\; ½\, C_D\, \rho V^{\,2}\, A_p \qquad \text{(Eq. D-12)}$$

where:

F_D = hydrodynamic drag force, in N (or lbs)
C_D = drag coefficient, function of structure shape and dimensions
V = velocity of flood water, in m/s (or ft/s)
A_p = the projected area of structure or structural element exposed to the moving water, including any erosion or scour

The application of the load shall occur at the mid-point of the water depth which assumes uniform flow velocity over the depth. The drag coefficient CD varies based on the geometry of the element or the ratio of the depth of water to the building width if the entire building is the object being impacted by the flow. Suggested CD values are shown in Tables D-4 and D-5.

Table D-4 Drag coefficients on structural elements



Structural Element Section	**Drag Coefficient C_d**
Round column or equilateral polygon with six sides or more	1.2
Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow	1.6
Triangular pointing into flow	1.6
Freestanding wall submerged in flow	1.6
Square or rectangular column with longer face oriented perpendicular to flow	2.0
Triangular column pointing away from flow	2.0
Wall or flat plate, normal to flow	2.0
Diamond-shape column, pointed into the flow (based on face width, not projected width)	2.5
Rectangular beam, normal to flow	2.0
I, L, and channel shapes	2.0

Table D-5 Drag coefficients for buildings[a]

Width to inundation depth ratio w/d_f[b]	**Drag Coefficient C_d**
< 12	1.25
16	1.3
26	1.4
36	1.5
60	1.75
100	1.8
≥ 120	2.0

[a] drag coefficient may be interpolated between tabular values based on w/d_f

[b] ratio shall be based on the design flood depth d_f

D.2.6 Debris Loads

Accumulation loads: Debris loads can occur as debris accumulation loads or debris impact loads caused by debris striking a structural element or building. There is limited guidance on predicting drift cause by debris accumulations. Guidance by the USACE (1995) suggests a uniform load of 1.48 kN/m (100 lb./ft) acting in a strip 0.3 m (1 ft) thick along the length of the structure at the waterline. A prediction of the forces that could cause drift can be determined by

$$F_{Da} = ½\, C_D\, \rho V^{2} A_a \qquad \text{(Eq. D-13)}$$

where:

F_{da} = drag force due to debris accumulation, in N (lb)

V = flow velocity upstream of debris accumulation, in m/s (ft/s)

A_a = projected area of the debris accumulation into the flow, approximated by depth of accumulation times width of accumulation perpendicular to flow, in m^2 (ft^2)

C_D = drag coefficient, assumed to equal 1

Impact loads: Impact loads occur from water-borne debris striking a structural element or the building. Lab test results form the basis of the method suggested below. The coefficients required for Equation D-14 are found in Figures D-3 and D-4, and Table D-6.

$$F_{di} = 1.57\, C_{de}\, C_{bl}\, C_{or}\, R_{max}\, W\, V \,/\, (g\, \Delta t) \qquad \text{(Eq. D-14)}$$

where:

F_{di} = impact force, in N (lbs)
W = weight of debris, in N (lbs)
V = velocity of floodwater propelling debris, in m/s (ft/s)
Δt = impact duration or time to reduce object velocity to zero, in sec, taken as 0.03 sec
C_{de} = depth reduction coefficient
C_{bl} = debris blockage coefficient
C_{or} = debris orientation coefficient, recommended as 0.8
R_{max} = structural response coefficient

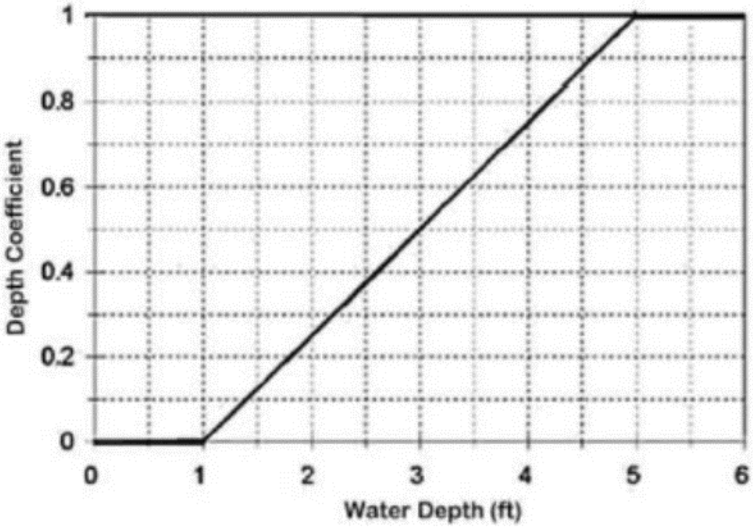


Figure D-3 Depth reduction coefficient for use in Equation D-14

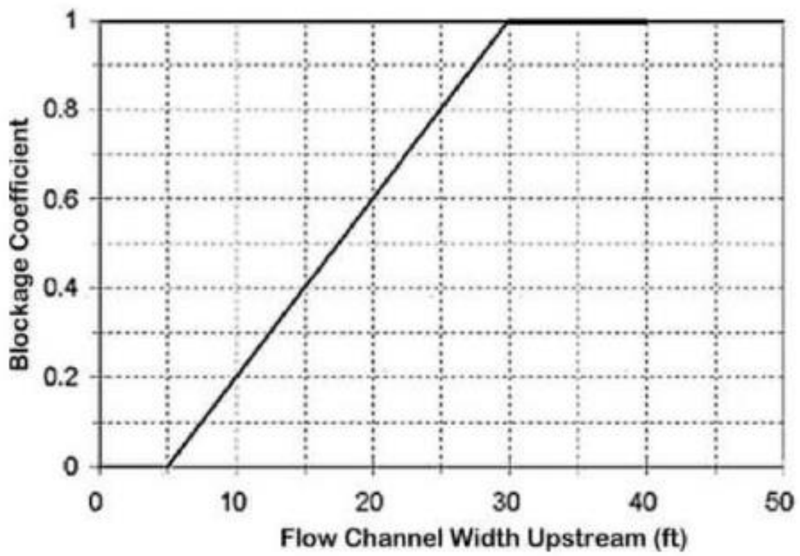


Figure D-4 Blockage coefficient for use in Equation D-14

Table D-6 Dynamic amplification factor R_{max} for use in Eq. D-14

Ratio of Impact Duration to Natural Period of the Impacted Structural Element	**R_{max} (Response Ratio)**
0.0	0.0
0.1	0.4
0.2	0.8
0.3	1.1
0.4	1.4
0.5	1.5
0.6	1.7
0.7	1.8
0.9	1.8
1.0	1.7
1.1	1.7
1.2	1.6
1.3	1.6
≥ 1.4	1.5

Ice crushing loads: Ice loads are treated as debris loads using equation D-13 or D-14. An upper limit on ice loads occurs based on the contact pressure at which the ice fails in crushing. These loads would most often be appropriate for large ice sheets that completely surround a structure, and which are driven by a large driving force such as wind or current. The suggested method is usually conservative as it has been adopted by bridge design codes.

$$F_c = C_a P_c t w \qquad \text{(Eq. D-15)}$$

where:

t = ice floe thickness in m (ft)
w = width of structural element in m (ft)
C_a = interaction coefficient = $(1+5t/w)^{1/2}$
P_c = crushing strength of ice, from about 400 kPa to 1500 kPa depending on properties of ice during breakup.

Debris impact loads shall be applied at the stillwater level to the structure. Impact loads shall assume to only strike one structural element at a time, and the structural analysis shall consider possible collapse prevention techniques to prevent collapse caused by the debris strike. Thus, a debris load shall not be included in a global force analysis on the building.

D.2.7 Wave Loads

Wave loads shall be considered in locations where waves can directly impact a structure, either through wave propagation over flooded ground or through wave runup on otherwise dry slopes above the still water flood level. The behaviour of waves and runup is complicated and may require a detailed coastal engineering study. In some cases, simple and conservative methods may be applied. See the technical guidance in Chapter 3 for information on these prescriptive methods. See the Commentary for more detailed descriptions of wave and runup phenomena and how to design for their conditions.

D.2.7.1 Wave Conditions

For application in flooded coastal regions, where the over-ground flooded depth, d_f, is expected to be small, random wave heights are likely limited by waves breaking. The upper limit on wave heights corresponds to the largest breaking wave height that can occur in any water depth, H_b. In shallow water, the USACE (2002) gives this limiting wave height as

$$H = 0.9 d_f \qquad \text{(Eq. D-16)}$$

where:

H = maximum breaking wave height in m (ft)
d_f = local still water depth in coastal location, m (ft)

The maximum amplitude or elevation that the crest of the wave above the still water level is given as

$$\eta_c = 0.7\, H_b \qquad \text{(Eq. D-17)}$$

where:

η_c = wave crest elevation, in m (ft), above the still water flood level

Estimates of wave runup are more complicated and require additional information on the incident wave conditions. Runup is a function of controlled by the design flood depth d_f, the wave period

T, the wave height H, the bottom slope $\tan\theta$, and ground or slope roughness. Methods of estimating runup are discussed in the Commentary.

D.2.7.2 Wave Loads

Wave loads shall be considered for the following conditions as they apply to the design and site conditions:

- breaking wave loads on piles and columns
- breaking wave loads on truncated piles and columns
- impulsive slamming loads on piles and columns
- breaking wave loads on vertical walls
- breaking wave loads on elevated walls

D.2.7.2.1 Wave Action on Vertical Piles and Columns

For structures with habitable spaces elevated above ground but subject to flood waters, a fundamental form of wave loading consists of wave action on vertical piles or foundation columns.

D.2.7.2.1.1 Breaking Wave Loads on Piles and Columns

For piles and columns subjected to breaking waves, the maximum drag force due to breaking waves shall be computed as

$$F_b = ½\, C_{Db}\, \rho g\, D\, H_b^2 \qquad \text{(Eq. D-18)}$$

where:

F_b = breaking wave force, in N (lb)
C_{Db} = breaking wave drag coefficient (1.75 for round piles, 2.25 for square piles)
D = pile diameter for round pile, or 1.4 times width of pile for square, in m (ft)
H_b = breaking wave height, in m (ft)

The point of application for the force vector shall be applied at the still water level, and the overturning moment about the ground level is $M_b = F_b{*}d_f$.

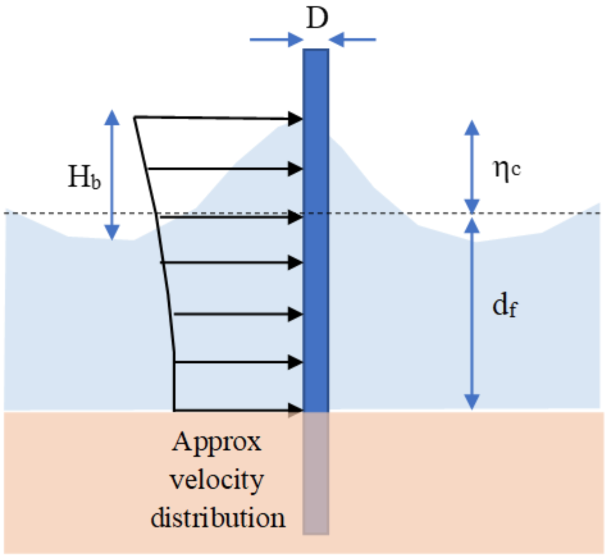


Figure D-5 Breaking waves forces on piles

D.2.7.2.1.2 Truncated Piles and Columns

When the first floor of a building is elevated on piles or columns, and the wave crest elevation is above the bottom of the floor, forces of breaking waves on the truncated piles and columns shall be computed as

$$F_{bt} = ½\, C_{Db}\, \rho g\, D\, H_b^2\ \left(h_f/(d_f + \eta_c)\right) \quad \text{(Eq. D-19)}$$

where:

F_{bt} = breaking wave force on truncated column
h_f = height of column above ground (presumed than df+ηc)

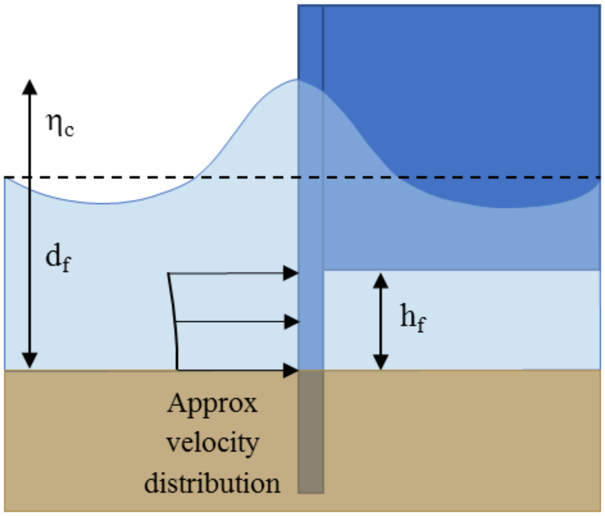


Figure D-6 Wave forces on truncated piles and column foundations

D.2.7.2.1.3 Impulsive Slamming Load on Piles and Columns

For buildings with High and Post-Disaster Importance Categories, an additional dynamic or impulsive wave slamming force shall be added to other breaking wave forces. This additional load shall be applied to a critical pile or column

$$F_S = \tfrac{1}{2}\, C_S \;\rho g\, D\, d_f \;\lambda\, \eta_C \qquad \text{(Eq. D-20)}$$

where:

F_s= slamming force due to direct breaking wave impact, in N (lb)
C_s = slamming coefficient = 3.1
λ = curling factor, as low as 0.1 for spilling breaking waves of low intensity and as high as 0.9 for plunging wave of high intensity.
η_c = wave crest amplitude

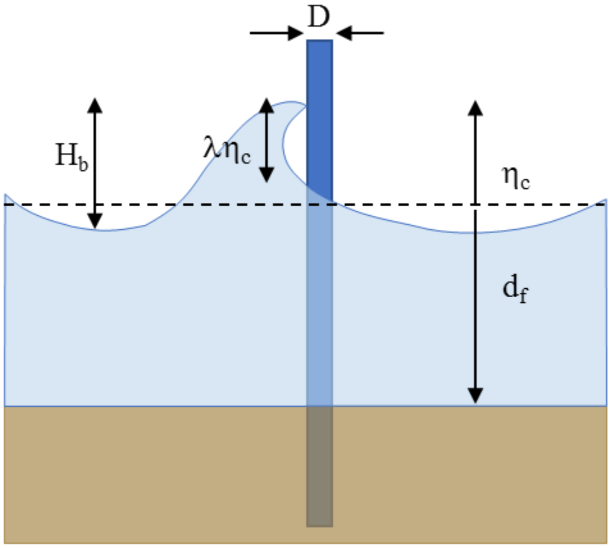


Figure D-7 Dynamic wave slamming on a pile, adapted from ISO Standard 21650 (2016)

D.2.7.2.2 Wave Action on Vertical Walls

For structures with vertical wall panels subjected to waves, a fundamental form of wave loading consists of wave action due to wave reflection and wave impact pressures on the vertical wall panels.

D.2.7.2.2.1 Breaking Wave Loads on Vertical Walls

Breaking wave loads on full depth vertical walls shall be based on the method of Goda (1985), as modified for shallow water breaking wave conditions. The wave crest elevation at the wall is elevated due to wave reflection, and the resulting pressure distribution is shown in Figure D-8. These shall be computed as

$$\eta_* = 0.75\,(1+cos\beta)\;H \qquad \text{(Eq. D-21a)}$$

$$P_1 = 0.5\,(1+cos\beta)\,(1.1 + \alpha\, cos^2\beta)\,\rho\, g\, H \qquad \text{(Eq. D-21b)}$$

where:

η_* = the elevation in m (ft) from the design stillwater level below which the wave pressure is assumed to act, or the elevation at which the wave pressure equals zero
H = design wave height in m (ft), taken as H_b or 1.5 H_s
P_1 = pressure in N/m^2 (lb/ft^2) at the design still water level
α= impulsive wave pressure coefficient, which can be taken as 0.8
β = wave direction relative to wall, 0^o for normal incidence

The wave-induced force shall be computed as

$$f_{wb} \ = \ 0.5\, P_1\, \eta_* \ + \ P_1 \ d_f \qquad \text{(Eq. D-22)}$$

Where:

f_{wb} = breaking wave force per unit width in N/m (lb/f^2)

The total force over the wall length would be given by

$$F_{wb} \ = \ w\, f_{wb} \qquad \text{(Eq. D-23)}$$

For overturning moments, the point of application of the wave load would correspond to the centre of pressure from Equation 3-32. For a full depth wall (not overtopped), a simple approximation is to apply the load at the still water level.

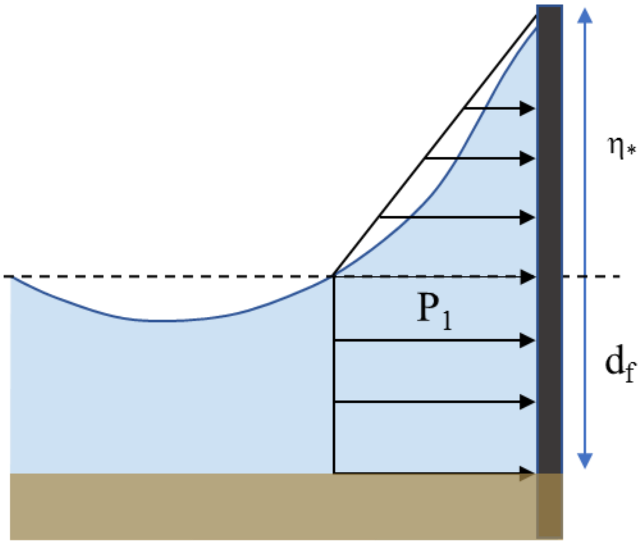


Figure D-8 Breaking wave pressures on vertical walls, adapted from Goda (1985)

D.2.7.2.2.2 Wave Loads on Elevated Walls

Breaking wave loads on wall panels elevated on foundation piles or columns are computed by truncating the pressure distribution in Figure D-9. The force per unit width is obtained as

$$f_{wb} = 0.5\, P_1\, \eta_* + P_1\, (d_f - h_f) \qquad \text{(Eq. D-24)}$$

where:

h_f = height of floor or wall element above ground level

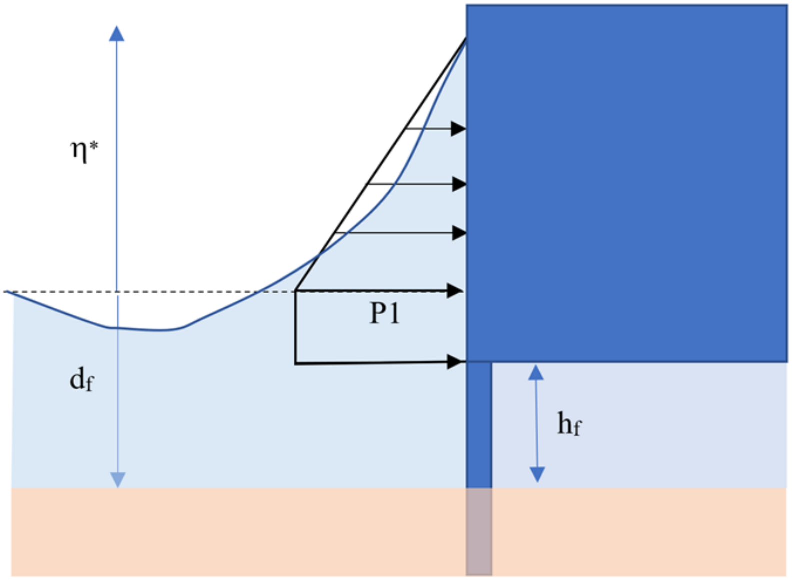


Figure D-9 Loads on elevated wall panel, from method of Goda

D.2.7.2.2.3 Wave Slam due to Wave Crest Impact

For walls that are elevated above the still water levels and subjected to impact from wave crests, the lateral wave slam shall be calculated as:

$$F_{ws} = w\, f_{ws} = w\, (0.5\, C_S\, \rho\, g\, d_f\, h) \qquad \text{(Eq. D-25)}$$

where:

F_{ws} = Lateral wave slam force, in N (lb)

C_S = wave slam coefficient, with recommended value of 2.0

h = vertical height of wall impacted from bottom of floor joist or beam to wave crest

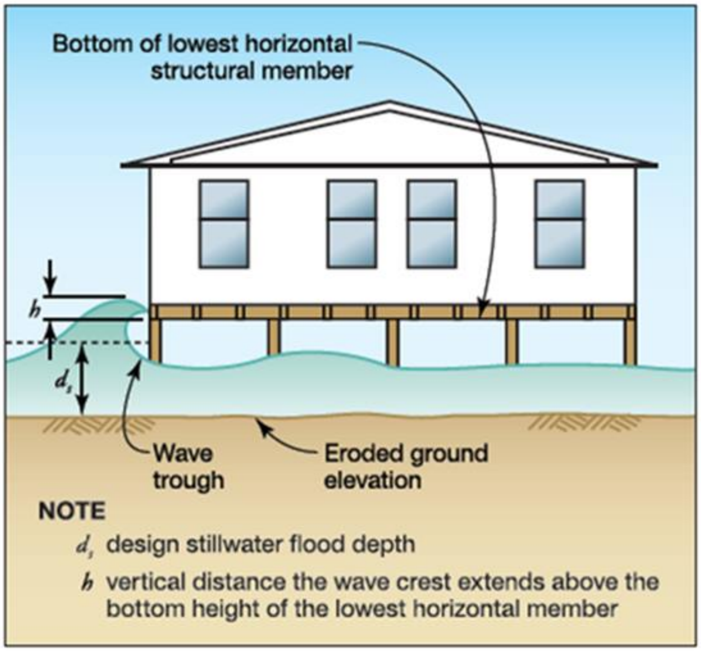


Figure D-10 Wave slam due to wave crest hitting side wall of elevated building, from FEMA (2011)

D.2.7.2.3 Wave Uplift Forces on Horizontal Floors

Methods of estimating vertical wave uplift forces on elevated floors are not well established for buildings. Methods developed for vertical loads on elevated bridge decks are reviewed in the Commentary.

D.2.7.2.4 Wave Forces on Horizontal Beams

Methods of estimating lateral wave forces on horizontal beams are not well established for buildings. Methods developed for horizontal beams on piers are reviewed in the Commentary.

D.2.7.2.5 Wave Runup Forces on Walls

Methods of estimating wave forces due to wave runup impact on wall panels require a detailed estimate of wave runup, which requires a coastal engineering analysis of incident wave height and wave period. Methods for estimating wave runup forces are reviewed in the Commentary.

D.2.8 Flood Load Combinations

Flood loads are to be combined as described below to create a total flood load to be applied to a structure.

For riverine conditions, two fundamental situations are common:

- Flood zones with no flow conveyance (flood storage only, standing water, no velocity) – to include the following loads:
  - hydrostatic only

- Flood zones with flow conveyance (flow velocity) – to include the following loads:
  - hydrostatic + hydrodynamic + debris (on critical structural element)

For coastal or lake conditions, a wider range of conditions are possible often depending on the conditions:

- Sheltered Flood zones with no exposed fetch for wave growth and no substantial net flows, probably in back bays and estuaries with less than 1 km of open water:
  - hydrostatic only
- Protected Flood zones with no exposed fetch for waves but with net flows, probably in narrow bays and estuaries with less than 1 km of open water but with tidal inflow or outflow, or near a river mouth
  - hydrostatic + hydrodynamic + debris (on critical structural elements)
- Exposed Flood zones with wave action but with no substantial net flows, probably on an open coast or along a bay or estuary with more than 1 km of open water:
  - hydrostatic + waves
- Exposed Flood zones with both wave action and net currents, probably on an open coast or along a bay or estuary but near a river mouth or some constriction causing currents due to tidal or surge
  - hydrostatic + hydrodynamic + waves + debris (on critical structural elements)

In most riverine cases, flood loads can be estimated from existing flood maps where flood depth and velocity are either directly indicated on flood maps or can be developed from flood maps using prescriptive methods outlined in section D.1.

In many coastal and lake conditions, when waves are not present, flood loads can be estimated from existing flood maps where flood depth is either directly indicated on flood maps or can be developed from flood maps using prescriptive methods outlined in section D.1.

In coastal and lake conditions where wave action is present, methods outlined in D.2.7.1 may be used with mapped flood elevations to develop a conservative estimate of wave forces. Methods outlined here assume depth-limited breaking waves which is a reasonable assumption on many exposed coasts when the offshore waves are large and waves in shallow flooded areas are most likely breaking. This assumption may overestimate waves conditions on protected coasts where offshore waves may be too small to break in all flooded depths. In these and other coastal cases, wave conditions may be best determined from a coastal engineering study that can establish wave heights, crest elevation, and wave runup.

D.3 Flood-Resistant Materials

Flood resistant materials shall be used for all construction below the FCL.

D.4 Flood Design for New Buildings

New buildings shall be built such that the FCL is at or above the recommended elevation of the design flood listed in Table D-1. Table D-7 provides alternative flood mitigation measures that may be considered when a new building cannot be built in accordance with the elevations of Table

D-1. If there are no alternative mitigation measures that should be considered, the measure is designated as N/A (not available). These measures may be used as part of a Performance Based-Design solution.

Table D-7 Alternative mitigation measures for achieving acceptable performance

Importance Category	**Minimum FCL Required**	**Alternative Mitigation Measures**
Low	1:100 AEP	N/A
Normal	1:500 AEP	1- and 2-family residential buildings – N/A Multi-family, multi-story buildings – floodproof lowest floor at FCL as long as lowest space is used for service or utility space
High	1:750 AEP	High occupancy space to be used for shelters should be above the FCL. Utility or other non-occupiable space may use alternative measures such as PBD
Post-disaster	1:1000 AEP	Critical use space shall have elements and spaces critical to safe operation and of community importance either elevated above the FCL or floodproofed. PBD may be used as an alternative measure to the prescriptive method proposed in this Guide

D.5 Flood Design for Existing Buildings

Renovations or rebuilding existing buildings located in floodplains shall be renovated or rebuilt such that the area below the design FCL is protected from future flood damage. Table D.8 provides minimum design flood elevations for each use and importance category in the NBC.

Table D-8 Minimum FCL required for renovated existing buildings

Importance Category	Minimum FCL Required
Low	1:100 AEP
Normal	1:250 AEP
High	1:500 AEP
Post-disaster	1:750 AEP

Achieving full compliance with elevation requirements established for new buildings in renovating or rebuilding existing buildings is recognized as difficult and costly. For that reason, the minimum FCL required for existing buildings is reduced below that required for new buildings. Improved building performance during flood conditions may be improved using an alternative design method such as Performance-based Design for floods.

D.6 Floodproofing Existing Buildings

Reducing flood losses on existing buildings can be accomplished by allowing water to enter the building and then clean it up after the flood (passive technique) or preventing water from entering the building (active technique) by blocking the pathways that allow water entry. The passive technique is called wet floodproofing. The active technique is called dry floodproofing.

D.6.1 Wet Floodproofing

Wet floodproofing shall be permitted for any existing building where flood water will not damage the interior or contents of the building. There shall be flood openings in the exterior walls of the building to allow flood water to flow freely into and out of the building and to reduce hydrostatic pressure against the building walls.

There must be at least two openings in the building walls. The bottom of the openings must be no more than 30 cm above grade. The openings must have a net open area of at least 70 cm^2 to 1 m^2 of building footprint. Alternatively, the quantity of net open area may be calculated using hydraulic engineering textbooks with an appropriate coefficient of discharge that is not greater than 0.40.

Wet floodproofing shall not be used in areas that house utilities without the utilities being placed at or above the FCL or protected by a dry floodproofing system.

D.6.2 Dry Floodproofing

Dry floodproofing shall be permitted for Normal use commercial buildings, and High and Post-disaster use buildings. Dry floodproofing shall not be used for Normal use one- and two-family residential buildings. Dry floodproofing systems may be used to protect utilities and other critically important operational systems within buildings.

The required FCL for dry floodproofing systems shall be in accordance with Table D-1. Dry floodproofing designs and installations must be certified by a design professional licensed in Canada. The certification must include a flood vulnerability assessment performed on the building by a design professional. Products used in the dry floodproofing installation shall be tested or certified to be used in flood water conditions including design depths and duration, and to minimize leakage into the building interior. There shall be a secondary water collection system installed inside the dry floodproofed perimeter to collect any seepage that occurs through the flood components.

A dry floodproofing installation shall have an operations and maintenance plan developed as well as an annual testing protocol used to test the deployment of the dry floodproofing components. The operations plan shall address, at a minimum, the storage of deployable flood protection components, the expected deployment time required for the components, the amount of flood warning time required to be able to adequately deploy the components, and the organization and leadership of the flood deployment team.

GLOSSARY/DEFINITIONS

Annual Exceedance Probability (AEP): the annual likelihood of a flood equal to or greater than a given magnitude occurring, expressed as a probability less than 1.0. The 0.01 AEP flood is equivalent to both the 1% annual probability flood, and the 100-year return period flood (or a flood with a 100-yr Mean Recurrence interval (MRI)). (See also Mean Recurrence Interval and Return Period)

Approved: Acceptable to the Authority Having Jurisdiction.

Area of future-conditions hazard: The land area that would be inundated by the-1-percent-annual chance (100-year) flood based on future conditions hydrology.

Authority Having Jurisdiction. Government entity responsible for approving flood risk guidelines and regulations, also termed **Approving Authority** or denoted **AHJ**.

Base Flood Elevation (BFE): Elevation of flooding, including wave crest or wave runup elevations, having a 1% chance of being equaled or exceeded in any given year, as used in US Flood Insurance Studies and maps.

Breakaway wall: Any type of wall subject to flooding that is not required to provide primary structural support to a building or other structure and that is designed and constructed such that it will collapse under specific lateral loads, to (1) allow the free passage of floodwaters, and (2) not damage the primary structure or supporting foundation system.

Breaking Wave Height: wave height when waves are breaking and at their maximum height, in context of this report refers to maximum wave height limited by water depth.

Catchment: Also known as drainage area, drainage basin or watershed. It is the area of land draining to a particular location and includes the upstream drainage area of the main waterway as well as any tributary streams.

Chart Datum: A common reference elevation used to define water elevations, water depths, and ground elevations. In Canada, this is typically LLWLT in coastal areas.

Climate Change: A change in the state of the climate that can be identified (e.g., by using statistical tests) by changes in the mean and/or the variability of its properties and that persists for an extended period, typically decades or longer. Climate change may be due to natural internal processes or external forcings such as modulations of the solar cycles, volcanic eruptions and persistent anthropogenic changes in the composition of the atmosphere or in land use.

Cross-section: A survey string of channel and floodplain elevations that is taken perpendicular to the main flow direction in a river.

Coastal Flooding: Flooding associated with a defined shoreline along an ocean. This can be due to a combination of high tides, storm surges, waves, rising sea levels and riverine flooding.

Debris accumulation load: Load on a structure, or element of a structure, caused by an accumulation of flood-borne debris.

Debris impact loads: Loads caused by a piece of flood-borne debris striking a structure, or a portion of a structure; often it is sudden in nature and large in magnitude.

Design flood: The flood corresponding to the greater or higher of the following: (1) the design mean recurrence interval flood, (2) the area designated on a flood hazard map, or (3) the flood elevation adopted by an Authority Having Jurisdiction

Design flood depth: The water depth in m (ft) at an individual building or other structure produced by the design flood, based on difference of the still water flood elevation minus ground elevation including any erosion or scour.

Design Flood Level (DFL): The elevation of the design flood in m (ft)), excluding wave crest or runup effects, relative to the datum specified on a community's flood hazard map or adopted by an Authority Having Jurisdiction.

Design Storm: A rainfall time-series input, based on an historic event or synthesized using intensity-duration-frequency curves, that is used to calculate flow and to delineate a Flood Hazard Area. In Canada, Design Storms are often used instead of Design Floods in jurisdictions where an historic event was of higher magnitude than the Design Flood.

Digital Elevation Model (DEM): A file with terrain elevations recorded for the intersection of a fine-grained grid and organized by quadrangle as the digital equivalent of the elevation data on a topographic base map.

Elevated Structure: A structure with its base or lowest inhabited floor positioned above ground, raised on foundation walls, shear walls, posts, piers, pilings or columns, such that water may flow below the base.

Encounter probability: probability of a flood event with an annual probability given by the AEP occurring at least once within a time period of multiple years.

Eroded ground elevation: Elevation of ground following erosion.

Erosion: Wearing away of the land surface by detachment and movement of soil and rock fragments during a flood or storm or over a period of years, through the action of wind, water, or other geologic processes.

Flood Awareness Map: Communication maps that serve to inform members of the public regarding the history of flooding in their communities, as well as the potential for future flooding and the risks that such flooding would pose to residential properties, businesses, cultural assets, infrastructure and human life. These poster-style maps include a range of additional content types, such as photographs, descriptive text and graphics.

Flood Construction Level: FCL a design flood elevation at or above the DFL including wave crest or runup effects, and including any freeboard as or adopted by an Authority Having Jurisdiction.

Flood Fringe Areas: The area between the Floodway and the delineated extent of flooding for a Design Flood. In Canada, the Flood Fringe Area is often defined as having a flood depth below 1 metre and a flood velocity less than 1 metre per second.

Flood Hazard Area: The delineated extent of flooding for a Design Flood (e.g. 0.01 AEP flood), which includes the 'Floodway' and the 'Flood Fringe Area'.

Flood Hazard Management: The operation of a program of corrective and preventative measures for reducing flood damage, including, but not limited to, development plans, emergency preparedness plans, flood-control works, and land use regulations.

Flood Hazard Map: A flood delineation at a given location, based on the flood's anticipated magnitude (e.g. its depth, horizontal extent, and flow velocity) and its annual exceedance probability. It shows the extent of the regulatory flood hazard, often including two zones: floodway and flood fringe areas. This type of map is used for regulatory planning purposes.

Flood Hazard Study: A compilation and presentation of flood risk data for specific watercourses, lakes, and coastal flood hazard areas. Typically contains flood elevation data in flood profiles and data tables.

Flood Inundation Map: Maps that show the extent of actual floods or potential floodwater coverage during flood events of different magnitudes (AEPs). They are intended to aid in the management of emergency preparedness plans for communities situated within floodplains and flood prone areas.

Flood Mitigation: A sustained action taken to reduce or eliminate long-term risk to people and property from flood hazards and their effects. Mitigation distinguishes actions that have a long-term impact from those that are more closely associated with preparedness for, immediate response to, and short-term recovery from specific events.

Floodplain: A low-lying, relatively flat area of land adjacent to a river or stream that is subject to flooding. Floodplains are generally made up of alluvium (sand, silt, and clay) deposited by past flood events.

Flood Protection: Any combination of structural and non-structural additions, changes, or adjustments to structures, which reduce or eliminate risk of flood damage to real estate or improved real property, water and sanitation facilities, or structures and their contents.

Flood Risk Map: Maps that contain the flood hazard or inundation delineations along with additional socio-economic values, such as potential loss or property vulnerability levels. These maps serve to identify the social, economic and environmental consequences to communities during a potential flood event.

Flood Risk: Flood risk is a combination of the likelihood of a flood event occurring (**Flood Hazard**) and the social or economic consequences of that event when it occurs (the exposure to the flood hazard).

Floodway: The channel and adjacent area where flood depths and velocities are greatest and most destructive. In Canada, the Floodway is often defined as having a flood depth above 1 metre and flood velocity greater than 1 metre per second and the floodway elevation represents the 1:20 year flood event.

Flow: The rate of flow of water measured in volume per unit time – for example, cubic metres per second (m^3/s). Flow is different from the speed or velocity of flow, which is a measure of how fast the water is moving – for example, metres per second (m/s).

Freeboard: A vertical height added to calculated flood elevations to provide additional protection from flooding, or to account for uncertainty from sources including climate change and data limitations, providing an air gap between the highest expected water level and the bottom of a structure.

Future Conditions: Quantification of anticipated future changes, including, at a minimum, the effects of relative sea level change, subsidence, erosion, and other factors expected to occur during the service life.

Higher High Water Large Tide Level (HHWLT): The elevation of the highest tide level which controls coastal flood elevations, based on averaging the highest annual times for 19 years. This level will usually be a historical tide level taken from a tide gauge.

Hydraulic Analysis: An engineering analysis of flow scenarios carried out to provide estimates of the water surface elevations and flow characteristics for selected recurrence intervals.

Hydrodynamic load: Load imposed on an object by water flowing against and around it.

Hydrologic Analysis: An engineering analysis of a flooding source carried out to establish peak flood discharges and their frequencies of occurrence.

Hydrostatic load: Load imposed on an object due to the effects of water height only not including any velocity or wave effects.

Importance Categories: Those building use categories defined by the National Building Code of Canada (Table 4.1.2.1 in the NBC of Canada) and shown in Figure 2-27 of the report.

Infiltration: The penetration of water through the ground surface into the sub-surface soil.

Lake Flooding: Flooding associated with a defined shoreline along a lake. This can be due to a combination of high-water levels, waves, storm surges and riverine flooding.

Lower Low Water Large Tide (LLWLT): The average of the lowest low waters, one from each of 19 years of predictions, often adopted.

Maximum Wave Height: The largest individual wave in a random sea. In shallow water typically limited by wave breaking.

Mean Recurrence Interval (MRI): The estimated average time or interval between events that equal or exceed so defined intensity, for events such as earthquakes, floods, wave heights, or a river discharge flows. The MRI is also known as the return period, and is the inverse of AEP.

Performance-Based Design: A design approach allowed by the building code that enables the development of structures that will have predictable performance when subjected to defined loading. This approach is an alternative to building prescriptively in strict adherence to flood elevation requirements of flood maps or flood studies.

Pluvial Flooding: The temporary inundation by water of normally dry land, usually caused by extreme rainfall events and not necessarily near to water bodies. Pluvial flooding is common in urban areas where water temporarily accumulates due to more rainfall entering an area than can be removed by infiltration into the ground and discharge through infrastructure (e.g. storm sewers).

Prescriptive method: a design method that is suitable for building codes, usually given in simple equation form and intended to be easily applied by a typical design engineer.

Regulatory Flood: A specific flooding event designated as the Design Flood in a certain jurisdiction.

Relative sea-level change: the change in mean sea level that is observed or experienced relative to a location on land, as the combination of absolute/global sea-level change and vertical land motion. Land uplift decreases relative sea-level rise and land subsidence increases relative sea-level rise.

Riverine Flooding: The temporary inundation by water of normally dry land adjacent to a river and caused by rainfall, snowmelt, stream blockages including ice jams, failure of engineering works including dams, or other factors.

Runoff: The amount of precipitation or water deriving from snowmelt and rainfall that drains into the surface drainage network to become streamflow; also known as rainfall excess.

Scour: The local removal of material of the land surface during flood conditions due to an abrupt change in flow direction or velocity around an object or structural element.

Sea-level Rise: the increase in mean sea level on a global, regional, or local scale due to climate effects including ocean thermal expansion, melting of land-based ice, modification of ocean currents, and other effects related to an average increase in temperature.

Significant Wave Height: A statistical wave height defined as: (1) the average height of the one-third highest waves of a given wave group or (2) four times the standard deviation of the fluctuation water surface. Also believed to be the wave height that would be estimated by eye by a trained observer.

Site-Specific Flood Study: An alternative study procedure using analytical, numerical, or experimental methods, approved by AHJ, to determine flood depth, associated conditions, or flood loads, for a site-specific location.

Stage: Water elevation measured relative to a local reference datum.

Stillwater depth: Vertical distance in m (ft) between the ground and the still-water elevation, excluding future conditions.

Stillwater Level (SWL): Elevation that the surface of the water would assume in the absence of waves referenced to a datum, excluding future conditions.

Storm Surge: The increases in coastal water levels above predicted astronomical tide level (i.e. tidal anomaly) resulting from a range of location-dependent factors, including low atmospheric pressure, wind and wave set-up and astronomical tidal waves, together with any other factors that increase tidal water levels.

Velocity of Floodwater: The speed at which flood waters are moving, typically measured in metres per second (ft/s).

Watershed: See **Catchment**.

Water level: The mean elevation of the water when averaged over a period of time long enough (about one minute) to eliminate oscillations caused by surface gravity waves which have periods in the order of a few seconds.

Wave setup: rise in mean water at the shoreline due to effect of breaking waves

Wave height: Vertical distance in m (ft) between the crest and the trough of a wave.

Design wave height: wave height adopted for determining design wave loads on a structure.

Wave crest: the highest part of an individual wave above still-water level

Wave load: Load imparted on a structure caused by wave interacting with the structure or a portion thereof.

Wave runup: Vertical height reached by water above the still water level due to rush of wave water running up a slope or structure.

Wave crest elevation: Elevation referenced to a datum reached by wave crest, or wave amplitude above still water level

Wave runup elevation: Elevation referenced to a datum reached by wave runup.

SYMBOLS

Symbol	Definition
A	cross sectional area of riverine transect below flood level, in m^2 (ft^2)
A_a	projected area of the debris accumulation into the flow, approximated by depth of accumulation times width of accumulation perpendicular to flow, in m^2 (ft^2)
A_f	floor area exposed to wave uplift, in m^2 (ft^2)
A_h	contact area in horizontal plane over which hydrostatic pressures act, in m^2 (ft^2)
A_p	projected area of structure or structural element exposed to the moving water, in m^2 (ft^2)
c	clearance of beam from still water level to bottom of beam, in m (ft)
C_a	interaction coefficient for ice crushing against structure
C_{bl}	blockage coefficient for debris impact loads
C_{de}	depth reduction coefficient for debris impact loads
C_i	impulsive uplift coefficient, suggested to be 3.0
C_{or}	debris orientation coefficient for debris impact loads, recommended as 0.8
C_p	dynamic pressure coefficient ($1.6 < C_p < 3.5$)
C_s	wave slamming coefficient
C_u	uplift coefficient, suggested to be 1.0
C_D	drag coefficient, function of structure shape and dimensions
C_{Db}	breaking wave drag coefficient on a vertical pile or column (1.75 for round piles, 2.25 for square piles)
C_L	lift coefficient, function of structure shape and dimensions
d	generic variable for flood depth in m (ft)
d_b	depth from still water level down to bottom of floor of elevated structure, in m (ft)

Symbol	Definition
d_f	design flood depth, based on design flood level minus ground elevation, including any added depth due to erosion or scour or future conditions
d_g	submerged depth below grade, in m (ft)
d_w	depth of wave uprush at wall, in m
$d_{2\%}$	depth of water at structure due to runup, expressed at 2% exceedance level
D	pile or column diameter in M (m) (ft) for circular sections, or 1.4 times the width of the pile or column for a square pile, in m (ft)
DFL	Design flood level, elevation of the design flood in m (ft), excluding wave crest or wave runup, relative to the datum specified on a community's flood hazard map
f	generic force per unit width or unit length, in N/m (lb/ft)
f_{surge}	force per unit width due to wave uprush, in N/m
f_{wb}	breaking wave force on wall per unit width of structure, in N/m (or lb/ft)
f_H	hydrostatic force per unit width along the wall, in N/m (or lb/ft)
F	generic force on structural element, in N (lb)
F_b	breaking wave force on pile or column, in N (lbs)
F_{bt}	breaking wave force on truncated column or pile, in N (lb)
F_B	vertical buoyant force due to hydrostatic pressures, in N (or lb)
Fc	Force due to crushing of ice on structure, in N (lb)
F_{di}	debris impact force in N (lb)
F_D	hydrodynamic drag force, in N (or lbs)
F_{Da}	drag force due to debris accumulation, in N (lb)
F_h	horizontal force of breaking wave on beam, in N (lb)
F_{h*}	characteristic force on horizontal beam, to be adjusted, in N (lb)

Symbol	Definition
F_H	horizontal or lateral force due to hydrostatic pressures, in N (lb)
F_i	rapidly varying impulsive uplift force on floor, in N (lb)
F_L	hydrodynamic lift force, in N (or lbs)
F_q	quasi-static or slowly varying uplift force on floor, in N (lb)
F_S	slamming force due to direct breaking wave impact, in N (lbs)
Fv	vertical hydrostatic force caused by hydrostatic pressures, in N (lb)
F_{wb}	Breaking wave force on wall of width w, in N (lb)
F_{ws}	Lateral wave slam force, in N (lbs)
FCL	Flood construction level, elevation used for regulating elevation of building components, including wave effects and freeboard in m (ft), relative to the datum specified on a community's flood hazard map
g	acceleration due to gravity, 9.81 m/s^{2} $^{(}$32.2 ft/s^{2})
G	ground elevation, in m (ft)
h	height of a horizontal beam in m (ft)
h_f	height of column above ground in m (ft)
h_w	height of water at wall for wave runup in m (ft)
H	Wave height, from trough to crest in m (ft)
H_s	Significant wave heit in m (ft), also H_{mo} or $H_{1/3}$
H_{sb}	significant wave height with depth limited wave breaking, in m (ft)
k	effective stiffness of impacting debris or impacted structural element(s) deformed by the impact, whichever is less
k	Coefficient used in Manning's equation associated with units (k=1 in SI units, k=1.486 in US Customary units)
K	conveyance of the cross section, with unit of discharge in m^3/s (ft^3/s)

Symbol	Definition
K_i	breaking wave coefficient for horizontal beam = 3.25
L_o	wavelength of design wave in deep water, $gT_p^2/2\pi$, in m (ft)
M	debris mass, also given as W/g
M_b	Moment of breaking wave force on pile or column acting about ground level, in N-m (lb-ft)
n	Manning's roughness coefficient, related to ground cover, flow turbulence, flow meandering, and flow obstructions (typical values are given in Appendix B)
P	Wetted perimeter of the cross-sectional area, in m (ft)
P_{bot}	hydrostatic pressure on bottom of structure or element, based on z_{bot}
P_c	crushing strength of ice, from about 400 kPa to 1500 kPa depending on properties of ice during breakup
P_h	hydrostatic pressure, in N/m^2 (or lb/ft^2)
P_s	stagnation pressure due to reduction in flow velocity at leading edge of structure, in N/m^2 (or lb/ft^2)
P_{top}	hydrostatic pressure on top of structure or element, based on depth to top Z_{top}
P_1	pressure in N/m^2 (lb/ft^2) at the design stillwater level for wave loads on walls or pressure at top of beam for horizontal beam
P_2	pressure in N/m^2 (lb/ft^2) at the ground level for vertical wall, or pressure at bottom of horizontal beam
R	vertical wave runup distance above the stillwater elevation in m (ft)
R_h	hydraulic radius (m or ft), given by $R_h = A/P$ or approximately by $R_h \sim A/W$
R_{max}	maximum response ratio for impulsive debris load
$R_{2\%}$	vertical runup above SWL at 2% exceedance level
S	slope of energy grade line (m/m or ft/ft), or energy gradient, approximated by the slope of the water surface at the cross section
S_n	flood elevation of return period of interest, n, in m (ft) above a defined reference datum
S_{R1}	flood elevation of lowest reference return period for MRI scaling

Symbol	Definition
S_{R2}	flood elevation of highest reference return period for MRI scaling
t	ice floe thickness in m (ft)
T_p	peak wave period corresponding to the significant wave height in seconds (s)
V	flow velocity, averaged over depth, in m/s (ft/s)
~~V~~	volume of water displaced by the structural element, in m^3 (or ft^3)
w	width of the wall or structural element perpendicular to the flow, in m (ft)
W	debris weight of debris, in N (lbs)
W	Width of the cross section (m or ft)
X_R	horizontal runup excursion landward from SWL shoreline
z	depth of submergence, down from SWL to point of interest (including in submerged soils)
Z	elevation of ground at wall location between shoreline and runup limit
Z_{bot}	submerged depth to bottom of structure or element, from free water surface
Z_{top}	submerged depth to top of structure or element, from free water surface
α	scaling factor used for MRI scaling
α	shape factor or drag coefficient = 1.25 (or from Table 3-1 below as a function of building aspect ratio)
$\alpha*$	impulsive wave pressure coefficient, which can be taken as 0.8
β	horizontal angle between the direction of wave approach and the vertical wall, 0^o for normal incidence
γ	water unit weight, 9,810 N/m^3 fresh or 10,055 N/m^3 salt (62.4 lb/ft^3 fresh, 64.0 lb/ft^3 salt)
γ_r	reduction factor for ground roughness, structure shape, and angle of wave attack
Δt	impact duration or time to reduce object velocity to zero, in sec, taken as 0.03 sec

Symbol	**Definition**
Δd_f	increase in water level on upstream side compared to downstream side, in m (ft)
ξ	Irribarren Number or Surf Similarity Parameter equal to $tan\theta / (Hs/Lo)^{1/2}$
η_c	wave crest elevation or amplitude, in m (ft), above the still water flood level
$\eta*$	elevation of wave crest on wall in m (ft) above design still water level, at which the wave pressure equals zero
θ	Slope angle of beach or structure, in radians or degrees
κ	breaker height to depth ratio, or breaker index
λ	curling factor, as low as 0.1 for spilling breaking waves of low intensity and as high as 0.9 for plunging wave of high intensity
ρ	water density, 1,000 kg/m^3 fresh, 1,025 kg/m^3 salt (1.94 slug/ft^3 fresh, 1.99 slug/ft^3 salt)