Building Designer's Guide to Calculating Flood Loads Using ASCE 7-22 Supplement 2

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Acronyms and Abbreviations

3DEP	USGS 3D Elevation Program
ABFE	Advisory Base Flood Elevation
ADCIRC	Advanced CIRCulation
AEP	Annual Exceedance Probability
AHJ	Authority Having Jurisdiction
ASCE	American Society of Civil Engineers
BFE	Base Flood Elevation
CAD	Computer-Aided Design
CHAMP	Coastal Hazard Analysis Modeling Program
DEM	Digital Elevation Model
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
ft	feet
ft^2	square feet
ft/s	feet per second
ft/s^2	feet per second squared
ft/yr	feet per year
GIS	Geographic Information System
H&H	Hydrologic And Hydraulic
in	inches
in^4	area moment of inertia
IBC	International Building Code

IRC	International Residential Code
kg/m^3	kilogram per cubic meter
kN	kilonewtons
kN/m	kilonewtons per meter
kN/m^2	kilonewtons per square meter
kN/m^3	kilonewtons per cubic meter
lb(s)	pounds
lb/ft	pounds per foot
lb/ft^2	pounds per square foot
lb/ft^3	pounds per cubic foot
lb/in	pounds per inch
$lb\ s^2/ft$	slugs (the mass of a one-pound object)
$lb\ s^2/ft^4$	mass density
LiMWA	Limit of Moderate Wave Action
m	meters
m/s	meters per second
m/s^2	meters per second squared
m/yr	meters per year
MRI	mean recurrence interval
NAVD 88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NRC	National Research Council

Pa	Pascals
PFD	primary frontal dune
psi	pounds per square inch
RSL	relative sea level
RSLC	relative sea level change
SWAN	Simulating Waves Nearshore
SWEL	stillwater flood elevation / stillwater elevation
TB	Technical Bulletin
TMAC	Technical Mapping Advisory Council
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WHAFIS	Wave Height Analysis for Flood Insurance Studies

Select Design Parameters used in the Determination of Flood Loads

- B = overall width of building perpendicular to the flow direction, in ft (m)
- C_d = drag coefficient for submerged objects subjected to currents
- C_{MRI} = Flood scale factor for mean recurrence interval (MRI); flood scale factor associated with the MRI from Table 7 for different locations when $SWEL_{100}$ is the starting stillwater elevation (SWEL)
- C_{MRI_500} = flood scale factor associated with the MRI from Table 8 for different locations when $SWEL_{500}$ is the starting stillwater elevation (SWEL)
- D = pile or column diameter, in ft (m) for circular sections, or the largest projected width of the pile or column in ft (length of plan diagonal) (m) for a square or rectangular pile or column
- d_f = design stillwater flood depth, in ft (m)
- g = Acceleration due to gravity, taken as 32.2 ft/s^2 (9.81 m/s^2)
- G_e = elevation of grade at the building or other structure inclusive of effects of erosion in ft (m)
- H_b = breaking wave height, in ft (m)
- H_C = controlling wave height, in ft (m)
- H_{C100} = controlling wave height for the 100-year MRI, in ft (m)
- H_{CMRI} = controlling wave height corresponding to the Risk Category and MRI, in ft (m)
- H_{design} = design wave height, in ft (m)
- H_s = significant wave height, in ft (m)
- p_h = hydrostatic pressure at a given depth z, in lb/ft^2 (kN/m^2)
- PL = project lifecycle, not less than 50 years
- SLR_A = the average annual rate of relative sea level change in ft/yr (m/yr); may be either the historic rate or a selected rate above the historic rate
- S_{RA} = average annual shoreline retreat in ft/yr (m/yr)
- S_{RTOT} = total expected shoreline retreat in ft (m)
- $SWEL_{design}$ = design stillwater flood elevation, in ft (m)
- $SWEL_{MRI}$ = stillwater elevation corresponding to the specified Risk Category and MRI, in ft (m)
- $SWEL_{100}$ = stillwater elevation for the 100-year MRI, in ft (m)
- $SWEL_{500}$ = stillwater elevation for the 500-year MRI, in ft (m)
- V = design flood velocity, in ft/s (m/s)
- z = depth below design stillwater flood elevation, in ft (m)
- Z_{datum} = elevation of mean water level based on local datum, in ft (m); for riverine sites, Z_{datum} shall be taken as the annual high-water level; for coastal sites, Z_{datum} shall be permitted to be taken as 0
- ρ = mass density of water, in lb s^2/ft^4, taken as 1.94 lb s^2/ft^4 (1000 kg/m^3) for freshwater and 1.99 lb s^2/ft^4 (1027 kg/m^3) for saltwater
- Δ_{SLR} = the total relative sea level change for coastal sites over the project lifecycle (PL), in ft (m); shall not be taken as less than 0
- γ_w = specific weight of water in lb/ft^3 (kN/m^3), taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater

1. Introduction

1.1. Introduction

In May 2023, ASCE/SEI 7-22 (ASCE 7-22), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Supplement 2, was published as an extension to ASCE 7-22. ASCE 7-22 Supplement 2 (ASCE 7-22-S2) provides enhanced flood load requirements by making an adjustment to the design flood based on the building Risk Category—to make the approach more consistent with the method to calculate other hazards such as wind and seismic loads—and by making changes to multiple flood load calculations. This design guide provides instructional guidance for design professionals for designing buildings or other structures for flood loads in accordance with the requirements of ASCE 7-22-S2. The design guide is intended to support the use of ASCE 7-22 and ASCE 7-22-S2. Design professionals will need a copy of ASCE 7-22 and ASCE 7-22-S2 in addition to this design guide to complete the ASCE 7-22 design calculations.

Following the relevant sections of this design guide will assist a design professional to complete a building design that meets the intent of the flood load provisions of the ASCE 7-22-S2. Some sections of this design guide will not be applicable in all cases. ASCE 7-22-S2 was developed to provide a modern approach to calculating flood loads and was developed independent of the National Flood Insurance Program (NFIP) requirements. ASCE 7-22-S2 exceeds many of the NFIP minimum requirements in accordance with Title 44 of the Code of Federal Regulations (CFR) § 60.3(a)(3). However, in the guide, there are two noted instances where ASCE 7-22-S2 does not comply with the flood load requirements of the NFIP. This design guide applies to new or Substantially Improved (including damaged) structures that are required to comply with ASCE 7-22-S2. The design guide also provides guidance for other structures where higher flood load performance is desired. Compliance with ASCE 7-22 only applies to flood loads and does not include floodplain management requirements. When designing a building or structure, it is important to contact the jurisdiction to determine whether and how the community's floodplain management regulations and building code requirements vary from the flood-resistant design and construction requirements in the International Residential Code® (IRC®), International Building Code® (IBC®), ASCE 24, *Flood Resistant Design and Construction*, and ASCE 7. The designer should also inquire whether the IBC Appendix G (Flood-Resistant Construction) has been adopted in the community where the building is located. The ASCE 7-22-S2 design calculations for flood loads exceed the existing flood load calculation requirements outlined in the 2021 and earlier International Codes.

This design guide does not follow the specific order in which ASCE 7-22-S2 is presented. The design guide was developed to allow design professionals to assemble data in the order in which it would be gathered and calculated to avoid having to go back to previously referenced documents. It highlights all the necessary information to be collected from each data source at one time in an efficient manner. Where other ASCE 7-22 chapters or the ASCE 7-22-S2 commentary are required to complete the design process or a calculation, the reference is provided within the design guide section. In addition to the concepts covered in ASCE 7-22-S2, the design guide also addresses some aspects of floodplain management that may influence design decisions and thus flood loads.

Examples and explanations are included to assist design professionals in understanding how to apply sources or the methodology necessary to complete a calculation.

The first section of the design guide introduces floodplain management requirements and sources of flood data and explains how to determine the design flood data needed to complete the flood load calculations. The remainder of the first section provides a detailed overview of the calculation of flood loads based on ASCE 7-22-S2. The second section includes design examples to answer additional questions designers may have about the process of completing flood load calculations. The two primary examples address the calculation of flood loads for a coastal building and a riverine building. These two examples are supplemented by additional discussions of variants of the primary examples in order to illustrate alternative design conditions. Equation numbers throughout the first section are used in the second section example calculations to help readers refer back to the descriptions provided with the equations.

Where indicated, additional guidance and detail are provided that may exceed the minimum requirements of ASCE 7-22-S2. Information or procedures that are not found in ASCE 7-22-S2 are differentiated in the narrative or provided in text boxes. The appendices provide lengthy procedures for estimating coastal erosion and an alternative calculation for riverine velocities. This design guide contains five text box categories: Clarification, Exceeding Minimums, Additional Considerations, Resources, and Examples. Each of these text boxes contains information that is not provided in ASCE 7-22-S2. The examples presented in Chapter 10 and Chapter 11 consider all of the information from the main Chapters 1 through 9. The examples use both the ASCE 7-22-S2 guidance, and when possible, the guidance presented in the Exceeding Minimums, Additional Considerations, and Clarification text boxes. The information in the appendices and Chapter 2 should be considered information not provided in ASCE 7-22-S2 except where indicated.

Common text boxes in this design guide:

CLARIFICATION **text boxes provide additional information on topics to elicit a deeper understanding.**

EXCEEDING MINIMUMS **text boxes provide methods and rationale to consider going above the minimums outlined in ASCE 7-22-S2.**

ADDITIONAL CONSIDERATIONS **text boxes provide additional guidance to practitioners to aid in the completion of load calculations or a compliant design.**

RESOURCES **text boxes provide resources for further details on a specific topic or for tools to perform specific tasks.**

EXAMPLE **text boxes provide example calculations of methods either defined by ASCE-22-S2, this design guide, or a combination thereof.**

1.2. Overview of Changes for 7-22-S2

The *Building Designer's Guide to Calculating Flood Loads using ASCE 7-22 Supplement 2* was written specifically to address the many changes in flood load calculation methods and processes adopted during the ASCE 7-22 Flood Load Committee deliberations that were not present in the ASCE 7-16 Chapter 5 (Flood Loads) and ASCE flood load provisions, which had not changed in many years. There have been changes in determining the flood hazard, how to determine various aspects of flood loads, and how to combine various flood loads to use in ASCE load combination equations shown in ASCE 7-22 Chapter 2.

The most significant change was the adoption of using the structure's risk category to assign a mean recurrence interval (MRI) flood event as the design flood. This change required the creation of scaling factors for greater return periods than 100 years because the current flood studies and maps provided by FEMA are all based on the 100-year return period, and they frequently provide little information about greater than 100-year return periods. There are scaling factors for 500-year, 750-year, and 1000-year return periods for coastal, Great Lakes, and riverine flood locations when these values are not readily available.

Based on university and U.S. Army Corps of Engineers (USACE) research and storm data collection efforts, many aspects of flood load design have also changed. The primary changes are as follows:

- The calculation of the coastal flood velocity upper limit has always been considered conservative. USACE data provided information that supported the implementation of a 0.5 multiplier to the coastal velocity equation and supported limiting coastal velocities to a maximum of 15 ft/s. These changes reduce the hydrodynamic and flood debris loads.
- Additional design information on debris damming and drag forces was added.
- Significant details on wave load calculations were added. These additions include how to treat both non-breaking and breaking waves and provide a decision tree on how to calculate wave loads based on the site and expected wave conditions.
- Debris impact loads were revised to include more details on the approach and follows the methodology provided in ASCE 7-22 Chapter 6 on Tsunami Loads. There are exceptions for single- and multi-family residential use structures and for Risk Category II structures outside the Special Flood Hazard Area (SFHA). Debris minimum weight and elastic stiffness values required

for calculating debris impact loads have been provided for various debris impact objects, including passenger vehicles, small vessels, wood poles, shipping containers, and ships/barges.

- Flood load combinations have been re-defined to clarify which flood loads are to be included and represented as F_a in ASCE 7, Chapter 2, Load Combinations and the load factors for V Zones or Coastal A Zones were reduced and brought into alignment with the noncoastal A Zones flood load factors.

This design guide includes several examples to illustrate the changes in ASCE 7-22 Supplement 2. These examples also illustrate how to apply the flood provisions to a variety of flood scenarios. Other resources with information on flood loads are included for the designer.

2. Understanding Floodplain Management Considerations and Available Flood Data Sources

2.1. Floodplain Management

Most communities that are identified by the Federal Emergency Management Agency (FEMA) as floodprone adopt and enforce floodplain management regulations in order to participate in the NFIP. Local floodplain management ordinances require communities to regulate buildings and other development in SFHAs. The minimum requirements are included in 44 CFR § 60.3. In return for local management of floodprone areas, FEMA accepts communities to participate in the NFIP and makes federal flood insurance coverage available to property owners and tenants.

As part of local ordinances, communities must adopt and use Flood Insurance Studies (FISs) provided by FEMA. Flood Insurance Rate Maps (FIRMs) are adopted as part of the FISs. The studies and maps provide regulatory flood data. Some states and communities adopt more restrictive maps that incorporate higher flood elevations and more expansive floodplains, which include areas outside of SFHAs shown on FIRMs and areas not studied by FEMA. Additionally, communities may require incorporation of future sea level rise, which exceeds FEMA requirements and also typically exceeds the minimum requirement of ASCE 7-22-S2 (incorporation of historic sea level rise).

This chapter can help designers in identifying the flood zone and other regulated areas where the building is located, identifying what flood data sources they should reference for gathering flood data to complete load calculations, and provides an overview of flood-resistant design. Designers should consult their local floodplain ordinance in order to understand whether additional design requirements have been adopted that might influence their building use and design decisions. The material in this chapter is largely not covered in ASCE 7-22-S2, so where ASCE 7-22-S2 is referenced, the information is shown in *italicized* text.

2.2. Determining the Flood Zone

Before calculating flood loads, it is important to understand how floodplain management requirements could influence minimum elevation requirements for the building and foundation requirements as these requirements could influence the calculated flood loads. Requirements could be related to the local floodplain ordinance, ASCE 24-14, *Flood Resistant Design and Construction* (ASCE/SEI 2014), or flood provisions of the locally adopted building code. Considerations of how floodplain management requirements dictate building design decisions, such as foundation type, and incorporation of these elements into the building design will help to limit the potential revisions to building plans and the need to recalculate flood loads following a plans review. Understanding the requirements for design considerations, such as the inclusion of breakaway walls, will influence foundation load calculations. The minimum requirements identified in either codes, standards, or the local floodplain ordinance can be applied by identifying the flood zone and other regulated areas using either FEMA or locally developed flood data products.

ASCE 7-22-S2 applies to areas defined as the Flood Hazard Area. Buildings and other structures located in the Flood Hazard Area per ASCE 7-22-S2 must be designed and constructed to resist flood loads as determined in accordance with ASCE 7-22-S2, where applicable. ASCE 7-22-S2, Section 5.3.1, defines the Flood Hazard Area as it pertains to a structure’s Risk Category as:

> *For Risk Categories II, III, and IV structures, the Flood Hazard Area shall be the 500-year floodplain designated as the Special Flood Hazard Area and the Shaded X-Zone. For Risk Category I structures, the Flood Hazard Area shall be the 100-year floodplain designated as the Special Flood Hazard Area.*

CLARIFICATION

Utilizing Community Adopted Flood Hazard Maps

If a community has developed and adopted its own flood maps that delineate areas in addition to those described in ASCE 7-22-S2, Section 5.3.1, designers should utilize ASCE 7-22-S2 to develop flood loading criteria for buildings in the community-designated Flood Hazard Areas as well as those defined in Section 5.3.1. See the text box “Exceeding Minimums: Elevation Height” in Section 3.2.4 of this design guide for additional clarification on how community elevation requirements may be utilized to derive flood loads.

Gaining a comprehensive understanding of the different areas delineated on a FIRM is an important first step in determining where ASCE 7-22-S2 applies and how applicable floodplain management requirements are also regulated. The SFHA is the land area subject to flooding by the base flood. The base flood is the flood that has a 1% chance of being equaled or exceeded in any given year (commonly called the “100-year flood”). SFHAs are shown on FIRMs prepared by FEMA as Zone A and Zone V. FIRMs also show Zone X, which are areas outside of the SFHA and include the 500-year floodplain.

Figure 1 shows the typical flood zones from coastal and riverine flood sources as they apply to NFIP minimum requirements for V Zone and A Zone and additional requirements of the International Codes and ASCE 24-14 with respect to Coastal A Zones. A Zone and X Zone exist in both coastal and riverine flood source environments.

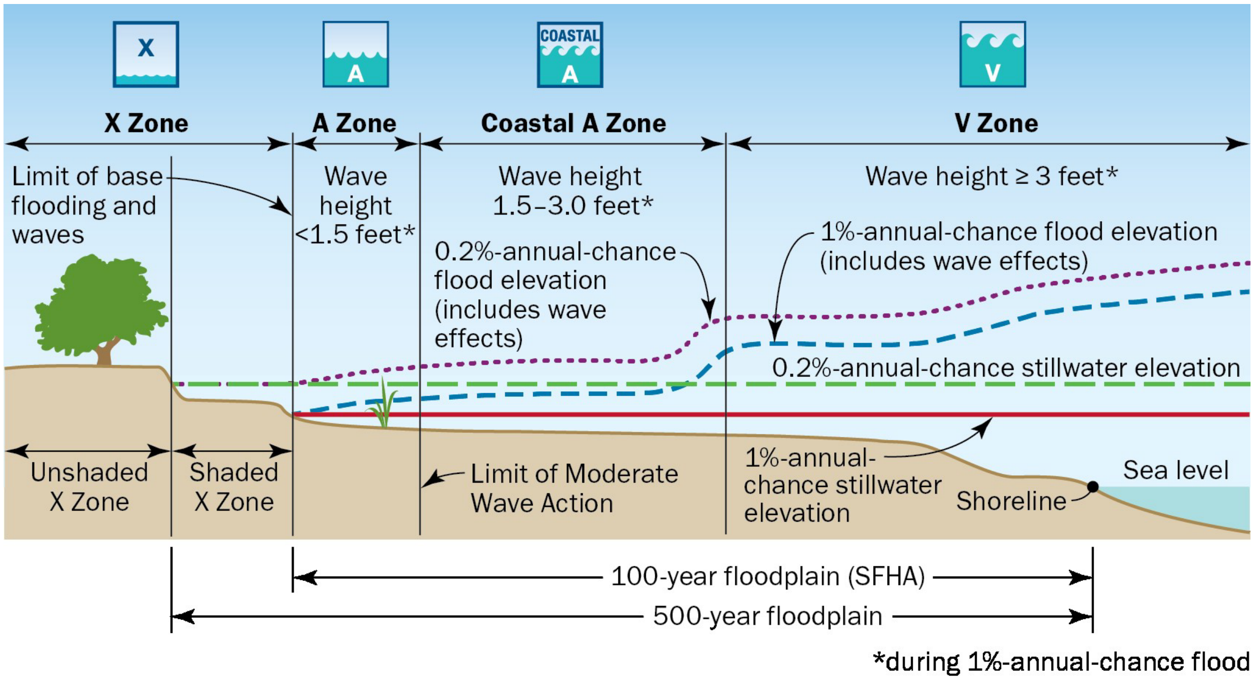


Figure 1. Typical flood zones from coastal flood sources

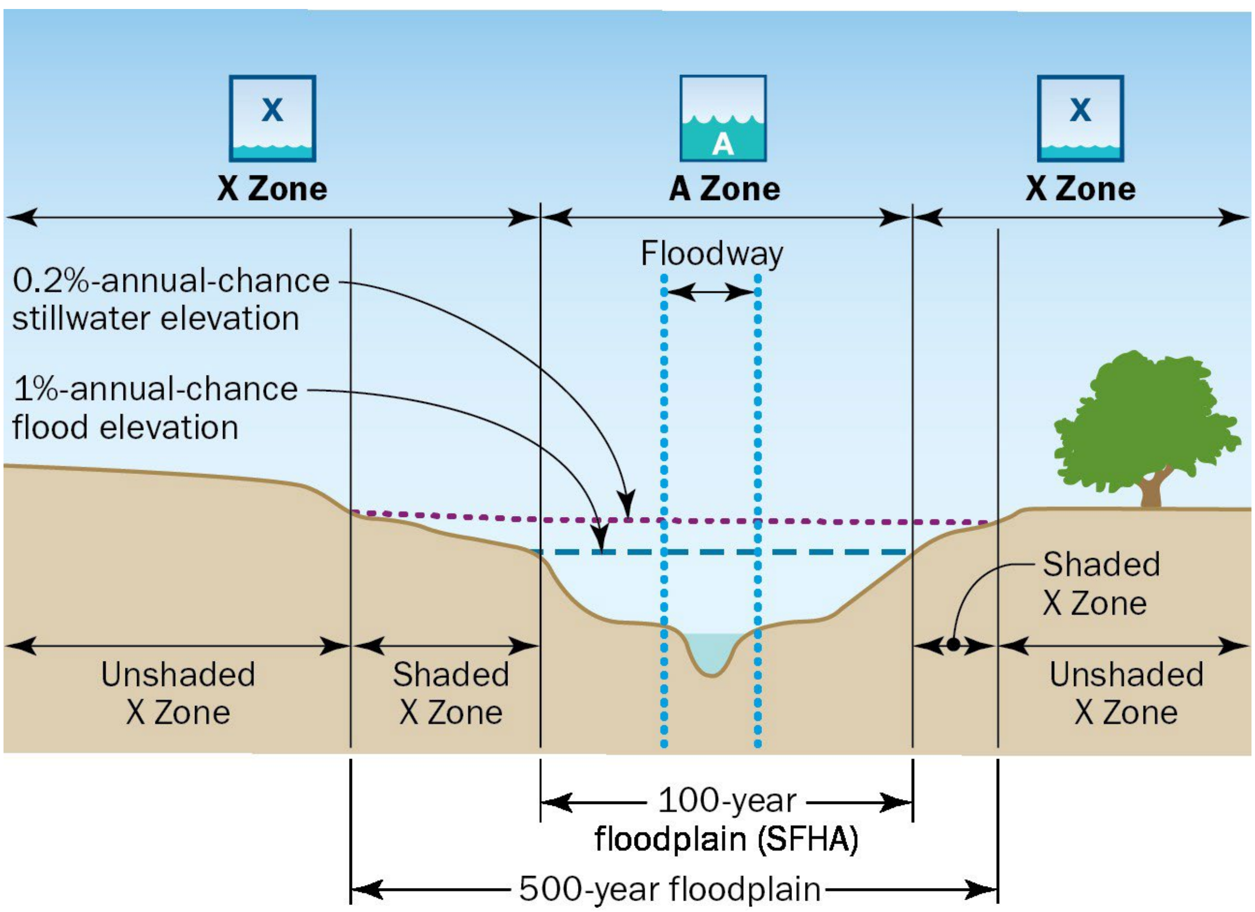


Figure 2. Typical flood zones from riverine flood sources

Flood zones are described in the following text—in some instances with more information than is provided in ASCE 7-22-S2. With the exception of the Coastal A Zone, which is delineated by the Limit of Moderate Wave Action (LiMWA), all of the flood zones can appear on a FIRM but are dictated by the mapped base flood conditions.

A Zone. In SFHAs identified as A Zones (Zones A, AE, A1–30, AR, AO, and AH), the principal source of flooding is runoff from rainfall, snowmelt, or coastal storms where the base flood wave height is less than 3.0 feet. A Zones have minimum design and construction requirements that specify the elevation of the lowest floor, including the basement, in relation to the base flood elevation (BFE) or depth of the base flood. Specific requirements apply to fully enclosed areas below the lowest floor.

Coastal A Zone. Coastal A Zones represent the portion of the A Zone where wave heights are expected to be 1.5 feet or greater during base flood conditions. Coastal A Zones are not formally delineated on a FIRM but include areas seaward of the LiMWA[1] (if delineated) and landward of the V Zone boundary or landward of the shoreline where a V Zone is not identified. Design and construction requirements in Coastal A Zones depend on the codes and regulations of the authority having jurisdiction (AHJ). Most building codes align Coastal A Zones requirements with those of V Zones. In the absence of building codes, the NFIP regulations may be the minimum requirement, which aligns Coastal A Zone requirements with A Zone requirements.

V Zone. V Zones (Zones V, VE, V1–30, and VO), also referred to as Coastal High Hazard Areas, extend from offshore to the inland limit of a primary frontal dune (PFD) along an open coast, and any other area subject to high-velocity wave action from storms (or in some locations tsunamis) where the potential base flood wave height is 3.0 feet or more. V Zones have more restrictive requirements than A Zones pertaining to the siting of buildings, elevation of the lowest horizontal structural member of the lowest floor in relation to the BFE, foundation design, enclosures below the lowest floor, and alterations of sand dunes and mangrove stands.

Shaded X Zone. X Zones identify areas outside the SFHA. Shaded X Zones identify areas of moderate flood hazard subject to inundation by the flood that has a 0.2% chance of being equaled or exceeded during any given year (commonly called the "500-year flood").

Unshaded X Zone. X Zones identify areas outside the SFHA. Unshaded X Zones identify areas of minimal flood hazard, which are outside the 500-year floodplain.

ADDITIONAL CONSIDERATIONS

Federal Compliance Notices

Communities that participate in the NFIP must review and issue permits for development of any kind in the SFHA, including buildings and structures, to ensure compliance with the NFIP minimum requirements. FEMA recommends that designers work with the local floodplain

[1] The LiMWA represents the landward limit of the 1.5-foot wave (FEMA 2011).

administrator to verify that all NFIP minimum requirements and any additional locally enforced requirements are also being met.

The NFIP floodplain management requirements do not apply in X Zones. However, projects funded by federal grants may be required to utilize the 500-year floodplain as the minimum floodplain of concern, see 44 CFR § 9.6.

2.3. Acquiring Data and Reviewing Applicable Regulations

Flood data can come from a variety of sources. Communities regulate to a locally adopted flood map, which is either a FIRM or a community flood hazard map. The locally adopted and enforced FIRMs and associated FIS are referred to as being "effective" maps and an effective study. Communities may have newer mapping products that are available, but have not been formally adopted, which are referred to as "preliminary" FIRMs and a "preliminary" study. These preliminary FIRMs and studies contain more recent data. If both preliminary and effective FIRMs and studies exist, FEMA recommends that design professionals consider using the preliminary FIRMs and studies for design values if they exceed the values in the effective FIRMs and studies. However, designers should verify that the BFE and the extent of the SFHA on the preliminary FIRM are not lower or less restrictive than the effective FIRM. If the preliminary FIRM and study are lower or less restrictive, the effective FIRM must be used until the preliminary FIRM is formally adopted, because the effective FIRM represents a higher minimum requirement. Some communities may adopt community flood hazard maps, which can incorporate higher BFEs than those indicated on the FEMA study and they can also reflect a larger extent of the floodplain.

Following some federally declared disasters, an Advisory Base Flood Elevation (ABFE) map may be temporarily enforced during the recovery process until revised FIRMs can be produced. In order to expedite the mapping process, assumptions are included in the models that may be revised during future mapping updates. Later, more detailed mapping products may result in flood zone designations and BFEs that are less restrictive than those shown on ABFE maps. These ABFE products indicate the extent of the SFHA and sometimes include the 0.2% floodplain and contain the BFE, and if the 0.2% floodplain is delineated, they also contain the 0.2% flood elevations.

When available, Base Level Engineering provides high-resolution data that will eventually be used to determine flood elevations. Once a Base Level Engineering assessment is completed, FEMA releases the flood risk information on the Estimated Base Flood Elevation Viewer (https://go.usa.gov/xsGdn). The Estimated Base Flood Elevation Viewer, an interactive web portal, allows federal, state, regional, and local entities; industry professionals; and the public at large to interact with the Base Level Engineering results. Base Level Engineering (BLE) data are not regulatory but may be considered for design values when determining flood conditions if the BLE data provide an elevation for the 1% annual chance flood event that exceeds the BFE on the effective FIRM.

2.3.1. FIRM AND FIS DATA TO RECORD

Once a designer has determined that designing for flood loads is required, the information in Table 1 and Table 2 can be recorded from the FIRM and FIS for later use. The FIRM and FIS are available online through FEMA's Map Service Center at https://msc.fema.gov. Table 1 and Table 2 list the flood data necessary to complete flood loads calculations and the source of the data for coastal and riverine locations, respectively.

Table 1. Coastal Flood Data Items and Associated Source Documents

Item to Record[(a)]	*Source*	*Section*
FIRM Panel Number	FIRM	
FIRM Effective Date	FIRM	
Flood Zone	FIRM	
BFE	FIRM	
Transect	FIRM	
FIS Number and Effective Date	FIS	Cover page
1% Stillwater Elevation (SWEL)	FIS	Summary Of Coastal Stillwater Elevations Tables
0.2% Stillwater Elevation (SWEL) (if available)	FIS	Summary Of Coastal Stillwater Elevations Tables
0.2% Annual Chance Wave Envelope (if available)(b)	FIS	Transect Profiles

BFE = base flood elevation; FIRM = Flood Insurance Rate Map; FIS = Flood Insurance Study

(a) Record the datum (e.g., NAVD 88 or NGVD 29) for all elevations. Datums and datum conversions are discussed further in Appendix A.

(b) The 0.2% Annual Chance Wave Envelope provides the elevation of the 0.2% Annual Exceedance Probability (AEP) (500-year) wave.

Table 2. Riverine Flood Data Items and Associated Source Documents

Item to Record[(a)]	*Source*	*Section*
FIRM Panel Number	FIRM	
FIRM Effective Date	FIRM	
Flood Zone	FIRM	
BFE (if a Flood Profile is provided in the FIS, use the Flood Profile to determine the BFE)	FIRM	
Nearby Cross-Sections with 1% Annual-Chance Water Surface Elevations	FIRM	

Item to Record[a]	***Source***	***Section***
1% Annual-Chance Flood or 1% Annual-Chance Water Surface Elevation(b)	FIS	Flood Profiles (if no Flood Profile is provided in the FIS, use the BFE from the FIRM)
FIS Number and Effective Date	FIS	Cover page
0.2% Annual Chance Flood	FIS	Flood Profiles – if available
Stream Bed Elevation	FIS	Flood Profiles – if available
Mean Velocity in the Floodway, where delineated (feet per second)	FIS	Floodway Data Tables – if available
Floodway Width	FIS	Floodway Data Tables – if available

BFE = base flood elevation; FIRM = Flood Insurance Rate Map; FIS = Flood Insurance Study

[a] Record the datum (e.g., NAVD 88 or NGVD 29) for all elevations. Datums and datum conversions are discussed further in Appendix A.

[b] When flood profiles are available in the FIS, the flood profile data should be used to determine flood elevations and not the nearby BFEs shown on the FIRM. The flood profiles provide more accurate site-specific flood elevation data.

RESOURCES

FIRM and FIS Guidance

FEMA 480, *National Flood Insurance Program (NFIP) Floodplain Management Requirements: A Study Guide and Desk Reference for Local Officials* (FEMA 2005), Unit 3 and Unit 4, provide detailed information on how to read NFIP Studies and maps.

FEMA also provides flood data tutorials, which provide information on FIRMs and FISs. Tutorials can be accessed here: https://www.fema.gov/flood-maps/tutorials.

CLARIFICATION

Identifying Flood Data in Shaded X Zones

Because the initial intent of FIRMs and associated FISs was to regulate the SFHA, less flood information is available for buildings located in Shaded X Zones. The X Zone represents an area outside of the SFHA, so the ground elevation is above the 1% annual-chance flood elevation. ASCE 7-22-S2 flood load calculations instruct the designer to use the $SWEL_{MRI}$ where available, but if it is not provided to use the 1% annual chance stillwater elevation ($SWEL_{100}$) as a starting point. While the ground elevation of a Shaded X Zone is above the 1% annual-chance flood elevation, the 1% annual chance stillwater elevation ($SWEL_{100}$) can still be used as the basis for ASCE 7-22-S2 flood elevation scaling calculations where the 0.2% (500-year) annual-chance stillwater elevation is not available.

Identifying the $SWEL_{100}$ in Coastal Flood Areas:

Identify the location of the building on the FIRM. Identify the highest BFE in the A Zones that are adjacent to the Shaded X Zone that the building is located within. Next, identify the closest transect associated with the A Zone showing the highest BFE. Use the Summary of Coastal Stillwater Elevations Tables in the FIS to obtain the 1% annual-chance stillwater elevation and if available the 0.2% annual-chance stillwater elevation. Generally, the 0.2% data will be available where Shaded X Zones are delineated.

Identifying the $SWEL_{100}$ in Riverine Flood Areas:

Similar to buildings located in riverine A Zones, for buildings located in Shaded X Zones, use the centerline of the stream or river to identify where a line can be drawn perpendicular from the stream centerline to intersect with the building footprint. The line should intersect the most upstream portion of the building footprint. Then, take a measurement to the nearest stream cross-section (represented by a letter surrounded by a hexagon) on the FIRM. Next, find the flood profile for the stream in the FIS and take a measurement from the identified cross-sections (along the X-axis of the profile) to identify the building location along the stream. The Y-axis provides the elevation of at least the 1% annual-chance flood but may also provide the elevation of the 0.2% annual-chance flood for the building location. Generally, the 0.2% data is available where Shaded X Zones are delineated. To gather flood velocity data, use a similar process for identifying the closest available cross-section for velocity data from the Floodway Data Table. Then, identify the 1% annual-chance flood velocity from the Floodway Data Table and apply the value based on Section 5.1.2 of this guide.

2.3.2. REGULATIONS AND GUIDANCE PERTAINING TO FLOOD-RESISTANT DESIGN

Participating NFIP communities are required to meet or exceed the NFIP minimum requirements outlined in 44 CFR § 60.3. Communities adopt a local floodplain ordinance that outlines all their minimum requirements. These locally adopted floodplain ordinances regulate all development within the floodplain. In addition to these requirements, many communities also enforce a state or locally adopted building code. Codes often reference ASCE 24-14 (or ASCE 24-05),[2] which outlines additional flood-resistant design requirements for buildings.

Designers will need to investigate both the locally adopted floodplain management ordinance and, when applicable, ASCE 24-14 to determine where there are more restrictive requirements. ASCE 24-14 was developed to incorporate the potential for locally enforced minimum elevation requirements, which could be a specified elevation from a Community Flood Hazard Map or the ordinance provides provisions for a calculated freeboard value, often provided in feet, that is added to the BFE. ASCE 24-14 refers to this locally adopted minimum elevation requirement as the design flood elevation (DFE); however, that definition is somewhat different than how this guide refers to the DFE, see Section 3.2.4 for more information. ASCE 24-14 provides minimum elevation requirements based on the Flood Design Class and the flood zone. Additional flood-resistant design provisions are provided in

[2] ASCE 24-05, *Flood Resistant Design and Construction* (ASCE/SEI 2005).

ASCE 24-14 based on the flood zone and hazard. However, FEMA recommends that designers still evaluate the local floodplain ordinances as some communities may have more restrictive requirements in their ordinance.

Riverine and Associated Floodway Requirements

When evaluating the project site, it is important to evaluate the placement of the structure in relation to the information provided on the FIRM. One item to assess is the structure's location relative to the floodway; see Figure 3 for an example of how a floodway is delineated on a FIRM. Floodways represent delineated areas where there are restrictions on placing fill or obstructions in the floodplain. Before structures or development can be placed in the floodway, an analysis must be conducted to determine the impact of the structure/development/fill and whether it increases the elevation of flooding in the surrounding area, including upstream and downstream. A hydrologic and hydraulic (H&H) analysis must be conducted to verify that the development within the floodway will not increase base flood heights. In a regulatory floodway, there is no acceptable increase. A "no rise" certificate and associated documentation must be submitted to the local jurisdiction to confirm that the verification has been completed. If an H&H analysis is required, then the analysis may provide an opportunity to make a better determination of the flood velocities for the base flood and the design flood heights than are available in the FIS. More information on these requirements can be found in FEMA 480.

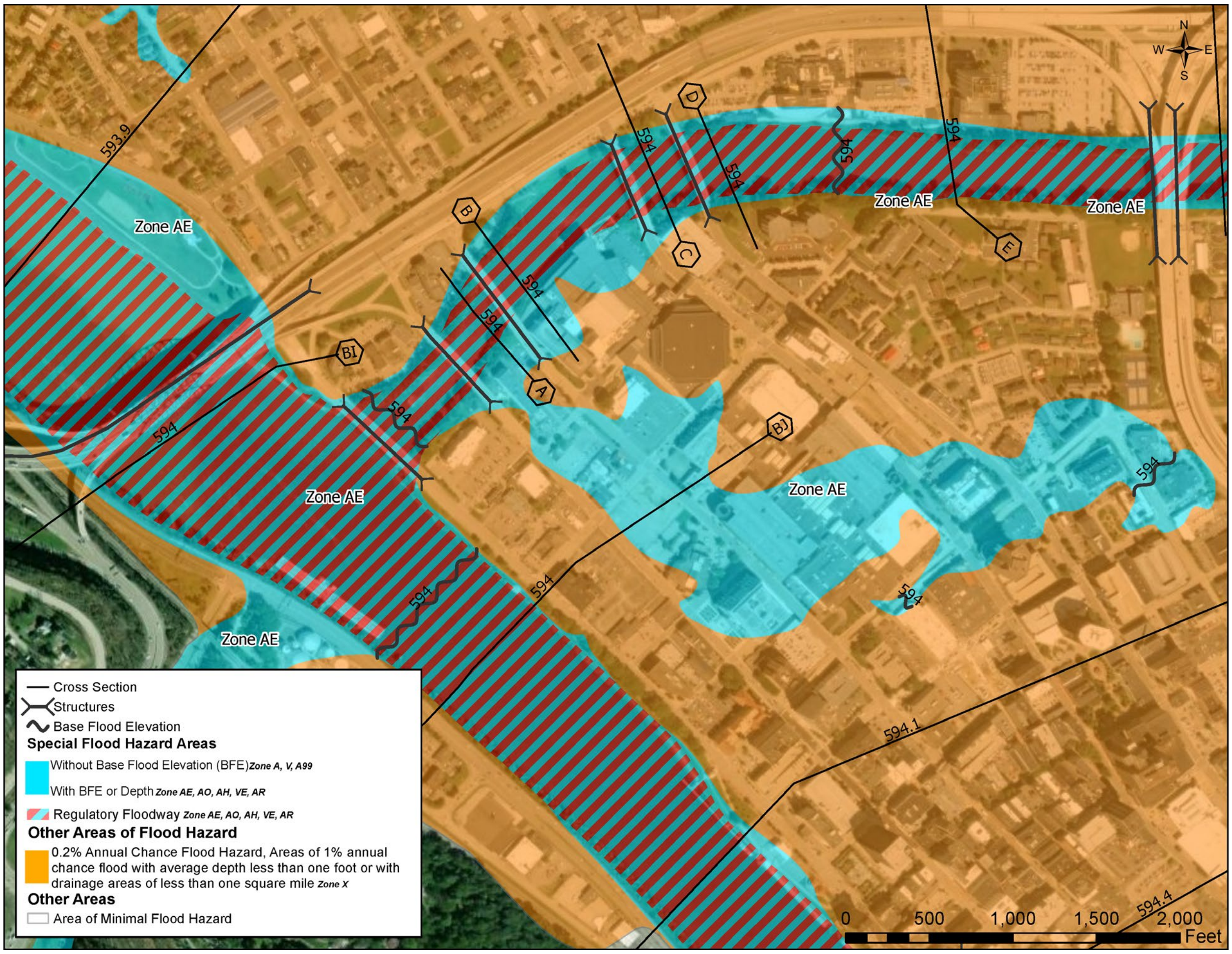


Figure 3. Riverine flood map panel with a delineated floodway

Coastal Floodplain Requirements

Coastal structures have requirements specific to allowable foundation types in portions of the coastal floodplain. In areas designated as V Zones (referred to in many requirements as the Coastal High Hazard Area), the NFIP requires that structures have open foundations; this can be accomplished with piles, columns, or in some instances shear walls (according to NFIP Technical Bulletin 5 (TB 5), *Free-of-Obstruction Requirements for Buildings Located in Coastal High Hazard Areas in Accordance with the National Flood Insurance Program* [FEMA 2020a]), which should be oriented parallel to the direction of flow. Information on the use of shear walls in V Zones is addressed in Section 6.8 of NFIP TB 5. Additionally, in V Zones, the NFIP minimum elevation requirement dictates that the bottom of the lowest horizontal structural member supporting the lowest floor shall be at or above the BFE. Where the NFIP references the BFE for building requirements, these requirements are superseded if the building is required to comply with ASCE 24-14 or the local jurisdiction has higher minimum elevation requirements. The highest elevation requirement takes precedence.

There are instances where structure elements may be perpendicular to the direction of flow in V Zones. In these instances, a “free-of-obstruction” analysis must be conducted to verify that the elements (e.g., a retaining wall or pool) would not divert floodwaters into surrounding buildings and increase the flood risk to those buildings. NFIP TB 5 addresses what is needed to comply with a “free-of-obstruction” analysis. When buildings with open foundations are constructed in V Zones, enclosed areas below the lowest floor must be constructed with breakaway walls (or, alternatively, insect screening or latticework). These breakaway walls must be designed to fail during a base flood or lesser conditions. Requirements for these walls and design methods are outlined in NFIP Technical Bulletin 9 (TB 9), *Design and Construction Guidance for Breakaway Walls Below Elevated Buildings Located in Coastal High Hazard Areas in Accordance with the National Flood Insurance Program* (FEMA 2021a). Although not required by the NFIP, ASCE 24-14 requires that breakaway walls in V Zones have flood openings.

The NFIP treats all buildings in the A Zone the same, but ASCE 24-14, many building codes, and some local floodplain ordinances may have more restrictive requirements for portions of the A Zone. ASCE 24-14 has more restrictive requirements for structures located in the Coastal A Zone when it is delineated. Buildings are addressed similarly to those located in the V Zone with requirements for open foundations. Minimum elevation requirements in the Coastal A Zone differ between the NFIP and ASCE 24-14. The NFIP references minimum elevation requirements to the top of the lowest floor in all A Zones, but ASCE 24-14 references minimum elevation requirements in Coastal A Zones to the bottom of the lowest horizontal structural member of the lowest floor, the same as V Zone requirements. ASCE 24-14 allows for the use of stem wall foundations[3] in the Coastal A Zone provided that it can meet predicted scour and erosion and all stability requirements consistent with shallow open foundations. Measurement to the lowest horizontal structural member on stem wall foundations is typically referenced to the bottom of the floor slab. Open foundations are allowed to incorporate enclosures below the lowest floor. However, ASCE 24-14 requires these enclosures to be either lattice, insect screening, or breakaway walls. Both the NFIP and ASCE 24-14 require breakaway walls in the Coastal A Zone to include flood openings. ASCE 24-14 also requires flood openings in V zone breakaway walls, while the NFIP does not.

Prior to beginning the design process, it is important to understand how the floodplain management requirements and provisions of ASCE 24-14 can impact design decisions. Although the process for calculating flood loads has undergone significant changes with ASCE 7-22-S2, and those changes are not included in FEMA P-55, *Coastal Construction Manual* (FEMA 2011), FEMA P-55 remains a fundamental source for design considerations and guidance. Additionally, FEMA P-550, *Recommended Residential Construction for Coastal Areas* (FEMA 2009), may also provide valuable insights into the design of coastal foundations, but readers should be aware of changes in flood load calculations and adjustments to wind design procedures (and associated load combination factors) since release of the guidance.

[3] A stem wall is a solid perimeter wall that is backfilled and topped with a concrete slab.

ADDITIONAL CONSIDERATIONS

Breakaway Walls

Based on the regulatory requirements, buildings in V Zones (and Coastal A Zones when required) will be required to be constructed on open foundations. If enclosures are incorporated into the design, the enclosure walls must be breakaway walls. NFIP TB 9 provides three design methods for breakaway walls. The prescriptive and simplified designs rely on tables and illustrations provided in TB 9 to meet the breakaway walls requirements. In some instances, either because of design decisions or based on design wind speed, the performance-based design method may be required for breakaway wall design. Designers should note that ASCE 7-22-S2 references a minimum design load of 16 lb/ft^2, while TB 9 uses a value of 17 lb/ft^2. This value was previously 10 lb/ft^2 based on an allowable stress design approach but has been updated to be consistent with modern wind design approaches. Designers should use the more restrictive value between ASCE 7-22-S2 and TB 9. Designers are encouraged to read the performance-based design approach in its entirety before beginning a design. When designing the breakaway walls and their connection to the structure, the design of the breakaway walls should not be made overly conservative or stronger than necessary. The intent is to provide for an intentional failure (thus breakaway) for flood loads during a base flood event (100-year flood). When designing the foundation, the lateral flood loads on the foundation members should consider two design conditions: one considers the probable capacity of the foundation (including the loads on the breakaway wall system and connections) just prior to failure of the breakaway wall (during a base flood event) and another one considers the capacity of the foundation during a design flood event (as discussed in Section 3.1) where the breakaway wall has failed and, thus, does not transfer additional load to the primary structure. The larger of the loads for the two conditions should be used for the foundation design.

ASCE 7-22-S2, Section 5.3.10, states:

> *Where required by ASCE/SEI 24 to break away, walls and partitions, including their connections to the structure, shall be designed in accordance with this section. The wall shall be designed to resist the following loads acting perpendicular to the plane of the wall:*
>
> 1. *The wind load specified in this standard,*
> 2. *The seismic load specified in Chapter 12*
> 3. *The lateral earth pressure specified in Chapter 3, and*
> 4. *16 lb/ft^2 (0.76 kN/m^2).*
>
> *If the largest of the loads above is less than the 100-year flood load, the wall shall be designed to fail during the 100-year flood condition.*

Breakaway walls are not permitted to be designed to a load that exceeds the 100-year flood load. Thus, in some instances (such as areas with high wind speeds and small wave heights), breakaway walls may not be a feasible design option as it may not be possible to design them to both survive the loads noted in ASCE 7-22-S2, Section 5.3.10, while also failing at or below the 100-year flood loads.

ADDITIONAL CONSIDERATIONS

Foundation Considerations Based on Wave Heights

Requirements for building in V Zones and Coastal A Zones in ASCE 24-14 are based upon building performance during design flood events in areas of high and moderate wave action. FEMA recommends evaluating wave heights when evaluating an appropriate building foundation. As discussed in Section 3.1 of this design guide, most structures designed using ASCE 7-22-S2 are designed using a higher MRI than the 100-year base flood event. Consequently, wave heights are often greater than those in the base flood condition. When wave heights (see Section 6.1) are greater than 3 feet, and buildings are located outside of the delineated V Zone and Coastal A Zone, FEMA recommends that consideration be given to incorporating open foundations and elevating the lowest horizontal structural member supporting the lowest floor above the wave crest elevation similarly to Chapter 4 of ASCE 24-14. For light-frame construction buildings, the same recommendations should be considered when wave heights are greater than 1.5 feet.

NFIP Technical Bulletins

FEMA developed NFIP Technical Bulletins (TBs) to provide guidance for complying with the NFIP building performance requirements in 44 CFR § 60.3, Floodplain Management Criteria for Flood-prone Areas. TBs can help inform design professionals and others of the minimum floodplain management requirements for buildings and other structures. FEMA still recommends that designers meet with local floodplain administrators when structures are proposed to be located in the SFHA. Table 3 lists the title for each of the 12 TBs. At a minimum, designers should consult NFIP Technical Bulletin 0 (TB 0), *User's Guide to NFIP Technical Bulletins* (FEMA 2021c), to gain additional familiarity with the TBs and understand which TBs might apply to the structures they are designing. When applicable, TBs are referenced throughout this design guide.

Table 3. NFIP Technical Bulletin Overview

Technical Bulletin (TB)	*Title*
TB 0	*User's Guide to NFIP Technical Bulletins*
TB 1	*Requirements for Flood Openings in Foundation Walls and Walls of Enclosures*
TB 2	*Flood Damage-Resistant Materials Requirements*
TB 3	*Requirements for the Design and Certification of Dry Floodproofed Non-Residential and Mixed-Use Buildings*
TB 4	*Elevator Installation*
TB 5	*Free-of-Obstruction Requirements*

Technical Bulletin (TB)	***Title***
TB 6	*Requirements for Dry Floodproofed Below-Grade Parking Areas Under Non-Residential and Mixed-Use Buildings*
TB 7	*Wet Floodproofing Requirements and Limitations*
TB 8	*Corrosion Protection for Metal Connectors and Fasteners in Coastal Areas*
TB 9	*Design and Construction Guidance for Breakaway Walls*
TB 10	*Reasonably Safe From Flooding Requirement for Building on Filled Land Removed From the Special Flood Hazard Area*
TB 11	*Crawlspace Construction for Buildings Located in Special Flood Hazard Areas (Interim Guidance)*

Key NFIP Floodplain Management Terminology

Table 4 provides additional descriptions of key floodplain management terms that are not defined in ASCE 7-22-S2. These additional details are provided to allow designers to consider how the terms potentially apply to floodplain management. Reference ASCE 7-22-S2, Section 5.2.1, for additional definitions.

Table 4. Key Floodplain Management Terms and Design Considerations

Term	***CFR Definition***	***Design Considerations***
Basement	"Area of the building having its floor subgrade (below ground level) on all sides" (44 CFR § 59.1).	In the Special Flood Hazard Area (SFHA), National Flood Insurance Program (NFIP) regulations do not allow basements to extend below the base flood elevation (BFE) except in dry-floodproofed, non-residential buildings. FEMA TB 3 describes requirements for dry floodproofing of non-residential portions of mixed-use buildings.
Development	"Any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation or drilling operations or storage of equipment or materials" (44 CFR § 59.1).	All buildings and structures constructed in the SFHA must obtain permits from the authority having jurisdiction (AHJ).

Term	***CFR Definition***	***Design Considerations***
Floodway	"The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height" (44 CFR § 59.1 – Defined as Regulatory Floodway). "Prohibit encroachments, including fill, new construction, substantial improvements, and other development within the adopted regulatory floodway unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed encroachment would not result in any increase in flood levels within the community during the occurrence of the base flood discharge;" (44 CFR § 60.3(d)(3) – Community Requirements)	Any development constructed in a floodway (e.g., buildings, structures, fill material, etc.) is required to submit a "no rise" certification as discussed in Section 2.3.2, subsection Riverine and Associated Floodway Requirements, of this design guide. When a floodway is not delineated, new construction or Substantial Improvements or repair of Substantially Damaged buildings can be permitted only when analysis demonstrates that the cumulative effect of the proposed development will not increase the base flood more than 1 foot. The terms "substantial improvement" and "substantial damage" are defined in 44 CFR § 59.1 and ASCE 24-14.
Lowest Floor	"The lowest floor of the lowest enclosed area (including basement). An unfinished or flood resistant enclosure, usable solely for parking of vehicles, building access or storage in an area other than a basement area is not considered a building's lowest floor; *Provided*, that such enclosure is not built so as to render the structure in violation of the applicable non-elevation design requirements of §60.3" (44 CFR § 59.1 – Defined as Lowest Floor).	The lowest floor is provided as a reference for multiple floodplain management requirements. In A Zones, the minimum elevation requirements are measured to the top of the lowest floor. In V Zones, the minimum elevation requirement is measured to the bottom of the lowest horizontal structural member supporting the lowest floor.
Primary frontal dune (PFD)	"A continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope" (44 CFR § 59.1).	Buildings or structures located on or near a PFD should consider the potential impacts of erosion. This can be estimated based on the procedure outlined in Appendix A. Man-made alterations of sand dunes and mangrove stands within V Zones are prohibited if they would increase potential flood damage. Additional information is provided in NFIP TB 5.

Term	*CFR Definition*	*Design Considerations*
Enclosure (enclosed area)	"An area below an elevated building that is enclosed by walls on all sides" (NFIP TB 1). The term applies to buildings within the SFHA that are not dry floodproofed (see more information on dry floodproofing in NFIP TB 3).	Enclosures below buildings in the SFHA must allow the automatic entry and exit of floodwaters, be constructed of flood damage–resistant materials, have breakaway walls (in V Zones), and have the use limited to building access, parking of vehicles, and storage. Additional information can be found in NFIP TBs 1, 2, and 5. ASCE 24-14 defines "enclosed area or enclosure."

3. Defining the Flood Hazard

3.1. Association Between Risk Category and Design Flood Mean Recurrence Interval

As with other sections of ASCE 7-22, flood loads in ASCE 7-22-S2 are now calculated using the Risk Category associated with the building. The Risk Category is developed using ASCE 7-22 Section 1.5, Classification of Buildings and Other Structures. This information is used to inform the governing Flood Hazard Area and the design flood mean recurrence interval (MRI) as shown in Table 5. The Flood Hazard Area includes the 100- and 500-year floodplains, while the design flood MRIs include the 100-, 500-, 750-, and 1000-year MRIs. The MRI was assigned to each Risk Category using a target reliability by considering the probability of a design flood occurring over the minimum project useful life of 50 years (e.g., a Risk Category III structure uses a 750-year MRI, which has a 6% chance of experiencing a design flood event over 50 years). The chance of experiencing a design flood event is calculated as an Annual Exceedance Probability (AEP) and then the inverse of that value is the MRI expressed as years. Although the MRI is expressed in years, it represents the probability of the event occurring over a period of time and should not be taken as the number of years between the design events for the building. Table 6 indicates the flood loads that should be calculated per Risk Category using the ASCE 7-22-S2. Not all of the loads shown may apply, depending on the flood conditions at the structure site. The Risk Category and MRI should be noted on the design drawings along with other important flood design information. See Appendix B for a checklist of recommended items to include on design drawings for buildings in Flood Hazard Areas.

Table 5. Governing Flood Hazard Areas and Mean Recurrence Intervals per Risk Category [Based on ASCE 7-22-S2, Section 5.3.1 and Table 5.3-1]

ASCE 7-22-S2 Design Flood Requirements	*Risk Category I*	*Risk Category II*	*Risk Category III*	*Risk Category IV*
Governing Flood Hazard Area	100-year floodplain[(a)]	500-year floodplain[(b)]	500-year floodplain[(b)]	500-year floodplain[(b)]
Design Flood Mean Recurrence Intervals (MRIs) and Annual Exceedance Probabilities (AEPs)	100-year MRI (1% AEP)	500-year MRI (0.2% AEP)	750-year MRI (0.13% AEP)	1000-year MRI (0.1% AEP)

(a) 100-year floodplain is represented by the SFHA on a FIRM.
(b) 500-year floodplain includes the SFHA and Shaded X Zone on a FIRM.

Table 6. Flood Loads Required per Risk Category

Loads and Forces Covered in ASCE 7-22-S2	*ASCE 7-22-S2 Section*	*Section in this design guide*	*Risk Category I*	*Risk Category II*	*Risk Categories III and IV*
Lateral Hydrostatic (F_h)	5.4.2.2	4.1.1	Yes	Yes	Yes
Uplift Hydrostatic, Buoyancy (F_B)	5.4.2.1	4.1.1	Yes	Yes	Yes
Drag Force on Components (F_{drag})	5.4.3.1	5.3.1	Yes	Yes	Yes
Drag Force on Lateral Force Resisting System (F_{drag})	5.4.3.2	5.3.2	Yes	Yes	Yes
Debris Damming Drag Force (F_{drag})	5.4.3	5.3.2	No	Yes, if d_f > 3' and clear spacing between vertical structural elements fails to meet ASCE 7-22-S2 criteria Exception: Enclosed buildings without breakaway walls	Yes, if d_f > 3' and clear spacing between vertical structural elements fails to meet ASCE 7-22-S2 criteria Exception: Enclosed buildings without breakaway walls
Lateral Wave	5.4.4	6.3.1	Yes, if waves present[(a)]	Yes, if waves present[(a)]	Yes, if waves present[(a)]
Uplift Wave (F_L)	5.4.4.3	6.3.2	Yes, if waves present[(a)]	Yes, if waves present[(a)]	Yes, if waves present[(a)]
Debris Impact	5.4.5	8.2	No	Yes, if df > 3' Exceptions: 1. Detached one- and two-family dwellings are excluded	Yes, if d_f > 3'

Loads and Forces Covered in ASCE 7-22-S2	*ASCE 7-22-S2 Section*	*Section in this design guide*	*Risk Category I*	*Risk Category II*	*Risk Categories III and IV*
				2. Risk Category II Structures outside of the 100-year floodplain (SFHA)[b] are excluded	

(a) Per ASCE 7-22-S2, Section 5.3.7.1, the effects of waves shall be included for both V Zones and A Zones. In areas subjected to riverine flooding only, the effects of waves are permitted to be neglected.

(b) The Special Flood Hazard Area (SFHA) is the land in the floodplain subject to a 1% or greater chance of flooding in any given year. These areas are delineated on a community's FIRM as A Zones (A, AE, A1–30, A99, AR, AO, or AH) or V Zones (V, VE, VO, or V1–30). Thus, Risk Category II buildings in Shaded X Zones, Unshaded X Zones, and unmapped zones are not required to account for debris impact forces.

3.2. Determining Stillwater Elevation ($SWEL_{MRI}$) and Design Stillwater Flood Depth (d_f)

Most flood loads within ASCE 7-22-S2 are derived from the stillwater elevation corresponding to the risk category and MRI ($SWEL_{MRI}$) and design stillwater flood depth (d_f). Some loads use wave heights, which are either calculated using the $SWEL_{MRI}$ or, when available, wave data that may be provided in a study.

Sections 3.2.1, 3.2.2, and 3.2.3 provide details for calculating the variables required to calculate the design stillwater flood depth (d_f) in Section 3.2.4.

3.2.1. CALCULATING STILLWATER ELEVATION ($SWEL_{MRI}$) WHEN MRI DATA ARE NOT AVAILABLE

This section discusses the methods for calculating the stillwater elevation ($SWEL_{MRI}$) when MRI data are not available for the required MRI event. If the stillwater elevation for the required MRI event is known, the designer should use the known value and does not need to apply Section 3.2.1.

Equation 1 defines how to determine the stillwater elevation (SWEL) for a higher MRI design event when the 100-year flood stillwater elevation ($SWEL_{100}$) is provided in the FIS or other study. **Equation 1** utilizes a flood scale factor (scaling factor) for MRI termed C_{MRI}. C_{MRI} values are defined in Table 7.

Where the $SWEL_{100}$ is available, ASCE 7-22-S2 states the $SWEL_{100}$ shall be used to determine the $SWEL_{MRI}$ corresponding to the MRI design flood event using Table 7 and **Equation 1**.

$SWEL_{MRI} = C_{MRI} (SWEL_{100} - Z_{datum}) + Z_{datum}$ **Equation 1** [ASCE 7-22-S2, Eq. 5.3-2]

where,

$SWEL_{MRI}$ = stillwater elevation corresponding to the specified risk category and MRI, in ft (m)C_{MRI} = flood scale factor associated with the MRI from Table 7 for different locations when $SWEL_{100}$ is the starting SWEL

$SWEL_{100}$ = stillwater elevation for the 100-year MRI, in ft (m)

Z_{datum} = elevation of mean water level based on local datum, in ft (m). For riverine sites, Z_{datum} shall be taken as the annual high-water level. Z_{datum} shall be permitted to be taken as 0 for coastal sites.

CLARIFICATION
Z_{datum} for Riverine Sites

The Z_{datum} for riverine sites may be taken as the annual high-water level, which is equivalent to the 1-year MRI flood. Often, 1-year MRI flood data are not readily available. Appendix C provides methods to aid in the estimation of the 1-year MRI flood elevation when data are not available.

Table 7. Stillwater Elevation Scaling Factors Starting with $SWEL_{100}$ [ASCE 7-22-S2, Table 5.3-1]

Risk Category	*MRI (years)*	*Annual Exceedance Probability (AEP)*	*C_{MRI} Gulf of Mexico Coastal Sites[(a)] (for $SWEL_{100}$)*	*C_{MRI} All Other Coastal Sites[(a)] (for $SWEL_{100}$)*	*C_{MRI} Great Lakes Sites[(b)] (for $SWEL_{100}$)*	*C_{MRI} Riverine Sites (for $SWEL_{100}$)*
I	100	1.00%	1.00	1.00	1.00	1.00
II	500	0.20%	1.35	1.25	1.15	1.35
III	750	0.13%	1.45	1.35	1.20	1.45
IV	1,000	0.10%	1.50	1.40	1.25	1.50

(a) Gulf Coast site scale factors are for coastlines of Texas, Louisiana, Mississippi, Alabama, and Florida west of 80.75 deg W. All other coastlines shall be taken as Other.
(b) If flood loading is being considered on other lakes, the scale factors for riverine sites shall be used.

CLARIFICATION
Design Stillwater Flood Elevation

ASCE 7-22-S2 uses the term design stillwater flood elevation. ASCE 7-22-S2 defines the design stillwater flood elevation as, "The elevation, relative to the adopted datum, of the stillwater during the design flood, including relative sea level change. See [ASCE 7-22-S2] Figures 5.2-1 and 5.2-2."

Note that the design stillwater flood elevation is different than $SWEL_{MRI}$ in coastal areas since $SWEL_{MRI}$ does not include relative sea level change (RSLC). The design stillwater flood elevation can be calculated by adding the design flood depth (see section 3.2.4) and the eroded ground elevation (see section 3.2.3) or by adding RSLC (Δ_{SLR}, see section 3.2.2) to $SWEL_{MRI}$ in coastal areas. Where the design stillwater flood elevation is used in equations in this guide, it is termed $SWEL_{design}$.

EXCEEDING MINIMUMS

Calculating Required $SWEL_{MRI}$ with $SWEL_{500}$

ASCE 7-22-S2 does not provide scaling factors based on the 500-year design flood. If designing for a 750- or 1000-year MRI and the 500-year stillwater elevation ($SWEL_{500}$) is specified by a flood hazard study, such as a Flood Insurance Study (FIS), then the $SWEL_{500}$ should be used to determine the $SWEL_{MRI}$ corresponding to the MRI design flood event using Table 8 and **Equation 2.** In these instances, the SWELMRI should be calculated with both **Equation 1** and **Equation 2,** and the higher of the two resulting elevations should be selected. Calculating the $SWEL_{MRI}$ with both **Equation 1** and **Equation 2** allows the designer to ensure compliance with ASCE 7-22-S2 Equation 5.3-2 (**Equation 1**) and to obtain a more accurate $SWEL_{MRI}$ estimate through **Equation 2.**

$SWEL_{MRI} = C_{MRI\_500} (SWEL_{500} - Z_{datum}) + Z_{datum}$ **Equation 2** [Based on ASCE 7-22-S2, Eq. 5.3-2]

where,

C_{MRI_500} = flood scale factor associated with the MRI from Table 8 for different locations when $SWEL_{500}$ is the starting SWEL

$SWEL_{500}$ = stillwater elevation for the 500-year MRI, in ft (m)

Table 8. Stillwater Elevation Scaling Factors Starting with $SWEL_{500}$ [Derived from ASCE 7-22-S2, Table 5.3-1]

Risk Category	MRI (years)	Annual Exceedance Probability (AEP)	C_{MRI_500} Gulf of Mexico Coastal Sites[(a)] (for $SWEL_{500}$)	C_{MRI_500} All Other Coastal Sites[(a)] (for $SWEL_{500}$)	C_{MRI_500} Great Lakes Sites[(b)] (for $SWEL_{500}$)	C_{MRI_500} Riverine Sites (for $SWEL_{500}$)
II	500	0.20%	1.00	1.00	1.00	1.00
III	750	0.13%	1.07	1.08	1.04	1.07
IV	1,000	0.10%	1.11	1.12	1.09	1.11

(a) Gulf Coast site scale factors are for coastlines of Texas, Louisiana, Mississippi, Alabama, and Florida west of 80.75 deg W. All other coastlines shall be taken as Other.

(b) If flood loading is being considered on other lakes, the scale factors for riverine sites shall be used.

CLARIFICATION

Using Equation 1 and Equation 2

When using **Equation 1** or **Equation 2**, the MRI subscripts should be replaced with the design MRI. For example, if designing for a 750-year MRI with the 100-year SWEL as the starting point for scaling, **Equation 1** should be modified to read: $SWEL_{750} = C_{750}\ (SWEL_{100} - Z_{datum}) + Z_{datum}$.

3.2.2. ESTIMATING RELATIVE SEA LEVEL CHANGE

Per ASCE 7-22-S2, Section 5.3.4,

> *The effects of relative sea level change shall be included in the calculation of flood conditions and flood loads for sites whose flooding comes from coastal sources. A project lifecycle 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 sea level change rate for the site over a 50-year period. The increase in relative sea level during the project lifecycle of the structure shall be added to the design stillwater flood elevation as required by Section 5.3.3.*

CLARIFICATION

Relative Sea Level Change

Rising sea levels have been well documented at National Oceanic and Atmospheric Administration (NOAA) tide gages in many locations along the U.S. Coastline. If the rate of sea level rise continues, the frequency of coastal flooding will increase. The elevation of today's base flood event (100-year flood or 1% AEP), in those locations, will occur more frequently in the future, and future 100-year flood elevations will be higher than today's level. It is important to think of the amount of sea level change as a time-dependent rate, based on a historic trend or future projection, rather than a specific value. Rates of sea level change are not the same in all locations, so the term relative sea level change (RSLC) is used to indicate the location-specific conditions that contribute to the amount of rise specific to an area. The RSLC accounts for factors such as subsidence or, conversely, glacial rebound, as well as ocean currents, ocean temperatures, and other contributary factors. This section provides the minimum requirements to account for historic sea level change outlined in ASCE 7-22-S2 but also provides Exceeding Minimums recommendations to account for increased rates of sea level change. It is important to consult the most current sources of RSLC data to incorporate the latest science into the design process. Additionally, consider that states, local authorities, grant requirements, owner preferences, or other factors may dictate the use of a specific RSLC trend or projection.

Relative Sea Level Change Approximation Method:

The total RSLC, or as noted in the equations, Δ_{SLR}, over the project lifecycle can be determined by applying **Equation 3**, which utilizes the average annual rate of relative sea level rise. At a minimum, the historic RSLC rate must be used for SLR_A. Consider that future projections often account for a varying rate of sea level change and the projected total sea level change may be a specific value provided for a future year rather than an average rate. In these instances, see the "EXCEEDING MINIMUMS: Future Sea Level Change Projections" textbox for details on determining Δ_{SLR}.

$$\Delta_{SLR} = SLR_A * PL \qquad \textbf{Equation 3}$$

where,

Δ_{SLR} = the total relative sea level change for coastal sites over the project lifecycle, in ft (m); shall not be taken as less than 0.

SLR_A = the average annual rate of RSLC in ft/yr (m/yr). May be either the historic rate, or when available, a selected rate above the historic rate.

PL = the project lifecycle, not less than 50 years.

Refer to ASCE 7-22-S2, Section C5.3.4, for more information how to consider RSLC.

Refer to the Sea Level Change Resources and Exceeding Minimums boxes below for further information regarding the estimation of RSLC.

RESOURCES
Historic Sea Level Change

NOAA's Tides and Currents' site provides historic RSLC trends: https://tidesandcurrents.noaa.gov/sltrends/sltrends.html

NOAA's Seal Level Calculator discussed in the "EXCEEDING MINIMUMS: Future Sea Level Change Projections" text box may also be used to determine historic sea level change. NOAA's Seal Level Calculator provides observed sea level trends, also referred to as historic sea level rise rates.

EXCEEDING MINIMUMS
Future Sea Level Change Projections

Designers should use future RSLC projections for a scenario where future conditions represent an increased rate of change beyond the historic rate of change. When using future RSLC projections, Δ_{SLR} is equal to the total projected RSLC during the project lifecycle.

Considerations associated with the use of future RSLC projections should be discussed with the client. The considerations discussed may include the building owner and operator's tolerance for damage, displacement, and downtime, the cost associated with utilizing future RSLC, the age of the effective flood analysis, and residual risk. Refer to Hurricane Ian in Florida

Recovery Advisory 1, *Designing for Flood Levels Above the Minimum Required Elevation After Hurricane Ian* (FEMA 2023), Section 4.1 for additional information on these considerations.

Future RSLC projections should be obtained from reliable sources. Check to see if a local, regional, or state planning agency has adopted future RSLC projections. If so, consider using the adopted projections. If local, regional, or state planning agencies have not adopted RSLC projections, alternate authoritative sources, such as NOAA's Sea Level Calculator, may be consulted for RSLC projections.

NOAA's Sea Level Calculator (https://coast.noaa.gov/sealevelcalculator/) provides projected sea level change curves for various RSLC data sources and scenarios. NOAA's Interagency Sea Level Rise Scenario Tool (https://sealevel.nasa.gov/task-force-scenario-tool) provides projections for global, regional, and local sea level rise scenarios from 2020 to 2150, relative to a 2000 baseline. The following keywords may be used in search engines to identify other sources: sea level tracker, sea level change calculator.

If the project, owner, or firm does not have a preferred RSLC data source, FEMA recommends that the most current "source" be utilized to enable implementation of the latest science that has been integrated into the projections.

To determine the total RSLC (Δ_{SLR}) predicted at a site:

1. Acquire RSLC data for the location nearest the project site with available RSLC data
2. Document the relative sea level (RSL) at the site for the study date of the flood data (e.g., FIS study date of 2012, what is the RSL is 2012)
3. Document the RSL at the site for the future date associated with the project useful life (e.g., project useful life (project lifecycle) end date using a minimum of 50 years from the year of design)
4. Subtract the RSL for the study date from the RSL at the future date
5. The difference is the total expected RSLC, Δ_{SLR}, at the site

Note: Steps 1–4 also apply when using RSLC projections, which use a date earlier than the study date as the baseline for the projections. The RSLC projected for the study date should be subtracted from the future RSLC for the project useful life end date to obtain the total expected RSLC.

Table 9 is presented to provide examples of how historic RSLC may differ from future RSLC projections.

Table 9. Comparison of Relative Sea Level Change values for a 50-year project lifecycle; using historic rates and future projections

Location	Relative sea level trend, historic RSLC Rate (mm/yr)[a] [ft/yr]	Total RSLC over 50 years using historic RSLC rates, Δ_{SLR} (ft)	Total RSLC over 50 years using future projections[b],[c], Δ_{SLR} (ft)
Naples, FL	3.21 [0.011] (from 1965 to 2022)	0.5	2.9
Galveston, TX Pier 21	6.63 [0.022] (from 1947 to 2022)	1.1	3.5
Duck, NC	4.74 [0.016] (from 1978 to 2022)	0.8	3.1
The Battery, NY	2.9 [0.01] (from 1856 to 2022)	0.5	3.0

[a] Data from NOAA's Tides & Currents Relative Sea Level Trends site, accessed on May 25, 2023. (https://tidesandcurrents.noaa.gov/sltrends/sltrends.html)
[b] Data from USACE's Sea Level Tracker, accessed on May 25, 2023. (https://climate.sec.usace.army.mil/slr_app/)
[c] Utilized NOAA et al. 2022 Scenario 1.5 (Intermediate-High) with a 0.83 non-exceedance probability

3.2.3. ESTIMATING EROSION

Per ASCE 7-22-S2, Section 5.3.5,

> *The effects of erosion shall be included in the calculation of flood conditions and flood loads. The effects of erosion need not exceed the depth of non-erodible strata.*

CLARIFICATION
Erosion

The two primary types of erosion are erosion associated with long-term shoreline retreat in coastal areas or long-term riverbank movement in riverine areas, and sudden erosion caused by storms or floods (episodic erosion). The effects of erosion on flood conditions and flood loads are accounted for by increasing the design stillwater flood depth (d_f) based on an eroded ground calculation.

CLARIFICATION
Erosion vs Scour

Erosion is a lowering of the ground surface over a large area, usually brought on by a coastal storm or long-term shoreline recession. Erosion increases the unbraced length of vertical foundation elements, increases the stillwater depth at the building, and in some instances, allows larger waves to reach the foundation.

Scour is a localized loss of soil immediately around an object or obstruction. Scour also increases the unbraced length of vertical foundation elements but does not act to increase the stillwater flood depth across which waves propagate. Thus, scour can be ignored for wave height and flood load calculation purposes. Walls, columns, pilings, pile caps, footings, slabs, and other objects found under a coastal building can contribute to localized scour. See Figure 4 for depictions of erosion and scour.

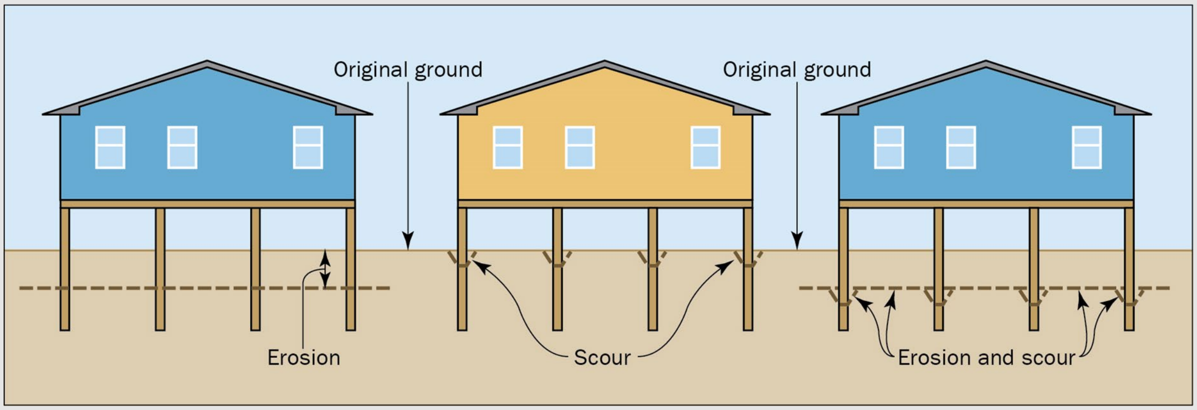


Figure 4. Depiction of erosion, scour, and combined erosion and scour

Depending on the building location, soil characteristics, and flood conditions, a building may be subject to either coastal erosion or scour, or both. See Chapter 7 of this design guide for further information on scour.

ADDITIONAL CONSIDERATIONS

Coastal Erosion

In coastal areas, erosion associated with shoreline retreat refers to the inland movement of the shoreline due to various processes. These processes include sea level rise, subsidence (or conversely uplift), longshore drift (transport of beach sediment), and the interruption of littoral transport. Whereas coastal storm-induced erosion mainly focuses on storm erosion effects on dunes and bluffs. Coastal flood maps may include storm-driven dune erosion but do not include long-term erosion.

See Appendix A for applicable coastal erosion methodologies.

ADDITIONAL CONSIDERATIONS

Riverine Erosion

In riverine areas, erosion is a complex process that includes many factors, such as fluvial hydraulics, soil stability, sediment transport, and watershed characteristics, including hydrology and sediment yield, past and future land use, and vegetation, among others. FEMA riverine flood maps do not include storm- or event-driven erosion, nor long-term erosion. The Technical

Mapping Advisory Council (TMAC), a federal advisory committee, developed some guidance on addressing projected riverine erosion with the incorporation of future conditions. Sections 4.5, 5.3, and 5.4.2 in *TMAC Future Conditions Risk Assessment and Modeling* (TMAC 2015) provide some insights for projecting future erosion. While the report was primarily developed to recommend changes to FIRMs, the discussion could be helpful to decide whether to conduct a site assessment of a potential building site or a more formal hydrologic and hydraulic (H&H) study to understand the impacts of future conditions on a building site.

If the site has a known long-term erosion rate, the method described for shoreline retreat in coastal areas in Appendix A may be applied to estimate the new riverbank position caused by erosion.

3.2.4. CALCULATING DESIGN STILLWATER FLOOD DEPTH (D_F)

Per ASCE 7-22-S2, Section 5.3.3, design stillwater flood depth (d_f) shall be calculated per Equation 5.3-1 [**Equation 4**].

$$d_f = (SWEL_{MRI} - G_e) + \Delta_{SLR}$$ **Equation 4** [ASCE 7-22-S2, Eq. 5.3-1]

where,

d_f = design stillwater flood depth, in ft (m)

G_e = elevation of grade at the building or other structure inclusive of effects of erosion in ft (m). See Section 3.2.3 of this design guide for guidance.

EXCEEDING MINIMUMS

Elevation Height

Building elevation and protection heights are currently governed by local requirements, the NFIP, and state and local building codes that reference ASCE 24-14. The NFIP regulates to the Base Flood Elevation (BFE), whereas many communities and ASCE 24-14 use a combination of freeboard (BFE + 1 or 2 feet) requirements, the local Design Flood Elevation (DFE), and in some instances, the 500-year (0.2% annual chance) flood elevation to determine the minimum flood elevation. The DFE can either be determined by a locally adopted freeboard value or defined by locally adopted maps that supersede or supplement the FIRM. Projects supported with federal grant funds may have additional elevation requirements.

ASCE 7-22-S2 does not dictate the minimum building elevation, but only the flood depth used to calculate flood loads. In most instances, ASCE 7-22-S2 does not determine flood loads based on the current BFE or DFE. For coastal structures, it uses future conditions such as RSLC and, for all structures, it includes erosion as well as varying MRIs based on the building's Risk Category to determine a design flood depth. Given the differences in methods between the determination of the minimum building elevation or protection height and the determination of a design flood depth to calculate flood loads, the two approaches may not align. Because of the different methods, it is possible to elevate a building to the regulatory minimum required elevation while also being required to calculated flood loads for a flood depth or wave crest height that equates to an elevation above the minimum required flood elevation (usually the lowest floor elevation or bottom of the lowest horizontal member elevation), thus using ASCE 7-22-S2, it is possible for the design flood to impart loads on the elevated structure above the lowest floor (or above the bottom of the lowest horizontal member). While not required, FEMA recommends that designers and owners use the higher of the two elevations for both the building elevation (or protection height) and for determining the design flood depth or wave crest height for the calculation of flood loads.

FEMA recommends using the AHJ's elevation requirement for flood characteristics to determine loads when the elevation exceeds the ASCE 7-22-S2 calculated flood depth/wave crest elevation because the AHJ's elevation requirement may include added elevation for future conditions. These future conditions could include factors such as increased runoff due to land development or increased precipitation, which cause an increase in flood depth that may not be captured in the ASCE 7-22-S2 calculation. However, in some cases, AHJs may adopt such large freeboard amounts that using the required elevation to determine the design flood depth for the calculation of flood loads may be impractical. When large freeboard amounts are adopted by the AHJ, the designer should consult with the local floodplain administrator to understand how the freeboard value was determined. This determination may dictate how much additional height above the ASCE 7-22-S2 minimum should be accounted for in the DFE.

Selecting the higher of the two elevations as the minimum elevation will reduce risk and will also aid in reducing some flood loads. Many uplift loads due to wave action may be eliminated with sufficiently elevated structures; see Section 6.3.2 of this design guide for further information.

Although ASCE 24-14 uses the term DFE to refer to elevation requirements dictated by the community or AHJ, this design guide uses DFE to refer to the minimum required elevation or flood protection elevation.

Calculating Design Flood Elevation Utilizing Freeboard or Local Adoption and ASCE 7-22-S2 Methods

Design Flood Elevation (DFE): The elevation of the highest flood that a structure is designed to protect against, which could be the BFE plus freeboard, the 500-year flood elevation based on the ASCE 24-14 Flood Design Class, or the minimum elevation requirement adopted in local floodplain ordinance. Also referred to as Flood Protection Elevation. Note: The DFE is not used to determine the design stillwater flood depth defined by ASCE 7-22-S2.

MRI based Design Flood Elevation (DFE_{MRI}): The DFE, as described above, when incorporating the ASCE 7-22-S2 flood depth and wave height determination methods for use as the minimum elevation for floor heights, bottom of lowest horizontal structural member, or flood protection.

Base Flood Elevation (BFE): The elevation of surface water resulting from a flood that has a 1% chance of being equaled or exceeded in any given year. In situations where waves are present, this value is a wave crest elevation.

Freeboard: An additional amount of height above the BFE (e.g., 2 feet above the BFE) used as a factor of safety in determining the level at which a structure's lowest floor must be elevated or floodproofed to be in accordance with state or community floodplain management regulations. Freeboard is not required by NFIP standards, although FEMA encourages communities to adopt at least a 1-foot freeboard. In riverine areas, 1 foot of freeboard would only account for the 1-foot rise built into the concept of a delineated floodway if the remainder of the floodplain is filled up to the extent of the floodway. In coastal areas, the BFE is represented as a value rounded to the nearest whole foot. This means that the modeled BFE could range from either 6 inches less than the stated BFE or almost 6 inches above the stated BFE. One foot of freeboard helps to account for the range in possible flood elevations as the BFE is a rounded value.

Additionally, users should consider that the BFE represents the median 1% annual-chance flood event (this is the case for both riverine and coastal BFEs). This means that it is possible that a 1% flood event could be higher (or lower) than the BFE provided in the flood data, so a selected freeboard should also ideally account for the uncertainty in the flood models. Other considerations for freeboard could include other factors that influence flood elevations, such as future changes in precipitation rates or future increased development that could influence runoff rates into the floodplain.

Method 1: DFE Calculation Based on Freeboard:

DFE = BFE + Freeboard

where,

DFE = Design Flood Elevation in ft (m)

In some instances, use of the 500-year flood elevation is required when it is higher than BFE + freeboard. In these cases, the 500-year flood elevation should be used for the DFE. Additionally, some AHJ's may simply provide a regulatory DFE; the AHJ's regulatory DFE should be used for the DFE when it exceeds all other regulatory elevation requirements, such as those required by the NFIP and locally adopted codes.

Method 2: DFE_{MRI} Calculation Based on ASCE 7-22-S2 Calculations:

Per ASCE 7-22-S2, Section C5.3.3,

> *In ASCE 7-22 Supplement 2, loads in Chapter 5 are based on the stillwater elevation. In prior editions, flood loads also were based on stillwater elevation, but the chapter referenced a DFE in some load calculations. ASCE 7-22 Supplement 2 drops the reference to the DFE.*
>
> *If needed for comparison purposes, the ASCE 7-22 Supplement 2 coastal DFE can be determined in accordance with Equation (C5.3-1).*

...The ASCE 7-22 Supplement 2 riverine DFE is the same as the Design Stillwater Flood Elevation.

$DFE_{MRI} = d_f + G_e + 0.7H_{design}$ **Equation 5** [ASCE 7-22-S2, Eq. C5.3-1]

where,
DFE_{MRI} = MRI-based Design Flood Elevation in ft (m)

d_f = design stillwater flood depth, in ft (m)

G_e = elevation of grade at the building or other structure inclusive of effects of erosion in ft (m)

H_{design} = design wave height, ft (m), as calculated in ASCE 7-22-S2, Section 5.3.7.1, and Chapter 6 of this design guide. For a riverine site, H_{design} may be taken as 0 for the DFE_{MRI} calculation.

Utilizing ASCE 7-22-S2 When the Higher Method 1 Results in the Higher DFE

FEMA recommends that the user calculate DFE and DFE_{MRI} per the previously described Method 1 and Method 2 and use the higher of the two for design. If Method 1 results in the higher DFE, the following method may be used to determine the flood characteristics required for ASCE 7-22-S2 calculations. If DFE_{MRI} (Method 2) is equal to or higher than DFE (Method 1), document the calculation to demonstrate compliance with the requirements (freeboard, local adoption, etc.) and proceed with the calculation methods herein (skip "If DFE is higher than DFE_{MRI}").

If DFE is higher than DFE_{MRI}:

This method is provided for situations in which the DFE (Method 1) exceeds the DFE_{MRI} (Method 2) and the DFE will be used to determine the flood characteristics required for ASCE 7-22-S2 calculations. This method requires a design stillwater flood depth to be calculated for use with ASCE 7-22-S2 load calculations.

Calculate the design stillwater flood depth (d_f) as shown in **Equation 6**. The stillwater flood depth (d_f) is calculated by effectively increasing the stillwater elevation while the wave height remains the same. The calculation method for the new d_f enables the incorporation of the increased freeboard associated with the DFE without impacting the wave height. The design wave height, H_{design}, may remain the same and may equal the H_{design} used in calculating DFE_{MRI}. H_{design} is limited to prevent overburdening the loads when large freeboard values are used.

$d_f = DFE - G_e - 0.7H_{design}$ **Equation 6**

4. Hydrostatic Flood Loads

4.1. Hydrostatic Loads

Per ASCE 7-22-S2, Section 5.4.2,

> *Hydrostatic loads caused by a depth of water to the design stillwater flood elevation shall be applied over all surfaces contacted, both above and below ground level.*
>
> *…Reduced hydrostatic uplift and lateral loads on surfaces of enclosed spaces below the design stillwater flood elevation shall apply only if provision is made for entry and exit of floodwater.*
>
> *Hydrostatic loads below grade shall be calculated assuming the soils are fully saturated and in accordance with Sections 5.4.2.1 and 5.4.2.2 unless the degree of soil saturation and below grade porewater pressures during a flood event are determined in accordance with Section 5.4.2.3.*

See Figure 5 for a depiction of the various hydrostatic load conditions discussed in ASCE 7-22-S2, Section 5.4.2.

CLARIFICATION

Flood Openings

Enclosures that incorporate flood openings for automatic entry and exit of floodwater will have reduced hydrostatic uplift and lateral loads on surfaces of enclosed spaces below the design stillwater flood elevation due to equalization of pressures. NFIP Technical Bulletin 1 (TB 1), *Requirements for Flood Openings in Foundation Walls and Walls of Enclosures Below Elevated Buildings in Special Flood Hazard Areas In Accordance with the National Flood Insurance Program* (FEMA 2020b), provides guidance on the NFIP regulations concerning the requirement for openings in below-BFE foundation walls and walls of enclosures for buildings located in Zones A, AE, A1–A30, AR, AO, and AH. Flood openings are also discussed in ASCE 24-14, Section 2.7.2.2.

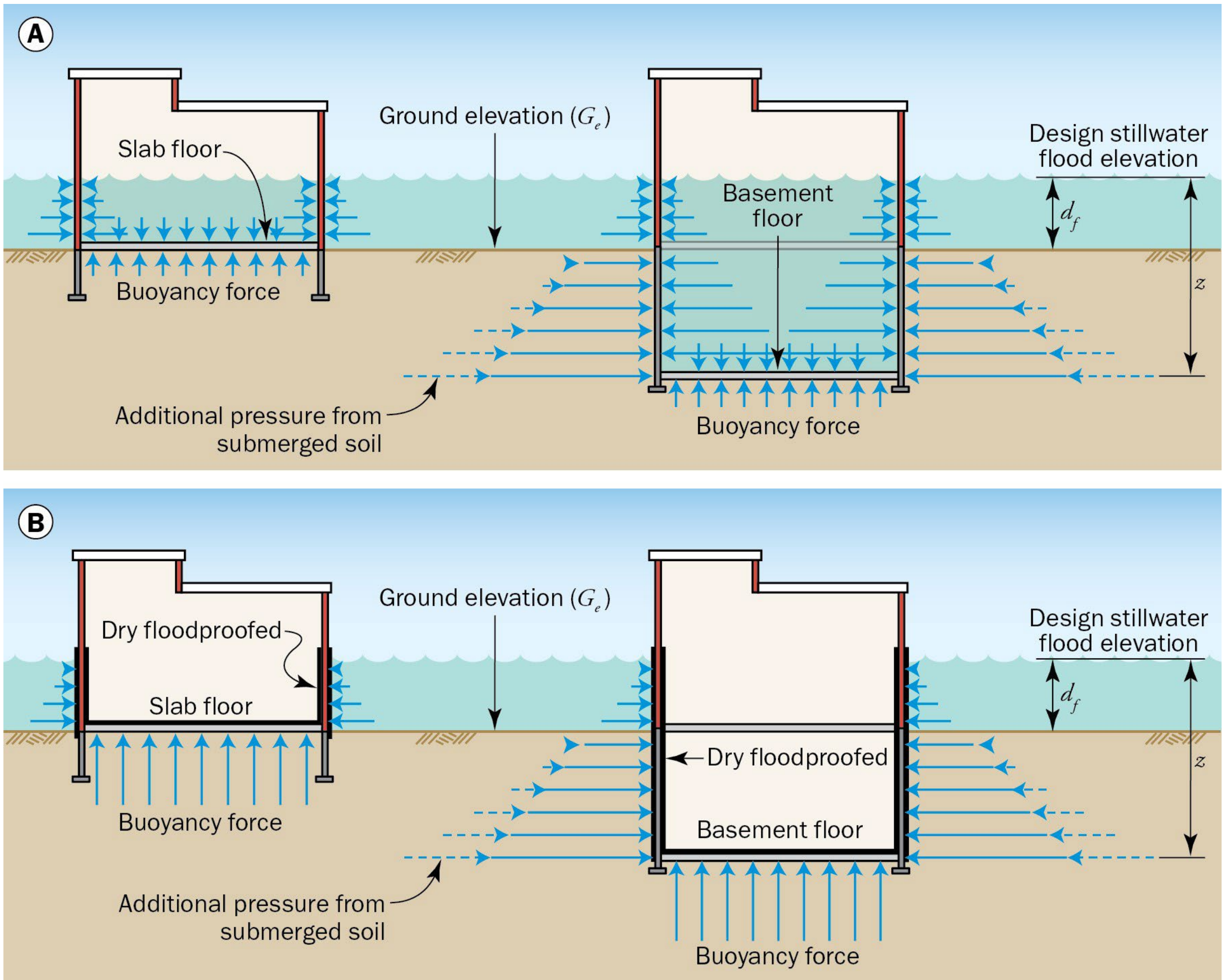


Figure 5. Equal (A) and unequal (B) hydrostatic pressures applied to the exterior elements of a building; (B) shows the building protected by dry floodproofing methods applied

4.1.1. HYDROSTATIC PRESSURES

Per ASCE 7-22-S2, Section 5.4.2,

> *Hydrostatic forces shall be calculated in accordance with Sections 5.4.2.1 and 5.4.2.2. The hydrostatic pressures shall be calculated utilizing basic fluid mechanics by applying pressures perpendicular to wetted surfaces proportional to the depth of water such that*

$p_h = \gamma_w z$ **Equation 7** [ASCE 7-22-S2, Eq. 5.4-1]

where,

p_h = hydrostatic pressure at a given depth z, in lb/ft^2 (kN/m^2)

γ_w = specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater

z= depth below design stillwater flood elevation, in ft (m), [see Figure 5]

Vertical Hydrostatic Force (Buoyancy)

Per ASCE 7-22-S2, Section 5.4.2.1,

> *The vertical uplift force caused by buoyancy for determination of structure uplift shall be applied at the centroid of the submerged volume of the structure and shall be calculated using Equation (5.4-2) [***Equation 8***].*

$$F_B = \gamma_w V_w$$ **Equation 8** [ASCE 7-22-S2, Eq. 5.4-2]

where,

F_B = uplift force caused by buoyancy, in lb (kN)
V_w = volume of displaced water, in ft^3 (m$_3$)

Lateral Hydrostatic Force

Per ASCE 7-22-S2, Section 5.4.2.2,

> *The lateral force, F_h, caused by the hydrostatic pressure on one side of a vertical wall per unit width, lb/ft (kN/m) shall be calculated by Equation (5.4-3) [***Equation 9***].*

$$F_h = (1/2)\ \gamma_w\ d_f^2$$ **Equation 9** [ASCE 7-22-S2, Eq. 5.4-3]

where,

F_h = lateral force caused by hydrostatic pressure, in lb/ft (kN/m)
d_f = design stillwater flood depth, in ft (m)

ADDITIONAL CONSIDERATIONS

Lateral Hydrostatic Force with Subgrade Hydrostatic Forces

Per ASCE 7-22-S2, Section 5.4.2,

> *Hydrostatic loads below grade shall be calculated assuming the soils are fully saturated and in accordance with Sections 5.4.2.1 and 5.4.2.2 unless the degree of soil saturation and below grade porewater pressures during a flood event are determined in accordance with Section 5.4.2.3.*

When subgrade hydrostatic forces are present, **Equation 10** can be utilized to calculate the lateral hydrostatic force on one side of a vertical wall per unit width, lb/ft (kN/m).

$$F_{hs} = (1/2)\ \gamma_w\ (z)^2$$ **Equation 10**

where,

F_{hs} = lateral force caused by hydrostatic pressure including subgrade forces, in lb/ft (kN/m)

ASCE 7-22-S2, Sections 5.4.2.3 and C5.4.2.3 discuss seepage and acceptable methods for reducing below-grade hydrostatic force and pressure calculations.

Equation 10 is derived from ASCE 7-22-S2, Eq. 5.4-3 (**Equation 9** in this design guide) and replaces d_f with z to enable hydrostatic calculations that include subgrade forces.

Submerged and Fully Saturated Soil Forces

ADDITIONAL CONSIDERATIONS

Submerged and Fully Saturated Soil Forces

This entire section, Submerged and Fully Saturated Soil Forces, addresses additional considerations that are not detailed in ASCE 7-22-S2. The entire section is not captured in a text box for readability purposes, but all of the content should be treated as additional considerations.

If any portion of the structure is below grade, submerged or saturated soil forces must be considered in addition to the hydrostatic forces above grade. However, if a structural member, such as a pile, is surrounded by soil on all sides, the lateral soil loads will cancel out. **Equation 9** and **Equation 10** do not include lateral soil forces. Lateral soil loads should be calculated in addition to lateral hydrostatic loads as depicted in Figure 5. Lateral soil loads are addressed in Chapter 3 of ASCE 7-22 and herein.

During conditions with above-grade flooding, the common conservative assumption is to assume that all soils below the flood level are submerged soils. ASCE 7-22-S2 requires the user to assume the soils are fully saturated "unless the degree of soil saturation and below grade porewater pressures during a flood event are determined in accordance with Section 5.4.2.3 [in ASCE 7-22-S2]." Submerged soils and fully saturated soils are synonymous when soils are below the water table. Soils can be considered as being below the water table during above-grade flood conditions when it is assumed that soils are submerged. Given these assumptions, this section will use the term "submerged soil" to refer to both submerged soils and fully saturated soils under flood conditions. Submerged/soils contain pores that are filled with water and no air is present. When submerged soils are present, sub-grade hydrostatic forces, which are influenced by above-grade water (if present), are included in addition to soil forces.

For soil unit weight calculation purposes, the total unit weight of fully saturated soil (γ_{sat}) and the effective unit weight of buoyant/submerged soil (γ_b) are different. The effective unit weight of buoyant soil (γ_b) is the total unit weight of fully saturated soil (γ_{sat}) minus the unit weight of water (γ_w) due to buoyancy. The effective unit weight of buoyant soil (γ_b) is used to calculate the differential soil/water force. The total unit weight of fully saturated soil (γ_{sat}) for various soil types are presented in Table 10 and Table 11 (FEMA 2012). The total unit weights provided in Table 10 are for fully saturated soils (γ_{sat}) inundated with freshwater (equivalent fluid weight of submerged soil and water). The values in Table 10 represent minimum (loose soil) and maximum (dense soil) weights within the specified range. In the absence of geotechnical site data, the maximum values should be used to obtain a conservative estimate.

Table 10. Total Unit Weight of Fully Saturated Soil

Soil Type[a]	*γ_{sat}, Total Unit Weight of Fully Saturated Soil (Submerged Soil and Water Weight) (lb/ft^3)*[b][c][d]
Sand and gravel (GW, GP, GM, GM-GP, SW, SP, SM, SM-SP)	119–154
Mixed soils - silts and clays, silty fine sands, clayey sands, and gravels (CL, ML, CH, MH, SM, SC, GC)	124–156
Clay and organic soils (CL, ML, OL, CH, OH)	93–133

(a) Soil types are based on USDA Unified Soil Classification System; see Table 11 for soil type definitions.
(b) γ_{sat} values derived by adding 62 lb/ft^3 to the submerged weight from Table 6 in Soil Mechanics DESIGN MANUAL 7.01 by Naval Facilities Engineering Command (NAVFAC) (NAVFAC 1986).
(c) Values represent minimum (loose soil) and maximum (dense soil) weights within the specified range.
(d) Add 1.6 lb/ft^3 to the equivalent fluid weight of submerged soil and water if flooding is due to saltwater.

Table 11. Soil Type Definitions Based on USDA Unified Soil Classification System [FEMA P-259 Table 4-4]

Soil Type	*Group Symbol*	*Description*
Gravels	GW	Well-graded gravels and gravel mixtures
	GP	Poorly graded gravel-sand-silt mixtures
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
Sands	SW	Well-graded sands and gravelly sands
	SP	Poorly graded sands and gravelly sands
	SM	Silty sands, poorly graded sand-silt-mixtures
	SC	Clayey sands, poorly graded sand-clay mixtures
Fine Grain Silt and Clays	ML	Inorganic silts and clayey silts
	CL	Inorganic clays of low to medium plasticity
	OL	Organic silts and organic silty clays of low plasticity
	MH	Inorganic silts, micaceous or fine sands or silts, elastic silts
	CH	Inorganic clays of high plasticity, fine clays
	OH	Organic clays of medium to high plasticity

Source: FEMA 2012

When a structure is subject to hydrostatic forces from both submerged soil and standing water (illustrated in Figure 6), the resultant combined hydrostatic lateral force is the sum of the lateral water hydrostatic force, F_{hs}, and the differential between the water and soil pressures, f_{dif}. The basic equation for computing f_{dif} is shown in **Equation 11** The conditions illustrated in Figure 6 may vary based on site-specific conditions such as a sloping grade.

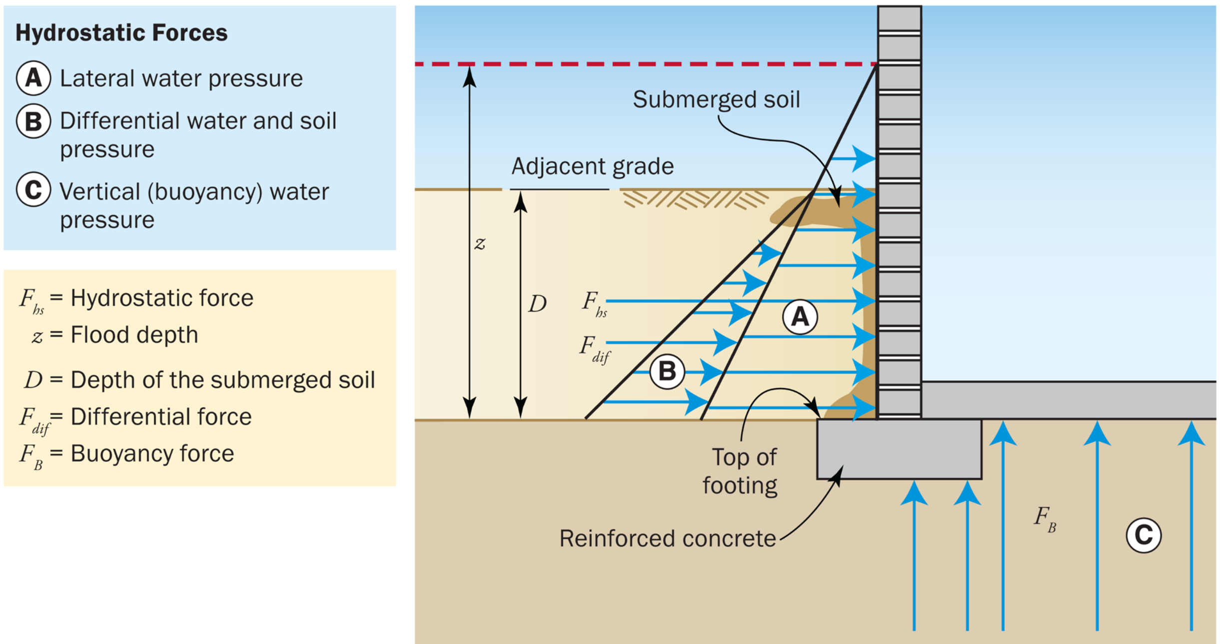


Figure 6. Combination soil/water hydrostatic and buoyancy forces for submerged soils

Equation 11 [FEMA P-936 Eq. 2-3]

$$f_{dif} = \frac{1}{2}(\gamma_{sat} - \gamma_w)D^2$$

where,

f_{dif} = differential soil/water force acting at a distance D/3 from the point under consideration (lb/lf)

D = depth of submerged soil from adjacent grade to the top of the footer (ft)

γ_{sat} = total unit weight of fully saturated soil (equivalent fluid weight of submerged soil and water) (lb/ft^3) as shown in Table 10

Refer to FEMA P-936, *Floodproofing Non-Residential Buildings* (FEMA 2013), Section 2.2.4 for further discussion on submerged soil loads.

ADDITIONAL CONSIDERATIONS

Geotechnical Engineer Consult

Geotechnical engineers can provide site-specific soil information for parameters such as soil types and soil unit weights, as well as soil permeability. Information received from a geotechnical engineer will enable more accurate calculations. Also, contacting a geotechnical engineer may be simpler than determining the soil design conditions based on the guidance herein.

5. Hydrodynamic Flood Loads

5.1. Determining Design Flood Velocity (V)

Design flood velocity (V) is required to calculate various types of hydrodynamic loads. ASCE 7-22-S2, Section 5.3.6, provides guidance for determining design flood velocity.

CLARIFICATION
Determination of Flow Direction

Flow direction refers to the direction of moving floodwaters during a design event. Although not addressed in ASCE 7-22-S2, Section 5.3.6, or the associated commentary, the commentary related to debris impact does provide some insight on the intent of the ASCE 7-22-S2 with respect to how flow direction should be addressed. For buildings subject to coastal flooding, ASCE 7-22-S2, Section C5.4.5.1.2, *Elastic Debris Impact Loads*, indicates, "*However, unless a site-specific study is performed to establish the flow direction, flow is required to be considered in all directions for coastal sites.*" This suggests that without a site-specific study, floodwaters must be assumed to be striking the building from any side of the building. For buildings subject to riverine flooding, ASCE 7-22-S2, Section C5.3.9.1, *Debris Impact*, indicates, "*The second exception occurs for riverine conditions where the flow direction is generally known, the debris strikes can be assumed to occur from the upstream direction with a +/-22.5 degree arc to account for some variation in flow.*" This suggests that moving floodwaters will primarily strike the building on the upstream side of the building.

5.1.1. VELOCITY IN COASTAL AREAS

Per ASCE 7-22-S2, Section 5.3.6.1,

> *For coastal areas, the velocity of water V in the absence of neighboring structures shall be obtained by one of the following three methods: (1) by using Equation (5.3-4), (2) by numerical modeling, or (3) by laboratory testing (physical modeling). When Method 2 or 3 are used, design flood parameters shall be determined using site-specific studies in accordance with Section 5.3.11.*

CLARIFICATION
Influence of Surrounding Structures on Design Flood Velocity

Per ASCE 7-22-S2, Section C5.3.6,

> *The magnitude and direction of the flow can be influenced by the presence of large buildings, such as enclosed structures of concrete, masonry, or structural steel construction located in close proximity to the site, that can accelerate the flow between buildings. Some buildings or other obstacles such as coastal forests can provide shielding*

to reduce the flow speed. The magnitude and direction of the flow can be affected by local changes in topography and changes in surface roughness such as pavement and vegetation.

Per ASCE 7-22-S2, Section C5.3.11,

Site-specific studies may account for the effects of local topography and development/land use on water surface elevations and flow fields, and in coastal areas, wave fields. Site-specific studies can reveal local variations in design flood conditions and erosion, including those resulting from channeling and sheltering by buildings and other structures.

Based on ASCE 7-22-S2 commentary, in some instances, the design flood velocity may be reduced by surrounding structures that restrict the flow of floodwaters reaching the structure or site being evaluated. In other instances, the design flood velocity may increase due to a channeling effect created by buildings seaward of the site. Methods 2 and 3 described in ASCE 7-22-S2, Section 5.3.6.1, may aid in evaluating the decrease or increase in the design flood velocity due to the nature of the built environment. Note that ASCE 7-22-S2, Section 5.3.11, Site Specific Studies, provides a list of maximum allowable reductions for site-specific studies, users should also reference ASCE 7-22-S2, Section C5.3.11, for additional information on site-specific studies.

The design flood velocity (V) may be calculated with **Equation 12**.

$V = C_V * \sqrt{(g * d_f)}$ **Equation 12** [ASCE7-22-S2, Eq. 5.3-4]

where,

V = Design flood velocity, in ft/s (m/s),
g = Acceleration due to gravity, taken as 32.2 ft/s^2 (9.81 m/s^2)
d_f = Design stillwater flood depth, in ft (m), and
C_V = Velocity coefficient, taken as 0.5.

Per ASCE 7-22-S2, Section 5.3.6.1,

*The maximum velocity of water, V_{max}, for coastal areas need not be greater than C_{VMAX} *10 ft/s (C_{VMAX} * 3.05 m/s) where C_{VMAX} is the coefficient obtained from Table 5.3-2 [*Table 13*] used to scale to the maximum velocity.*

Table 12 outlines the process to determine whether the design flood velocity, V, should equal the velocity calculated by **Equation 12** or if it should be set equal to V_{max}. Table 13 lists the velocity coefficients for maximum velocity and the calculated V_{max} for coastal sites by Risk Category.

Table 12. Process for Determining the Design Velocity (V) in Coastal Areas

Step	*Objective*	*Method*
Step 1	Calculate design velocity (V)	$V=C_V*\sqrt{(g*d_f)}$ **Equation 12**
Step 2	Calculate V_{MAX}	$V_{MAX} = C_{VMAX}$ *10 ft/s, given in Table 13 ($V_{MAX} = C_{VMAX}$ *3.05 m/s) Note: *C_{VMAX} defined by Risk Category in* Table 13
Step 3	Check for V_{MAX}	If $V \leq V_{MAX}$, then, V=V, V calculated in Step 1 is design velocity (V) If $V > V_{MAX}$, then, $V=V_{MAX}$, set design velocity (V) equal to V_{MAX}

Table 13. Scaling Factors for Maximum Velocity and Maximum Velocity [Derived from ASCE 7-22S Table 5.3-2]

	All Coastal Sites	
Risk Category	*C_{VMAX}*	*V_{max}*
I	1.00	10 ft/s (3.05 m/s)
II	1.35	13.5 ft/s (4.12 m/s)
III	1.45	14.5 ft/s (4.42 m/s)
IV	1.50	15 ft/s (4.58 m/s)

5.1.2. VELOCITY IN RIVERINE AREAS

Per ASCE 7-22-S2, Section 5.3.6.2,

> *For riverine areas, the average velocity of water V shall be obtained by one of the following four methods: (1) from a flood hazard study, (2) by analytical methods using open channel flow hydraulics, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Methods 2, 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11.*

ADDITIONAL CONSIDERATIONS

Riverine Velocity

ASCE 7-22-S2 does not provide guidance on scaling the design flood velocity to the needed MRI in riverine areas. Appendix D outlines two approaches to determine the design flood velocity (V) of water for the needed MRI when the velocity is not known for the required MRI.

ADDITIONAL CONSIDERATIONS

Designing for Floodway Locations

When a structure is located in a delineated floodway, the required H&H study for the "no rise" analysis may provide the necessary velocity values. The "no rise" analysis requires calculations for the 1% annual-chance event. It is important to stipulate during scoping of the H&H study what design flood MRI event is required so that the study can also provide the required elevation ($SWEL_{MRI}$) and velocity information.

5.2. Determining Drag Coefficient (C_d), Average Clear Spacing (s), and Debris Damming Closure Ratio (C_{cx})

Drag Coefficient (C_d), Average Clear Spacing (s), and Debris Damming Closure Ratio (C_{cx}) are all variables that are required to calculate various types of hydrodynamic loads.

5.2.1. DEBRIS DAMMING CLOSURE RATIO (C_{CX}) AND AVERAGE CLEAR SPACING (S)

Refer to ASCE 7-22-S2, Sections 5.3.9.2 and C5.3.9.2 and Figure 5.3-1 for guidance.

Per ASCE 7-22-S2, Section 5.3.9,

> *Risk Categories II, III, and IV structures shall be designed for debris impact and debris damming in accordance with this section where the Design Stillwater Flood Depth (d_f) is greater than 3 ft (0.91 m).*

Debris damming need not be considered if the site has a design stillwater flood depth less than 3 feet (0.91 meters) or if the structure being designed is a Risk Category I structure. If debris damming is not required for consideration, set C_{CX} in the drag equations equal to 0.

ADDITIONAL CONSIDERATIONS

Debris Damming for Lateral Force Resisting System Drag Force Calculations for Open Buildings or Buildings With Breakaway Walls

ASCE 7-22-S2, Section 5.4.3.2, states:

> *For open buildings or buildings with breakaway walls as defined by Section 5.3.10…The effects of debris damming shall be considered on the side of the structure exposed to the flow direction, considering damming on any two adjacent bays or a minimum of a 50 ft (15.2 m) width, whichever produces the largest base shear.*

Although not stated in ASCE 7-22-S2, assuming the effects of debris damming need not exceed the total length of the building is reasonable, and the previous statement may be interpreted to include the following bracketed and underlined caveat:

The effects of debris damming shall be considered on the side of the structure exposed to the flow direction, considering damming on any two adjacent bays or a minimum of a 50 ft (15.2 m) width [<u>or the width of the structure if the side of the structure exposed to the flow direction is less than 50 ft wide</u>], whichever produces the largest base shear.

5.2.2. DRAG COEFFICIENT (C_D)

Refer to ASCE 7-22-S2, Section 5.4-1 and C5.4-1 and Tables 5.4-1 and 5.4-2, for guidance.

5.3. Hydrodynamic Loads (F_{drag})

Hydrodynamic loads occur when water flows around a building (or a structural element or other object). Hydrodynamic loads are functions of flow velocity and structural geometry. Hydrodynamic loads are represented by drag forces in ASCE-7-22-S2.

CLARIFICATION

Hydrodynamic Loads

As shown in Figure 7, water flowing around a building (or a structural element or other object) imposes loads on the building. Open foundation systems, unlike that shown in Figure 7, can greatly reduce hydrodynamic loading. Hydrodynamic loads, which are a function of flow velocity and structural geometry, include frontal impact on the side perpendicular (normal) to flow (upstream side), drag along the sides, and suction on the downstream side (backside) (FEMA 2011). The hydrodynamic loads (drag forces) presented in ASCE 7-22-S2 represent frontal impact forces and do not address drag along the sides or suction on the downstream side.

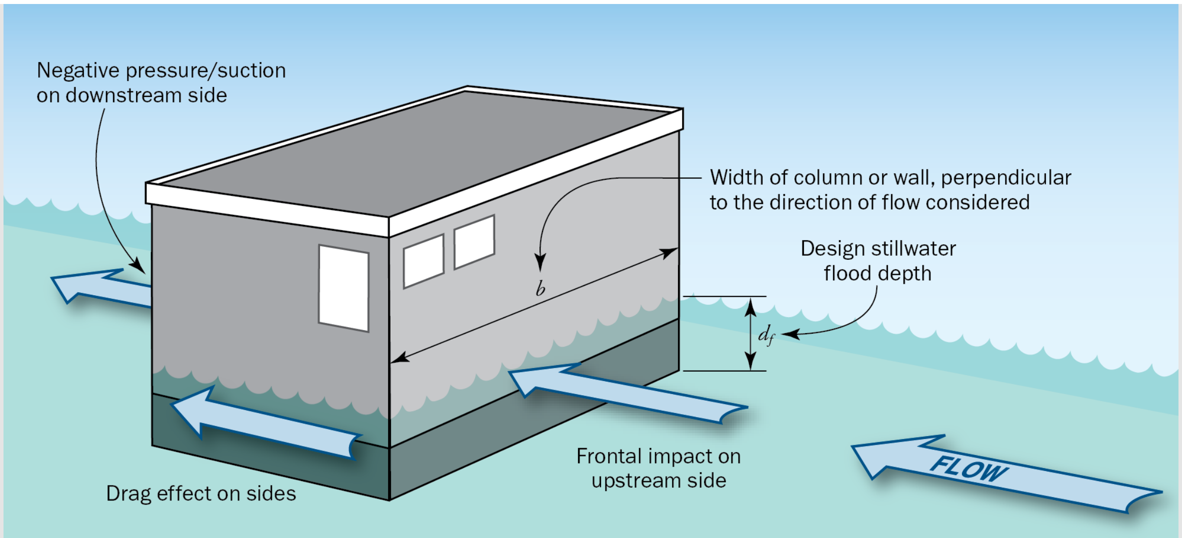


Figure 7. Hydrodynamic loads on a building

5.3.1. DRAG FORCE (F_{DRAG}) ON COMPONENTS (WALLS, COLUMNS, ETC.)

Refer to ASCE 7-22-S2, Section 5.4.3.1 and C5.4.3.1 and Table 5.4-1, for guidance.

Highlights:

- This section provides guidance for applying drag forces on components, and the forces shall be applied to individual components of the structure (i.e., walls and columns).
- Table 5.4-1 contains Drag Coefficients (C_d) for structural components, including those with debris damming.

CLARIFICATION

Section 5.4.3.1 Drag Force on Components

ASCE 7-22-S2, Section 5.4.3.1 states:

> *For geometries not provided in Table 5.4-1 and for structures with openings, the dynamic effects of moving water shall be determined by a detailed analysis utilizing concepts of fluid mechanics.*

Clarifications:

- "Structures with openings" refers to walls with large openings or open foundations, or combinations of the two.
- Table 5.4-1 and equation 5.4-4 may be utilized to develop component drag forces as detailed in Section 5.4.3.1 for structures with openings.
- "Structures with openings" is in reference to completing Section 5.4.3.2 Drag Force (F_{drag}) on Lateral Force Resisting System, not section 5.4.3.1.

5.3.2. DRAG FORCE (F_{DRAG}) ON LATERAL FORCE RESISTING SYSTEM

Refer to ASCE 7-22-S2, Section 5.4.3.2 and C5.4.3.2 and Table 5.4-2, for guidance.

CLARIFICATION

Drag Force (F_{drag}) on Lateral Force Resisting System

Equation 5.4-5 presented in ASCE 7-22-S2 only applies to enclosed buildings. From a floodplain management perspective, an enclosed building would be referred to as a building having an "enclosure" below the lowest floor. These calculations could also apply to buildings having the first floor below the design stillwater flood elevation (which could be protected using dry floodproofing). See Section "Drag Force (F_{drag}) on Lateral Force Resisting System for Non-Enclosed Buildings" in this design guide for non-enclosed buildings.

For walled structures, the calculated drag load can be evenly distributed throughout the area normal to the flow of the water. When different flow directions are possible, independent load cases for drag forces should be considered for each direction. Loads on piles and columns are often calculated on a per pile or column basis, but the load can further be distributed throughout the area of the pile or column normal to the direction of flow.

Drag Force (F_{drag}) on Lateral Force Resisting System for Non-Enclosed Buildings

ASCE 7-22-S2, Section 5.4.3.2 states,

> *For open buildings or buildings with breakaway walls as defined by Section 5.3.10, the building lateral force resisting system shall be designed to resist the summation of the drag loads of each exposed vertical and horizontal element as required by Section 5.4.3.1 using Equation 5.4-4. The effects of debris damming shall be considered on the side of the structure exposed to the flow direction, considering damming on any two adjacent bays or a minimum of a 50-ft (15.2-m) width, whichever produces the largest base shear.*

ASCE 7-22-S2, Section C5.4.3.2 states,

> *While for component design all vertical elements need to consider this damming it is not reasonable to consider every column with this increased drag simultaneously thus the requirements of Section 5.3.9.2 limit the number of bays that need to consider damming for the design of the lateral force resisting system.*

EXAMPLE

Equation Derivation for Drag Forces on Lateral Force Resisting System for Non-enclosed Buildings

Equation 13, an adaptation of ASCE 7-22-S2, Equation 5.4-4, is applicable to non-enclosed foundations (including those with breakaway walls) with debris damming and a non-breakaway enclosed space. The equation sums the drag forces for debris damming, the enclosed space, and the exposed piles, respectively. Users should consider how the equation may need to be adjusted based on their building foundation layout. **Equation 13** is one example of how ASCE 7-22-S2, Equation 5.4-4 may be used for non-enclosed buildings. The following Example text box provides a worked example of how **Equation 13** may be applied. For alternate configurations and geometries, ASCE 7-22-S2, Equation 5.4-4 and **Equation 13** can be leveraged to create site-specific calculations. Additional functions may be needed for **Equation 13** when drag coefficients vary for an element type. For example, if a building has both round and square columns, two functions will be needed to account for drag on columns. Likewise, if a building has freestanding walls parallel to the flow as well as walls perpendicular to the flow, two functions will be needed to account for drag on walls. The following worked example problem explains the origin of each equation term.

$$F_{drag} = (1/2)\rho C_d V^2 h(n_d b_c + C_{cx} s_L) + (1/2)\rho C_d V^2 h(b_{w1} + b_{w2} + \ldots + b_{wn}) + (1/2)\rho C_d V^2 h b_c n_e$$

Equation 13 [Derived from ASCE 7-22-S2 5.4-4]

where,

C_d = drag coefficient for submerged objects subjected to currents, defined in ASCE 7-22-S2, Table 5.4-1. Note: There is a specific C_d for structural components with debris damming.

h = submerged height of column or wall above its foundation or structural floor level, in ft (m)

Notes:

1. The submerged height is in reference to d_f and does not include wave height.
2. For open pile foundations without grade beams, h is taken as the submerged height above the eroded ground elevation.

s_L = clear spacing of column to the adjacent column plus the clear spacing to the next adjacent column in ft (m) or the clear spacing behind a minimum 50 ft (15.2-m) debris dam in width [or the width of the structure if the side of the structure exposed to the flow direction is less than 50 ft wide], whichever produces the largest base shear. If the minimum 50 ft debris dam is used, the s_L should be calculated as (50-ft – $n_d b_c$) to obtain the clear spacing behind the 50 ft debris dam. If the width of the structure is used, s_L should be calculated as (building width in feet – $n_d b_c$). Note that ASCE 7-22-S2 does not state that the structure width may be used as s_L if it is less than 50 ft. However, it is included as an option in this example as this is a reasonable assumption given the intent of the calculation. Portions of the s_L definition that use this assumption are underlined.

b_c = width of column, perpendicular to the direction of flow considered, in ft (m)

b_w = width of wall, perpendicular to the direction of flow considered, in ft (m)

C_{cx} = debris damming closure ratio as determined in ASCE 7-22-S2, Section 5.3.9.2. Based on "s" as determined in ASCE 7-22-S2, Sections 5.3.9.2 and 5.4.3.2, not "s_L" utilized for lateral loading.

n_d = number of submerged vertical foundation elements (piles, columns, etc.) covered by damming debris. If the clear spacing of column to the adjacent column plus the clear spacing to the next adjacent column is utilized for s_L, n_d = 3. If s_L is calculated with the 50-ft minimum assumption, the total number of submerged vertical foundation elements (piles, columns, etc.) covered by damming debris should be used.

n_e = number of submerged vertical foundation elements (piles, columns, etc.) exposed to flow. This is the total number of submerged vertical foundation elements minus the elements that have flow blocked by an attached wall or by debris.

V = design flood velocity, in ft/s (m/s)

ρ = mass density of water, in lb s^2/ft^4, taken as 1.94 lb s^2/ft^4 (1000 kg/m^3) for freshwater and 1.99 lb s^2/ft^4 (1027 kg/m^3) for saltwater

EXAMPLE

Calculating Drag Forces on Lateral Force Resisting System for Non-enclosed Buildings Using Equation 13

Calculate: Drag force (F_{drag}) on a lateral force resisting system for a non-enclosed (open) foundation exposed to saltwater flooding as shown in Figure 8. The foundation plan includes a non-breakaway enclosure. The foundation does not have additional bracing below the flood elevation. The piles are 12-inch-square concrete piles with a spacing of 11 feet on center. Note: The calculation of drag forces on individual components is also required but is not shown as part of this example; refer to Section 5.3.1 of this design guide.

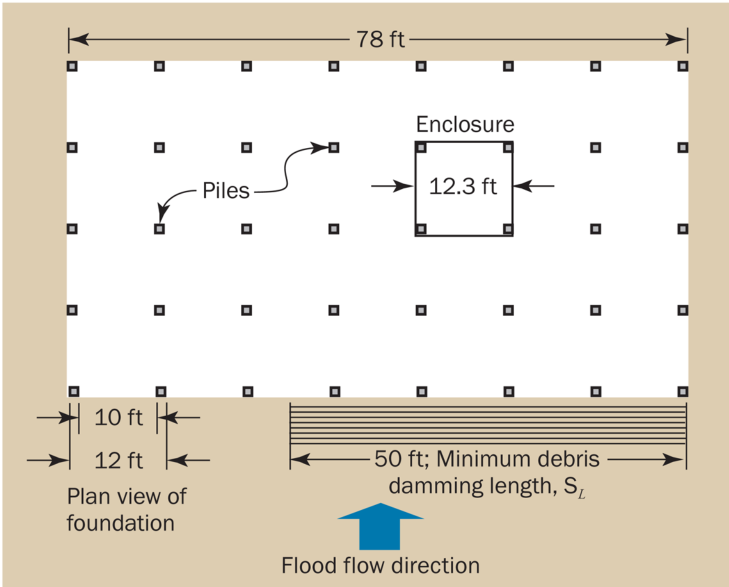


Figure 8. Foundation plan for the non-enclosed foundation

Given:

ρ = 1.99 lb s^2/ft^4

V = 5 ft/s

h = 4 ft

s = 10 ft

b_c = 1 ft

b_{w1} = 12.3 ft

Find drag force due to debris damming:

Portion of **Equation 13**: $(1/2)\rho C_d V^2 h(n_d b_c + C_{cx} s_L)$ = 7,264 lbs

C_d = 2 [Table 5.4-1, Structural Components with debris damming]

C_{cx} = 0.70 [Figure 5.3-1, for s = 10 ft]

s_L = 45 ft [2 bays = 23 ft, therefore, select 50-ft minimum. Subtract $n_d * b_c$ from 50 ft to obtain clear spacing]

n_d = 5 [maximum number of piles covered by debris using the 50 ft minimum for s_L]

Find drag force on enclosed area:

Portion of **Equation 13**: $(1/2)\rho C_d V^2 h(b_{w1}+b_{w2}+\ldots+b_{wn}) = (1/2)\rho C_d V^2 b_{w1} h$ = 2,448 lbs

C_d = 2 [Table 5.4-1, Wall or flat plate, normal to flow]

Note: The drag force on the side walls is included in the width of b_w and, therefore, does not need to be calculated separately.

Find drag force on exposed piles:

Portion of **Equation 13**: $(1/2)\rho C_d V^2 h b_c n_e$ = 6,169 lbs

C_d = 2 [Table 5.4-1, Square or rectangular column with longer face oriented perpendicular to flow]

n_e = 31 [40 total piles − 4 piles inside enclosure − 5 piles blocked by debris]

Find total drag force on lateral force resisting system:

F_{drag} = 6,169 + 2,448 + 7,264 = 15,881 lbs

6. Wave Variables and Loads

6.1. Wave Height (H_{design}) and Type Determination

Refer to ASCE 7-22-S2, Section 5.3.7.1 and C5.3.7.1, for further guidance.

Per ASCE 7-22-S2, Section 5.3.7,

> *The effects of waves shall be included for both V-Zones and A-Zones. In areas subjected to riverine flooding only, the effects of waves are permitted to be neglected.*

EXCEEDING MINIMUMS

Waves in X Zones

Although not required by ASCE 7-22-S2, FEMA recommends that the effects of waves also be included for X Zones subjected to coastal flooding.

Per ASCE 7-22-S2, Section 5.3.7.1,

> *The design wave height H_{design} at the site in ft (m) shall be obtained by one of the following four methods: (1) by assuming depth-limited breaking wave conditions, (2) from a flood hazard study, (3) by numerical modeling, or (4) by laboratory testing (physical modeling). When Methods 3 or 4 are used, design flood parameters shall be determined using the site-specific hazard procedures of Section 5.3.11*

Per ASCE 7-22-S2, Section C5.3.7.1,

> *Because the assumption of depth-limited waves may be overly conservative in some cases, ASCE 7-22 Supplement 2 allows for the computation of scour depth and wave loads based on nonbreaking waves of a wave height lower than the depth-limited wave height. Figure C5.3-9 [Figure 9] illustrates a procedure to utilize information that may yield a wave height lower than the conservative estimate of a depth-limited breaking wave at the site and a procedure to determine if the computed wave height at a site is a breaking or nonbreaking wave. The figure is divided into three branches. To start, the designer must determine if the wave height information is available at the site from a flood hazard study. If the answer is yes, then the designer proceeds to Branch 1. If no, then the designer must determine if the wave height information is available at the shoreline. If yes, then the designer proceeds to Branch 2. If no, then the designer proceeds to Branch 3.*

Per ASCE 7-22-S2, Section C5.4.4.2.4,

> *Wave loads on vertical walls reach a maximum when the waves are normally incident (i.e., direction of wave approach is perpendicular to the face of the wall with wave crests parallel to the face of the wall). Obliquely incident waves may be identified by a site-specific study. In*

the absence of a site-specific study and as guidance for designers of coastal buildings or other structures on normally dry land (i.e., flooded only during coastal storm or flood events), it can be assumed that waves will approach the shoreline within a 22.5-degree angle from perpendicular. Therefore, the direction of wave approach relative to a vertical wall will depend upon the orientation of the wall relative to the shoreline.

CLARIFICATION

Wave Height Parameters

ASCE 7-22-S2 discusses various wave height parameters. Each parameter is detailed in Table 14.

Table 14. Wave Height Parameters

Wave Height Parameters	Definitions	Relationships with Other Wave Height and Flood Design Parameters
Significant Wave Height, H_s	The wave height associated with the mean of the highest one-third of waves associated with the design flood, used to determine Controlling Wave Height when not known (ASCE 7-22-S2).	$H_s = 0.625H_c$ *where,* H_C = controlling wave height, in ft (m) H_s = significant wave height, in ft (m)
Controlling Wave Height, H_c	The wave height associated with the mean of the highest 2% of waves associated with the design flood used to determine design wave height when the waves are not depth-limited (ASCE 7-22-S2). The average height of the 1% highest waves (National Academy of Sciences 1977). Approximately, the average height of the highest 1% of waves during storm conditions (FEMA 2016).	$H_c = 1.6H_s$ *where,* H_C = controlling wave height, in ft (m) H_s = significant wave height, in ft (m)
Breaking Wave Height, H_b	The controlling wave will break when it reaches a height equal to about 0.8 of the depth of water (National Academy of Sciences 1977). This height is termed the breaking wave height.	$H_b = C_{br} * d_f$ *where,* C_{br} = wave height coefficient for depth-limited breaking, taken as 0.78 d_f = design stillwater flood depth in ft (m)
Design Wave Height, H_{design}	Wave height defined by ASCE 7-22-S2, Section 5.3.7.1. Used for calculating wave loads.	See Chapter 6

A procedure to calculate the design wave height (H_{design}) and to define the wave type (breaking or non-breaking) is outlined in the commentary of ASCE 7-22-S2. This procedure has three branches and coincides with Figure 9 (Figure C5.3-9 in ASCE 7-22-S2). Numerical modeling and laboratory testing are also permitted by ASCE 7-22-S2 as methods to determine the design wave height; these modeling and testing methods are not detailed herein.

- Branch 1 – Utilizes Site Data
  - Wave height information required: wave height information available at the project site.
  - See Section 6.1.4 of this design guide for further information.

ADDITIONAL CONSIDERATIONS

Branch 1

An alternate method is presented in Section 6.1.4 that enables Branch 1 to be completed with site data from the FIRM and FIS.

This alternate method can be used for Branch 1 unless the site is in AO Zone.

- Branch 2 – Utilizes Overland Wave Transformation Theory
  - Wave height information required: wave height information available at shoreline. Use an applicable Flood Hazard Study to define H_s.
  - See Section 6.1.5 of this design guide for further information.

ADDITIONAL CONSIDERATIONS

Branch 2

The H_s given in a FEMA FIS cannot be used for this step if the header contains "Starting Wave Conditions." See Figure 10. This H_s is the significant wave height outside the surf zone and not applicable at the shoreline. Use Branch 1 and follow "Alternate Step 2" to use FIS and FIRM data.

- Branch 3 – Utilizes Depth-Limited Breaking Waves
  - Wave height information required: none.
  - See Section 6.1.6 of this design guide for further information.

ADDITIONAL CONSIDERATIONS

Branch 3

Depth-limited breaking waves are assumed in Branch 3. Assuming depth-limited breaking waves will yield a conservative estimate.

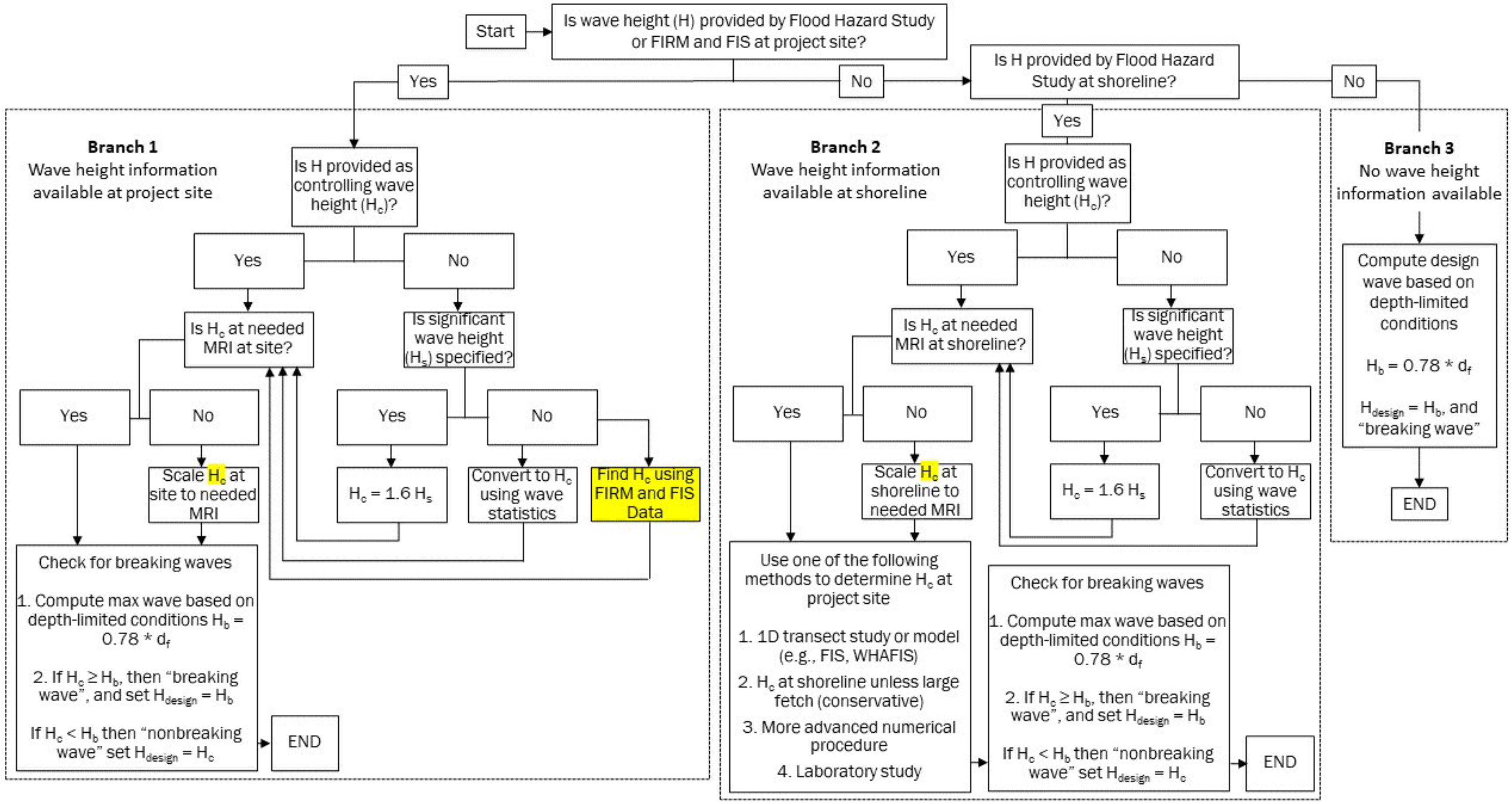


Figure 9. Methods (branches) to determine wave height at site for breaking and nonbreaking waves [Based on ASCE 7-22-S2 Figure C5.3-9 with corrections and additions highlighted in yellow]

CLARIFICATION
Figure 9 Differences from ASCE 7-22-S2 Figure C5.3-9

Figure 9, which is based on ASCE 7-22-S2, Figure C5.3-9, includes two corrections and one addition that are highlighted in yellow.

Both Branch 1 and Branch 2 correct "H_s" to be "H_c" when scaling the wave height to the needed MRI at the site.

Branch 1 has an additional option of finding H_c with the FIRM and FIS when the H_s is not specified at the site.

6.1.1. SCALING H_C TO NEEDED MRI

Branch 1 and Branch 2 include provisions to scale H_c (controlling wave height) to the required MRI. This section outlines the method for scaling H_c to the required MRI, which provides H_{CMRI}, the controlling wave height based on the MRI associated with the applicable Risk Category.

If the controlling wave height is specified by a 100-year design flood, then the controlling wave height must be adjusted to the controlling wave height corresponding to the MRI design flood event using Table 15 and **Equation 14**.

$H_{CMRI} = C_{HC} * H_{C100}$ **Equation 14** [ASCE 7-22-S2, Section 5.3.7.1]

where,

H_{CMRI} = controlling wave height corresponding to the Risk Category and MRI, in ft (m)
C_{HC} = scaling factor for controlling wave height
H_{C100} = controlling wave height for the 100-year MRI, in ft (m)

Table 15. Design Flood MRI and Scaling Factors for Controlling Wave Height [Based on ASCE 7-22-S2, Table 5.3-3]

Risk Category	*MRI*	*All Coastal Sites C_{HC} Scaling from H_{c100}*
I	100-year	1.00
II	500-year	1.30
III	750-year	1.35
IV	1,000-year	1.40

EXCEEDING MINIMUMS

Calculating H_{CMRI} Based on H_{C500}

If the controlling wave height is specified by a 500-year design flood, then the controlling wave height must be adjusted to the controlling -wave height corresponding to the MRI design flood event using Table 16 and **Equation 15.**

$H_{CMRI} = C_{HC\_500} * H_{C500}$ **Equation 15** [Derived from ASCE 7-22-S2, Section 5.3.7.1]

where,

H_{C500} = controlling wave height for the 500-year MRI, in ft (m)
C_{HC_500} = scaling factor for controlling wave height with the MRI from Table 16 when H_{c500} is the starting controlling wave height

Table 16. Design Flood MRI and Scaling Factor for Controlling Wave Height for Use with H_{c500} [Derived from ASCE 7-22-S2, Table 5.3-3]

Risk Category	MRI	All Coastal Sites C_{HC_500} Scaling from H_{c500}
I	100-year	N/A
II	500-year	1.00
III	750-year	1.04
IV	1,000-year	1.08

6.1.2. CONVERTING SIGNIFICANT WAVE HEIGHT, H_S, TO CONTROLLING WAVE HEIGHT, H_C

If the wave height is provided as a significant wave height (H_s), it can be converted to a controlling wave height (H_c) with the same MRI by utilizing **Equation 16.**

$H_c = 1.6\ H_s$ **Equation 16** [ASCE 7-22-S2, Eq. 5.3-8]

where,

H_s = significant wave height, in ft (m)
H_c = controlling wave height, in ft (m)

6.1.3. DEPTH-LIMITED WAVES

CLARIFICATION

Depth-Limited Waves

Wave heights are limited by numerous factors including water depth, fetches, and obstructions. Wave heights which are limited by depth are referred to as depth-limited waves. Depth-limited (i.e., breaking) waves occur when the wave height exceeds the height that can be supported by the local water depth. Thus, when the wave height exceeds the maximum supported wave height, the wave breaks. Assuming a wave height is depth-limited is a conservative assumption as it assumes the maximum possible wave height.

The breaking wave height (H_b) may be calculated by **Equation 17.** If the calculated H_{design} at the site is equal to or exceeds H_b from **Equation 17**, then the wave must be considered as breaking and H_{design} = H_b.

$H_b = C_{br} * d_f$ **Equation 17** [ASCE 7-22-S2, Eq. 5.3-6]

where,

C_{br} = wave height coefficient for depth-limited breaking (which represents the ratio of the wave height to the stillwater depth), taken as 0.78

d_f = design stillwater flood depth in ft (m)

6.1.4. BRANCH 1: WAVE HEIGHT INFORMATION AVAILABLE AT PROJECT SITE OR CAN BE DERIVED FROM FIS AND FIRM

This section steps through Branch 1 of the *Procedure to Determine Wave Height at Site for Breaking and Nonbreaking Waves*. These steps assume the significant wave height, H_s, is provided for the site. If the controlling wave height, H_c is provided, skip Steps 1 and 2. If a wave height is provided, but it is neither the H_s or H_c, wave statistics may be used to convert it to H_c. Branch 3 is often the path employed in the absence of H_s or H_c. See Table 17 for the Branch 1 Procedure.

Table 17. Branch 1 Procedure

Step	*Objective*	*Method*
Step 1	Obtain Significant Wave Height (H_s) from the Flood Hazard Study	Use an applicable Flood Hazard Study to define H_s. **Note:** The H_s given in FEMA Flood Insurance Study (FIS) Transect Data **cannot** be used for this step. See Figure 10. See "Alternate Step 2" to utilize FIS and Flood Insurance Rate Map (FIRM) data.
Step 2	Convert H_s to H_c	$H_c = 1.6\ H_s$ **Equation 16**

Step	Objective	Method
Alternate Step 2	Calculate Controlling Wave Height (H_c) based on FIRM and FIS	See "Additional Considerations: Estimating the Controlling Wave Height (H_c) Based on the FIRM and FIS" text box for detailed method. **Note:** This step results in either the 1% annual chance controlling wave height (H_{c100}) or the 0.2% annual chance controlling wave height (H_{c500}) depending on the method selected
Step 3	Scale H_c to needed mean recurrence interval (MRI) (as required)	$H_{CMRI} = C_{HC}*H_{c100}$ **Equation 14** $H_{CMRI} = C_{HC\_500}*H_{c500}$ **Equation 15** C_{HC} and C_{HC_500} are defined in Table 15 and Table 16 **Note:** *If H_s or H_c were given in terms of the design MRI, scaling is not required and $H_c = H_{cMRI}$. Skip this step.*
Step 4	Calculate breaking wave height (H_b)	$H_b = C_{br}*d_f$ **Equation 17** *where,* C_{br} = wave height coefficient for depth-limited breaking, taken as 0.78
Step 5	Check if H_{design} is limited by water depth	If $H_{cMRI} < H_b$, then nonbreaking wave and $H_{design} = H_{cMRI}$ If $H_{cMRI} \geq H_b$, then breaking wave and $H_{design} = H_b$ **Note:** The breaking wave height is the maximum wave height supported by the water depth.

CLARIFICATION

Starting Wave Conditions in FIS

The "Starting Wave Conditions" shown in the Transect Data Table in Figure 10 are for waves outside of the surf zone. The given Significant Wave Height (H_s) cannot be used as a shoreline or inland wave height.

TABLE 9 – TRANSECT DATA (continued)

			Starting Wave Conditions for the 1-percent Annual Chance		Starting Stillwater Elevations (ft NAVD88) Range of Stillwater Elevations* (ft NAVD88)			
Flood Source	Transect	Coordinates	Significant Wave Height H_s (ft)	Peak Wave Period T_p (sec)	10-percent Annual Chance	2-percent Annual Chance	1-percent Annual Chance	0.2-percent Annual Chance
Atlantic Ocean	38	N 38.682166 W 75.069399	16.67	12.80	6.1 3.9 - 6.2	7.5 4.6 - 7.6	8.1 4.9 - 8.2	9.7 5.5 - 9.8
Atlantic Ocean	39	N 38.674456 W 75.068224	16.11	12.87	6.1 3.9 - 6.2	7.5 4.6 - 7.6	8.1 4.9 - 8.2	9.7 5.6 - 9.7
Indian River Bay	40	N 38.671883 W 75.073050	2.90	3.36	4 4 - 5.3	4.6 4.6 - 7.7	4.9 4.9 - 8.3	5.6 5.6 - 9.8

Figure 10. Graphic depicting FIS transect data (source: FEMA 2018a)

ADDITIONAL CONSIDERATIONS

Alternate Step 2: Estimating the Controlling Wave Height (H_c) Based on the FIRM and FIS

This method is valid in areas subject to overland waves.

This method is not valid in AO Zone. Per FEMA, on flood maps in coastal communities, AO Zone usually marks areas at risk of flooding from wave overtopping, where waves are expected to wash over the crest of a dune or bluff and flow down into the area beyond (FEMA 2021b). Thus, the AO Zone exclusion is placed on this method because this method involves scaling of an existing wave height and sites classified as AO Zone do not have existing wave heights due to those areas being flooded by wave overtopping. Further, wave conditions may be present in the future at sites classified as AO Zone if the dune or bluff protecting the area is eroded.

Depending on the available data, H_{c100} or H_{c500} may be calculated. Review each calculation option first to determine which is most appropriate given the available data.

Calculate H_{c100}:

Obtain the BFE from the FIRM panel associated with the site location.

Obtain the 1% annual-chance stillwater elevation ($SWEL_{100}$) at the site from the FIS.

Use **Equation 18** to calculate H_{c100}.

$$H_{c100} = (BFE + 0.5\text{ ft} - SWEL_{100}) / 0.7 \qquad \textbf{Equation 18}$$

where,

BFE = base flood elevation in ft (m)
H_{c100} = controlling wave height for the 100-year MRI, in ft (m)
$SWEL_{100}$ = stillwater elevation for the 100-year MRI, in ft (m)

Calculate H_{c500}:

Some FISs include the 0.2% annual-chance wave envelope elevation, which can be used to obtain the H_{c500}.

Obtain the 0.2% annual-chance wave envelope (crest) elevation (WE_{500}) from the FIS associated with the site location.

Obtain the 0.2% annual-chance stillwater elevation ($SWEL_{500}$) at the site from the FIS.

Use **Equation 19** to calculate H_{c500}.

$$H_{c500} = (WE_{500} - SWEL_{500}) / 0.7 \qquad \textbf{Equation 19}$$

where,

WE_{500} = 0.2% annual-chance wave envelope (crest) elevation, in ft (m)
H_{c500} = controlling wave height for the 500-year MRI, in ft (m)
$SWEL_{500}$ = stillwater elevation for the 500-year MRI, in ft (m)

Commentary:

Equation 18 and **Equation 19** utilize 0.7 in the denominator to obtain the total wave height. As depicted in Figure 11, 70% of the wave is above the SWEL.

The BFEs on FIRMs are rounded to the nearest whole foot. **Equation 18** adds 0.5 foot to the BFE to account for potential rounding down. The addition of the 0.5 foot exceeds minimums and is not required. The practitioner may elect to remove the 0.5 foot from **Equation 18**.

Figure 12 depicts a coastal transect profile with 0.2% annual-chance wave envelope (crest) elevations. Figure 12 is provided for demonstration purposes only and must not be used for deriving load calculations.

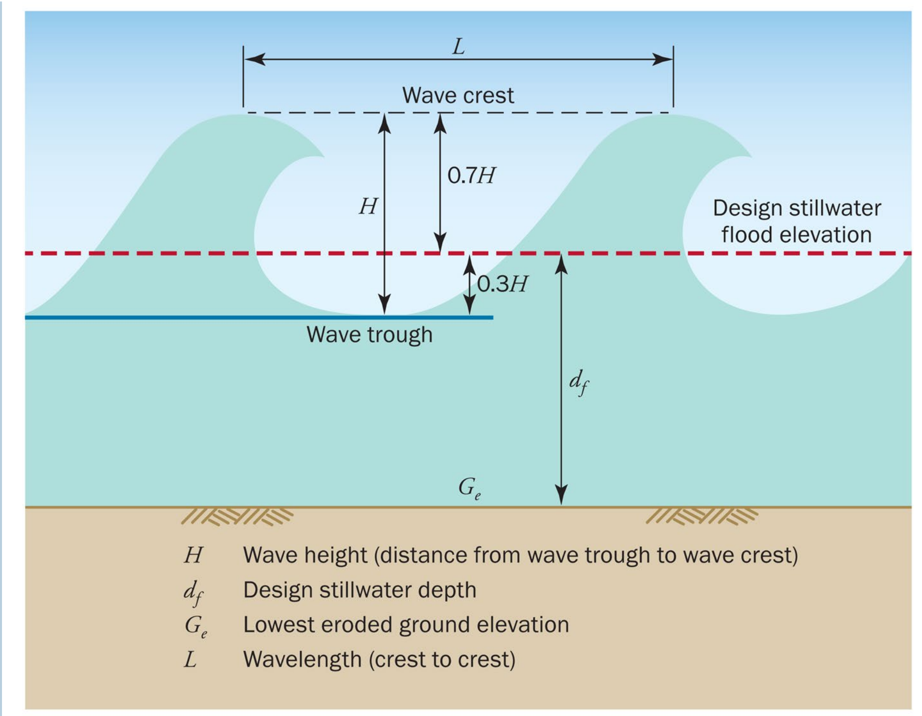


Figure 11. Wave characteristics

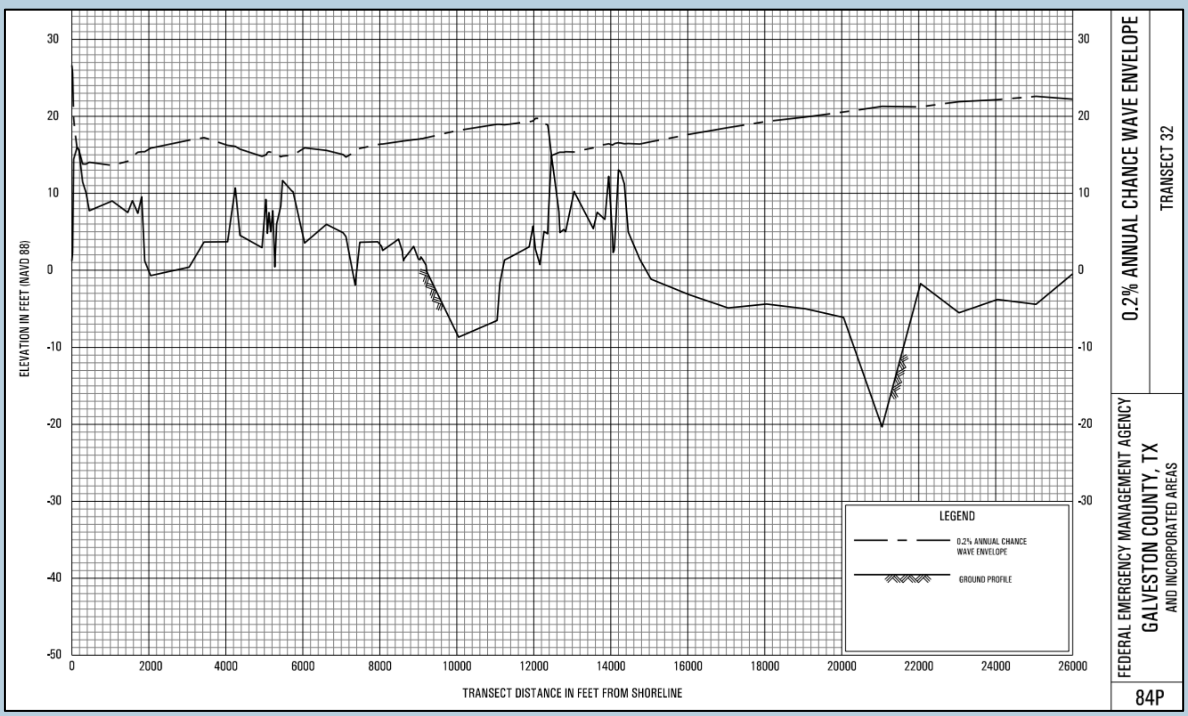


Figure 12. Example of a coastal transect profile with 0.2% annual-chance wave envelope elevations

6.1.5. BRANCH 2: WAVE HEIGHT INFORMATION AVAILABLE AT SHORELINE

This section steps through Branch 2 of the *Procedure to Determine Wave Height at Site for Breaking and Nonbreaking Waves*. If the controlling wave height (H_c) at the shoreline is provided, skip Steps 1 and 2. If a wave height is provided, but it is neither the H_s or H_c, wave statistics may be utilized to convert it to H_c. Branch 3 is often the preferred path in the absence of H_s or H_c. See Table 18 for the Branch 2 Procedure.

Table 18. Branch 2 Procedure

Step	*Objective*	*Method*
Step 1	Obtain Significant Wave Height (H_s) from the Flood Hazard Study	Use an applicable Flood Hazard Study to define H_s. **Note:** The H_s given in a FEMA Flood Insurance Study (FIS) cannot be used for this step if the header contains "Starting Wave Conditions." See Figure 10. Starting Wave Conditions are for waves outside of the surf zone and are not applicable at the shoreline. Use Branch 1 and follow "Alternate Step 2" to utilize FIS and Flood Insurance Rate Map (FIRM) data.
Step 2	Convert H_s to H_c	$H_c = 1.6\ H_s$ **Equation 16**
Step 3	Scale H_c to needed MRI	$H_{cMRI} = C_{HC}*H_{c100}$ **Equation 14** $H_{cMRI} = C_{HC\_500}*H_{c500}$ **Equation 15** C_{HC} and C_{HC_500} are defined in Table 15 and Table 16 **Note:** *If H_s or H_c were given in terms of the design MRI, scaling is not required and $H_c = H_{cMRI}$. Skip this step.*
Step 4	Transform H_{cMRI} to H_{cSITE} at Project Site	<u>Method 1</u> Use a one-dimensional transect model such as WHAFIS. This may include using the method outlined in the section Transforming Wave Height from Shoreline to Inland Site to transform the wave height from the shoreline to an inland site.
		<u>Method 2</u> Use H_{cMRI} at the shoreline as the H_{cSITE} at the site. **Note:** *This method is conservative unless there is a large unobstructed fetch between the shoreline and the site.*
		<u>Methods 3 and 4</u> Advanced methods, refer to ASCE 7-22-S2 for instruction.
Step 5	Calculate breaking wave height (H_b)	$H_b = C_{br}*d_f$ **Equation 17** *where,*

Step	Objective	Method
Step 6		C_{br} = wave height coefficient for depth-limited breaking, taken as 0.78
	Check if H_{design} is limited by water depth	If $H_{cSITE} < H_b$, then nonbreaking wave and $H_{design} = H_{cSITE}$ If $H_{cSITE} \geq H_b$, then breaking wave and $H_{design} = H_b$ **Note:** The breaking wave height is the maximum wave height supported by the water depth

Transforming Wave Height from Shoreline to Inland Site

Per ASCE 7-22-S2, Section C5.3.7.1,

> *In Method 1, the designer can use a one-dimensional transect model such as WHAFIS that is used in FEMA Flood Insurance Studies. WHAFIS is based on the 1977 National Academy of Science (NAS) study titled "Methodology for Calculating Wave Action Effects Associated with Storm Surges." Using the methods presented in the NAS study, a designer could use Table C5.3-2 to transform the wave height from the shoreline to an inland site. Table C5.3-2 provides example wave height factors. The wave height factors in Table C5.3-2 uses the experimental results and NAS procedure, and accounts for some conservatism related to incident wave direction and building array geometry (staggering of buildings within the array).*

CLARIFICATION

Wave Height Analysis for Flood Insurance Studies (WHAFIS)

As noted by ASCE 7-22-S2, FEMA uses WHAFIS for development of FIRMs and FISs. WHAFIS is a one-dimensional model. Two-dimensional models such as ADCIRC (Advanced CIRCulation) and Simulating Waves Nearshore (SWAN) are often utilized to develop the WHAFIS inputs such as the starting wave height, wave setup, and stillwater depth (FEMA 2015). Currently, WHAFIS is typically used as a module within The Coastal Hazard Analysis Modeling Program (CHAMP).

More information on how WHAFIS models are developed can be found here: https://www.fema.gov/sites/default/files/documents/fema_coastal-overland-wave-propagation_112022.pdf.

ASCE 7-22-S2, Section C5.3.7.1, presents Table C5.3-2 [Table 19], which may be used to transform the wave height from the shoreline to an inland site. The "Additional Considerations: Wave Transformation Using Table 19" text box provides a brief procedure to implement Table 19.

Table 19. Number of Shielding Rows of Structures and Wave Height Factor [Based on ASCE 7-22-S2, Table C5.3-2]

Number Of Shielding Rows	*Wave Height Factor (C_{wH})*
0 or 1	1.0
2 or 3	0.7
4 or 5	0.5
6 or more	0.3

Per ASCE 7-22-S2, Section C5.3.7.1, the following requirements must be met to be considered a shielding row:

- *A seaward shielding structure must be robust enough to remain intact during the design storm event. A building whose façade fails while the primary frame does not, is not considered a shielding structure. Buildings or barriers constructed of wood, light gage steel, aluminum, or any similarly performing material, shall be considered nonshielding unless proven otherwise by an engineering analysis.*
- *Only rows with less than 70% open distance between buildings relative to total distance measured parallel to shore can be considered as a shielding row.*

ADDITIONAL CONSIDERATIONS

Breakaway Walls

When buildings include breakaway walls, which allow the free passage of waves and surge, the building should not be considered a shielding structure. Note that because of free-of-obstruction requirements, buildings located in the V Zone are typically constructed with breakaway walls.

ADDITIONAL CONSIDERATIONS

Wave Transformation Using Table 19

The transformation method described in this section outlines the transformation method noted in Method 1 of Branch 2. If there are no buildings between the project site and the shoreline, this method will provide the same result as Method 2 in Branch 2, the H_{cMRI} at the shoreline will equal the H_{cSITE} at the site.

The shoreline wave height can be transformed to an inland site wave height by utilizing the wave height factors in Table 19 and **Equation 20**.

$$H_{cSITE} = H_{cMRI} * C_{wH} \quad \textbf{Equation 20}$$

where,

H_{cSITE} = the controlling wave height at the site in ft (m)
H_{cMRI} = controlling wave height corresponding to the risk category and MRI, in ft (m)
C_{wH} = wave height factor

6.1.6. BRANCH 3: NO WAVE HEIGHT INFORMATION AVAILABLE

If no wave height information is available at either the shoreline or the site, depth-limited breaking waves can be assumed. Assuming depth-limited breaking waves will yield a conservative estimate. See Table 20 for the Branch 3 Procedure.

Table 20. Branch 3 Procedure

Step	*Objective*	*Method*
Step 1	Calculate breaking wave height (H_b)	$H_b = C_{br}*d_f$ Equation 17 *where,* C_{br} = wave height coefficient for depth-limited breaking, taken as 0.78
Step 2	Set the design wave height (H_{design}) equal to the breaking wave height (H_b)	$H_{design} = H_b$

6.2. Wave Period (T_p) and Wavelength (L) Determination

The wave period (T_p) and wavelength (L) variables are required for scour and wave pressure calculations. Refer to ASCE 7-22-S2, Section 5.3.7.2, for applicable procedures.

CLARIFICATION
Wave Period and Wavelength

The wave period and wavelength define a single wave cycle. The wave period is the time interval between the passage of two successive wave crests or troughs at a given point. The wavelength is the horizontal distance between two identical reference points on two successive wave crests or two successive wave troughs (USACE IWR n.d.). Wavelength is depicted in Figure 11.

ADDITIONAL CONSIDERATIONS
Wave Period Provided in FIS

If the wave period is provided with transect data in a FIS, the provided wave period may be used, but it must only apply to the specified MRI. For example, a "*Peak Wave Period*" listed under the "*Starting Wave Conditions for the 1-percent Annual Chance*" heading must only be applied to load calculations requiring a 1% AEP (100-year MRI) design flood. Wave period data are often provided for the 1% AEP or the 100-year flood. Transect data may be available for the 0.2% annual chance or the 500-year event but are unlikely to be available for other MRIs.

6.3. Wave Loads

ADDITIONAL CONSIDERATIONS

Selecting Structure and Overhang Elevations Based on Wave Height

As described in the "Exceeding Minimums: Elevation Height" text box in Section 3.2.4, elevating the structure above the wave crest elevation defined by ASCE 7-22-S2 is a design option which reduces flood damage risks.

Further, the designer will not have to account for many of the wave uplift loads as defined in ASCE 7-22-S2, Section 5.4.4.3, if the structure is elevated above the wave crest elevation, which is defined in **Equation 21**. Eliminating the necessity to calculate many of the wave uplift loads may be an additional benefit for the designer as the wave uplift loads described in ASCE 7-22-S2 may be challenging to calculate because they require empirical data to define various variables.

If the lowest horizontal structural member of a structure is elevated to an elevation equal to or above E_h defined in **Equation 21**, then wave uplift loads per ASCE 7-22-S2, Section 5.4.4.3, are only required to be calculated for the vertical foundation members of the structure and applicable overhangs.

$$E_h = d_f + G_e + 0.7H_{design} \qquad \textbf{Equation 21}$$

where,

E_h = Minimum elevation of the lowest horizontal structural member supporting the lowest floor to avoid wave uplift loads on the main structure per ASCE 7-22-S2, Section 5.4.4.3

G_e = Elevation of grade at the building or other structure inclusive of effects of erosion in ft (m), per ASCE 7-22-S2, Section 5.3.5; see Section 3.2.3 in this design guide for additional guidance

In addition to the structure elevation, overhang elevations must also be considered per ASCE 7-22-S2, Section 5.4.4.3. If an overhang is below the $SWEL_{MRI}$ or elevated to an elevation equal to or above E_o defined in **Equation 22**, then wave uplift loads per ASCE 7-22-S2, Section 5.4.4.3, are not required to be calculated for the overhang.

$$E_o = d_f + G_e + 1.5H_{design} \qquad \textbf{Equation 22}$$

where,

E_O = Minimum overhang elevation to avoid wave uplift loads on the overhang per ASCE 7-22-S2, Section 5.4.4.3, if the overhang is located at an elevation above the $SWEL_{MRI}$

6.3.1. BREAKING AND NON-BREAKING WAVE LOADS

Breaking and non-breaking wave loads should be developed as described in the applicable sections of ASCE 7-22-S2. Refer to Table 21 and Table 22 for an overview of criteria and wave loads discussed in ASCE 7-22-S2. Table 21 provides an overview of the criteria for determining if a structural element must be analyzed as a pile or column, or a wall for wave loads. Table 22 provides an overview of the sections in ASCE 7-22-S2 relating to wave loads. Table 22 also provides additional criteria and notes as they pertain to specific sections.

CLARIFICATION

Equation 5.4-13, Lateral Breaking Wave Loads on Non-elevated Vertical Walls

ASCE 7-22-S2 Equation 5.4-13 for lateral breaking wave loads on non-elevated vertical walls points to ASCE 7-22-S2, Section 5.4.4.2.1, for the development of p_2, p_3, and η^*. The p_2 and p_3 equations (5.4-10 and 5.4-11) in ASCE 7-22-S2, Section 5.4.4.2.1, use p_1, but users should apply the modified p_{1B} instead of p_1 when calculating F_{BRK} with Equation 5.4-13. The modified pressure p_{1B} calculation is described in ASCE 7-22-S2, Section 5.4.4.2.2.

Table 21. Criteria for Determining if a Structural Element is Analyzed as a Pile or Column, or a Wall for Wave Loads

Use pile or column equations (Section 5.4.4.1)	*Use wall equations (Section 5.4.4.2)*
If, $d_f/w \geq 3$	If, $d_f/w < 3$
If, $s \geq w/2$	If, $s < w/2$

where,

w = the lateral dimension of a structural element facing the wave

s = average clear spacing of column or wall to the adjacent column or wall in ft (m)

CLARIFICATION

Use of Table 21

When a design has mixed results for determining whether a structural element must be analyzed as a pile or column or a wall for wave loads per Table 21, the more conservative calculation method should be used.

Table 22. Overview of Wave Load Sections in ASCE 7-22-S2 by Support Element Type, Wave Type, and Additional Criteria

Support Element	*ASCE 7-22-S2 Section*	*Wave Load*	*Additional Criteria and Notes*
Vertical Piles & Columns	5.4.4.1.1	Nonbreaking Wave Loads	if nondimensional parameter W = $C_M D/(C_D H_{design})$ > 1.0, then the wave load must be calculated using the requirements of Section 5.4.4.2.1. where C_M is: 2.0 for round piles or columns 2.5 for square piles or columns where C_D is: 0.7 for round piles or columns 2.25 for square or rectangular piles or columns
Vertical Piles & Columns	5.4.4.1.2	Breaking Wave Loads	None
Non-elevated Vertical Walls	5.4.4.2.1	Lateral Nonbreaking and Broken Wave Loads	Lateral wave loads must be modified for nonvertical walls and obliquely incident waves as specified in Section 5.4.4.2.3 and Section 5.4.4.2.4, respectively.
Non-elevated Vertical Walls	5.4.4.2.2	Lateral Breaking Wave Loads	Lateral wave loads must be modified for nonvertical walls and obliquely incident waves as specified in Section 5.4.4.2.3 and Section 5.4.4.2.4, respectively. See “CLARIFICATION: Equation 5.4-13, Lateral Breaking Wave Loads on Non-elevated Vertical Walls” textbox.
Non-elevated Non-vertical Walls	5.4.4.2.3	Lateral Breaking Wave Loads	None

Support Element	*ASCE 7-22-S2 Section*	*Wave Load*	*Additional Criteria and Notes*
Walls	5.4.4.2.4	Lateral Breaking Wave Loads from Obliquely Incident Waves	None
Elevated Vertical Walls	5.4.4.2.5	Lateral Wave Loads	None

6.3.2. WAVE UPLIFT FORCES (F_L)

Refer to ASCE 7-22-S2, Section 5.4.4.3 and C5.4.4.3, for further guidance on wave uplift forces.

Per ASCE 7-22-S2, Section 5.4.4.3,

> *Wave uplift forces on elevated structures shall be considered on portions of elevated structures located less than 0.7H_{design} above the design stillwater flood elevation.*
>
> *Wave uplift forces on horizontal overhangs shall be considered when overhangs are positioned within a height of η^* above the design stillwater flood elevation and a solid wall below directs water up against the underside of the overhang.*

Per ASCE 7-22-S2, wave uplift forces shall be considered for:

- Portions of elevated structures located less than 0.7H_{design} above the design stillwater flood elevation.
- Horizontal overhangs positioned within a height of η^* above the design stillwater flood elevation with a solid wall below. Per Section 5.4.4.2.1 of ASCE 7-22-S2, $\eta^* = 1.5H_{design}$.

CLARIFICATION

Wave Uplift Requirements and Equations in ASCE 7-22-S2

ASCE 7-22-S2, Section C5.4.4.3, provides two equations which may be used to calculate wave uplift forces. The equations provided in Commentary are for guidance only and are not governing.

The following guidance is provided for use of the equations in ASCE 7-22-S2, Section C5.4.4.3:

Equation C5.4-13 Guidance:

- C_L is an emperical lift coefficient. Values are not readily defined for most structural components. Values for vertical cylindrical piles are provided in EM 1110-2-1100, *Coastal Engineering Manual* (USACE 2002), Part VI, figure VI-5-141.
- If the lift force calculated by this equation is applied to a vertical pile, the result is a horizontal force perpendicular to pile and flow direction.
- If the lift force calculated by this equation is applied to a horizontal member, the result is an upward, or upward and downward in oscillatory flow, force.
- The magnitude of the flow velocity (u) may be taken as the calculated design velocity (V).
- This equation appears in the *Coastal Engineering Manual*, Part II, as equation VI-5-196 (USACE 2002).

Equation C5.4-14 Guidance:

- The vertical component of the flow velocity (w) is not equal to the calculated design velocity (V).
- This equation appears in the *Coastal Engineering Manual*, Part II, as equation VI-5-197 (USACE 2002).

ADDITIONAL CONSIDERATIONS

Wave Uplift Calculations – Additional Guidance Source

National Research Council (NRC) of Canada published a flood design guide, *Guide for Design of Flood-Resistant Buildings* (NRC 2021), which provides interim guidance for wave uplift loads. Refer to Sections 3.7.4 and 3.9.2.7 of the NRC document for guidance related to the loads presented in Section C5.4.4.3 of ASCE 7-22-S2. As stated in the NRC document, the procedures outlined in Sections 3.7.4 and 3.9.2.7 should be regarded as interim until more appropriate guidance is developed.

Document access: https://nrc-publications.canada.ca/eng/view/object/?id=96b3275c-b731-4fa6-847e-e2a9a0f080d8

7. Scour Depth (S_m) Determination

CLARIFICATION
Scour

Scour occurs when waves and water currents create turbulence around foundation elements, causing localized scouring of soils around those elements. Determining potential scour is critical in designing flood-resistant foundations to ensure that failure does not occur as a result of the loss in either bearing capacity, lateral resistance/capacity, or anchoring resistance around the posts, piles, piers, columns, footings, or walls.

Per ASCE 7-22-S2, Section 5.3.8,

> *Scour shall be considered for surfaces subject to hydrodynamic forces above the non-erodible strata.*
>
> *EXCEPTION: Analysis of scour is not required if soils adjacent to structural foundations are non-erodible or protected against scour by structures designed for anticipated flood loads.*

Per ASCE 7-22-S2, Section C5.3.8,

> *Scour is considered for evaluation of unbraced pile length, pile capacity, and review of undermining of foundations and walls. Unlike the effects of erosion, local scour does not affect the calculation of the design flood depth at a site or increase the wave height.*

7.1. Scour in Coastal Areas

Scour depth in coastal areas must be calculated by utilizing the procedures outlined in ASCE 7-22-S2, Sections 5.3.8.1 and 5.3.8.2. Refer to Table 23 to determine the applicable scour section. Scour is different than erosion, refer to Section 3.2.3 for more information.

CLARIFICATION
Breakaway Walls

Designers of buildings with breakaway walls should calculate scour caused by the breakaway wall before failure. The flood characteristics for breakaway wall design (wave height, stillwater flood depth, etc.) to calculate the failure point of the breakaway wall may be used to calculate the expected scour due to the breakaway wall. The ASCE 7-22-S2 design flood characteristics (e.g., H_{design}) will need to be used to determine the scour caused by the structural, non-breakaway elements (e.g., piles).

Table 23. Scour Depth Applicable Section Selection for Coastal Areas

Support Element	*ASCE 7-22-S2 Section*	*Wave Type*	*Additional Criteria and Notes*
Walls	5.3.8.1.1	Nonbreaking	If, $d_f/b_w < 3$, then element is considered to act as a wall[(a)] If, $S_m > H_{design}\ per\ eq. 5.3-11$, then, $S_m = H_{design}$
Walls	5.3.8.1.2	Breaking	If, $d_f/b_w < 3$, then element is considered to act as a wall[(a)]
Piles & Columns	5.3.8.2	Breaking & Nonbreaking	If, $d_f/b_w < 3$, then element is considered to act as a wall[(a)] If, $s < b_c/2$, then element is considered to act as a wall[(b),(c)] Scour at embedded piles (i.e., buried under pile caps) need only be considered if piles are determined to be exposed, based on scour of walls and pile caps above the piles. Effects of pile groups on scour should be considered. See "Additional Considerations: Calculating Effects of Pile Groups on Scour" text box for guidance. Per ASCE 7-22-S2, Section C.5.3.8.2, ASCE 7-22-S2 is applicable to small-diameter piles where the pile diameter is less than one-tenth of the incident wavelength.

(a) b_w is equal to the lateral dimension of a structural element facing the wave in ft (m)
(b) s is equal to the average clear spacing of column or wall to the adjacent column or wall in ft (m)
(c) b_c is equal to the lateral dimension of an individual pile or column facing the wave in ft (m)

ADDITIONAL CONSIDERATIONS

Calculating Effects of Pile Groups on Scour

Per ASCE 7-22-S2, Section C5.3.8.2,

> *The potential for group effects in scour depth calculation should be evaluated based on available historical records and factors affecting pile scour. An estimate for scour around a*

pile group is given in Chapter 8 of FEMA P-55 (FEMA, 2011), based on field measurements taken after flood events.

Per FEMA P-55 Chapter 8, scour around pile groups may be calculated with **Equation 23** or **Equation 24.**

$S_{TOT} = 6D + 2\ ft$ (if grade beam and/or slab-on-grade present) **Equation 23**

$S_{TOT} = 6D$ (if no grade beam and/or slab-on-grade present) **Equation 24**

where,

S_{TOT} = total localized scour depth (ft)

D = diameter of a round foundation element or the maximum diagonal cross-section dimension for a rectangular element

2 ft = allowance for vertical scour due to presence of grade beam or slab-on-grade

7.2. Scour in Riverine Areas

Per ASCE 7-22-S2, Section C5.3.8,

> *Scour at riverine sites is different from coastal sites and is a common aspect of bridge foundation design. FHWA provides an industry standard resource for evaluating scour at bridges (2012) that can be adapted for scour calculations at riverine sites. Since the FHWA standard is intended for bridges, adaptation of these methods for buildings should be used with caution by the designer.*

CLARIFICATION

Scour in Riverine Areas

Similar to coastal areas, in riverine areas, scour is caused by the velocity of the flood waters. The soil type at the project site influences the extent of the scour. Design professionals should consider the location of the structure with respect to the delineated floodway, or when a floodway is not delineated, to the primary stream channel as velocities can be higher in and near the floodway. When an H&H study is required for buildings located in a floodway or based on concern due to a potentially high flood velocity, FEMA recommends combining the H&H study with an evaluation of the soil to determine whether it is an erodible soil under a design flood condition. This may require consultation with a geotechnical engineer on the soils at the project site and consideration of the design flood velocity.

8. Debris Impact Variables and Loads

Per ASCE 7-22-S2, Section 5.3.9,

> *Risk Category II, III and IV structures shall be designed for debris impact in accordance with this section [5.3.9] where the Design Stillwater Flood Depth (d_f) is greater than 3-ft (0.91-m).*

Per ASCE 7-22-S2, Section 5.3.9.1,

> *Structures within the Flood Hazard Area shall be designed for debris impact loads as determined by Section 5.4.5.1. Debris impact loads shall be considered in any direction and at heights as required per Section 5.4.5.2. Debris impact loads need not be considered on multiple structural elements simultaneously.*
>
> *EXCEPTIONS:*
>
> *1. Where a site-specific study provides flow directions, debris impacts need only be considered from the directions shown in the site-specific study +/-22.5 degrees.*
>
> *2. For riverine sites, debris strikes need only be considered from the upstream direction with a strike direction of +/-22.5 degrees from the primary direction of flow.*
>
> *3. Design for debris impact is not required for detached one- and two-family dwellings.*
>
> *4. Design for debris impact is not required for Risk Category II structures outside of the Special Flood Hazard Area.*

ADDITIONAL CONSIDERATIONS

Debris Loads on Residential Structures in V Zones

Buildings in V Zones must comply with Title 44 of the Code of Federal Regulations (CFR) § 60.3(a)(4). Relative to the provisions of 44 CFR 60.3(e)(4), designs must be engineered and are required to consider all water loads for a base flood event, which would include debris impact loads. FEMA has long established that floodborne debris should be considered when designing for water/flood loads (e.g., FEMA P-55, *Coastal Construction Manual* [FEMA 2011], FEMA P-499, *Home Builder's Guide to Coastal Construction* [FEMA 2010], and FEMA P-550, *Recommended Residential Construction for Coastal Areas* [FEMA 2009]). Based on this requirement, all buildings, including residential, should account for debris loads for flood design for compliance with the minimum requirements of the NFIP.

8.1. Debris Impact Objects

Table 24 summarizes much of the information in ASCE 7-22-S2, Sections 5.4.5.2 and 5.4.5.3 and Tables 5.3-4 and 5.4-4. Designers should consult these sections to understand potential exceptions to Table 24, which may be particularly important for Risk Category III and IV structures. ASCE 7-22-S2, Section 5.4.5.3, provides additional guidance for Risk Category IV structures for Extraordinary Debris Impact.

Table 24. Debris Type Applicability and Minimum Debris Properties
[Based on ASCE 7-22-S2, Sections 5.4.5.2 and 5.4.5.3 and Tables 5.3-4 and 5.4-4]

Debris Type	*Applicable Risk Categories*	*Threshold Depth*[a]	*Impact on columns, piles, bearing walls and transfer beams*	*Impact on non-load bearing elements*[b]	*Minimum debris weight (W_{debris})*[e]	*Minimum elastic debris stiffness (k_e)*[e]	*Impact Area*	*Impact Location*[i]
Passenger Vehicles	II/III/IV	3 ft (0.91 m)	Yes	Yes	2,400 lb (12.455 kN)	72,000 lb/ft (1,051 kN/m)	5 ft wide x 2 ft high (1.5 m x 0.61 m)	Centered 3 ft (0.91 m) above grade to 1 ft (0.3 m) below d_f
Small Vessels[g]	II/III/IV	3 ft (0.91 m)	Yes[c]	Yes[c]	2,500 lb (11.121 kN)	360,000 lb/ft (5,254 kN/m)	4 ft wide x 2 ft high (1.2 m x 0.61 m)	Centered 3 ft (0.91 m) above grade to 3 ft (0.91 m) above d_f
Wood Log/Poles	III/IV	3 ft (0.91 m)	Yes	Yes	1,000 lb (4.448 kN)	4,200,000 lb/ft (61,300 kN/m)	1.5 ft x 1.5 ft (0.46 m x 0.46 m)	Centered 3 ft (0.91 m) above grade to d_f
20 ft Shipping Containers	III/IV	3 ft (0.91 m)	Yes[c]	n/a	5,000 lb (22.241 kN)	2,940,000 lb/ft (42,900 kN/m)	1 ft x 1 ft (0.3 m x 0.3 m)[f]	bottom corner rails, 3 ft (0.91 m) above grade to d_f
40 ft Shipping Containers	III/IV	3 ft (0.91 m)	Yes[c]	n/a	8,400 lb (37.365 kN)	2,040,000 lb/ft (29,800 kN/m)	1 ft x 1 ft (0.3 m x 0.3 m)[f]	bottom corner rails, 3 ft (0.91 m) above grade to d_f

Debris Type	Applicable Risk Categories	Threshold Depth[(a)]	Impact on columns, piles, bearing walls and transfer beams	Impact on non-load bearing elements[(b)]	Minimum debris weight (W_{debris})[(e)]	Minimum elastic debris stiffness (k_e)[(e)]	Impact Area	Impact Location[(i)]
Ships[(h)]/ barges	III/IV	6 ft (1.8 m)	Yes[(c)]	n/a	See ASCE 7-22-S2, Section 5.4.5.2	See ASCE 7 22-S2, Section 5.4.5.2	See ASCE 7-22-S2, Section 5.4.5.2	See ASCE 7-22-S2, Section 5.4.5.2
Extra-ordinary Debris[(c)]	IV	12 ft (3.7 m)	Yes[(c), (d)]	n/a	See ASCE 7-22-S2, Section 5.4.5.3	See ASCE 7-22-S2, Section 5.4.5.3	See ASCE 7-22-S2, Section 5.4.5.3	See ASCE 7-22-S2, Section 5.4.5.3

[(a)] Threshold depth is the minimum design stillwater flood depth required for a debris object to be considered for design.
[(b)] Elements that are part of a dry floodproofing system, including temporary flood barriers, as defined by ASCE 24-14.
[(c)] As required by Section 5.3.9.1.2
[(d)] See Section 5.3.9.1.3 for applicable impacted elements.
[(e)] These values are deemed as reasonable minimum thresholds for design. A designer should always consider local conditions that may result in a higher weight or effective stiffness.
[(f)] These values are specific to the bottom corner rails of shipping containers.
[(g)] Typically 16 to 30 ft (4.9 to 9.1 m) in length, 6 to 8 ft (1.8 to 3.6 m) in width; constructed of aluminum, wood, or fiberglass; are not ocean-going vessels; and limited to 2,500 lbs (11.1 kN).
[(h)] Typically, ocean-going vessels greater than 30 ft (9.1 m) in length that weigh between 2,500 pounds (11.1 kN) and 88,000 lbs (391 kN).
[(i)] Since d_f is a depth and not an elevation, it is recommended that d_f in relation to impact location requirements be taken as d_f + ground elevation to obtain the design stillwater flood elevation. Note that the design stillwater flood elevation is a different elevation than $SWEL_{MRI}$ in coastal areas since d_f includes Δ_{SLR}. See ASCE 7-22-S2 Figures 5.2-1 and 5.2-2.

8.1.1. SITE HAZARD ASSESSMENT

Per ASCE 7-22-S2, Section 5.3.9.1.2,

> *Nearby container yards, ports/harbors, marinas, or other sources shall be evaluated as potential debris origins for shipping containers, ships, small vessels, and barges according to the following procedure.*
>
> *Debris travel from their source location shall be based on a two-step process. Debris shall be assumed to travel over water, beach or open land and shall travel a minimum of a 10,000 ft (3.03-km) radius from the source or until rougher land surface exists. Then, from any point within the initial debris spread area the debris can travel an additional distance into a developed environment in accordance with Table 5.3-5 [Table 25].*

Table 25 outlines the distance debris can travel in a developed environment.

Table 25. Debris Travel in Urban Environment [ASCE 7-22-S2, Table 5.3-5]

Debris Type	*Travel distance in moderate density environment[(a)]*	*Travel distance in heavy density environment[(a)]*
Small vessels	2,000 ft (604 m)	1,000 ft (304 m)
Shipping Containers	2,000 ft (604 m)	1,000 ft (304 m)
Ships/barges	1,000 ft (302 m)	500 ft (152 m)
Extraordinary debris	1,000 ft (302 m)	500 ft (152 m)

(a) Heavy density environments are areas in which the density of structures with a height of at least 75% of the design flood depth is greater than 30% of the plan area within the Flood Hazard Area. All other areas shall be considered moderate density.

ADDITIONAL CONSIDERATIONS

Site Hazard Assessment for Risk Category II Structures

For Risk Category II structures,[4] an alternative to the Site Hazard Assessment is to simply assume the presence of small vessels with the understanding that this could increase the sizing of foundation elements because of the more conservative approach. Assuming the presence of small vessels for a Risk Category II structure enables the design professional to forgo the Site Hazard Assessment described in this section. Note that passenger vehicles are always considered as debris impact objects if the threshold depth is met or exceeded and, therefore, they are not evaluated during the site hazard assessment.

4 Design for debris impact is not required for detached one- and two-family dwellings. Design for debris impact is not required for Risk Category II structures outside of the Special Flood Hazard Area.

CLARIFICATION

Process for Determining Debris Sources Other Than Passenger Vehicles and Wooden Poles

Table 26 provides an outline of a process to determine debris sources other than passenger vehicles and wooden poles. In some instances, consulting a local building official may be advisable to ensure that all appropriate debris sources have been considered. This information should be recorded in the project documentation.

Table 26. Process for Determining Debris Sources Other Than Passenger Vehicles and Wooden Poles[a]

Step	Objective	Method
Step 1	Identify plan area	Draw a circle centered on the site with a 1,000 ft radius.
Step 2	Identify plan area within the Flood Hazard Area	Delineate the areas within the circle that are also within the Flood Hazard Area.
Step 3	Calculate plan area within the Flood Hazard Area	Calculate the area inside of the 1,000 ft radius that is also within the Flood Hazard Area.
Step 4	Identify applicable structures	Identify all buildings that have a height equal to or greater than 75% of the design flood depth.
Step 5	Calculate total area of applicable structures	Sum the areas of the building footprints for the structures identified in Step 4.
Step 6	Divide total area of applicable structures (Step 3) by plan area within the Flood Hazard Area (Step 5)	If result ≥ 0.30, then heavy-density environment. If result < 0.30, then moderate-density environment.
Step 7	Identify debris within land travel distance	Draw two circles centered on the site with radii equal to the travel distances as defined in Table 25 plus the distance from the center of the building to the outer most point of the building.[b] Identify all small vessels and shipping containers within the larger circle and all ships and barges within the smaller circle.[c] Identified items are debris sources unless limited by obstructions as described in Section 5.3.9.1.2 of ASCE 7-22-S2.[d]
Step 8	Identify nearby open land or open water	Identify all locations where either of the two circles cross or encompass open land or open water.

Step	Objective	Method
Step 9	Identify debris within open land or open water travel distance	Draw circles with radii of 10,000 ft at the edges and center of the open land or open water boundaries within the two circles as identified in Step 8. Note that these may be the same for both circles. Identify all small vessels and shipping containers within any of the 10,000 ft radius circles.[c] Identify all ships and barges within the 10,000 ft radius circles associated with the smaller initial circle.[c] Identified items are debris sources unless limited by obstructions as described in Section 5.3.9.1.2 of ASCE 7-22-S2.[d]

(a) Passenger vehicles and wooden poles shall always be considered as debris sources where required by the Risk Category designation. These debris sources are not considered to be location specific.
(b) The building width is added to ensure the debris analysis encompasses the required distance from all edges of the building. This method is conservative, especially for oblong buildings. An alternative method is to offset the perimeter of the building by the required travel distances in Step 7.
(c) Refer to Table 24 for the depth threshold required for each debris type. If the depth threshold is not met, the specified debris is not required to be considered.
(d) Debris impact loads from shipping containers, ships, and barges need not be considered where the flood depth at the structure is less than the draft of the debris object plus 2.0 ft (0.61m), or where the path of the debris object is blocked by a structure or topographic feature that results in inadequate draft.

ASCE 7-22-S2, Section C5.4.5.3, which provides guidance for determining the size of extraordinary debris (large marine vessels), states:

> *The size vessel to be used depends on the most probable size vessel typically present at the port or harbor. The harbormaster or port authority can be consulted to determine typical vessel sizes, ballasted drafts, and weight displacement under ballasted draft. Typical vessel sizes are also provided in PIANC (2014).*

ADDITIONAL CONSIDERATIONS

Recommended Documentation

Because surrounding site conditions are subject to change, project documentation and communication with the client and local officials are critical. FEMA recommends that the designer document all data and methods used in the site assessment and discuss the information with the client for transparency and concurrence. Documentation should include the date the source data were collected (e.g., date of the aerial/satellite image, date of conversations with officials), and the date of the analysis, so that the designer is protected against the appearance of a potential debris source following the analysis.

EXAMPLE

Identifying Potential Sources of Debris

Problem: Identify the potential debris sources for a new Risk Category III building in a moderate-density environment. The site has a stillwater design flood depth (d_f) of 3.5 feet. Based on the building's dimensions, the building's outermost edge will be 50 feet from the center of the building.

Solution: Figure 13 depicts an evaluation of the site and the surrounding area.

Because the building is Risk Category III, the site assessment for debris must include assessment of small vessels, ships, shipping containers, and barges. Each of these debris types may travel 10,000 feet over water, beach, or open land. Per Table 25, in moderate-density environments, small vessels and shipping containers can travel 2,000 feet over land and ships and barges can travel 1,000 feet over land.

Small vessels and ships are located within a 2,000-foot radius of the outer most building edge. Small vessels can travel 2,000 feet in a moderate-density area, so they are an easily identified potential debris source. Ships have a travel distance of 1,000 feet in a moderate-density area and can travel 10,000 feet over water, beach, or open land. Because the 1,000-foot radius is over the water in the ship harbor, it can be concluded that the ships may travel over water toward the shoreline and then inland toward the site, which is less than 1,000 feet away. However, the stillwater design flood depth (d_f) of 3.5 feet excludes the ships as potential debris sources because ships have a depth threshold of 6 feet per Table 24.

Shipping containers and a barge are located within the 10,000-foot radius circles that are centered on the outer edge of the 1,000-foot, site-centered circle. Thus, either object is within travel distance of the site both over water and over land. However, the stillwater design flood depth (d_f) of 3.5 feet excludes the barge as a potential debris source because barges have a depth threshold of 6 feet per Table 24.

Therefore, shipping containers and small vessels must be considered as potential flood debris at the site. Now that the potential debris has been identified, further topographic evaluations can be conducted to determine if either type of debris is limited by depth relative to the debris draft per ASCE 7-22-S2, Section 5.3.9.1.2. Assuming there are no topographic limitations, the Risk Category III building must be designed to withstand the impact forces of small vessels. shipping containers, passenger vehicles, and wooden poles as outlined in Sections 5.3.9 and 5.4.5 in ASCE 7-22-S2.

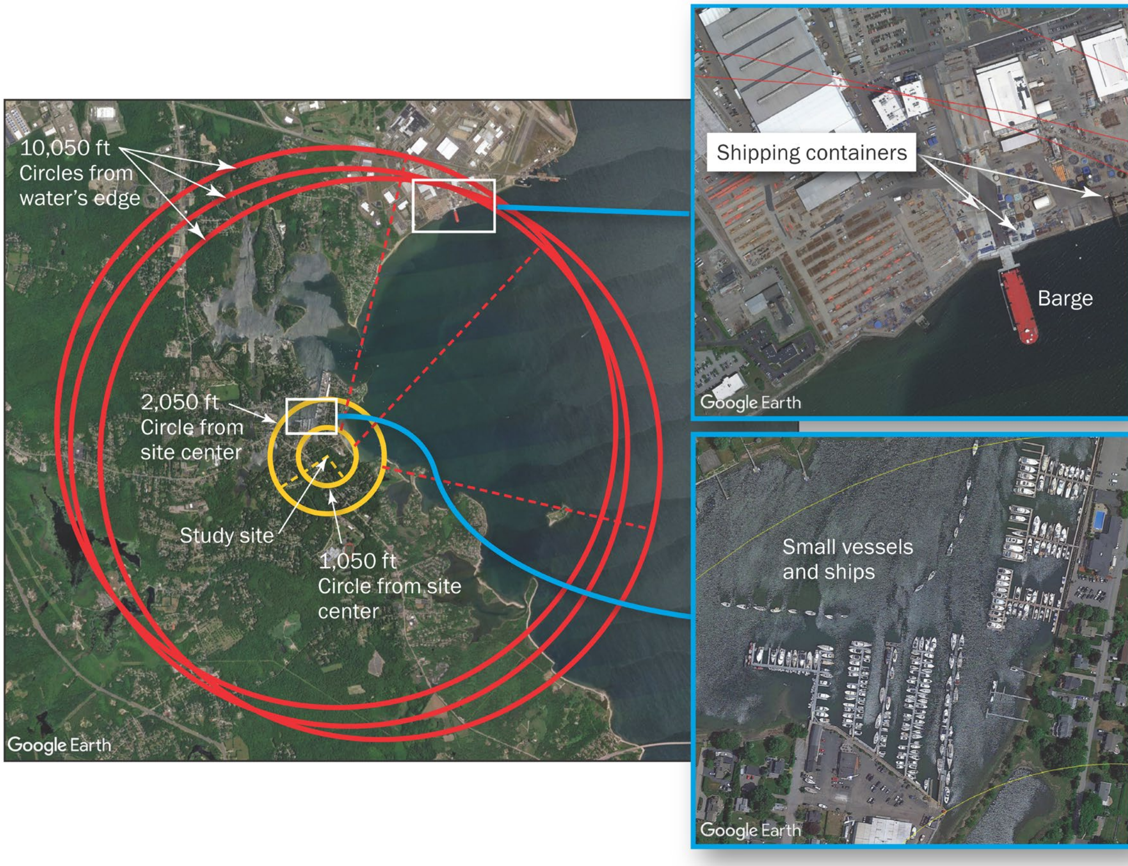


Figure 13. Visual representation of solution for "Example: Identifying Potential Sources of Debris"

8.2. Debris Impact Loads (F_{di})

8.2.1. SIMPLIFIED DEBRIS IMPACT LOAD FOR PASSENGER VEHICLES OR SMALL VESSELS

Per ASCE 7-22-S2, Section 5.4.5.1.1,

> *Debris impact forces for passenger vehicles or small vessels shall be permitted to be determined by applying a static lateral force given by Equation (5.4-19) [***Equation 25***] as the load in lieu of the loads based on the other methods of this section:*

$F_{di} = C_o * 51{,}000$ (lb) **Equation 25** [ASCE 7-22-S2, Eq. 5.4-19]

$F_{di} = C_o * 227$ (kN) **Equation 25 SI** [ASCE 7-22-S2, Eq. 5.4-19 SI]

where,

F_{di} = debris impact force in lb (kN)
C_o = debris orientation coefficient, taken as 0.80

ADDITIONAL CONSIDERATIONS

Simplified Debris Impact Load vs. Elastic Debris Impact Loads

In certain cases, the force derived from the simplified method per **Equation 25** is overly conservative when compared with the elastic debris impact method, which is defined by **Equation 26** in Section 8.2.2. The force resulting from the simplified method is:

F_{di} = 0.8 * 51,000 (lb) = 40,800 lb
F_{di} = 0.8 * 227 (kN) = 181.6 kN

While more complex, the elastic debris impact method may be preferred to avoid imposing overburdening loads on the design. The elastic debris impact method presented in Section 8.2.2 will result in a more accurate impact force and its use is recommended whenever feasible.

Below are examples of loads derived from the elastic debris impact method. For illustration purposes only, these can be compared against the simplified method, which results in a 40,800 lb force.

Example: Passenger Vehicle
Given: d_f = 8 ft, V = 8.0 ft/s, Risk Category II

$F_{di} = 0.8\ V\ C_R\ C_S\ (k_e\ m_{debris})^{0.5}$

F_{di} = 0.8*8.0*1.0*1.0*(72,000*74.5)$^{0.5}$ = 14,823 lb for elastic debris impact method

Example: Small Vessel
Given: d_f = 6 ft, V = 6.9 ft/s, Risk Category II

$F_{di} = 0.8\ V\ C_R\ C_S\ (k_e\ m_{debris})^{0.5}$

F_{di} = 0.8*6.9*1.0*1.0*(360,000*77.6)$^{0.5}$ = 29,176 lb for elastic debris impact method

8.2.2. ELASTIC DEBRIS IMPACT LOADS

Per ASCE 7-22-S2, Section 5.4.5.1.2,

> *Debris impact forces, F_{di} in lbs (kN), are permitted to be calculated using the elastic debris impact method per Equation (5.4-20) [**Equation 26**].*

$F_{di} = C_o\ V\ C_R\ C_s(k_e\ m_{debris})^{0.5}$ **Equation 26** [ASCE 7-22-S2, Eq. 5.4-20]

where,

C_o = debris orientation coefficient, taken as 0.80

C_R = debris depth coefficient taken as 1.0 for design flood depths greater than 5 ft (1.52 m) and taken as 0.0 for design flood depths less than 1 ft (0.3 m). Linear interpolation is permitted between design flood depths of 1 ft (0.3 m) and 5 ft (1.52 m). See "Clarification: Debris Depth Coefficient" text box for additional information.

k_e = effective stiffness of the impacting debris or the effective lateral stiffness of the impacted structural element(s) deformed by the impact, in lb/ft (kN/m) determined in accordance with ASCE 7-22-S2, Section 5.4.5.2. Using the combined elastic stiffness of the debris and the impacted element in series is permissible.

Per ASCE 7-22-S2, Section C5.4.5.1.2,

> *"…The stiffness is typically taken of the debris object assuming the impacted component is completely rigid, however the impacted component stiffness can be significantly less than the debris (in the case of a column or out of plane wall). If desired the lateral stiffness of the component can be included to determine an effective stiffness using the equation below (C5.4-15).*
> $k_{eff} = 1/(1/k_{debris} + 1/k_{structure})$ *[ASCE 7-22-S2, Equation (C5.4-15)]*

C_s = debris velocity stagnation coefficient per ASCE 7-22-S2, Table 5.4-3 [Table 27]. Applicable for non-load bearing elements on the exterior of buildings along the front face of a building wider than 30 ft (9.14-m). Walls must extend from grade to above the design stillwater flood elevation and be designed for the flood loads of this chapter. For load bearing elements C_s shall be taken as 1.0.

EXCEEDING MINIMUMS

Dry Floodproofing and The Debris Velocity Stagnation Coefficient

ASCE 7-22-S2 allows dry floodproofing elements to incorporate the debris velocity stagnation coefficient, C_s, to reduce the debris impact load. To increase the resilience of the building, FEMA recommends that dry floodproofing elements (including all parts of the system such as walls, sealing interfaces, and barriers) be designed to withstand the same debris impact load as the building's load-bearing elements.

Designers should discuss this recommendation with the owner/operator. The decision to implement this recommendation may be weighed against the owner/operator's tolerance for damage, displacement, and downtime.

m_{debris} = mass of the debris (W_{debris}/g) in lb s^2/ft (kg) determined in accordance with ASCE 7-22-S2, Section 5.4.5.2.

V = design flood velocity, in ft/s (m/s)

CLARIFICATION

Determining the Lateral Stiffness of the Impacted Component

ASCE 7-22-S2, Section 5.4.5.1.2, allows the use of the lateral stiffness of the impacted component, $k_{structure}$, to be used in series with the debris object stiffness to determine the effective stiffness. ASCE 7-22-S2, Section C5.4.5.1.2, elaborates on the use of the lateral stiffness of the impacted component. Implementing the lateral stiffness of the impacted component is likely to reduce the effective stiffness. As the effective stiffness, k_{eff}, decreases, the debris impact force, F_{di}, also decreases.

For instances where the lateral stiffness of the impacted component cannot be determined from an analysis model, the following method may be used to determine the lateral stiffness of the impacted component. See the following "Example: Determining Effective Stiffness using the Lateral Stiffness of the Impacted Component" text box for an effective stiffness example in which the lateral stiffness of the impacted component is calculated and implemented.

STEP 1. Identify the maximum beam deflection equation that best represents the impacted component. The diagram selection should consider the impact point load and the end (boundary) conditions. Note: The method presented excludes the consideration of wave loads, uniformly varying hydrostatic water load (if present, e.g., dry floodproofed buildings), and hydrodynamic loads. Wave loads can be excluded because ASCE 7-22-S2, Section 5.5, indicates that the load cases do not consider wave loads and impact loads in the same flood load, F_a, combinations. Hydrostatic and hydrodynamic loads are not considered in order to simplify the solution. This simplification is considered acceptable because the hydrostatic and hydrodynamic loads are constant and would pre-load the exposed columns similarly, whereas the stiffness relative to the impact force is focused on a single column and a non-constant load.

STEP 2. Alter the maximum beam deflection equation to obtain the beam stiffness equation. Below is an example for a beam fixed at one end and supported at the other with a center-point load.

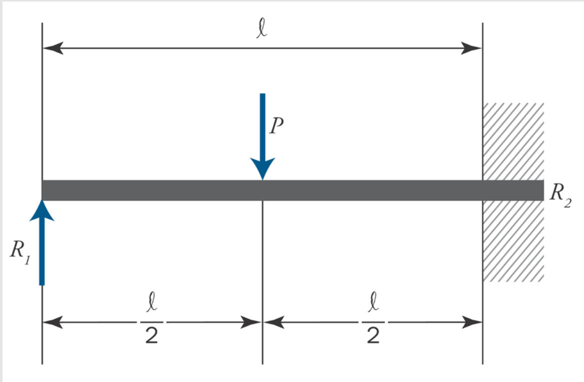


Figure 14. Reaction and load diagram for a beam fixed at one end and supported at the other with a center-point load.

- $y_B = 0.009317PL^3 / EI$

  where,

  y_B = maximum deflection, in

  P = force applied, lb (kN)

  L = length of beam (column), in

  E = Young's modulus (modulus of elasticity), psi (Pa)

  I = moment of inertia, in^4

- $P = k*y_B$

  where,

  k = stiffness, lb/in

- Re-arrange $P = k*y_B$ to be $y_B = P/k$
- Set $y_B = y_B$,
  - $0.009317PL^3 / EI = P/k$
- Solve for k, $1/k = 0.009317L^3 / EI$
  - $k_{structure} = EL / 0.009317L^3$

Note: The stiffness for the impact object, k_{debris}, is given in lb/ft, whereas $k_{structure}$ is typically calculated in lb/in. When $k_{structure}$ is calculated in lb/in, it will need to be converted to lb/ft before combining k_{debris} and $k_{structure}$ with ASCE 7-22-S2, Equation C5.4-15, to obtain k_{eff}.

EXAMPLE

Determining Effective Stiffness Using the Lateral Stiffness of the Impacted Component

Problem:

Part A: Solve for the effective stiffness in the described scenario. The effective stiffness should incorporate the lateral stiffness of the impacted component. The Risk Category II structure is supported by 12x12 (11.25-inch by 11.25-inch nominal) wood piles with a Young’s modulus of 1,500,000 psi and a moment of inertia of 1,335 in^4. The piles have a point of fixity 4 feet below-grade. The lowest horizontal structural member supporting the lowest floor is 8 feet above grade. The site has a calculated design stillwater flood depth of 4 feet and a velocity of 5.7 ft/s. The debris impact object is a small vessel. Per ASCE 7-22-S2, Section 5.4.5.2, and Table 24, small vessels are expected to “strike the structure centered at any elevation from 3 ft (0.91 m) above grade to 3 ft (0.91 m) above the design stillwater flood depth with an impact area of 4 ft (1.2 m) wide by 2 ft (0.61 m) high.” For this example, the small vessel is assumed to impact the structure centered at the design stillwater flood elevation of 4 feet above grade. Selecting an impact location closer to grade would increase the effective stiffness resulting in a more conservative calculation.

Part B: After finding the effective stiffness, solve for the debris impact force, F_{di}, with the elastic debris method and compare it to the debris impact force for the same scenario without using the lateral stiffness of the impacted component. Figure 15 shows the load and support diagram for this problem.

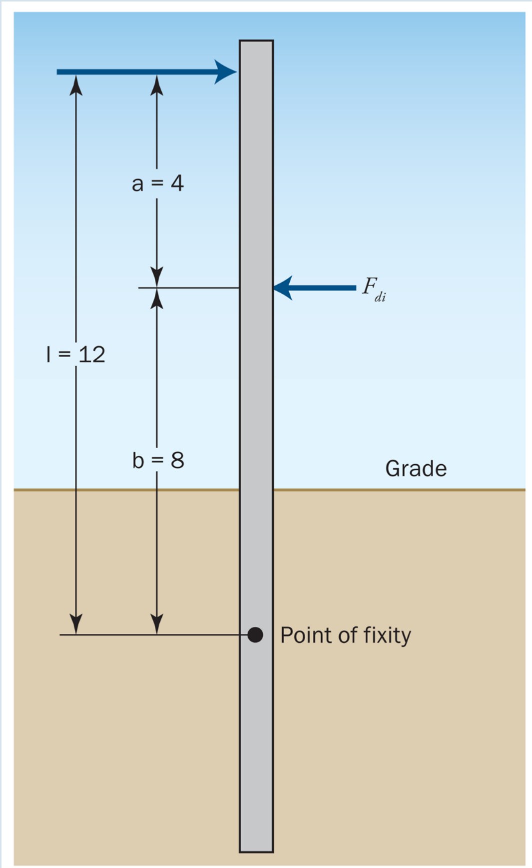


Figure 15. Load and support diagram for effective stiffness example

Given:

E = 1,500,000 psi

I = 1,335 in^4

W_{debris} = 2,500 lb (Table 24)

k_{debris} = 360,000 lb/ft (Table 24)

V = 5.7 ft/s

d_f = 4 ft

C_s = 1.0 (For load bearing elements C_s must be taken as 1.0)

C_R = 0.75

C_o = 0.8

Solution Part A: To determine the impact force applied to a pile, the boundary conditions of fixed at the soil and free at the top of the pile (cantilever condition) or fixed at the soil and pinned of the top of the pile (fixed-pinned) can be used. In both scenarios, the end condition in the soil is assumed to be fixed as deflection and the rotation of the pile is near zero. Where the pile connects to the structure, a pinned support can be assumed for the initial time of impact, as the elevated floor system of the structure and the connection to the structure itself provides some level of support that momentarily may prevent the top of the pile from deflecting, until the impact load can be transferred into the overall structure. If the connection has sufficient capacity to transfer the impact force, the overall structure and system of piles may act as an inverted pendulum, with the pile resembling a cantilever. Utilizing a pinned condition where the pile connects to the structure is conservative, as a free condition at the top of the pile may understate the stiffness and the impact loads. For this example, the pinned condition is used as it is more conservative and is more representative of the load path within the structure. ASCE 7-22-S2 provides the impact locations as areas, to simplify the calculations, this example resolves the load into a centered point load.

Note: The values in the example are rounded and represent outputs derived with unrounded numbers.

Beam deflection equation:

$$y_B\ (for\ a < 0.414 * l) = \frac{Pa(l^2 + a^2)^3}{3EI\ (3l^2 - a^2)^2}$$

Re-arrange the beam deflection equation to obtain $k_{structure}$:

$P = k * y_B$

$$y_B = \frac{P}{k} = \frac{Pa(l^2 + a^2)^3}{3EI\ (3l^2 - a^2)^2}$$

$$k_{structure} = \frac{3EI\ (3l^2 - a^2)^2}{a(l^2 + a^2)^3} = 36{,}717\,\frac{lb}{in}$$

Convert to $k_{structure}$ lb/ft:

$k_{structure}$ = 36,717 lb/in * 12 in/ft = 440,601 lb/ft

Calculate k_{eff}:

k_{eff} = 1/(1/360,000 + 1/440,601) = 198,122

Solution Part B:

Convert W_{debris} to m_{debris}:

m_{debris} = 2,500 lb / 32.2 ft/s^2 = 77.6 lb s^2/ft (slugs, unit of mass)

Calculate F_{di} for using k_{eff} from Part A:

$F_{di} = C_o\ V\ C_R\ C_S(k_e\ m_{debris})^{0.5}$ **Equation 26**

$F_{di} = 0.8*5.7*0.75*1*(198{,}122*77.6)^{0.5}$ = 13,410 lb

Calculate F_{di} without considering the lateral stiffness of the impacted component:

F_{di} = 0.8*5.7*0.75*1*(360,000*77.6)$^{0.5}$ = 18,076 lb

Compare the two F_{di} values:

1 – (13,410 / 18,076) = 0.26

Considering the lateral stiffness of the impacted component results in a debris impact load reduction of 26% for this example.

Table 27. Debris Velocity Stagnation Coefficient, C_s [Derived from ASCE 7-22-S2, Table 5.4-3]

Element Type	*Location on Building*	*Debris velocity stagnation coefficient (C_s)*
Load-bearing elements	All locations	1.00
Non-load-bearing elements	Within a distance of the greater of 0.2*B* or 10 ft (3.05 m) at edges of building	1.00
Non-load-bearing elements	Middle 0.6*B* of building	0.50

B = Overall width of building perpendicular to flow direction, in ft (m)

CLARIFICATION

Debris Depth Coefficient

Per ASCE 7-22-S2, linear interpolation of the debris depth coefficient, C_R, is permitted between design flood depths of 1 feet (0.3 meter) and 5 feet (1.52 meters). See Figure 16.

Note: C_R values are extended to a depth of 1 foot, but application of debris impact loads is only required if the d_f is above 3 feet.

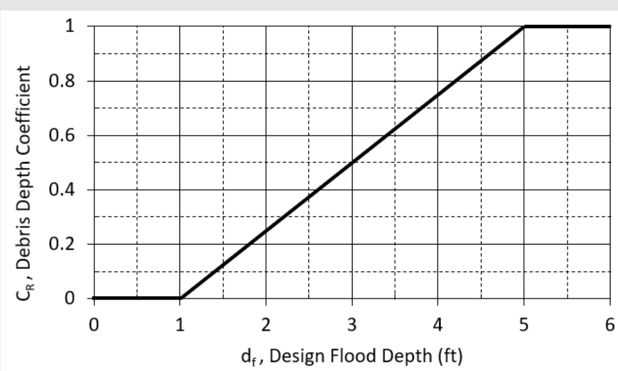


Figure 16. Debris depth coefficient, C_R

ASCE 7-22-S2 permits alternate load path progressive collapse provisions for the reaction of some debris impact loads. Per ASCE 7-22-S2, Section 5.4.5.2,

> *Where debris impacts from shipping containers, ships or barges exceed the capacity of the structural element, it shall be permitted to accommodate the impact through the provisions in Section 5.4.5.4. The provisions of Section 5.4.5.4 may be used for all debris impact types on individual piles.*

The progressive collapse approach is also permitted for extraordinary debris impact. See ASCE 7-22-S2, Section 5.4.5.3, for additional information.

9. Flood Load Combinations

ASCE 7-22-S2, Section 5.5, addresses which load combinations should be considered for coastal flooding and riverine flooding.

Flood loads are designated as F_a in Chapter 2 of ASCE 7-22-S2. Chapter 2 was updated in ASCE 7-22-S2 to reflect the adjustments to Chapter 5 of ASCE 7-22-S2. Incorporated in these changes is the removal of the load factor of 2 that was previously applied to F_a in higher-risk coastal flood zones (V Zones and Coastal A Zones).

Per ASCE 7-22-S2, Section C2.3.2,

> *In ASCE 7-22 Supplement 2, the flood load requirements of Chapter 5 are updated from a 100-year hazard basis, which was developed for the FEMA National Flood Insurance Program (NFIP), to a Risk Category (RC) targeted return period basis (100-year, 500-year, 750-year, and 1,000-year flood for RC I, II, III, and IV, respectively). At the time of the change, longer return period flood maps (specifically 750-year and 1,000-year maps) are not available for all communities. Therefore, a scale factor for the flood loading is provided in Section 5.3, which approximates the longer return period floods from the 100-year mapped flood. These scale factors were established based on data from simulated storms impacting the Gulf and Atlantic coasts based on USACE mapping projects (Ref. Section C5.3). This change in approach, along with revised loading equations, is a significant departure from previous versions of ASCE 7. With this change, the load factor of 2.0 previously used for coastal flood hazard areas, with significant uncertainty relative to the base flood, is no longer necessary. Based on the increased return period and the inherent conservatism in some of the equations of Chapter 5, a load factor of 1.0 is appropriate for LRFD design. Similar to other hazards, the ASD load factor is taken as 0.7 (1/1.5 rounded to one significant digit) of the LRFD load factor.*

Given this change, load combinations 4b and 5b in ASCE 7-22-S2, Section 2.3.2, and 5b, 6b, and 7b in ASCE 7-22-S2, Section, 2.4.2, have been consolidated into single sets of load combinations. See ASCE 7-22-S2, Sections 2.3.2 and 2.4.2, for exceptions and additional information.

The symbols used in the load combinations are:

D = Dead Load
F_a = Flood load as defined by ASCE 7-22-S2, Section 5.5
L = Live load
R = Rain load
S = Snow Load
W = Wind Load

Per ASCE 7-22-S2, Section 2.3.2, the applicable load combinations when a structure is located in a Flood Hazard Area are as follows:

- 4b. $1.2D + 1.0W + 1.0F_a + 1.0L + (0.5L_r$ or $0.3S$ or $0.5R)$
- 5b. $0.9D + 0.5W + 1.0F_a$

Per ASCE 7-22-S2, Section 2.4.2, the applicable load combinations when a structure is located in a Flood Hazard Area are as follows:

- 5b. $D + 0.6W + 0.7F_a$
- 6b. $D + 0.75L + 0.75(0.6W) + 0.75(L_r$ or $0.7S$ or $R) + 0.7F_a$
- 7b $0.6D + 0.6W + 0.7F_a$

ASCE 7-22-S2, Section 5.5, also includes two additional stability checks that should be conducted in addition to the load combinations provided in Chapter 2.

- Section 5.5.1 is a stability check against uplift (Commentary is provided in C5.5.1)
- Section 5.5.2 is a stability check against sliding (Commentary is provided in C5.5.2)

10. Coastal Flood Design Example

The intent of the coastal flood design example is to provide flood load calculations based on ASCE 7-22-S2 using realistic flood data with fictitious structures. The examples do no incorporate jurisdictional requirements in the calculations; however, the examples may demonstrate best practices to minimize flood risk to structures. References made throughout these examples are to ASCE 7-22-S2. All elevation data provided in the examples are provided in NAVD 88.

The structure is a Risk Category II, non-residential, structure located in the town of Topsail Beach, NC. The structure is a two-story, flat roof, wood-framed structure, with a deep open foundation. The building is approximately 40 feet by 40 feet with a gross building area of 1,600 square feet. supported by 18-inch-diameter concrete drilled piers with cast-in-place columns above grade. The structure is located in the Special Flood Hazard Area (SFHA) in VE Zone (see Figure 17) with a BFE of 15 feet NAVD 88 (see Figure 18.). The owner has requested that the designer use the minimum required project lifecycle of 50 years. Additionally, the owner requested that the building be elevated to account for future conditions and consistent with the wave crest elevations based on the wave height calculated in accordance with ASCE 7-22-S2.

Figure 17. Coastal FIRM for project location

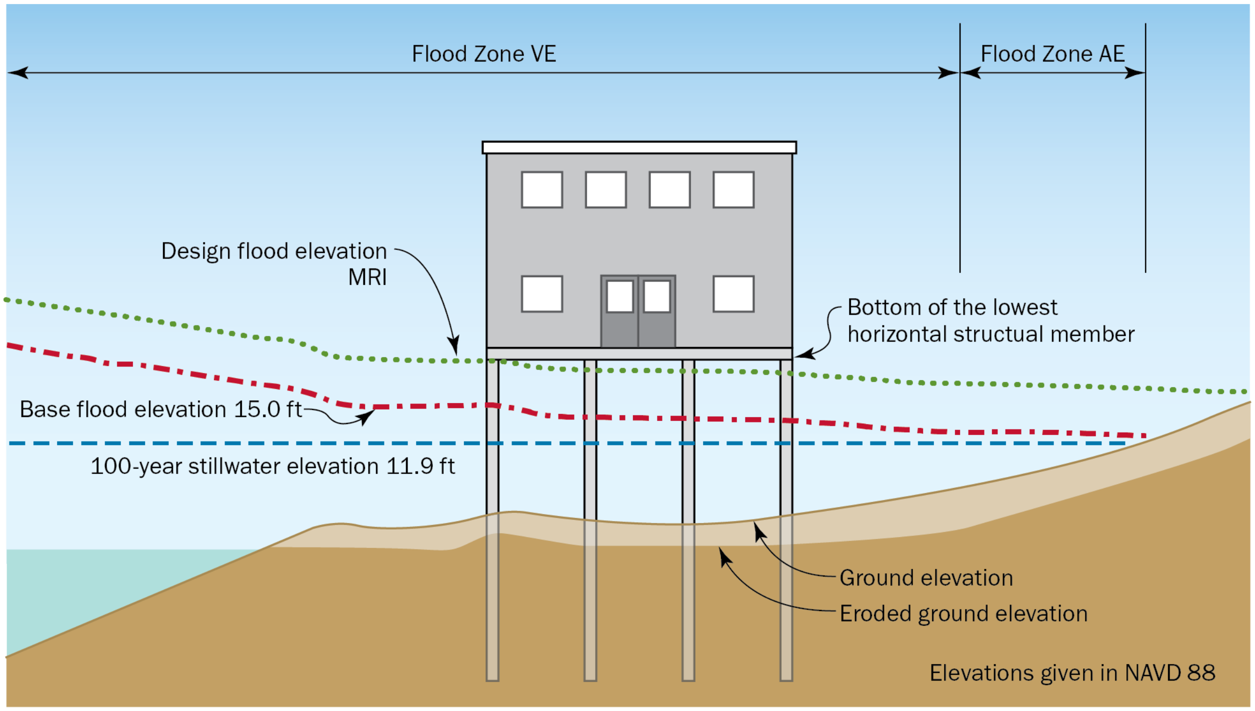


Figure 18. Coastal example building site and flood conditions

Step 1) Determine Design Stillwater Depth

The Risk Category II building requires that a 500-year MRI be considered per Table 5. Per Figure 19, which depicts a table extracted from the FIS associated with the site, the stillwater elevation (SWEL) for a 500-year MRI (0.2% annual chance) is 12.5 feet. If both the $SWEL_{100}$ and $SWEL_{500}$ are available, the $SWEL_{500}$ should be used to determine the $SWEL_{MRI}$ unless the design MRI is 100 years. The owner wants the designer to consider an RSLC rate above the historic rate of rise and use an annual rate of relative sea level change (SLR_A) of 0.04 ft/yr.

Table 20: Coastal Transect Parameters

Flooding Source	Transect No.	Transect Location	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance	Wave Runup Analysis: Zone Designation and BFE in feet NAVD 88	Wave Height Analysis: Zone Designation and BFE in feet NAVD 88	Primary Frontal Dune Identified
Atlantic Ocean	13	Approximately 700 feet southwest of the intersection of Darden Avenue and Ocean Boulevard	5.1	8.5	11.9	12.5	N/A	VE 12-19 AE 12-13	Yes
Topsail Sound			*	*	9.3	*	N/A	VE 11-12 AE 9-11	N/A
Atlantic Ocean	14	Approximately 700 feet northeast of the intersection of Scott Avenue and Ocean Boulevard	5.1	8.5	11.9	12.5	N/A	VE 13-19 AE 12-13	Yes
			5.1	8.5	9.8	12.5	N/A	AE 10-12	N/A
Topsail Sound			*	*	9.0	*	N/A	VE 11-12 AE 9-11	N/A
Atlantic Ocean	15	Approximately 600 feet northeast of the intersection of Flake Avenue and Ocean Boulevard	5.1	8.5	11.9	12.5	N/A	VE 14-19 AE 11-14	Yes
			5.1	8.5	9.8	12.5	N/A	AE 10-11	N/A
Topsail Sound			*	*	8.8	*	N/A	VE 11-12 AE 9-11	N/A

Figure 19. Coastal transect data for the example building site, 10% through 0.2% annual-chance elevations represent the stillwater elevations

ADDITIONAL CONSIDERATIONS

Coastal Erosion

See Appendix A of this design guide for explanation of coastal erosion methodologies demonstrated in this example.

Determine Average Annual Shoreline Retreat (S_{RA})

- S_{RA} = 2.3 ft/yr (0.7 m/yr) landward per short-term change rates of shorelines near the site available per the U.S. Geological Survey's (USGS's) Coastal Change Hazards Portal, see Figure 20 for a depiction of the portal for this site.

ADDITIONAL CONSIDERATIONS

Annual Shoreline Retreat

National resources are demonstrated in the example and jurisdictional requirements and data are not used. Designers should consider state and local erosion rates when they are available. In some instances, the use of the state or local erosion rates may be required per local ordinances and codes. If multiple erosion rates are available, it is recommended that the higher erosion rate be used.

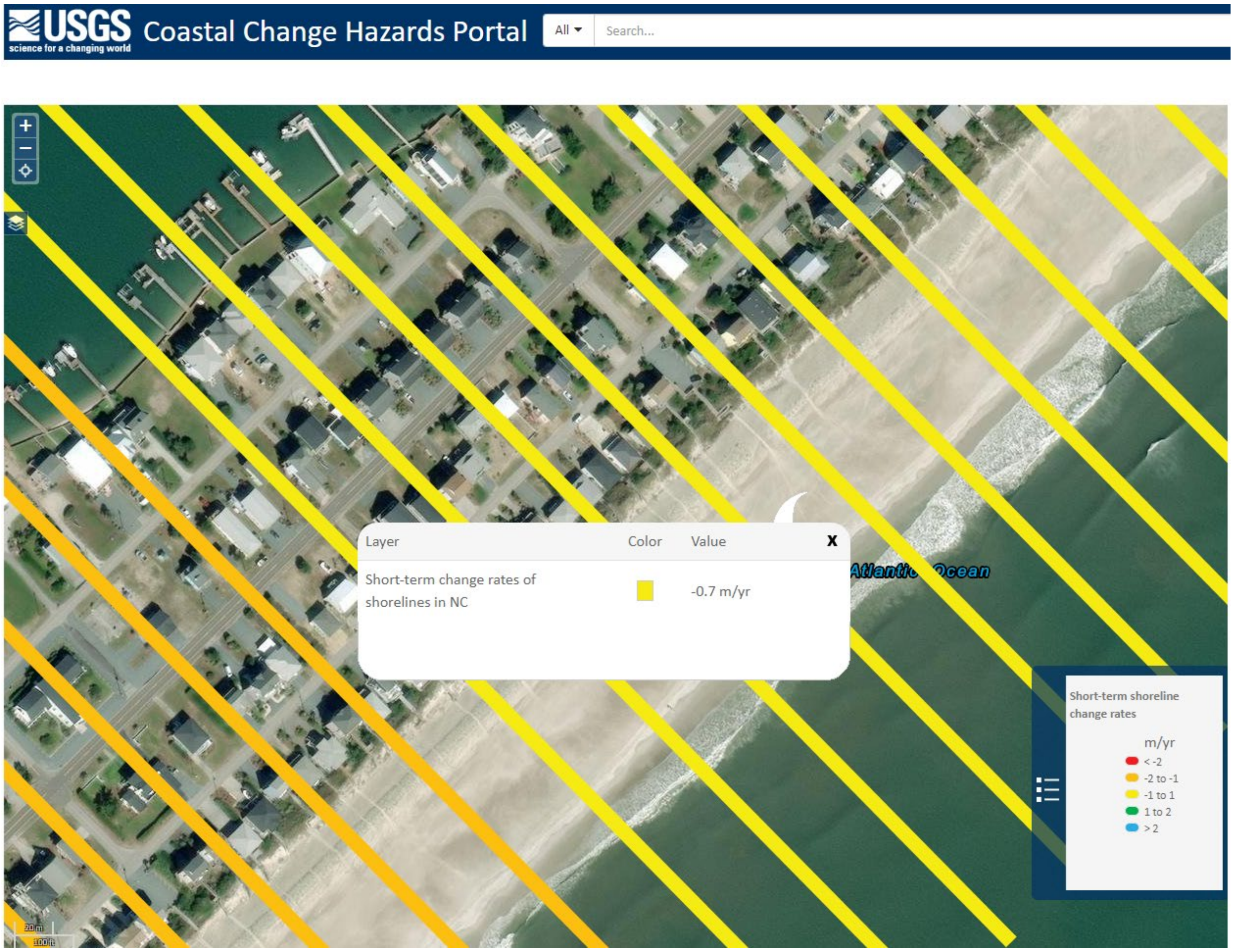


Figure 20. USGS's Coastal Change Hazards Portal short-term shoreline change rate for site

<u>Calculate Total Expected Shoreline Retreat (S_{RTOT})</u>

The total expected shoreline retreat is based on the average annual shoreline retreat adjusted for the minimum required project lifecycle of 50 years. The average annual shoreline retreat rate is multiplied by the project lifecycle to determine the total expected shoreline retreat over the life of the project. The beach and dune profiles are translated landward by the total expected shoreline retreat.

- $S_{RTOT} = S_{RA} * PL$ = 2.3 ft/yr * 50 years = 115 ft

  PL = the project lifecycle in years

<u>Obtain an Elevation Profile from the Shoreline through the Project Site</u>

As discussed in the "Additional Considerations: Elevation Profiles" text box, there are various approaches for obtaining an elevation profile through the project site. The dune profile in Figure 21 was developed using project site survey information.

- Dune profile is shown in Figure 21.

- Dune Toe = 7.2 ft
- Dune Max Height = 14 ft
- Dune Heel = 11.2 ft

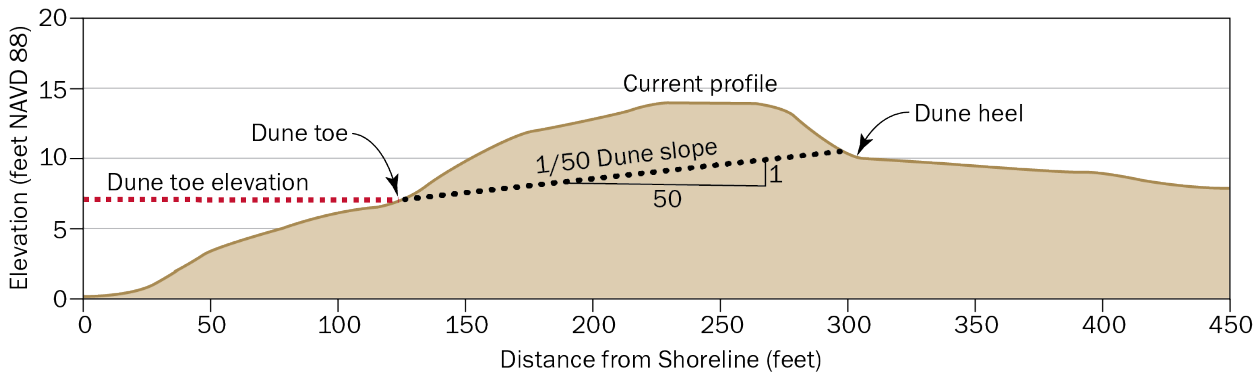


Figure 21. Elevation profile depicting the dune toe and dune heel

<u>Determine Stillwater Elevation for the Required MRI ($SWEL_{MRI}$)</u>

- The SWEL at the site for a 500-year MRI is defined as 12.5 feet by the FIS adopted by the AHJ.

- $SWEL_{MRI}$ = 12.5 ft

- Note: If MRI data are not available, refer to Section 3.2.1 "Calculating Stillwater Elevation ($SWEL_{MRI}$) When MRI Data Are Not Available."

<u>Calculate Δ_{SLR}</u>

- $\Delta_{SLR} = SLR_A * PL$ = 0.04 ft/yr * 50 years = 2 ft

<u>Determine Design Stillwater Flood Elevation ($SWEL_{design}$)</u>

- $SWEL_{design} = SWEL_{MRI} + \Delta_{SLR}$ = 12.5 ft + 2 ft = 14.5 ft

<u>Calculate dune reservoir cross-section</u>

- The design stillwater flood elevation of 14.5 feet exceeds the maximum dune crest height of 14 ft. Calculation of the dune reservoir cross-section is not required based on the existing project conditions.

<u>Determine if Dune Retreat or Removal Treatment is Required</u>

- Per Table 31 (see Appendix A), a minimum of 1030 square feet of reservoir cross-section is required for dune survival for a 500-year MRI. Since the design stillwater flood elevation exceeds the maximum dune crest height, the reservoir cross-section is 0 and dune removal treatment must be implemented.

Apply Appropriate Dune Treatment and Adjust the Elevation Profile Based on Shoreline Retreat

- For dune removal treatment, the dune area above the 1/50 slope is removed.
- For shoreline retreat, the eroded elevation profile is advanced landward by the total expected shoreline retreat (S_{RTOT}) distance of 115 feet.
- The dune treatment and shoreline retreat are shown in Figure 22.

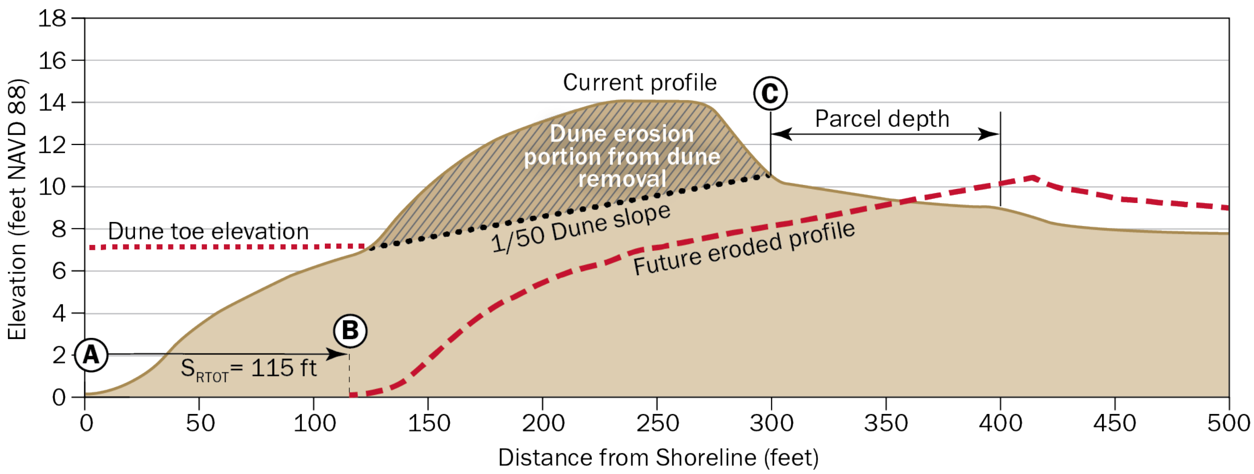


Figure 22. Current elevation profile and shoreline retreat profile with dune erosion for Coastal Flood Design Example; (A) original shoreline station, (B) future shoreline station, (C) seaward extent of the site

Determine Future Minimum Ground Elevation (G_e)

- The future minimum eroded ground elevation (G_e) is determined by locating the minimum ground elevation at the site.
- As shown in Figure 23, G_e is equal to 8 feet for this project site.

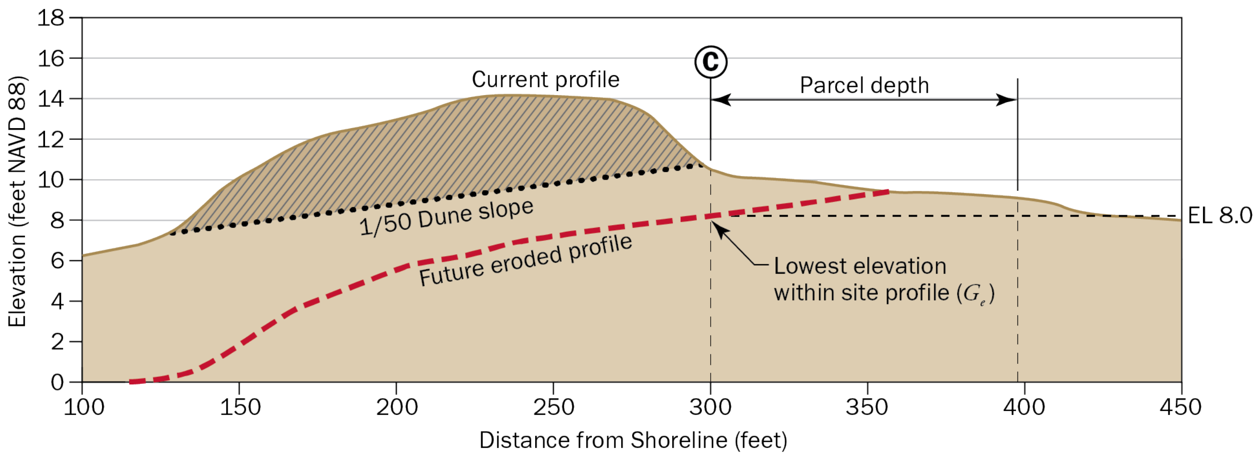


Figure 23. Elevation profile denoting the lowest elevation, of either the current or future profile, within the site profile

<u>Design Stillwater Flood Depth</u>

$d_f = (SWEL_{MRI} - G_e) + \Delta_{SLR} = (12.5ft - 8\ ft) + 2\ ft = 6.5\ ft$ **Equation 4** [ASCE 7-22-S2, Eq. 5.3-1]

Step 2) Determine Design Velocity

In this step, the design velocity, which is an important surge characteristic when analyzing fluid-structure interaction, is calculated.

$V=0.5*\sqrt{(g*df)} = 0.5*\sqrt{(32.2\ ft/s^2 *6.5\ ft)} = 7.2\ ft/s$

$V_{MAX} = C_{VMAX} *10\ ft/s = 1.35 * 10\ ft/s = 13.5\ ft/s$

C_{VMAX} = 1.35 for Risk Category II structure, see Table 13

If $V \leq V_{MAX}$, then, V=V, V calculated previously = 7.2 ft/s

Step 3) Determine Design Wave Height and Type of Wave Determinations

The method selected to determine wave height is per Branch 3 detailed in Section 6.1.6. The Branch 3 methodology is based on depth-limited breaking wave assumptions.

<u>Calculate breaking wave height (H_b)</u>

$H_b = 0.78*d_f = 0.78 * 6.5\ ft = 5.1\ ft$

<u>Set the design wave height (H_{design}) equal to the breaking wave height (H_b)</u>

$H_{design} = H_b = 5.1\ ft$

<u>Optional Step: Determine Design Flood Elevation MRI (DFE_{MRI})</u>

As discussed in the main body of this design guide, ASCE 7-22-S2 is not meant to regulate elevation. However, elevating or protecting to the higher of the regulatory DFE or the DFE_{MRI}, where the DFE_{MRI} is based on the calculations herein and in ASCE 7-22-S2, is recommended for consideration.

$DFE_{MRI} = d_f + G_e + 0.7H_{design}$ **Equation 5** [ASCE 7-22-S2, Eq. C5.3-1]

$DFE_{MRI} = 6.5 + 8 + 0.7 * 5.1 = 18.1$ ft

Compare the DFE_{MRI} with the regulatory DFE.

DFE = BFE + freeboard

freeboard = 2 ft, note that freeboard varies by location and is determined by local building codes and floodplain management regulations

BFE = 15 ft [determined from FIRM]

DFE = 15 + 2 = 17 ft

DFE_{MRI} > DFE, therefore, FEMA recommends consideration of elevating to the DFE_{MRI} of 18.1 ft.

Step 4) Determine Scour Around Piles

Step 4a) Find scour around individual piles

Scour around embedded columns (or piles) needs to be considered if the columns (or piles) are exposed. Scour is considered for evaluation of unbraced column length, column capacity, and review of undermining of foundations and walls. Unlike the effects of erosion, local scour does not affect the calculation of the design flood depth at a site or increase the wave height.

Determine applicable scour calculation method:

Per ASCE 7-22-S2, Section 5.3.8.1, a structural element with a ratio of the design stillwater flood depth to the lateral dimension facing the wave less than 3 shall be considered to act as a wall.

Check, If, $d_f/b_w < 3$

where,

b_w = 1.5 ft – the lateral dimension of a structural element facing the wave

d_f = 6.5 ft – the design stillwater flood depth

6.5 / 1.5 = 4.3 – which is greater than 3; thus, the element can be considered to act as a column for scour calculation purposes.

Per ASCE 7-22-S2, Section 5.3.8.1, tightly spaced columns (or piles) shall be considered to act as a wall. If $s < b_c / 2$, then the column (or pile) is considered to act as a wall per ASCE 7-22-S2, Section 5.3.8.1.

where,

s = 8.1 ft — the clear distance between columns or piles

b_c = 1.5 ft — the lateral dimension of an individual column or pile facing the wave

8.1 ft > 1.5 ft / 2; thus, the column can be considered to act as a column.

Per ASCE 7-22-S2, Section C5.3.8.2, Section 5.3.8.2, Scour at Vertical Piles and Columns, is applicable to small-diameter piles where the pile diameter is less than one-tenth of the incident wavelength. Thus, the wavelength needs to be calculated to ensure ASCE 7-22-S2, Section 5.3.8.2, is applicable.

The wavelength, L, in ft (m) must be calculated by ASCE 7-22-S2, Eq. 5.3-10.

$$L = \frac{gT_p{}^2}{2\pi}\left(1 - e^{-\left(\frac{2\pi}{Tp}\sqrt{\frac{d_f}{g}}\right)^{\frac{5}{2}}}\right)^{\frac{2}{5}} = \frac{32.2\frac{ft}{s^2} * 4.82^2\ sec^2}{2\pi}\left(1 - e^{-\left(\frac{2\pi}{4.82sec}\sqrt{\frac{6.5ft}{32.2\frac{ft}{s^2}}}\right)^{\frac{5}{2}}}\right)^{\frac{2}{5}} = 66.2\ ft$$

where,

$$T_p = C_T\sqrt{\frac{H_{design}}{g}} = 12.1\sqrt{\frac{5.1\,ft}{32.2\,\frac{ft}{s^2}}} = 4.82\ seconds$$ [ASCE 7-22-S2, Eq. 5.3-9]

where,

T_p = wave period corresponding to the wave height, in sec (s)

C_T = 12.1 — wave period coefficient

g = 32.2 ft/s^2 — acceleration due to gravity

<u>Check whether the pile diameter, D, is less than one-tenth of the incident wavelength, L.</u>

1/10 * L = 6.6 ft

D = 1.5 ft < L = 6.6 ft; thus, ASCE 7-22-S2 can be used to calculate scour at vertical piles or columns.

<u>Calculate Scour</u>

S_m = 2.0*D = 2.0 * (18inches/(12in/ft)) = 3 ft [ASCE 7-22-S2, Eq. 5.3-13]

D = 18 inches / (12in/ft) = 1.5 ft — pile or column diameter, in ft (m) for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in ft (m).

Step 4b) Find scour effects around pile groups (recommended by ASCE 7-22-S2, but not required)

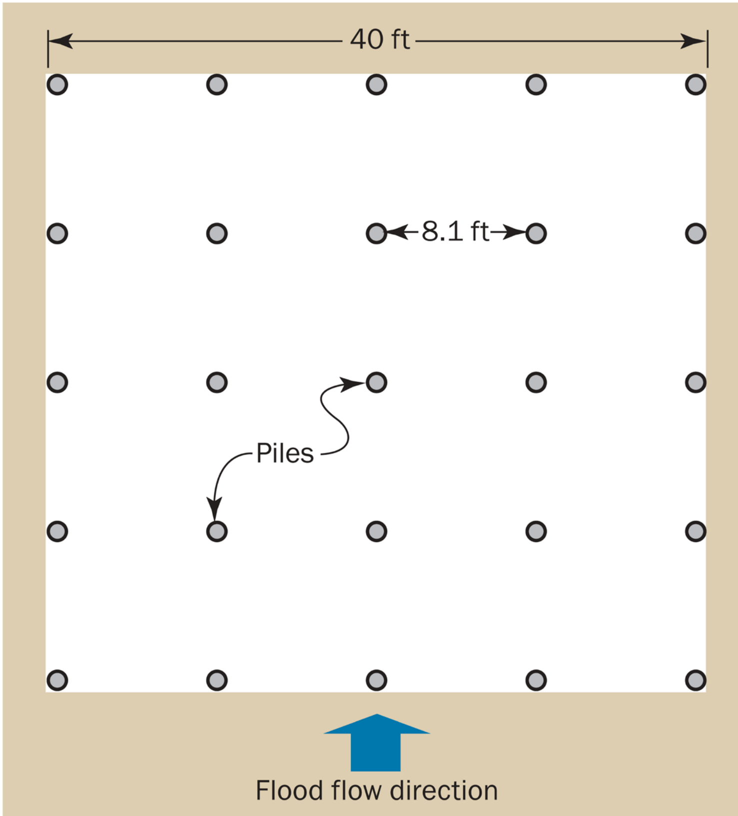
Per ASCE 7-22-S2, Section C5.3.8.2, pile group effects in scour depth may be considered.

Equation 24 provides a method for calculating the total localized scour depth (ft), S_{TOT}, around pile groups where no grade beam and/or slab-on-grade is present.

$$S_{TOT} = 6D = 6 * 1.5\ ft = 9\ ft$$ **Equation 24**

Step 5) Determining Drag Loads on Vertical Columns (Hydrodynamic)

Debris damming considerations are required for Risk Category II, III, and IV structures where the design stillwater flood depth (d_f) is greater than 3 feet (0.91 meters) and the average clear spacing (s) is less than 30 feet. So, debris damming must be considered in drag load calculations for this example. The foundation layout is shown in Figure 24 and there is no enclosed area beneath the structure.



Plan view foundation

Figure 24. Foundation plan for coastal example

Given:

ρ = 1.99 lb s^2/ft^4 [for saltwater]

V = 7.2 ft/s

h = 6.5 ft

s = 8.1 ft [For corner columns, one-half the clear distance must be used]

b = 1.5 ft

<u>Find drag force on individual components (vertical columns):</u>

Drag force on components is calculated with ASCE 7-22-S2, Equation 5.4-4.

$F_{drag} = (1/2)\ \rho\ C_d\ V^2\ h\ (b+C_{cx}\ s)$ ASCE 7-22-S2, Equation 5.4-4

C_d = 2.0 [ASCE 7-22-S2, Table 5.4-1, Structural components with debris damming per Section 5.3.9.2]

C_{cx} = 0.70 [ASCE 7-22-S2 Figure 5.3-1, for s = 8.1 ft]

b = b_c = 1.5 ft

Step 5a) Find drag force on corner columns:

F_{drag} = 0.5 * *1.99 lb s²/ft⁴* * 2.0* (7.2 ft/s)² **6.5 ft (1.5 ft+ (0.7 * 8.1 ft * 0.5))* = ***2,907 lbs***

Step 5b) Find drag force on internal (non-corner) columns:

F_{drag} = 0.5 * *1.99 lb s²/ft⁴* * 2.0* (7.2 ft/s)² **6.5 ft (1.5 ft+ (0.7 * 8.1 ft))* = ***4,808 lbs***

Find drag force on lateral force resisting system:

Step 5c) Find drag force due to debris damming:

Portion of **Equation 13**: $(1/2)\ \rho\ C_d\ V^2\ h\ (n_d\ b_c + C_{cx}\ s_L)$

F_{drag} = (1/2) * *1.99 lb s²/ft⁴* * 2 * (7.2 ft/s)² * 6.5 ft (5 * 1.5 ft + 0.7 * 32.5 ft) = **20,284 lbs**

C_d = 2 [Table 5.4-1, Structural Components with debris damming]

C_{cx} = 0.70 [Figure 5.3-1, for s = 8.1 ft]

s_L = 32.5 ft [2 bays = 17.75 ft; therefore, select 50 ft minimum or building width. Building width (40 ft) is less than 50 ft, so, select building width. Subtract $n_d\ b_c$ from 40 ft to obtain clear spacing]

n_d = 5 [minimum number of columns covered by debris using the 50 ft minimum for s_L]

Step 5d) Find drag force on exposed piles (piles without debris damming):

Portion of **Equation 13**: $(1/2)\ \rho\ C_d\ V^2\ b_c\ n\ h$

F_{drag}= (1/2) * *1.99 lb s²/ft⁴* * 1.2 * (7.2 ft/s)² * 1.5 ft * 20 * 6.5 ft = **12,070 lbs**

C_d = 1.2 [Table 5.4-1, Round column]

n = 25 (total piles) – 5 (piles considered for debris damming) = 20 piles

Step 5e) Find total drag force on lateral force resisting system:

F_{drag} = 20,284 + 12,070 = **32,354 lbs**

Step 6) Determine Breaking Wave Load

Use Table 21 to determine whether the columns can be analyzed as columns or if they need to be considered walls.

d_f/w = 6.5 ft / 1.5 ft = 4.3

- $d_f/w > 3$; therefore, use pile or column equations (ASCE 7-22-S2, Section 5.4.4.1)

$w/2$ = 1.5 ft / 2 = 0.75 ft

- $w/2 < s$, *where* s= 8.1 ft; therefore, use pile or column equations (ASCE 7-22-S2, Section 5.4.4.1)

Because both criteria checks establish that the columns may be analyzed as columns, the breaking wave loads on vertical columns are to be calculated in accordance with ASCE 7-22-S2, Section 5.4.4.1.2.

$F_{bw} = \phi_m C_{bw} \gamma_w H_{design}^2 D$ [ASCE 7-22-S2, Eq. *5.4-7]*

F_{bw} = 0.5 * 1.75 * 64 lb/ft^3* (5.1ft)^2 * 1.5 ft = **2,185 lbs**

where,

ϕ_m = 0.5 — force coefficient, taken as 0.5 for round or square piles and round or square columns.

C_{bw} = 1.75 — coefficient for breaking waves, shall be taken as 1.75 for round piles or columns and shall be taken as 2.25 for square or rectangular piles or columns.

H_{design} = 5.1 ft — design wave height in ft (m) as defined in ASCE 7-22-S2, Section 5.3.7.

γ_w = 64 lb/ft^3 — specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater

D = 18-inches = 1.5 ft — pile or column diameter, in ft (m) for circular sections, or the largest projected width of the pile or column in ft (length of plan diagonal) (m) for a square or rectangular pile or column.

Lateral Breaking Wave Loads on Non-Elevated Vertical Walls

Lateral breaking wave loads on non-elevated vertical walls are not required for completion of this example but are included to provide an example of the methods required for such loads. Breaking waves are considered for this example because the wave height was derived under the breaking wave height assumption outlined in Branch 3 of Figure 9. Each site is unique and the designer will need to follow the procedures outlined in ASCE 7-22-S2 and herein to determine whether breaking or non-breaking waves should be used for load calculations.

The lateral breaking wave force per unit length on a vertical wall, F_{BRK} in lb/ft (kN/m) must be calculated by ASCE 7-22-S2, Eq. 5.4-13.

$$F_{BRK} = \begin{cases} \left(\frac{1}{2}(p_{1B} + p_2)h_c + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* > h_c \\ \left(\frac{1}{2}p_{1B}(\eta^*) + \frac{1}{2}(p_{1B} + p_3)d_f\right) & \text{for } \eta^* \le h_c \end{cases}$$ [ASCE 7-22-S2, Eq. 5.4-13]

p_{1B} = the breaking wave pressure acting at the design stillwater flood elevation in lb/ft^2 (kN/m^2) calculated by ASCE 7-22-S2, Eq. (5.4-14).

p_2 = pressure at the top of the vertical wall or structure, in lb/ft^2 (kN/m^2)

p_3 = pressure at the eroded ground elevation, in lb/ft^2 (kN/m^2)

h_c = height to the top of the vertical wall or structure above the design stillwater flood elevation, in ft (m)

η*= height in ft (m), measured above the design stillwater flood elevation, below which the wave pressure is assumed to act, or the minimum height at which the wave pressure equals zero

H_{design} = design wave height as defined in ASCE 7-22-S2, Section 5.3.7, in ft (m)

d_f =design stillwater flood depth as defined in ASCE 7-22-S2, Section 5.2, in ft (m)

<u>Determine applicable F_{BRK} equation:</u>

h_c = building height – d_f = 48.5 – 6.5 = 42 ft
*η^* = 1.5H_{design} = 1.5 * 5.1 ft = 7.65 ft above the design stillwater flood elevation [ASCE 7-22-S2 Eq. 5.4-8]*

$\eta^* < h_c$, therefore, F_{BRK} = $\left(\frac{1}{2}p_{1B}(\eta^*) + \frac{1}{2}(p_{1B} + p_3)d_f\right)$

p_{1B}= [0.6 + 0.5 $\left(\frac{4\pi d_f/L}{sinh\ (\pi 4 d_f/L)}\right)^2$ *+α*]$\gamma_w H_{design}$* [ASCE 7-22-S2, Eq. 5.4-14]

In Equation 5.4-13, p_2, p_3, and η^* are calculated as prescribed in ASCE 7-22-S2, Section 5.4.4.2.1, and the parameter $\left(\frac{4\pi d_f/L}{sinh\ (4\pi d_f/L)}\right)^2$ may be conservatively taken as 1. For this example, $\left(\frac{4\pi d_f/L}{sinh\ (4\pi d_f/L)}\right)^2$ will be calculated and not taken as 1 in order to provide the user an option to calculate a simple comparison between the example loads if $\left(\frac{4\pi d_f/L}{sinh\ (4\pi d_f/L)}\right)^2$ were conservatively taken as 1.

$$\left(\frac{4\pi d_f/L}{sinh\ (4\pi d_f/L)}\right)^2 = \left(\frac{4*\pi*6.5/66.2}{sinh\ (4*\pi*6.5/66.2)}\right)^2 = 0.616$$

*α**= impulsive wave pressure coefficient, which shall be taken as 0.8
L = 66.2 ft, wavelength as specified in ASCE 7-22-S2, Section 5.3.7, in ft (m)
H_{design} = H_b = 5.1 ft
γ_w = 64 lb/ft^3 for *saltwater*
d_f = 6.5 ft

*p_{1B}= [0.6 + 0.5 (0.616)+0.8]*64 lb/ft^3 * 5.1ft = 557.5 lbs/ft^2* [ASCE 7-22-S2, Eq. 5.4-14]

- As a comparison, when 1 is used for the parameter $\left(\frac{4\pi d_f/L}{sinh\ (4\pi d_f/L)}\right)^2$, p_{1B} equals:

  p_{1B}= [0.6 + 0.5 (1)+0.8]*64 lb/ft^3 * 5.1ft = 620.2 lbs/ft^2

p_3 = $\left(\frac{1}{cosh\ (2\pi d_f/L)}\right)$ *p_1* [ASCE 7-22-S2, Eq. 5.4-11]

For breaking waves, p_{1B} should be used in place of p_1. See Section 6.3.1 for more information. Thus,

$$p_3 = \left(\frac{1}{cosh\ (2\pi d_f/L)}\right) p_{1B}$$

$$p_3 = \left(\frac{1}{cosh\ (2\pi * 6.5ft/74.1ft)}\right) *557.5\ lbs/ft^2 = 466\ lbs/ft^2$$

F_{BRK} = (½(557.5 lb/ft²)(7.65 ft)) + (½(557.5 lb/ft² + 466 lb/ft²) 6.5ft) = 5,458 lbs.

Step 7) Determine Impact Load

Structures within the Flood Hazard Area must be designed for debris impact loads as determined by ASCE 7-22-S2, Section 5.4.5.1. Debris impact loads must be considered in any direction and at heights as required per ASCE 7-22-S2, Section 5.4.5.2. Debris impact loads need not be considered on multiple structural elements simultaneously.

Identify potential sources of debris

Based on the project location, design flood depth, Risk Category, and proximity to Banks Channel, the debris types that must be considered for the Risk Category II structure are a passenger vehicle and a small vessel. Since the small vessel has a larger weight and stiffness and a smaller impact area than the passenger vehicle, it will result in a larger impact load than the passenger vehicle and is the controlling debris type for this structure. See Table 24 of this design guide for a summary of debris type requirements based on the Risk Category and design flood depth.

Calculate Debris Impact Using the Elastic Debris Impact Load Equation

Designers should consult Table 24 of this design guide for minimum design debris properties to be used when calculating the debris impact force. The following debris impact calculation uses the elastic debris impact load equation and the small vessel debris properties.

$F_{di} = C_o\ V\ C_R\ C_s\ (k_e\ m_{debris})^{0.5}$ **Equation 26** [ASCE 7-22-S2, Eq. 5.4-20]

F_{di} = 0.80 * 7.2 ft/s * 1.0 * 1.0 * [360,000 lb/ft * (2,500 lbs/32.2 ft/s²)]$^{0.5}$ = **30,452 lbs**

where,
C_o = debris orientation coefficient, taken as 0.80

V = 7.2 ft/s

C_R = 1.0 — debris depth coefficient taken as 1.0 for design flood depths greater than 5 ft (1.52 m) and taken as 0.0 for design flood depths less than 1 ft (0.3 m). Linear interpolation is permitted between design flood depths of 1 ft (0.3 m) and 5 ft (1.52 m).

C_s = 1.0 — debris velocity stagnation coefficient per ASCE 7-22-S2, Table 5.4-3. Applicable for non-load-bearing elements on the exterior of buildings along the front face of a building wider than

30 ft (9.14 m). Walls must extend from grade to above the design stillwater flood elevation and be designed for the flood loads of this chapter. For load-bearing elements C_s shall be taken as 1.0.

K_e = 360,000 lb/ft — effective stiffness of the impacting debris or the effective lateral stiffness of the impacted structural element(s) deformed by the impact, in lb/ft (kN/m), determined in accordance with ASCE 7-22-S2, Section 5.4.5.2. Using the combined elastic stiffness of the debris and the impacted element in series is permissible.

ADDITIONAL CONSIDERATIONS

Effective Stiffness

This example does not take advantage of the ASCE 7-22-S2 provision that allows for the combined elastic stiffness of the debris and the impacted element to be used as the effective stiffness, k_e. Using this provision would likely reduce the k_e and, in-turn, would reduce the debris impact load, F_{di}. See Section 8.2.2, "Clarification: Effective Stiffness" text box for more information.

$m_{debris} = W_{debris}/g$ = 2,500 lbs/32.2 ft/s^2 — mass of the debris (W_{debris}/g) in lb s^2/ft (kg) determined in accordance with ASCE 7-22-S2, Section 5.4.5.2.

The simplified debris impact load for passenger vehicles or small vessels is shown next for comparison purposes. There is no requirement to calculate the debris impact load with both methods. For this example, the Elastic Debris Impact Load calculation method resulted in a lower impact load (30,452lbs) than the simplified debris impact load (40,800 lbs). Using the Elastic Debris Impact Load is acceptable even though it is smaller than the simplified debris impact load because the simplified load is typically conservative. However, if the Elastic Debris Impact Load calculation method results in a higher load, the best practice would be to use the Elastic Debris Impact Load because the threshold of the Simplified Debris Impact Load method was likely exceeded by the input conditions (e.g. flood depth, velocity, etc.).

Simplified Debris Impact Load for Passenger Vehicles or Small Vessels

$F_{di} = C_o * 51{,}000$ (lb) Equation 25 [ASCE 7-22-S2, Eq. 5.4-19]

$F_{di} = 0.80 * 51{,}000 = 40{,}800$ lbs

C_o = debris orientation coefficient, taken as 0.80.

Table 28 summarizes the variables and loads solved for in the coastal design example.

Table 28. Coastal Design Example Summary

Design Step	***Design Component***	***Value***
Step 1	Design Stillwater Flood Depth	d_f = 6.5 ft

Design Step	***Design Component***	***Value***
Step 2	Design Velocity	V = 7.2 ft/s
Step 3	Exceeding Minimums: Design Flood Elevation for MRI	DFE_{MRI} = 18.1 ft
Step 4a	Scour Around Columns	S_m = 3 ft
Step 4b	Exceeding Minimums: Scour Around Columns with Group Effects	S_{TOT} = 9 ft
Step 5a	Drag Load on Individual Components (per each column) – Corner Column	F_{drag} = 2,907 lbs
Step 5b	Drag Load on Individual Components (per each column) – Internal (non-corner) Column	F_{drag} = 4,808 lbs
Step 5e	Drag Load on Lateral Force Resisting System	F_{drag} = 32,354 lbs
Step 6	Breaking Wave Load on Vertical Columns	F_{bw} = 2,185 lbs
Step 7	Impact Load (Small Vessel)	F_{di} = 30,452 lbs

11. Riverine Flood Design Example

The intent of the riverine flood design example is to provide flood load calculations based on ASCE 7-22-S2, utilizing realistic flood data with fictitious structures. The examples do not incorporate jurisdictional requirements in the calculations; however, the examples may demonstrate best practices to minimize flood risk to structures. References made throughout these examples are to ASCE 7-22-S2. All elevation data provided in the examples are provided in NAVD 88.

The structure is a two-story, Risk Category III, non-residential building located in Houston, Texas. The structure is composed of masonry shear walls, with a flat roof, and foundation walls that are used to elevate the lowest floor above floodwaters. The building is assumed to be 60 feet by 60 feet with a total square footage of 7,200 square feet. A recent site survey indicates the adjacent ground elevation is elevation 57 feet (NAVD 88). The structure is located within the A Zone with a BFE ($SWEL_{100}$)[5] of 60.3 feet (NAVD 88). In order to reduce hydrostatic loads and eliminate the need to dry floodproof the lowest floor, the bottom of the lowest floor system will be designed to an elevation equivalent to the $SWEL_{MRI}$. Figure 25 provides the project location on the FIRM, which is along Brays Bayou. The location is approximately at cross-section BU, between cross-section BT and BU. Figure 26 provides the flood profile, which can be used to determine the BFE or 1% annual-chance flood and if available the 0.2% annual-chance flood elevation. Figure 27 provides the Floodway Data Table from the FIS for the project location. The Floodway Data Table will be used for calculation of the flood velocity. The example building site and flood conditions are depicted in Figure 28.

[5] In riverine locations, because of the absence of waves, the 100-year stillwater elevation, $SWEL_{100}$, is equivalent to the BFE.

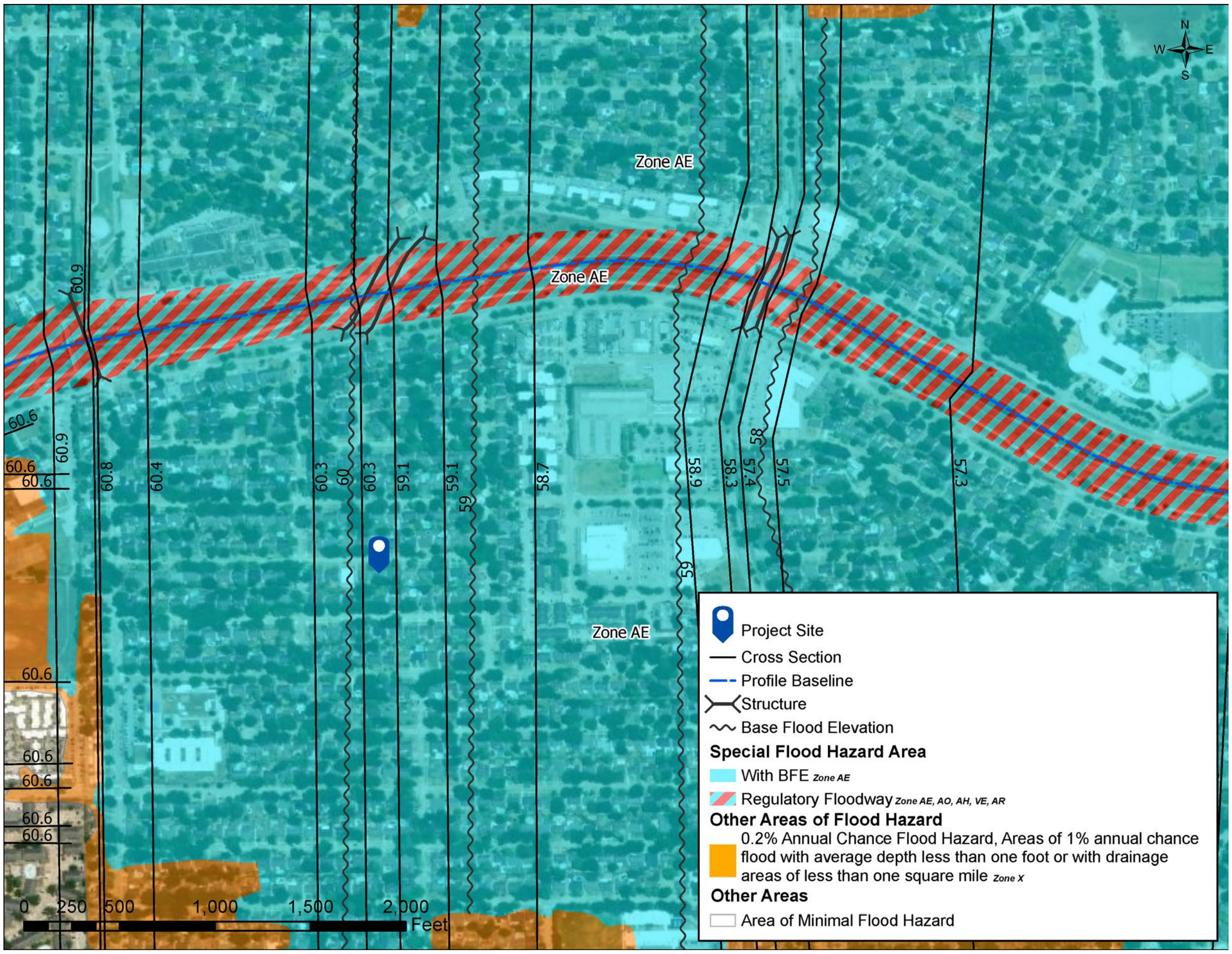


Figure 25. Riverine FIRM for project location

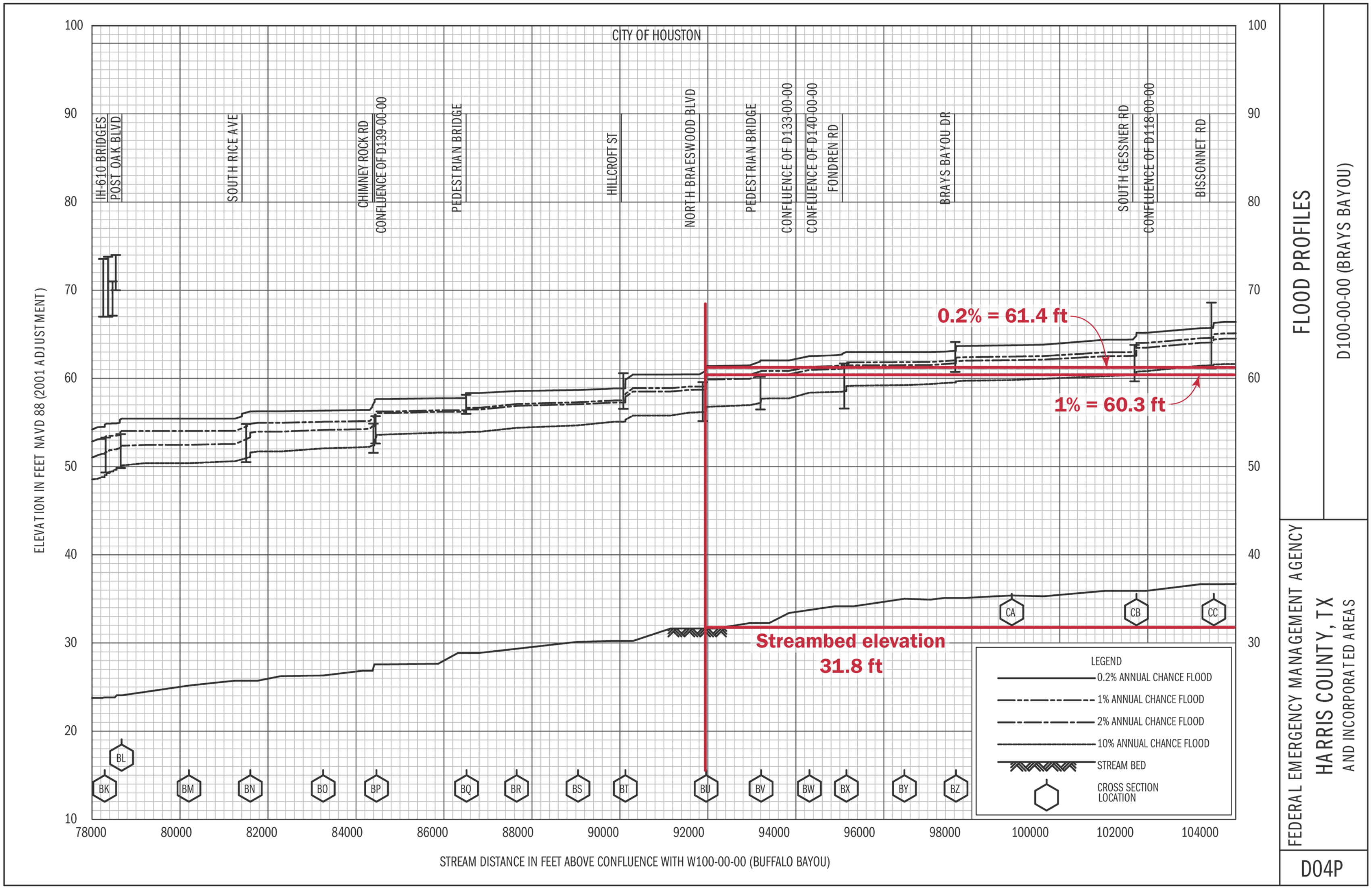


Figure 26. Flood profile for the project location, to determine 1% and 0.2% annual-chance flood elevations

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					FEET (NAVD 88, 2001 Adjustment)			
D100-00-00 (Brays Bayou)								
AY	62,151	278	4,856	6.4	48.1	48.1	49.1	1.0
AZ	63,605	399	5,201	5.9	48.3	48.3	49.2	0.9
BA	65,449	460	4,785	6.1	49.0	49.0	50.0	1.0
BB	67,310	347	4,686	6.4	49.3	49.3	50.2	0.9
BC	68,548	336	4,577	6.5	49.4	49.4	50.2	0.8
BD	69,468	385	4,831	6.3	49.5	49.5	50.3	0.8
BE	71,113	482	5,156	6.2	50.7	50.7	51.6	0.9
BF	73,154	386	4,389	7.2	50.8	50.8	51.7	0.9
BG	73,862	367	4,636	6.2	51.8	51.8	52.8	1.0
BH	75,383	363	4,592	5.8	52.6	52.6	53.5	0.9
BI	76,284	320	4,360	6.4	52.8	52.8	53.5	0.7
BJ	77,323	334	4,408	6.3	52.8	52.8	53.6	0.8
BK	78,288	517	5,289	5.5	53.2	53.2	53.9	0.7
BL	78,674	440	4,868	6.0	53.9	53.9	54.7	0.8
BM	80,203	400	4,302	7.2	54.0	540.0	54.7	0.7
BN	81,600	559	5,743	5.4	54.9	54.9	55.8	0.9
BO	83,257	400	4,326	7.6	55.1	55.1	55.9	0.8
BP	84,469	490	5,013	6.3	56.2	56.2	57.1	0.9
BQ	86,516	400	4,155	7.6	56.6	56.6	57.3	0.7
BR	87,656	350	3,815	8.2	57.1	57.1	57.6	0.5
BS	89,051	350	3,512	8.4	57.3	57.3	57.9	0.6
BT	90,129	350	3,663	8.1	58.3	58.3	59.2	0.9
BU	91,964	390	4,177	6.6	60.3	60.3	61.3	1.0
BV	93,213	475	4,923	5.6	60.9	60.9	61.8	0.9
BW	94,312	300	3,668	6.1	61.3	61.3	62.2	0.9

[1] Feet above confluence with G100-00-00 (Buffalo Bayou-Houston Ship Channel)
[2] Elevation computed using combined probability analysis

TABLE 8	FEDERAL EMERGENCY MANAGEMENT AGENCY **HARRIS COUNTY, TX AND INCORPORATED AREAS**	**FLOODWAY DATA**
		D100-00-00 (BRAYS BAYOU)

Figure 27. Floodway data table for the project location

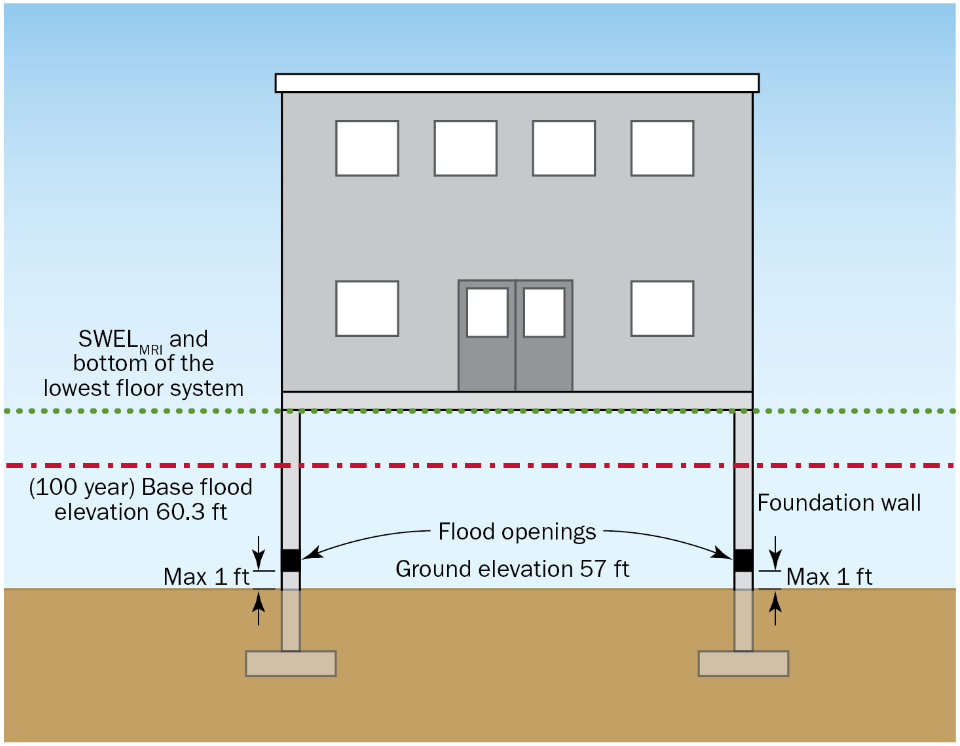


Figure 28. Riverine example building site and flood conditions

Step 1) Determine Flood Design Elevation

Risk Category III structures are to be designed for a 750-year MRI. Since flood data are not available for the 750-year MRI in the FIS, **Equation 1**, in conjunction with the scaling factors provided by Table 7, is to be used to calculate the stillwater design flood elevation, $SWEL_{MRI}$. The 1-year flood elevation was calculated as 53.7 feet, so, Z_{datum} will equal 53.7 feet. See Appendix C for more information on determining Z_{datum}. The 500-year stillwater elevation ($SWEL_{500}$) is specified as 61.4 feet by the FIS as shown in Figure 26. Since the 500-year stillwater elevation is known, the $SWEL_{MRI}$ will also be calculated with **Equation 2** and the higher of the two resulting elevations will be selected. This methodology is consistent with the explanation provided in textbox "EXCEEDING MINIMUMS: Calculating Required $SWEL_{MRI}$ with $SWEL_{500}$" in Section 3.2.1.

$$SWEL_{MRI} = C_{MRI} (SWEL_{100} - Z_{datum}) + Z_{datum}$$ **Equation 1** [ASCE 7-22-S2, Eq. 5.3-2]

where,

$SWEL_{100}$ = stillwater elevation for the 100-year MRI provided by a flood hazard study adopted by the Authority Having Jurisdiction in ft (m)

C_{MRI}= flood scale factor associated with the MRI from Table 7 for different locations. For a Risk Category III building in a riverine location, the factor is 1.45 to obtain the 750-year MRI elevation.

Z_{datum} = elevation of mean water level based on local datum, in ft (m). For riverine sites, Z_{datum} shall be taken as the annual high-water level.

$SWEL_{MRI} = C_{MRI\_500} (SWEL_{500} - Z_{datum}) + Z_{datum}$ **Equation 2** [Based on ASCE 7-22-S2, Eq. 5.3-2]

where,

C_{MRI_500} = flood scale factor associated with the MRI from Table 8 for different locations when $SWEL_{500}$ is the starting SWEL. For a Risk Category III building in a riverine location, the factor is 1.07 to obtain the 750-year MRI elevation.

$SWEL_{500}$ = stillwater elevation for the 500-year MRI, in ft (m)

Find the 750-year $SWEL_{MRI}$ with Equation 1:

$SWEL_{MRI} = 1.45 * (60.3\ ft - 53.7\ ft) + 53.7\ ft = 63.3\ ft,$

Find the 750-year $SWEL_{MRI}$ with Equation 2:

$SWEL_{MRI} = 1.07 * (61.4\ ft - 53.7\ ft) + 53.7\ ft = 61.9\ ft$

Compare the calculated 750-year $SWEL_{MRI}$ values:

Equation 1 $SWEL_{MRI}$ *: 63.3 ft > 61.9 :* **Equation 2** $SWEL_{MRI}$

Thus, the $SWEL_{MRI}$ for the 750-year MRI requirement for the Rick Category III structure is 63.3 feet.

Step 2) Determine Hydrostatic Load:

Because this structure is located within the SFHA, it must be designed to be compliant with the requirements of the NFIP. For non-dry floodproofed buildings, flood openings must be provided within 1 foot of the adjacent grade to allow the hydrostatic loads to equalize on the outside and inside of the foundation wall. Because the example building has flood openings and the bottom of the lowest floor system elevation will be equivalent to the $SWEL_{MRI}$, hydrostatic loads will not be calculated for the building in this example. Buildings that are not elevated to or above the $SWEL_{MRI}$ will need to consider hydrostatic loads. See the following "Additional Considerations: Dry Floodproofing" text box for an example of the development of hydrostatic loads.

ADDITIONAL CONSIDERATIONS

Dry Floodproofing

Structures that are dry floodproofed in accordance with the requirements of ASCE 24-14 and FEMA TB 3 will be subjected to hydrostatic and buoyant forces as water entry to the structure is resisted, with an allowable seepage rate of 4 inches in a 24-hour period. Figure 29 shows a building with passive dry floodproofing measures. These measures include flood windows and a barrier at the door that deploys automatically without human intervention. The dry floodproofing protection level exceeds the ASCE 24-14 requirement of BFE + 1 (for a Design Flood Class 3 building) and has a flood protection elevation equal to the $SWEL_{MRI}$ to align with the best practices within this design guide. Note that dry floodproofing is sometimes restricted based on the floodwater velocity at the site. For example, ASCE 24-14 does not allow dry floodproofing where the flood velocities adjacent to the building exceed 5 ft/s.

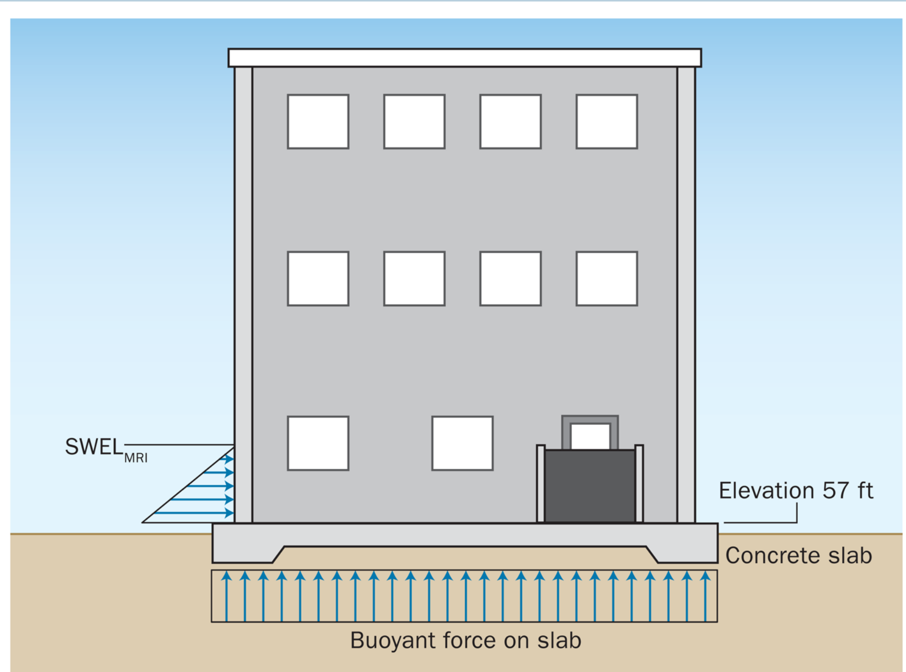


Figure 29. Dry floodproofed building subjected to riverine flooding

<u>Lateral Hydrostatic Force:</u>

Hydrostatic loading on wall elements is to be calculated in accordance with ASCE 7-22-S2, Section 5.4.2.

The lateral force, F_h, caused by the hydrostatic pressure on one side of a vertical wall per unit width, lb/ft (kN/m), must be calculated by substituting d_f in **Equation 9** for z, since d_f is calculated as the depth between $SWEL_{MRI}$ and eroded grade and the wall terminates above grade.

$F_h = (1/2)\, \gamma_w d_f^2$ **Equation 9** [ASCE 7-22-S2, Eq. 5.4-3]

$F_h = (1/2)\, \gamma_w z^2$

F_h = 0.5 * 62.4 lb/ft^3 * 6.3^2 ft = 1238 lb / ft (lb per linear foot)

γ_w = specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater

z = 63.3 ft – 57 ft = 6.3 ft (depth below design stillwater flood elevation, in ft [m])

Vertical Hydrostatic (Buoyant) Forces:

Buoyant forces, acting on the underside of the concrete slab are to be calculated in accordance with ASCE 7-22-S2, Section 5.4.2.1. The buoyant force is to be calculated using the exterior dimensions of the foundation and wall system. For the purpose of this example, the foundation slab is assumed to be 1 foot thick and the edge turndown is not considered in the calculation of buoyancy forces for simplicity purposes. Submerged soils have been assumed, which exceed the fully saturated soil assumption required by ASCE 7-22-S2, Section 5.4.2.1. See the subsection "Submerged and Fully Saturated Soil Forces" in Section 4.1.1 of this design guide for more information on submerged versus fully saturated soils.

$F_B = \gamma_w V_w$ **Equation 27** [ASCE 7-22-S2, Eq. 5.4-2]

F_B = 62.4 lb/ft^3 * 20,880 ft^3 = 1,303,000 lbs – uplift force caused by buoyancy, in lb (kN)

γ_w = 62.4 lb/ft^3 – specific weight of water, taken as 62.4 lb/ft^3 (9.81 kN/m^3) for freshwater and 64 lb/ft^3 (10.03 kN/m^3) for saltwater

V_w = 60 ft * 60 ft * (63.3 ft – 56 ft) = 26,280 ft^3 – volume of displaced water, in ft^3 (m^3)

Step 3) Determine Design Velocity

To determine the riverine velocity that corresponds to the 750-year MRI, the 100-year MRI obtained from the FIS is scaled using **Equation 30** and **Equation 31** contained within this design guide. This method results in conservative velocity values because the project location is approximately 1,400 feet from the flood source and the design flood depth (4.8 feet) is significantly shallower than the floodway depth (d_{f100} = 28.5 feet at the centerline of the floodway). A site-specific study could reduce the flood velocity that is applied to both the hydrodynamic loads and the debris impact loads.

ADDITIONAL CONSIDERATIONS

Riverine Velocity

See Appendix D of this design guide for explanation of the riverine velocity calculation methodologies demonstrated in this example.

$$C = \frac{V_{100}}{\left(\frac{d_{f100}}{w + 2d_{f100}}\right)^{2/3}}$$ **Equation 30**

C = site-specific constant as determined by **Equation 30** in ft/s (m/s)

where,

$d_{f100} = SWEL_{100} - G_r$ = 60.3 - 31.8 = 28.5 ft

d_{f100} = the 100-year flood depth taken at the center of the floodway, in ft (m). Note: *This value can usually be obtained from the flood profiles in the FIS by subtracting the riverbed (ground) elevation from the 1% annual-chance flood elevation.*

$SWEL_{100}$ = 60.3 ft NAVD 88 — stillwater elevation corresponding to the 100-year MRI, in ft (m).

G_r = 31.8 ft NAVD 88 — the riverbed (ground) elevation taken at the center of the floodway. This value may be obtainable from the flood profiles in the FIS.

V_{100} = 6.6 ft/s (Obtained from the FIS) — flood velocity for the 100-year MRI taken at the center of the floodway, in ft/s (m/s)

w = 390 ft (Obtained from the FIS) — width of the floodway, in ft (m)

$$C = \frac{6.6\ \text{ft/sec}}{\left(\frac{28.5\ ft}{390\ \text{ft} + 2 * 28.5\ ft}\right)^{2/3}} = 41.35$$

$$V_{MRI} = C\left(\frac{d_{fMRI}}{w + 2d_{fMRI}}\right)^{2/3} \qquad \textbf{Equation 31}$$

V_{MRI} = design flood velocity for a specified MRI, in ft/s (m/s)

where,

$$d_{fMRI} = SWEL_{MRI} - G_r = 63.3 - 31.8 = 31.5\ ft \qquad \textbf{Equation 32}$$

d_{fMRI} = the MRI design flood depth taken at the center of the floodway, in ft (m). This value may be obtainable from the flood profiles in the FIS by subtracting the riverbed (ground) elevation from the MRI flood elevation or it may be calculated with **Equation 32.**

$SWEL_{MRI}$ = 63.3 ft NAVD 88 — stillwater elevation corresponding to the specified risk category and MRI, in ft (m). See Section 3.2

G_r = 31.8 ft NAVD 88 — the riverbed (ground) elevation taken at the center of the floodway. This value may be obtainable from the flood profiles in the FIS.

C = 41.35 — site-specific constant as determined by **Equation 30**, in ft/s (m/s)

w = 390 ft — width of the floodway, in ft (m)

$$V_{MRI} = 41.35\left(\frac{31.5\ ft}{390\ ft + 2 * 31.5\ ft}\right)^{2/3} = 6.99\ ft/s$$

Step 4) Determine Hydrodynamic Load

In accordance with ASCE 7-22-S2, the hydrodynamic load on the wall can be calculated by ASCE 7-22-S2, Equation 5.4-5. The drag force on the components and the lateral force resisting system will be the same for this example as the building is a simple walled rectangular structure with grade acting as the lower elevation for both component and lateral force resisting system drag forces. Additionally, debris damming need not be considered since the structure is walled; whereas, an open foundation would require the consideration of debris damming. Alternate geometries may require separate calculations for the drag force on the components and the lateral force resisting system; refer to Section 5.3 in this design guide.

The design stillwater flood depth is required for drag force calculations and is calculated per **Equation 4**:

$d_f = (SWEL_{MRI} - G_e) = (63.3\text{ ft} - 57\text{ ft}) = 6.3\text{ ft}$ **Equation 4** [ASCE 7-22-S2, Eq. 5.3-1]

Next, the drag force is calculated per ASCE 7-22-S2, Eq. 5.4-5:

$F_{drag} = ½*\rho*C_d*V^2*B*d_f$ [ASCE 7-22-S2, Eq. 5.4-5]

$F_{drag} = ½*1.94\text{ lb s}^2/\text{ft}^4*1.23*(6.99\text{ ft/s})^2*60\text{ ft}*(6.3\text{ ft}) = 22{,}036\text{ lbs}$

where,

$\rho = 1.94\text{ lb s}^2/\text{ft}^4$ — mass density of water ($\text{lb s}^2/\text{ft}^4$) taken as $1.94\text{ lb s}^2/\text{ft}^4$ (1000 kg/m^3) for freshwater

$C_d = 1.23$ — drag coefficient per Table 5.4-2 for Rectilinear Buildings and Structures. A linear interpolation calculation may be used, or the FORECAST function in Microsoft® Excel®, as shown in Figure 30, may be used.

$V = 6.99$ ft/s — design flood velocity, in ft/s (m/s)

$B = 60$ ft — overall width of building perpendicular to the flow direction, in ft (m)

$d_f = 6.3$ ft — design flood depth (per Section 5.3.3), in ft (m)

$B/d_f = 60/6.3 = 9.5$

D6 =FORECAST(D5,D2:D3,C2:C3)

	A	B	C	D
1			Ratio of Structure width to design stillwater flood depth	Drag Coefficient C_d
2			12	1.25
3			120	2
4	B (ft):	60		
5	d_f (ft):	6.3	B/d_f:	9.52
6	B/d_f:	9.52	Drag Coefficient C_d:	1.233

Figure 30. Solving for drag coefficient with the Microsoft® Excel® FORECAST function

Drag forces are applied as described in Section 5.3 across the face of the building normal (perpendicular) to the flow of water.

<u>Step 5) Determine Debris Loads</u>

Using requirements outlined in Table 24 and Table 25, a Risk Category III structure should consider debris types based on the depth of flooding and distance from potential debris source. Risk Category III structures with a design stillwater flood depth of 3 feet or greater are required to consider passenger vehicles and wood poles. Other debris types are determined based on a site hazard assessment. A preliminary evaluation of aerial photographs suggests that the building density does not meet the requirements of a heavy-density environment. Therefore, the travel distances in the moderate-density column of Table 25 were considered for the site hazard assessment. Figure 31 provides a radius of 2,000 feet for the assessment of potential debris types. Based on the 4.8-foot flood depth, the assessment considers small vessels, and shipping containers. Given the high percent of residential structures and proximity to a waterway, a small vessel is likely to be within the 2,000-foot radius. An analysis of the aerial photographs also indicates the existence of a shopping center west of the project site within 2,000 feet of the project location. Within the shopping center, a 40-foot-long shipping container is visible on the aerial photographs, However, ASCE 7-22-S2, Section 5.3.9, lists the following exception, "For riverine sites, debris strikes need only be considered from the upstream direction with a strike direction of +/-22.5 degrees from the primary direction of flow." Since the shopping center is downstream of the project location, the shipping container that was identified is not required to be considered as a debris type.

Figure 31. Evaluation of project site debris sources

The debris impact load can be calculated with **Equation 26**.

$F_{di} = C_o V C_R C_s (k_e m_{debris})^{0.5}$ **Equation 26** [ASCE 7-22-S2, Eq. 5.4-20]

where,

C_o = debris orientation coefficient, taken as 0.80

C_R = debris depth coefficient taken as 1.0 for design flood depths greater than 5 ft (1.52 m) and taken as 0.0 for design flood depths less than 1 ft (0.3 m). Linear interpolation is permitted between design flood depths of 1 ft (0.3 m) and 5 ft (1.52 m). See Table 12. Note: *C_R values are extended to a depth of 1 ft, but debris impact is only required if the d_f is above 3 ft.*

C_s = debris velocity stagnation coefficient per Table 5.4-3, applicable for non-load bearing elements on the exterior of buildings along the front face of a building wider than 30 ft (9.14 m). Walls must extend from grade to above the design stillwater flood elevation and be designed for the flood loads of this chapter. For load bearing elements C_s shall be taken as 1.0.

k_e = effective stiffness of the impacting debris or the effective lateral stiffness of the impacted structural element(s) deformed by the impact, in lb/ft (kN/m), determined in accordance with Section 5.4.5.2. Using the combined elastic stiffness of the debris and the impacted element in series is permissible.

ADDITIONAL CONSIDERATIONS

Effective Stiffness

This example does not take advantage of the ASCE 7-22-S2 provision that allows for the combined elastic stiffness of the debris and the impacted element to be used as the effective stiffness, k_e. Using this provision would likely reduce the k_e and, in-turn, would reduce the debris impact load, F_{di}. See the "Clarification: Effective Stiffness" text box in Section 8.2.2 of this design guide for more information.

m_{debris} = mass of the debris (W_{debris}/g) in lb s²/ft (kg) determined in accordance with Section 5.4.5.2.

Table 29 provides a summary of the calculated debris impact loads and the values used in **Equation 26** to calculate the debris impact loads for each debris type. C_R (debris depth coefficient) is 1.0 since the design stillwater flood depth (6.3 feet) exceeds 5 feet. C_s applies the 1.0 coefficient for load-bearing walls. Values for k_e and m_{debris} are from Table 24 based on the debris type.

Table 29. Summary of Loads per Debris Source for this Example

Debris Type	*Velocity (ft/s)*	*C_R*	*C_S*	*W_{debris} (lbs)*	*m_{debris} (lb s²/ft)*	*K_{debris} (lbs/ft)*	*Load (lbs)*
Passenger Vehicles	6.99	0.95	1.0	2,400	74.5	72,000	12,951
Small Vessels	6.99	0.95	1.0	2,500	77.6	360,000	29,556
Wood Logs/Poles	6.99	0.95	1.0	1,000	31.1	4,200,000	63,910
Shipping container loads shown below for comparison purposes; loads not required for this site							
20-ft Shipping Containers	6.99	0.95	1.0	5,000	155.3	2,940,000	119,489
40-ft Shipping Containers	6.99	0.95	1.0	8,400	260.9	2,040,000	129,009

Table 30 summarizes the variables and loads solved for in the coastal design example.

Table 30. Riverine Design Example Summary

Design Step	*Design Component*	*Value*
Step 1	Design Stillwater Flood Depth	d_f =6.3 Feet
Step 2	Design Velocity	V = 6.99 ft/s
Step 3	Hydrostatic Load	F_h = 0 (flood openings in foundation walls)
Step 4	Hydrodynamic Loads on Walls	F_{drag} = 22,036 lbs
Step 5	Debris Loads on Walls	F_{di} = see Table 29 lbs

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13. Additional Resources

State-specific shoreline data published by state governments and universities:

North Carolina Erosion Map:
https://ncdenr.maps.arcgis.com/apps/webappviewer/index.html?id=f5e463a929ed430095e0a17ff803e156

Texas Gulf Shore Shoreline Change Map:
https://coastal.beg.utexas.edu/shorelinechange2019/

South Carolina Transect (i.e., Elevation Profiles) Data:
https://gis.dhec.sc.gov/bermexplorer/

California, Our Future Hazard Map (cliff retreat based on sea level rise):
https://ourcoastourfuture.org/hazard-map/

Guidance on barge impact forces:

USACE (U.S. Army Corps of Engineers). 2022. *Engineering and Design Barge Impact Forces for Hydraulic Structures*, USACE EM 1110-2-3402,
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Appendix A. Coastal Erosion Methodology

Introduction

The two primary types of coastal erosion are erosion associated with long-term shoreline retreat (from sea level rise and other chronic natural processes) and sudden erosion due to storm-induced dune loss. Methods for estimating these primary types of coastal erosion are outlined herein.

The effects of erosion on flood conditions and flood loads are accounted for by increasing the design stillwater flood depth (d_f) based on an eroded ground calculation, which creates the potential for increased wave heights, increased flood velocity, and increased potential for debris impact. This appendix outlines a process for estimating erosion associated with future conditions and mean recurrence intervals (MRIs) other than those modeled in the Flood Insurance Study (FIS).

Reminder: Scour should not be included in erosion calculations. Scour is not assumed to impact flood conditions or flood loads. Scour is calculated to check for structural stability of foundations after scour occurs.

RESOURCES

Coastal Erosion

For further information and discussion on coastal erosion, refer to FEMA P-55, *Coastal Construction Manual* (FEMA 2011), Volume I, Chapter 3, Sections 5 and 6 and Volume II, Chapter 8, Section 5.

Available at: https://www.fema.gov/emergency-managers/risk-management/building-science/publications

Estimating Coastal Erosion

ESTIMATING SHORELINE RETREAT METHODS

CLARIFICATION

Applicability of Appendix A Subsection, Estimating Shoreline Retreat Methods

The methods presented in Appendix A subsection, Estimating Shoreline Retreat Methods, are applicable for open coast sandy shorelines as well as bays and estuaries.

Shoreline retreat occurs due to subsidence, sea level rise, and other coastal processes that result in long-term erosion. Future shoreline retreat is not modeled or accounted for in FISs. Herein, a simplified method will be presented for determining the expected shoreline retreat and for translating the retreating profile. Methods that are more analytical may provide more precise results but will also add complexity; these methods will not be discussed herein.

The average annual shoreline retreat rate, also termed average annual shoreline erosion rate, can be determined from various sources outlined in Table 32. Table 32 outlines a process for estimating erosion from annual shoreline retreat and dune erosion. The process outlined in Table 32 aligns with the methods discussed in this appendix.

This average annual shoreline retreat rate includes effects of subsidence, historical sea level rise, and erosion from other coastal processes; thus, these various factors do not need to be estimated separately. As shown in **Equation 28**, the average annual shoreline retreat rate is multiplied by the project lifecycle to determine the total expected shoreline retreat over the life of the project. The beach and dune profiles are translated landward by the total expected shoreline retreat.

See Table 32 for a detailed process outline.

ESTIMATING STORM INDUCED DUNE LOSS EROSION METHODS

CLARIFICATION

Applicability of Appendix A Subsection, Estimating Storm Induced Dune Loss Erosion Methods

The methods presented in Appendix A subsection, Estimating Storm Induced Dune Loss Erosion Methods, are only applicable for open coast dune-backed sandy shorelines. This design guide does not provide guidance on the erosion of bluffs or cliffs.

Erosion due to storm-induced dune loss is usually modeled and accounted for in FISs for the 1% annual-chance (100-year) event and the 0.2% annual-chance (500-year) event but is not modeled for any other events (or recurrence intervals). This modeling accounts for the storm surge–associated sediment transport that can take place over a few hours and can remove small dunes and, in some cases, even large dune systems. The removal of a dune system can change the flood conditions for structures either located on or behind a dune system. Coastal flood zones are often delineated based on wave height (wave runup can be a factor) and the flood zones shown on Flood Insurance Rate Maps (FIRMs) are defined for the 1% annual-chance (100-year) event. Therefore, while the 0.2% annual-chance (500-year) event is studied for FISs, the FIRM flood zones do not indicate the expected wave heights or flood elevations during a 0.2% annual-chance (500-year) event. If a dune is mapped as surviving the 1% annual-chance event, but the modeled 0.2% annual-chance event will result in dune loss, the flood conditions landward of the dune would vary greatly between the two events.

Identification of a primary frontal dune (PFD) as well as mapping coastal areas with a PFD is regulated in Title 44 of the Code of Federal Regulations (CFR) as follows:

44 CFR § 59.1 defines a PFD as:

... a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope."

44 CFR § 65.11 (b) defines evaluation criteria for sand dunes used in mapping in coastal areas as:

Primary frontal dunes will not be considered as effective barriers to the base flood storm surges and associated wave action where the cross-sectional area of the primary frontal dune, as measured perpendicular to the shoreline and above the 100- year stillwater flood elevation and seaward of the dune crest, is equal to, or less than, 540 square feet.

FEMA P-55, *Coastal Construction Manual* (FEMA 2011) explains that mapping procedures require that a dune have a minimum frontal dune reservoir (dune cross-section above the 100-year stillwater level and seaward of the dune peak) of 540 square feet to be considered substantial enough to withstand erosion during a 1% annual-chance (100-year) flood event. According to Federal Emergency Management (FEMA) procedures, when considering a 1% annual-chance flood event, a frontal dune reservoir less than 540 square feet will result in dune removal (dune disintegration), while a frontal dune reservoir greater than or equal to 540 square feet generally will result in dune retreat (see Figure 32 and Figure 33). FEMA P-55 recommends that the size of the frontal dune reservoir used by designers to prevent dune removal during a 100-year storm be increased to 1,100 square feet to account for back-to-back storms and other factors. The 540-square-foot rule was initially derived from an equation presented by Hallermeir and Rhodes (Hallermeir and Rhodes 1988) and represents the median erosion area above the stillwater elevation during the 100-year event (MacArthur et al. 2005). Thus, not only will selecting a larger reservoir cross-section aid in protecting against back-to-back storms, but it will also help to protect dunes whose erosion areas exceed the median. Table 31 uses the Hallermeir and Rhodes equation to outline cross-section requirements and recommendations for dune reservoir cross-section areas for various MRIs. The recommended reservoir cross-section values represent the Hallermeir and Rhodes equation with a 2x factor applied; this mimics what FEMA P-55 recommends.

See Table 32 for a detailed process outline.

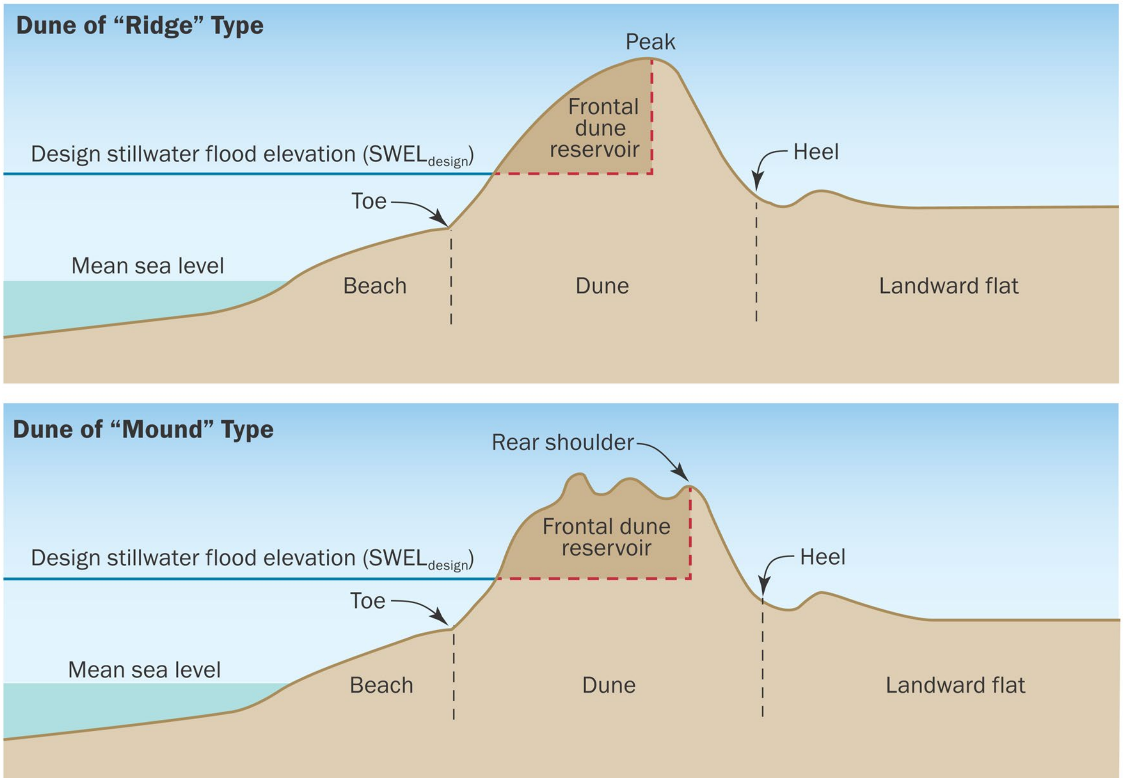


Figure 32. Definition sketch of frontal dune reservoir

Table 31. Primary frontal dune reservoir cross-section requirements and recommendations for varying MRIs

MRI (year)	*Annual Exceedance Probability (AEP)*	*Minimum Reservoir Cross-Section (ft^2)[a]*	*Recommended Reservoir Cross-Section (ft^2)[b]*
100	1.00%	540	1090[c]
500	0.20%	1030	2070
750	0.13%	1220	2430
1,000	0.10%	1360	2730

(a) Generated from the following equation: Erosion [m^2] = 8 (Recurrence Interval [yr])$^{0.4}$ (Hallermeir and Rhodes 1988) and m^2 values multiplied by 10.76 to convert to ft^2.
(b) Using the recommended values will result in a more conservative and robust design.
(c) Rounded to 1100 ft^2 in FEMA P-55. Either 1100 ft^2 or 1090 ft^2 may be used; 1100 ft^2 is used herein for consistency with FEMA P-55.

Post-Storm Profiles for Dune Removal and Dune Retreat

As described in FEMA P-55, the post-storm profile in the case of dune removal can be determined by drawing a straight line from the pre-storm dune toe landward at an upward slope of 1 on 50 (vertical to horizontal) until it intersects the pre-storm topography landward of the dune as shown in Figure 33. Any sediment above the line is assumed to be eroded.

The post-storm dune profile in the case of dune retreat can be determined by drawing two slopes: a straight line from the design stillwater flood elevation landward at an upward slope of 1 on 1 and a straight line from the design stillwater flood elevation seaward at a downward slope of 1 on 40 to balance erosion above the design stillwater flood elevation as shown in Figure 33. The horizontal position of these connected slopes, relative to the dune face, is determined by defining the position that results in erosion of the minimum reservoir cross-section above the design stillwater flood elevation (FEMA 2018b).

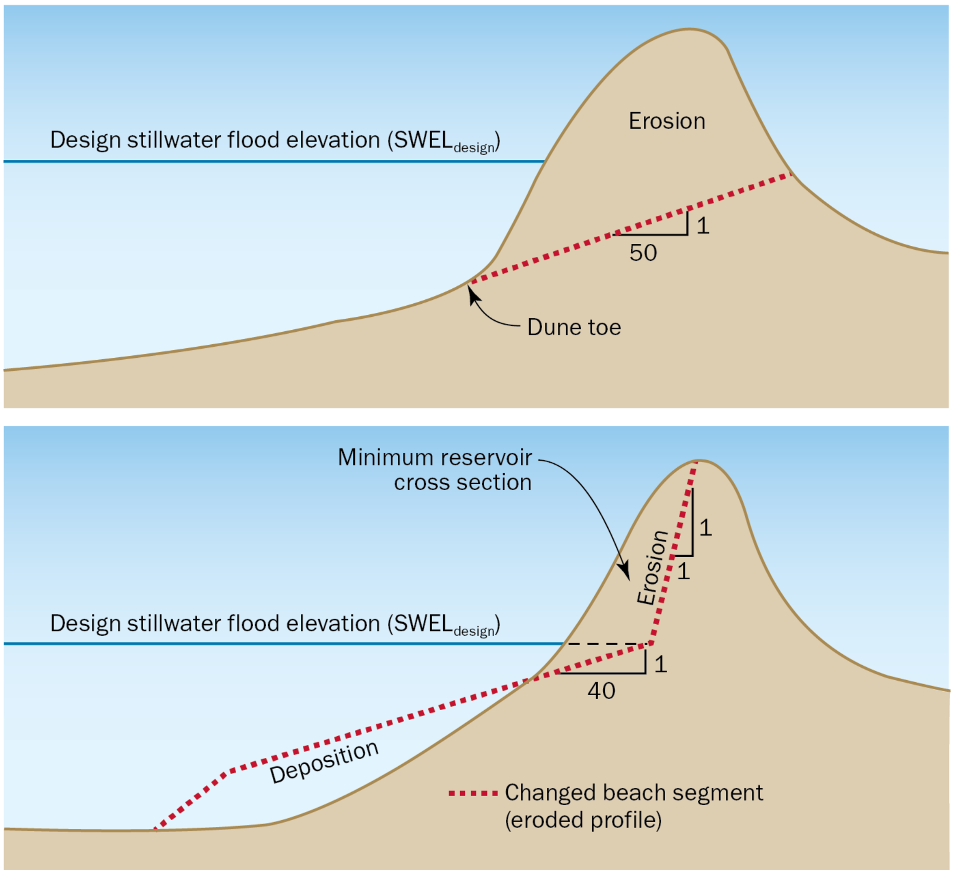


Figure 33. Dune removal (top) and dune retreat (bottom) treatment methodologies

COASTAL EROSION ESTIMATION PROCESS

Table 32 outlines a process for estimating erosion from annual shoreline retreat and dune erosion. The process outlined in Table 32 aligns with the methods discussed earlier in this appendix.

Table 32. Process for Estimating Erosion from Annual Shoreline Retreat and Dune Erosion

Step	*Objective*	*Method*
Step 1	Determine Average Annual Shoreline Retreat (S_{RA}) in ft/yr or m/yr[(a)] Note: If the shoreline is accreting (growing seaward), design should be based on current shoreline conditions.	Method 1 *(preferred method)*: Contact the authority having jurisdiction (AHJ) to obtain published erosion rates and guidelines. Many states and communities have calculated erosion rates to establish regulatory construction setback lines.
		Method 2: Obtain erosion rates from published reports and tools. Numerous tools and reports are publicly available for obtaining shoreline change estimates. When local studies are not available, see the "Resources: Shoreline Change and Dune Elevation (Nationwide)" text box in this appendix.
Step 2	Calculate Total Expected Shoreline Retreat (S_{RTOT})	$S_{RTOT} = S_{RA} * PL$ **Equation 28** *where,* S_{RA} = average annual shoreline retreat from Step 1 PL = project lifecycle in years
Step 3	Obtain an Elevation Profile from the Shoreline through the Project Site[6]	Method 1 *(recommended method)*: Obtain an elevation profile from a recent topographic survey. The profile should be oriented perpendicular to the shoreline and pass through the center of the project site.
		Method 2 *(only recommended for preliminary evaluations)*: Use a publicly available digital elevation model (DEM) or elevation profile. Elevation profiles and DEMs can be obtained from sources such as The U.S. Geological Survey (USGS) National Map Viewer, National Oceanic and Atmospheric Administration (NOAA) Data Access Viewer, or other publicly available sources. See the "Additional Considerations: Elevation Profiles" text box for more information. When using any public data source, verify that the data still represents the provided ground profile. If changes have occurred, use Method 1.

[6] This is a simplified method that assumes the dune is continuous with a constant shape. However, dunes along shorelines vary in shape and size and are not always continuous. Significant changes in the dune profile adjacent to the profile that is in alignment with the site may affect conditions at the site. This is an advanced topic and a coastal engineer may be consulted for an in-depth site study.

Step	***Objective***	***Method***
Step 4	Locate the Dune Toe[b]	Method 1 *(recommended method)*: Use the topographic information obtained in Step 3 to identify the junction of the gentle slope seaward of the dune and the dune face, which is marked by a slope of 1 on 10 or steeper. This is the dune toe. See Figure 32. Method 2: Use the USGS Coastal Change Hazards Portal as described in the "Resources: Shoreline Change and Dune Elevation Resource (Nationwide)" text box or use similarly available data from other reliable sources to obtain the dune toe elevation. Then, locate the intersection between the dune toe elevation and the dune elevation profile to determine the location of the dune toe. Method 3: Contact the AHJ to obtain published dune profile information that details the location of the dune toe.
Step 5	Locate the Dune Heel (the furthest landward point of the primary frontal dune [PFD])	Identify the approximate point where the landward dune face transitions from a relatively steep slope to relatively mild slope, this is the dune heel. See Figure 32. A FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report (Hatheway et al. 2005) explains that "in certain situations, man-made impacts to the landward dune face may have altered the terrain (excavations for homes and appurtenant structures) and will indicate a false transition point for the primary frontal dune heel. In those scenarios, the natural dune feature and primary frontal dune heel location may be evaluated based on historic data, unaltered portions of the dune, and/or aerial photography."

Step	*Objective*	*Method*
Step 6	Determine Stillwater Elevation for the Required MRI ($SWEL_{MRI}$)	$SWEL_{MRI} = C_{MRI} (SWEL_{100} - Z_{datum}) + Z_{datum}$ **Equation 1** Or, $SWEL_{MRI} = C_{MRI\_500} (SWEL_{500} - Z_{datum}) + Z_{datum}$ **Equation 2** *where,* $SWEL_{MRI}$ = stillwater elevation corresponding to the specified risk category and MRI, in ft (m) C_{MRI} = flood scale factor associated with the MRI from Table 7 for different locations when $SWEL_{100}$ is the starting SWEL $SWEL_{100}$ = stillwater elevation for the 100-year MRI, in ft (m) Z_{datum} = elevation of mean water level based on local datum, in ft (m). Z_{datum} shall be permitted to be taken as 0 for coastal sites. C_{MRI_500} = flood scale factor associated with the MRI from Table 8 for different locations when $SWEL_{500}$ is the starting SWEL $SWEL_{500}$ = stillwater elevation for the 500-year MRI, in ft (m)
Step 7	Identify or Calculate Δ_{SLR}	$\Delta_{SLR} = SLR_A * PL$ **Equation 3** Or exceed minimums by using the Δ_{SLR} identified for future RSLC projections *where,* Δ_{SLR} = the total relative sea level change for coastal sites over the project lifecycle, in ft (m). Shall not be taken as less than 0. SLR_A = the average annual rate of relative sea level change in ft/yr (m/yr). May be either the historic rate, or a selected rate above the historic rate. PL = project lifecycle See Section 3.2.2 for further details.
Step 8	Determine Design Stillwater Flood Elevation ($SWEL_{design}$)	$SWEL_{design} = SWEL_{MRI} + \Delta_{SLR}$ **Equation 29** *where,* $SWEL_{design}$ = design stillwater flood elevation in ft (m)

Step	***Objective***	***Method***
Step 9	Determine Dune Reservoir Cross-Section	Calculate dune reservoir cross-section as shown in Figure 32. Note: $SWEL_{design}$ is utilized for the horizontal elevation line.
Step 10	Determine if Dune Retreat or Removal Treatment[(c)] is Required	If dune reservoir cross-section, < applicable value in Table 31, then apply Removal Treatment $\geq$ applicable value in Table 31, then apply Retreat Treatment Note: Table 31 provides required values as well as recommended values. Using the recommended values will result in a more conservative and robust design.
Step 11	Apply Appropriate Dune Treatment	Refer to Figure 33 for dune removal and retreat methods described below. Removal Treatment Method: Remove dune area above the 1/50 slope starting from the toe. Retreat Treatment Method: Remove dune area seaward of the 1/1 slope and above the 1/40 slope. Location of the connected slopes must be adjusted to create a removal area above the stillwater elevation that matches the area in Table 31. The slopes must intersect at the $SWEL_{design}$.
Step 12	Shift the Elevation Profile Based on Shoreline Retreat[(d)]	Shift the elevation profile landward by the total expected shoreline retreat (S_{RTOT}) distance. Do not shift any portions that are landward of the dune heel. See Figure 36. Remove any portion of the new elevation profile that exceeds the elevation of the current elevation profile. If the shifted elevation profile is not at a lower elevation than the current ground elevation at any point along the project site, assume no erosion impact from shoreline retreat. *Do not continue to Step 13.*
Step 13	Determine Future Minimum Eroded Ground Elevation (G_e)	Identify the lowest ground elevation within the parcel depth. This is the future minimum eroded ground elevation (G_e). See Figure 37.

(a) If there are multiple published erosion rates, erosion from shoreline retreat should be based on the larger of the published erosion rates, unless there is compelling evidence to support a smaller erosion rate.
(b) It is important to ensure all elevations utilized are relative to the same datum. See the "Resources: Datum Conversion" text box in this appendix for more information.
(c) Dune treatment refers to the applicable theoretical dune loss calculation.
(d) Do not adjust the site elevation in locations where non-erodible strata are present. Locations with non-erodible strata shall be considered to have a constant elevation over the project lifecycle.

EXAMPLE

Estimating Eroded Profile from Annual Shoreline Retreat and Dune Removal

A property owner would like to build a Risk Category II structure on a parcel located near the beach along the Atlantic Ocean. The licensed professional hired by the owner has determined the average annual shoreline retreat (S_{RA}) for the area is 3.5 ft/yr, the dune toe elevation is 7.2 feet NAVD 88, and the current 1% annual-chance (100-year MRI) stillwater elevation is 11.3 feet NAVD 88. The elevation profile from the shoreline through the parcel is depicted in Figure 34. The owner has requested that the licensed professional account for a 50-year project lifecycle (PL). The owner and designer agreed to account for 2 feet of RSLC based on future projections over the 50-year PL.

Solution: The steps below outline the process for determining the estimated erosion from annual shoreline retreat over the PL and storm-induced dune erosion.

Step 1: Determine Average Annual Shoreline Retreat (S_{RA})

- S_{RA} = 3.5 ft/yr landward

Step 2: Calculate Total Expected Shoreline Retreat (S_{RTOT})

- $S_{RTOT} = S_{RA}$ * PL = 3.5 ft/yr * 50 yrs = 175 ft

Step 3: Obtain an Elevation Profile from the Shoreline through the Project Site

- Profile Shown in Figure 34.

Step 4 through 5: Locate the Dune Toe and the Dune Heel

- Dune toe elevation of 7.2 feet NAVD 88 intersects with the elevation profile 125 feet landward of the shoreline as shown in Figure 34. This is the dune toe.
- Identify the approximate point where the landward dune face transitions from a relatively steep slope to a relatively mild slope as defined in Step 5 of Table 32 to identify the dune heel as shown in Figure 34.

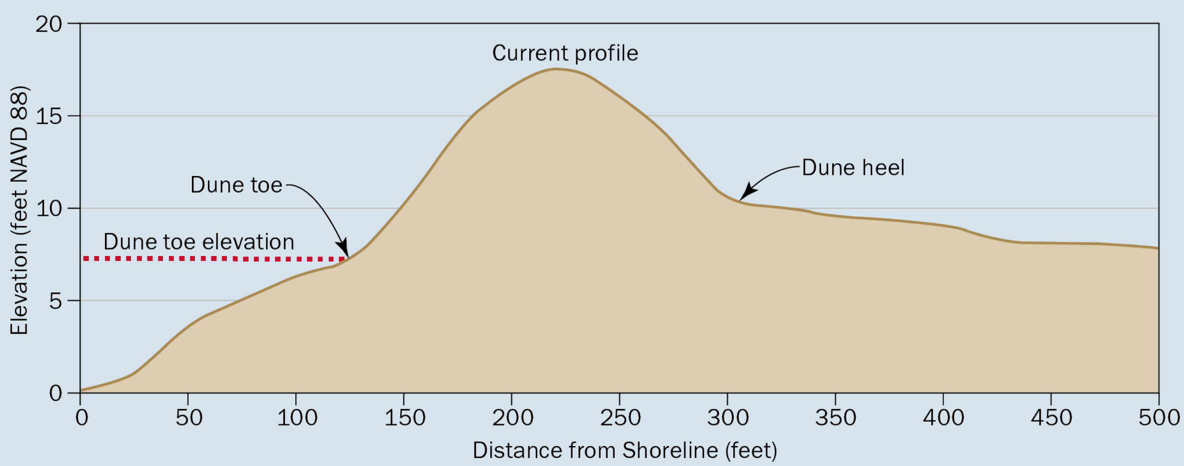


Figure 34. Elevation profile depicting the dune toe and dune heel

Step 6: Determine Stillwater Elevation for the Required MRI ($SWEL_{MRI}$)

- $SWEL_{MRI} = C_{MRI} (SWEL_{100} - Z_{datum}) + Z_{datum}$ **Equation 1**
- C_{MRI} is found in Table 7
- C_{MRI} for a Risk Category II structure (500-year MRI) in a coastal region other than the Gulf of Mexico is 1.25. C_{MRI_100} is used when the 1% annual-chance (100-year MRI) SWEL is used as the starting point.
- $SWEL_{MRI}$ = 1.25 (11.3 - 0) + 0 = 14.1 feet, the 500-year SWEL for the site

Step 7: Calculate Δ_{SLR}

- Δ_{SLR} = 2 feet, based on future RSLC projections over the 50-year PL

Step 8: Determine Design Stillwater Flood Elevation ($SWEL_{design}$)

- $SWEL_{design} = SWEL_{MRI} + \Delta_{SLR}$ = 14.1 feet + 2 feet = 16.1 feet NAVD 88

Step 9: Calculate dune reservoir cross-section as shown in Figure 32

- The frontal dune reservoir cross-section, which is shown in Figure 35, is approximately 38 square feet.

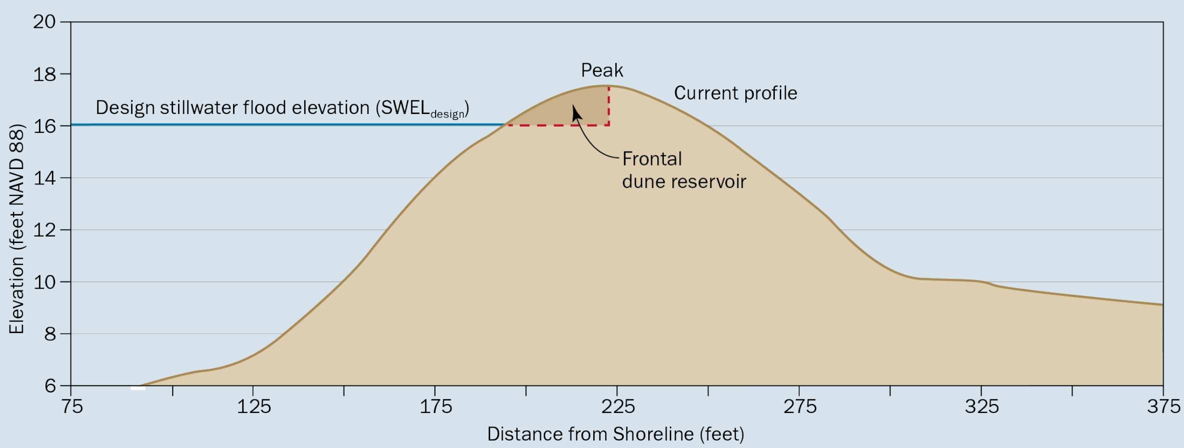


Figure 35. Primary Frontal Dune reservoir for Beach Town, USA

Step 10: Determine if Dune Retreat or Removal Treatment is Required

- Per Table 31, 1030 to 2070 square feet of reservoir cross-section is required for dune survival for a 500-year MRI. Because 38 square feet < the required square feet, dune removal treatment must be implemented.

Step 11: Apply Appropriate Dune Treatment

- For dune removal treatment, a straight line is drawn landward from the dune toe at an upward slope of 1 on 50 (vertical to horizontal) until it intersects the dune elevation profile 300 feet landward of the shoreline. The dune area above the 1/50 slope is removed as shown in Figure 36.

Step 12 and Step 13: Shift the Elevation Profile Based on Shoreline Retreat and Determine Future Minimum Ground Elevation (G_e)

The eroded elevation profile is shifted landward by the total expected shoreline retreat (S_{RTOT}) distance of 175 feet as shown in Figure 36. Note that only the elevation profile seaward of the dune heel is shifted.

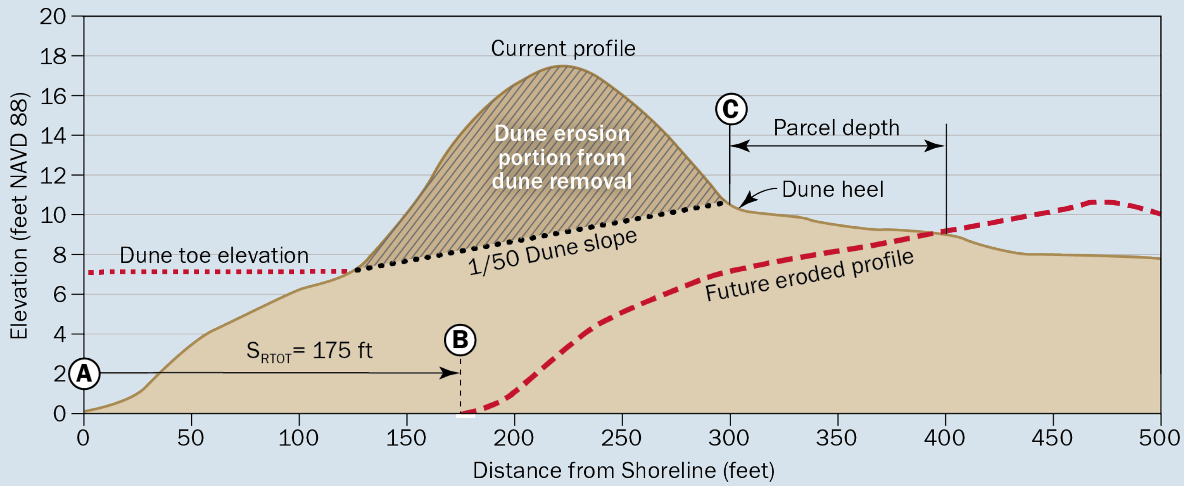


Figure 36. Current elevation profile and shoreline retreat profile with dune erosion for Beach Town, USA; (A) original shoreline station, (B) future shoreline station, (C) seaward extent of the site.

- The future minimum eroded ground elevation (G_e) is determined by locating the minimum ground elevation at the site.
- As shown in Figure 37, G_e is equal to 7.0 feet NAVD 88 for this project site.

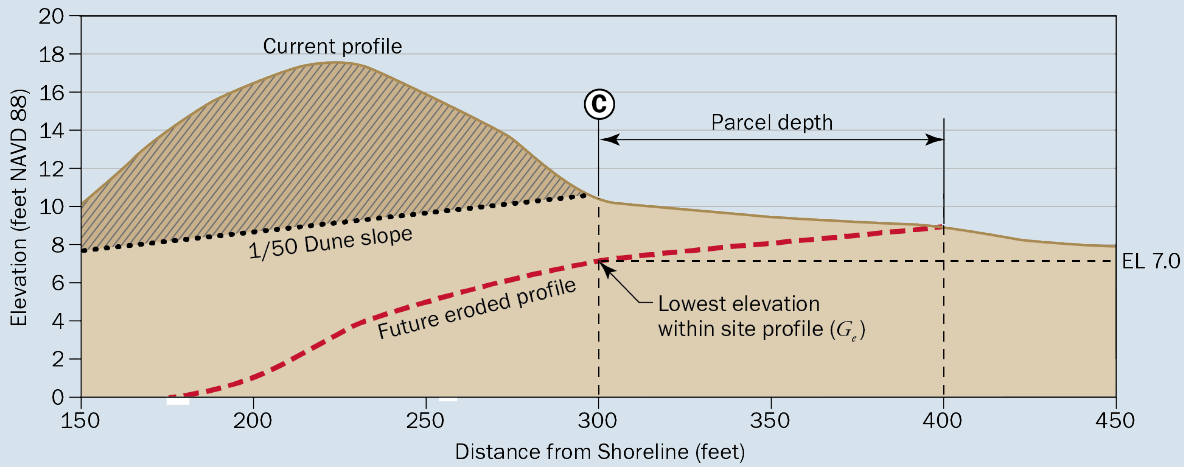


Figure 37. Elevation profile denoting the lowest elevation, of either the current or future profile, within the site profile

RESOURCES

Shoreline Change and Dune Elevation (Nationwide)

U.S. Geological Survey's (USGS's) Coastal Change Hazards Portal

Access: https://marine.usgs.gov/coastalchangehazardsportal/

Available data applicable to this design guide:

- Shoreline Change Rates
    - The tool refers to long-term (78+ years) and short-term (~30 years) change rates are available. Designers may elect to use either rate; a conservative selection would be to use the higher of the two. Short-term rates provide a better estimate of the current conditions, but using the higher of the two will aid in filtering out human processes (such as beach nourishment) that may have altered the rates. There is no issue with beach nourishment being inherently included in the shoreline change rates except that the rate will increase if nourishment is stopped. Designers are not expected to evaluate the influences of beach nourishment.
    - The bars, which correlate to shoreline change rates in terms of ranges, should be clicked to obtain more precise rates. Zoom in until the output text box no longer reads "Zoom in further for more accurate results."
- Dune Peak Elevation (Does not include CA, OR, WA, AK, HI, or locations without dunes)
- Dune Toe Elevation (Does not include CA, OR, WA, AK, HI, or locations without dunes)

Dune elevation information is listed under Extreme Storms → Baseline morphology

Keywords for search engines: coastal change, shoreline change, coastal erosion, shoreline retreat

Note: Designers should consider state and local erosion rates when they are available. In some instances, use of the state or local erosion rates may be required per local ordinances and codes. If multiple erosion rates are available, it is recommended that the higher erosion rate be used. See Chapter 13 for a non-exhaustive list of available statewide resources.

RESOURCES

Datum Conversion

FEMA currently uses the North American Vertical Datum of 1988 (NAVD 88) for the creation of FIRMs. Previously, National Geodetic Vertical Datum of 1929 (NGVD 29) was used to create FIRMs. Elevation data from FEMA and other sources may be provided in a variety of vertical datums, including, but not limited to, NAVD 88 and NGVD 29. The designer is responsible for ensuring a common datum is used throughout the calculation process. When varying data sources use different datums, the data should be converted to a common datum.

National Oceanic and Atmospheric Administration (NOAA) National Ocean Service's (NOS's) Vertical Datum Transformation tool (VDatum) is a conversion tool that may be used to convert data from various vertical datums into a common datum (subject to *Terms of Use* as outlined by NOAA/NOS).

NOAA/NOS's VDatum tool: https://vdatum.noaa.gov/runapp_agreement.php

ADDITIONAL CONSIDERATIONS

Elevation Profiles

Preliminary elevation profiles and Digital Elevation Models (DEMs) can be obtained from public data sources such as the USGS National Map Viewer and NOAA Data Access Viewer. Refer to Table 32, Step 3, to determine where these data are applicable.

The USGS National Map Viewer

Access the National Map Viewer at https://apps.nationalmap.gov/viewer/

Refer to Appendix C, Approach 1, Step 2, for details on obtaining elevations from the National Map Viewer.

NOAA Data Access Viewer

NOAA Data Access Viewer may be used to obtain Lidar contour data that can be viewed in computer-aided design (CAD) programs, such as AutoCAD Civil 3D (subject to *Terms of Use* as outlined by NOAA). The following outline provides an overview of the steps required:

- Access the Data Access Viewer at https://coast.noaa.gov/dataviewer/.
- Select "Elevation/Lidar."
- Enter an address or zoom to the site location on the map.
- Use the "Draw" button to draw a box around the area requiring evaluation. You may need to zoom out if you entered an address to identify the site location. The available data for the area, listed newest to oldest, will populate once the area is drawn.
- Select the desired data for download by clicking the cart icon next to the dataset.
- Navigate to the cart on the top right of the screen, select the cart, and click "Next" after verifying the desired data are in the cart.
- Modify any desired selections on the "Provision Your Data" screen and proceed through the request process. Note: FEMA recommends using GeoTIFF if it is available as an output option as it will enable the user to geolocate the data in CAD.
- The user will receive two emails. The first is a confirmation email; the second email will have a link to the downloadable data.

- Inputting the data into CAD will vary by program. For AutoCAD Civil 3D, a surface can be created with the data by selecting "Create a Surface by DEM." Further, if a GeoTIFF was used, additional geographic information system (GIS) data may be inserted to align additional information, such as property lines.

ADDITIONAL CONSIDERATIONS

Noting Eroded Ground Elevation on Design Drawings

The eroded ground elevation should be noted on the design drawings to ensure all foundation members are installed to depths that accommodate erosion from future conditions.

Appendix B. Design Drawing Notes Checklist

This appendix provides a list of recommended data that may be helpful to include as notes on design drawings. These data represent a combination of floodplain management data, building data, site data, and design values developed during the design process. Data provided can be helpful for local officials reviewing the plans set as well as provide the designer documentation of the date that data, such as sea level rise or debris source data, were collected, which could be helpful if conditions change following the data collection process. Although the list does not contain all of the calculated values, sufficient information is provided to communicate the design basis.

	Design Data	*Riverine Flooding Source*	*Coastal Flooding Source*	*Data Source*
Floodplain Management	Flood Insurance Rate Map (FIRM) / Flood Insurance Study (FIS) Date	✓	✓	FIS/FIRM
	FIRM Panel Number	✓	✓	FIRM
	Vertical Datum	✓	✓	FIRM
	Flood Zone (most restrictive flood zone that the building footprint is within)	✓	✓	FIRM
	Local Zoning Restrictions if applicable (e.g., Coastal Construction Control Line)	✓	✓	Local Requirements
	Cross-Section	✓		FIRM
	Streambed Elevation	✓		FIS (Flood Profile)
	1% Flood Elevation (Base Flood Elevation)	✓		FIS (Flood Profile)
	0.2% Flood Elevation (if available)	✓		FIS (Flood Profile)
	1% Flood Velocity	✓		FIS (Floodway Data Table)
	Floodway Width	✓		FIS (Floodway Data Table)
	Transect Number		✓	FIRM
	1% Flood Elevation (Base Flood Elevation)	If no Flood Profiles in FIS	✓	FIRM
	1% Stillwater Elevation		✓	FIS (Summary of Coastal Stillwater Elevations Tables)

	Design Data	***Riverine Flooding Source***	***Coastal Flooding Source***	***Data Source***
	0.2% Stillwater Elevation (if available)		✓	FIS (Summary of Coastal Stillwater Elevations Tables)
	0.2% Annual Chance Wave Envelope (if available)		✓	FIS (Transect Profiles)
	Risk Category	✓	✓	ASCE 7-22, Chapter 1
Building Data	Design Flood Mean Recurrence Interval (MRI)	✓	✓	ASCE 7-22-S2, Table 5.3-1
	Flood Design Class	✓	✓	ASCE 24-14, Chapter 1
	Local Datum (if applicable)	✓	✓	Site Survey
	Existing Ground Elevation (lowest point)	✓	✓	Site Survey
	Relative Sea Level Change (RSLC)		✓	User defined – see Section 3.2.2 of this design guide
	Historic RSLC or Future Projection Scenario		✓	User defined – see Section 3.2.2 of this design guide
Site Data	If a RSLC future projection was used, list scenario, version, date of model, and date of data gathering		✓	User defined – see Section 3.2.2 of this design guide
	Debris Source	✓	✓	User defined – see Section 8.1 of this design guide
	List how debris data were gathered (e.g., site visit or desktop study) and list date of data gathering	✓	✓	User defined – see Chapter 8 of this design guide
	Stillwater Flooding Depth	✓	✓	User defined – see Section 3.2.4 of this design guide
	Stillwater Flooding Elevation	✓	✓	User defined – see Section 3.2 of this design guide
Design Values	Velocity	✓	✓	User defined – for Coastal see Section 5.1.1 of this design guide and for Riverine see Section 5.1.2 of this design guide
	Wave Height		✓	User defined – see Section 6.1 of this design guide

	Design Data	***Riverine Flooding Source***	***Coastal Flooding Source***	***Data Source***
	Wave Crest Elevation		✓	User defined – see Section 6.1 of this design guide
	Wave Period		✓	User defined – see Section 6.2 of this design guide
	Wave Length		✓	User defined – see Section 6.2 of this design guide
	Depth of Erosion	If defined	✓	User defined – see Section 3.2.3 of this design guide
	Eroded Ground Elevation	If defined	✓	User defined – see Section 3.2.3 of this design guide
	Scour Depth	If defined	✓	User defined – for Coastal see Section 7.1 of this design guide and for Riverine see Section 7.2 of this design guide
	Lowest Floor Elevation	✓	✓	User defined – may be driven by flood load calculations or ASCE 24-14
	Flood Protection Elevation (for Dry Floodproofing Projects)	✓	✓	User defined – may be driven by flood load calculations or ASCE 24-14
	Elevation of Bottom of Lowest Horizontal Member Supporting the Lowest Floor		✓	User defined – may be driven by flood load calculations or ASCE 24-14

Appendix C. Determining Z_{datum} for Riverine Locations

Per ASCE 7-22-S2, the Z_{datum} is defined as the elevation of the mean water level based on local data, in feet. For riverine sites the Z_{datum} shall be taken as the annual high water level, which is the elevation of the 1-year flood. This appendix provides two approaches for estimating the Z_{datum} in riverine locations. Approach 1 is quantitative, and Approach 2 is qualitative. Designers should use Approach 1 when the necessary data are available. Approach 2 should only be used when Approach 1 is not an option due to a lack of data. Alternatively, while not discussed in this appendix, an H&H study may also be performed to determine the Z_{datum}. A site-specific H&H study will result in a more accurate Z_{datum} than Approach 1 or Approach 2 described herein.

Approach 1: Quantitative methodology to approximate the annual high water level elevation

This process has been developed to aid in determining the water surface elevation of the annual high water level. It is an approximation based on publicly available data. To determine the flow, the U.S. Geological Survey (USGS) Streamstats application is used (USGS n.d.). This web application uses regional regression equations to estimate hydrologic data for a selected watershed. In addition, a cross-section will need to be created to apply the hydrologic data to. A non-bathymetric cross-section can be created using a web application developed by the USGS 3D elevation program. The non-bathymetric dataset provided on this web application does not represent the channel bottom, rather it corresponds to the elevation of the water surface on the day that the aerial survey was conducted. Therefore, it is necessary to estimate the channel bottom elevation for proper calculation of the annual high water level. The channel bottom elevation can be determined from the Flood Insurance Study (FIS). For the channel cross-section that is below the water level, an extrapolated side slope will be used to connect the cross-section above the water surface (non-bathymetric) and the channel bottom elevation. In cases where survey data are available, the survey data should be used instead of the cross-section estimation methods presented herein. All of these data will be used in Manning's equation to determine a depth of flow, which will provide the water surface elevation associated with the selected recurrence interval data.

Prior to using this process, it is important to understand that this process will typically produce the 2-year (50% Annual Exceedance Probability [AEP]) recurrence interval water level elevation, not the annual (1-year) high water level elevation. In some instances, the 1.5-year (66.7% AEP) recurrence interval water level elevation will be calculated. Given the challenges of obtaining the annual high water level elevation, it is reasonable to use the 2-year water level elevation as an approximation of the annual high water level elevation. Note that using the 2-year water level elevation as an approximation of the annual high water level elevation will provide a less conservative stillwater elevation corresponding to the risk category and MRI ($SWEL_{MRI}$) value when compared to the use of the annual high water level elevation (1-year water elevation).

This process relies on USGS StreamStats data. USGS StreamStats data are fully implemented for the majority of the United States, but some areas still undergoing implementation. The StreamStats

About page has a map depicting the status of StreamStats data across the United States: https://www.usgs.gov/streamstats/about.

For regulated streams, StreamStats warns the user during the delineation process and within the report that the regression equations may not apply. The warning is produced because StreamStats filters all of the water from a designated watershed to one point, but regulated waterways capture water and cause it to not flow down. Because the regulation process is not being captured by StreamStats, StreamStats overestimates discharge, which in turn creates an underestimation of the 2-year or 1.5-year recurrence interval water level elevation with the method outlined in this appendix. Although the water level elevation may be underestimated, this method may be used for regulated streams if it is the best method available within the scope of the project.

Step 1) Delineate the watershed in StreamStats.

1A. Navigate to the USGS StreamStats Application: https://streamstats.usgs.gov/ss/

1B. Enter the address of the building in the search bar, see Figure 38.

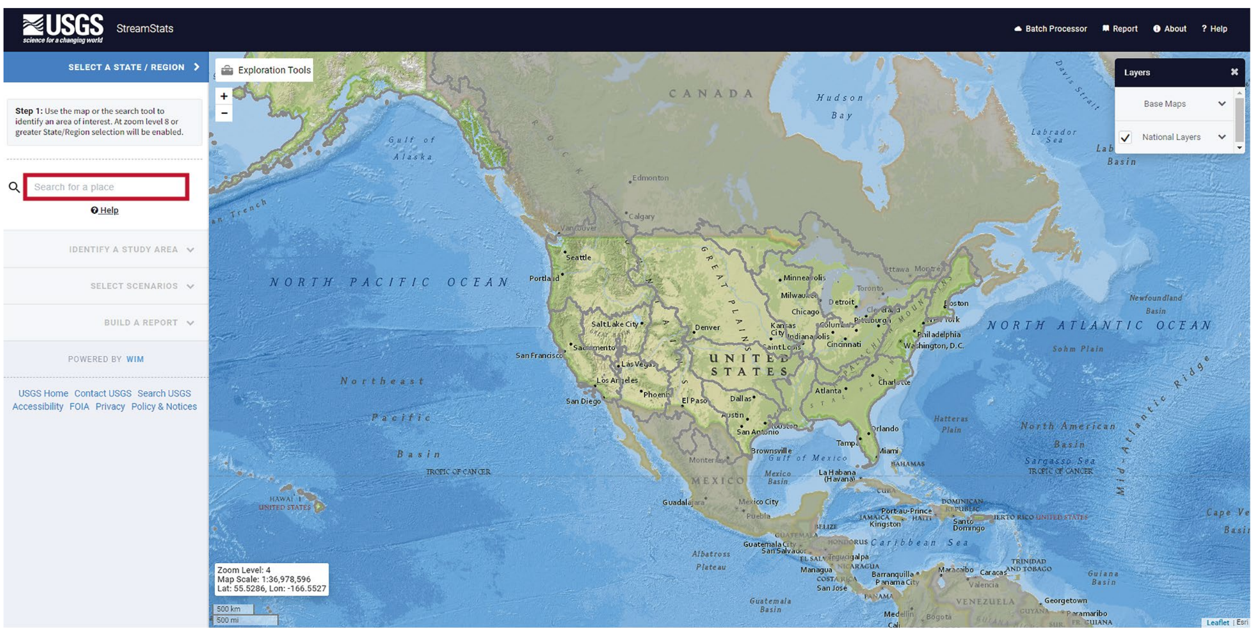


Figure 38. StreamStats page depicting the search bar location

1C. Once the page is zoomed into the area, select the state of the area of interest, see Figure 39.

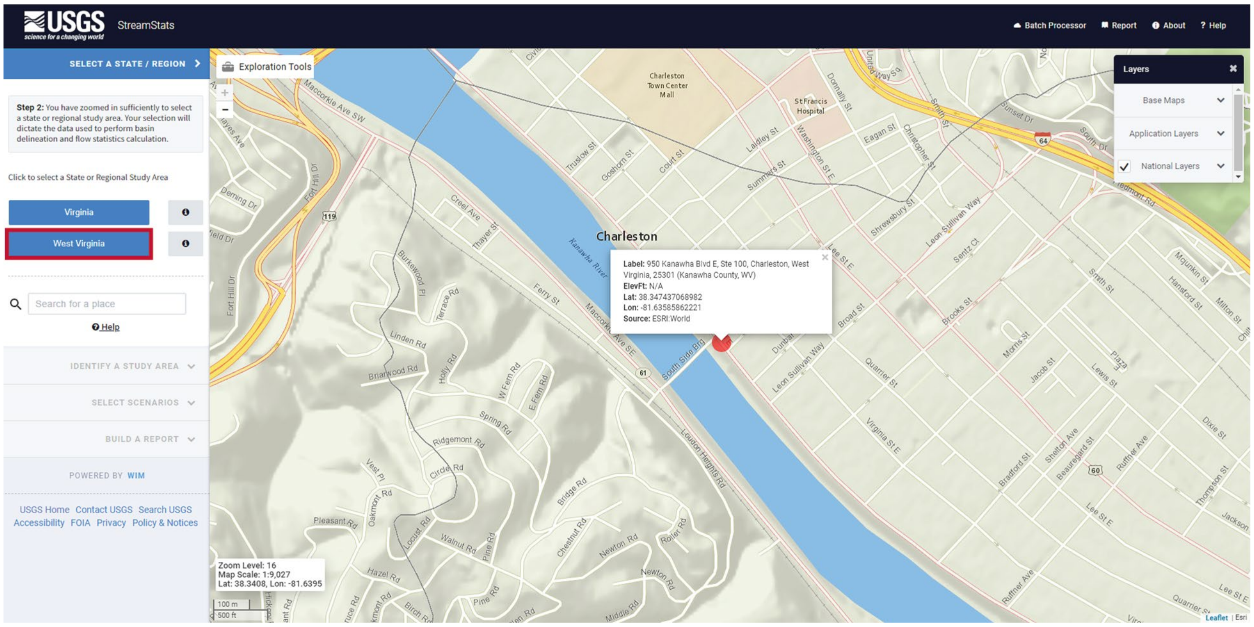


Figure 39. StreamStats page depicting state selection

1D. Once the state is selected, zoom in to the area of interest until the "Delineate" button appears. This is achieved by zooming in to at least "Zoom Level 15" as defined by the application. Click the delineate button (see Figure 40) and select a point on the stream that is most relevant to the area of interest (see Figure 41). The selected point must be a point on one of the blue lines shown in the application.

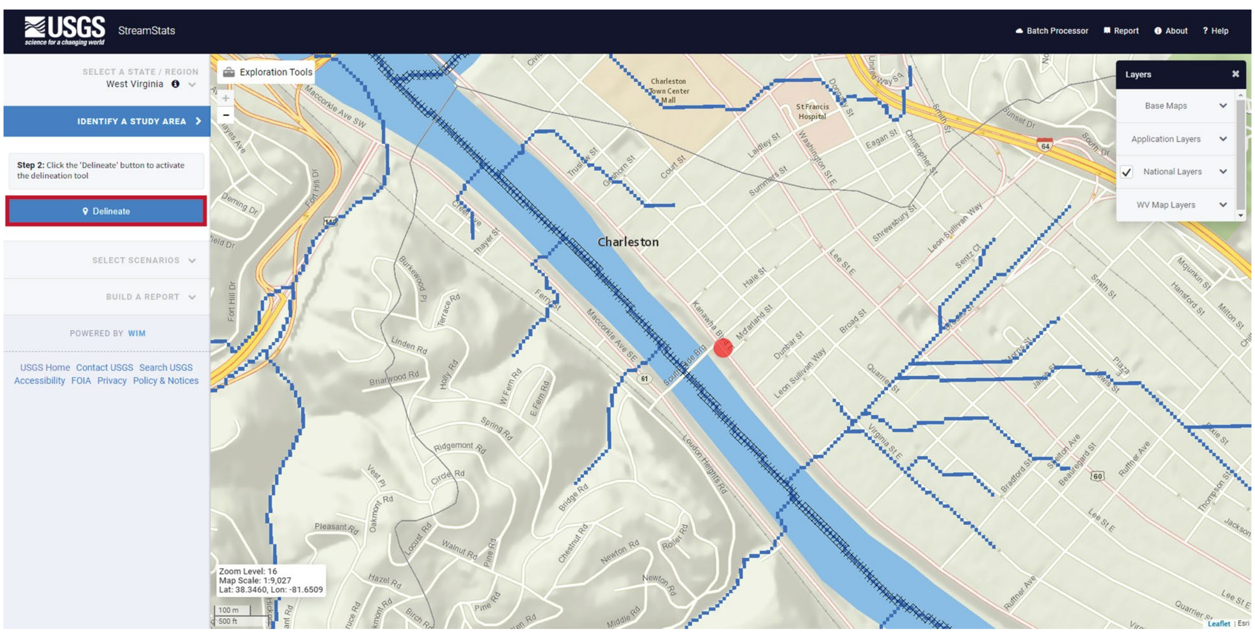


Figure 40. StreamStats page depicting "Delineate" button

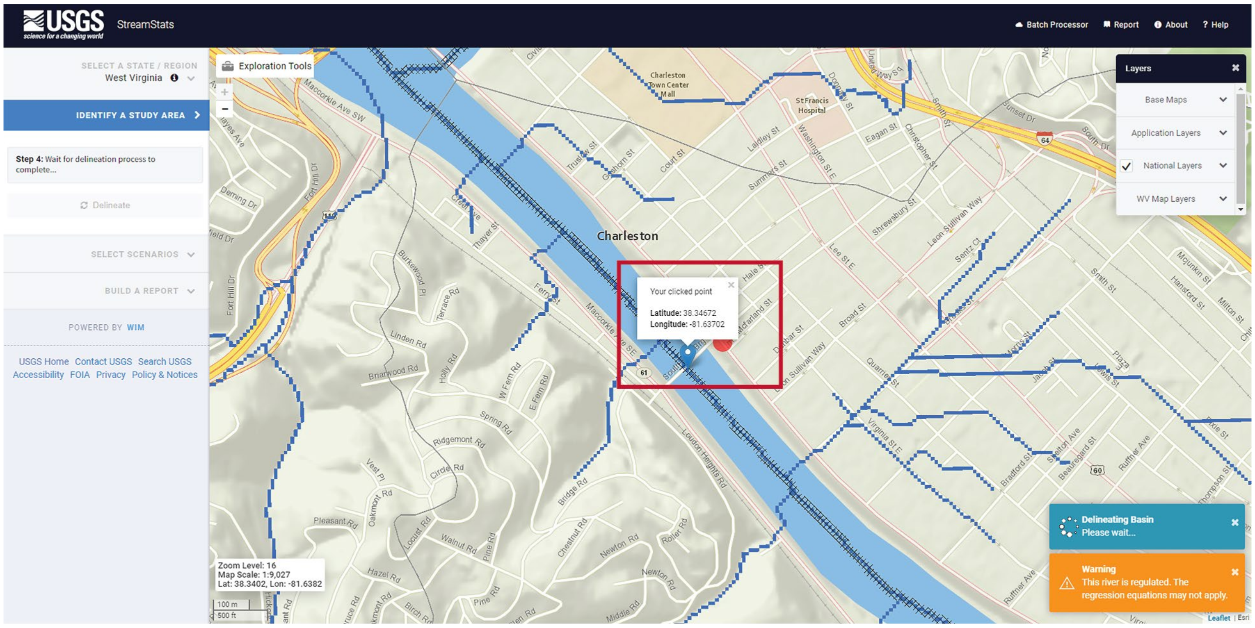


Figure 41. StreamStats page depicting point selection for the example location

1E. Step 1D returns a delineated basin. If it does not cover the area of interest, add the "Non-simplified Basin" layer. If the area of interest is still not covered, move the point. If it does cover the area of interest then click "Continue," see Figure 42.

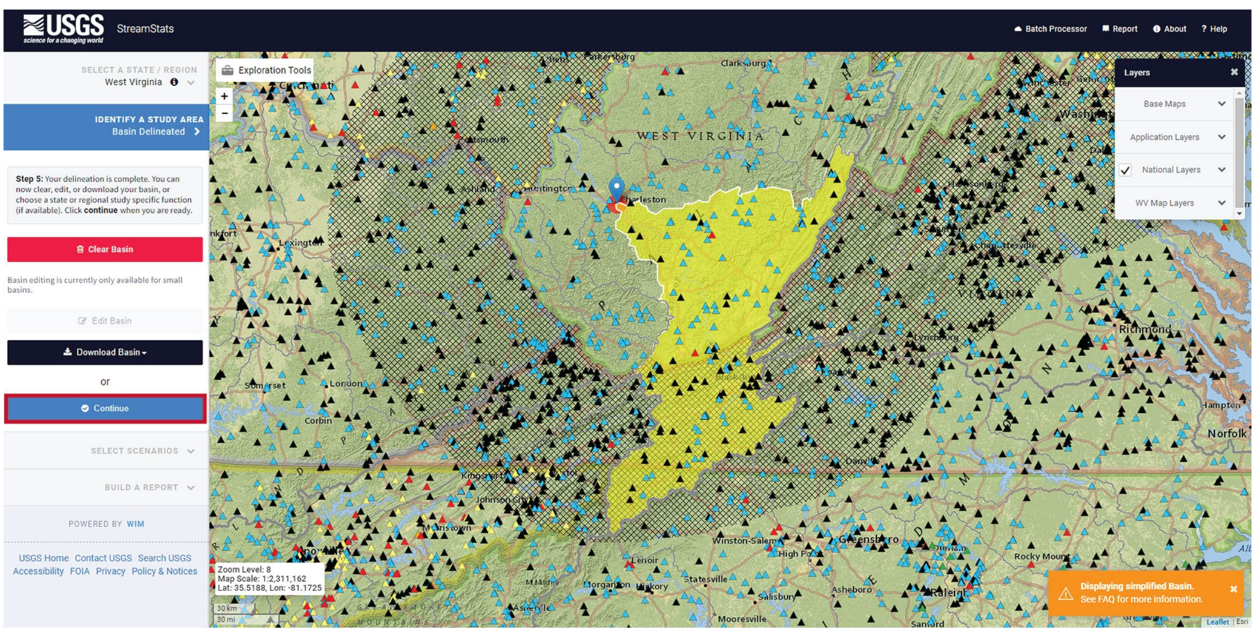


Figure 42. StreamStats page depicting "Continue" button

Figure 43 and Figure 44 show an example of an area where adding the "Non-simplified Basin" layer was required to show that the area of interest was captured.

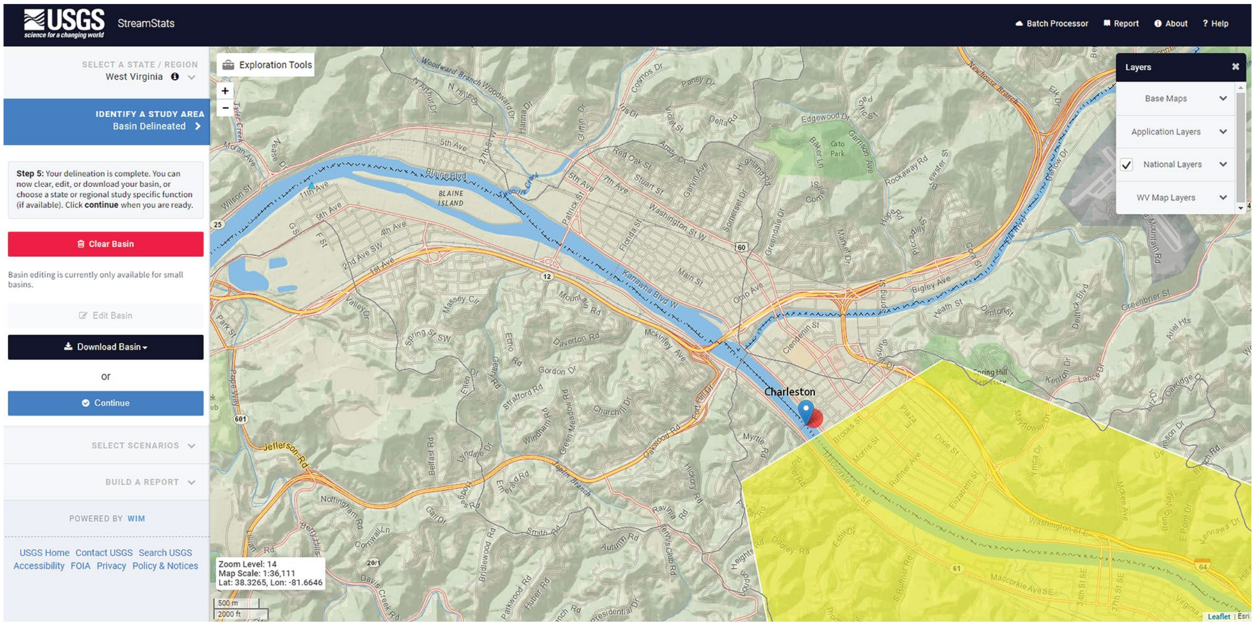


Figure 43. StreamStats page depicting simplified basin delineation

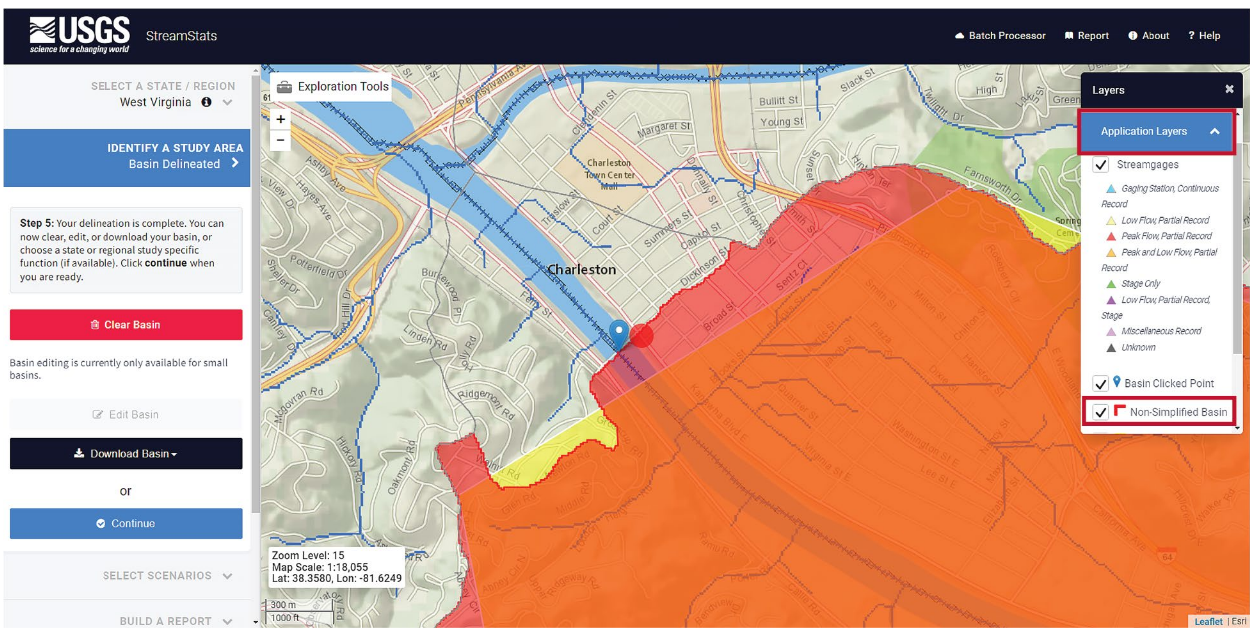


Figure 44. StreamStats page depicting non-simplified basin delineation

1F. Next, regression-based scenarios will be selected. StreamStats uses regional regression equations to determine peak flows. When selecting scenarios, it is important to select the correct option. The default scenario to select is "Peak-Flow Statistics." This scenario will be used for non-urbanized watersheds where the percent impervious land use is very low. In densely urban areas, or areas with high percentages of impervious surfaces, there are often not adequate data points (unregulated stream gages) to develop reliable regression equations. Urban areas experience higher peak flows because of a lack of infiltration of stormwater into the soil. In some cases, StreamStats will provide an option for an "Urban Peak-Flow" regression scenario. If the area of interest has an option to select "Urban Peak-Flow Statistics," select that regression scenario as well, see Figure 45. The "Urban Peak-Flow Statistics" will provide an option for flow values in an urbanized watershed. Selecting the "Regression Based Scenario" will automatically select the correct "Basin Characteristics" from the drop down; do not change these selections. Click "Continue."

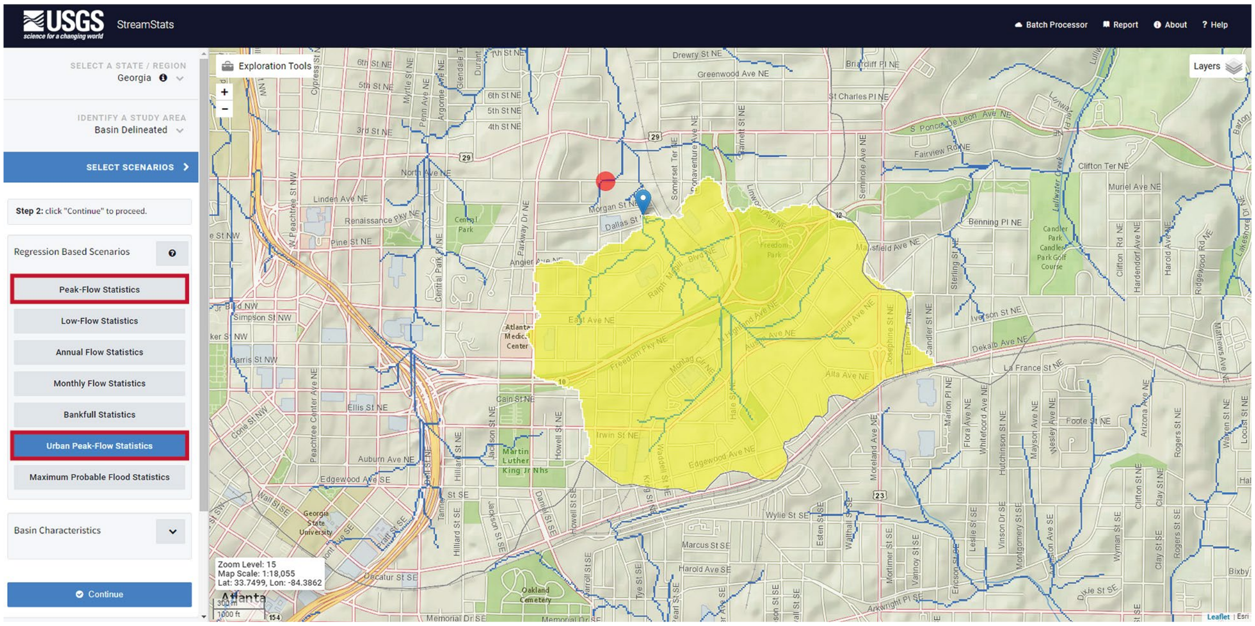


Figure 45. StreamStats page depicting an example of a "Regression Based Scenarios" menu and selections

1G. Once the report has been generated, click "Open Report," see Figure 46.

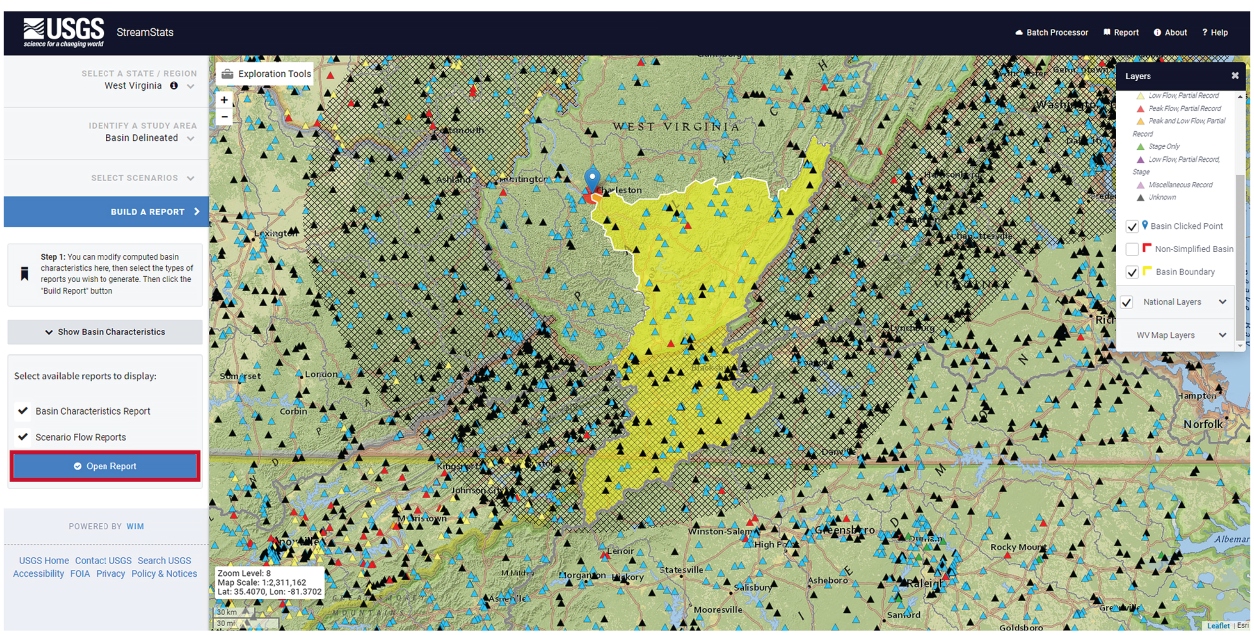


Figure 46. StreamStats page depicting "Open Report" button

Users are also encouraged to read the applicable State/Regional Info to gain an understanding of the applicable ranges of values provides in the report. State/Regional Info can be accessed by clicking "About" and navigating to the "State/Regional Info" tab, see Figure 47.

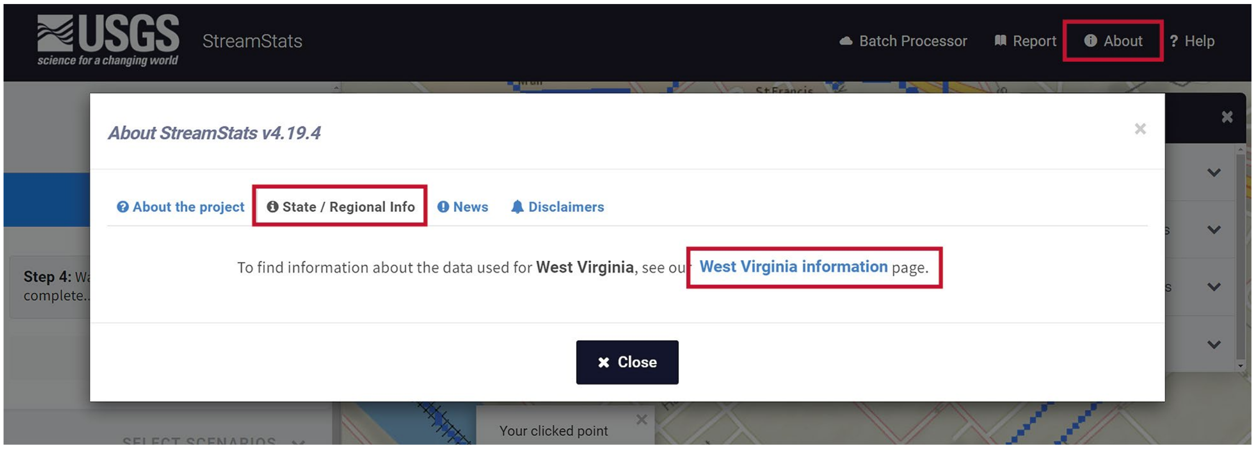


Figure 47. StreamStats page depicting navigation actions to access State/Regional Info

1H. Once the report is open, it can be re-named and notes can be added before saving it as a PDF. The whole report can also be viewed in this window. Scroll down to either the "Peak-Flow Statistics" or the "Urban Peak-Flow Statistics" to look at the flood recurrence intervals and associated values. In most instances, the value used for the analysis is the "50-percent AEP flood." This is the 50% AEP, otherwise known as the 2-year recurrence interval. If a higher-percent AEP is available, such as the "66.7-percent AEP" flood (otherwise known as the 1.5-year recurrence interval), use the higher-percent value.

In some cases, a watershed will span two or more physiographic provinces. Therefore, it will have two or more different regression equations. When this is the case, StreamStats will provide an "Area-Averaged" Peak-Flow Statistics Report, see Figure 48. Use the Area-Averaged values for flow in these cases.

Peak-Flow Statistics Flow Report [Area-Averaged]

Statistic	Value	Unit
66.7-percent AEP flood	165000	ft^3/s
50-percent AEP flood	188000	ft^3/s
20-percent AEP flood	244000	ft^3/s
10-percent AEP flood	284000	ft^3/s
4-percent AEP flood	334000	ft^3/s
2-percent AEP flood	374000	ft^3/s
1-percent AEP flood	417000	ft^3/s
0.5-percent AEP flood	457000	ft^3/s
0.2-percent AEP flood	514000	ft^3/s

Figure 48. StreamStats report depicting Area-Averaged Peak-Flow Statistics

If both "Peak-Flow Statistics" and "Urban Peak-Flow" were selected, the report and State/Regional Info can be used to determine which scenario should be used. In most cases, the report will provide a warning if the percent impervious land does not meet the required threshold. Figure 49 shows an example where the percent impervious is 0.1 and the allowable range for the Urban Peak-Flow is shown as 2 to 54.6. Since a percent impervious of 0.1 is outside of the allowable range for Urban Peak-Flow Statistics, StreamStats automatically highlighted the value and provided a warning message. If a warning is provided for the percent impervious for "Urban Peak-Flow," the "Peak-Flow Statistics" scenario should be used. Additionally, some states/regions provide guidance on the applicable uses of "Urban Peak-Flow" and "Peak-Flow" statistics, see Step 1G for access to "State/Regional Info."

EXAMPLE

State/Regional Info in StreamStats

The North Carolina StreamStats page provides an example of "State/Regional Info" that is useful for determining which Peak-Flow statistics to use. The North Carolina StreamStats page states, "Use of the urban peak-flow estimates is most appropriate when impervious areas are 10 percent or more. In basins with impervious areas of less than 10 percent, the computed urban peak discharge may be less than the computed rural peak discharge." So, for sites in North Carolina, it is advisable to use the "Peak-Flow Statistics" scenario when the impervious area is less than 10%.

Urban Peak-Flow Statistics

Urban Peak-Flow Statistics Parameters [Region 2 Urban Blue Ridge FS007-00]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	13.4	square miles	0.04	41
LC11IMP	Percent_Impervious_NLCD2011	0.1	percent	2	54.6

Urban Peak-Flow Statistics Disclaimers [Region 2 Urban Blue Ridge FS007-00]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Figure 49. StreamStats report depicting a warning for Urban Peak-Flow due to percent impervious

Step 2) Obtain a cross-section of the stream above the hydro-flattened surface

ADDITIONAL CONSIDERATIONS

River Cross-Section

Steps 2 through 4 outline a process to approximate the river cross-section. These steps may be skipped if a cross-section of the river near the site is available. A survey may be completed to obtain the river cross-section.

Step 2 is to obtain a cross-section of the stream of interest from the hydro-flattened Digital Elevation Model (DEM) available from the USGS 3D Elevation Program (3DEP), accessed in the National Map Viewer (USGS n.d.). The hydro-flattened DEM does not represent the channel because the LiDAR is unable to penetrate the water surface. Step 3 will provide a method to determine the cross-section below the water surface.

2A. Access the USGS 3DEP dataset at https://apps.nationalmap.gov/viewer/. Navigate to the area of interest. The search bar may be used for initial navigation, further zooming-in may be required for accuracy. If the display imagery disappears during navigation, open the layer list and select "Imagery (NAIP Plus)," see Figure 50.

RESOURCES

3DEP data

This appendix demonstrates use of The National Map Viewer for 3DEP data, but 3DEP data may also be accessed at https://apps.nationalmap.gov/3depdem/.

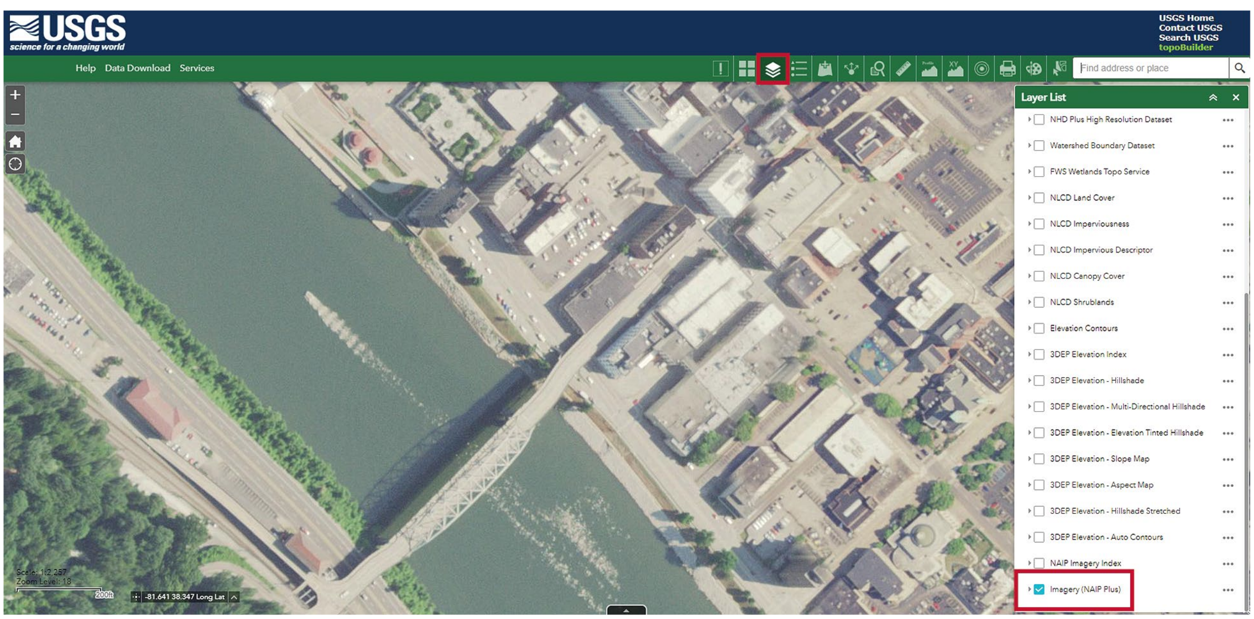


Figure 50. The National Map Viewer page depicting the layer list

2B. Click the "Spot Elevation" tool and select "Activate," see Figure 51. Use the tool to specify points to get station and elevation data. Space points tightly and in a straight line perpendicular to the channel, see Figure 52. The “Draw” tool represented by a painter’s pallet provides an option to draw a line, which can be helpful in making sure that the selected spot elevations are perpendicular to the channel/river centerline.

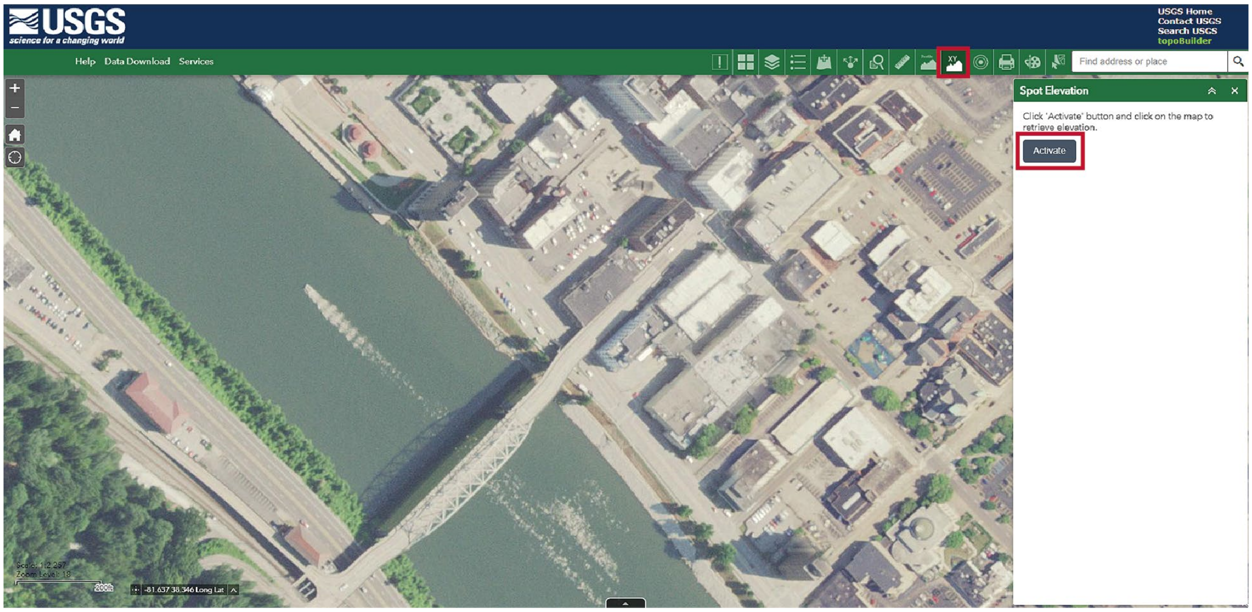


Figure 51. The National Map Viewer page depicting the Spot Elevation tool

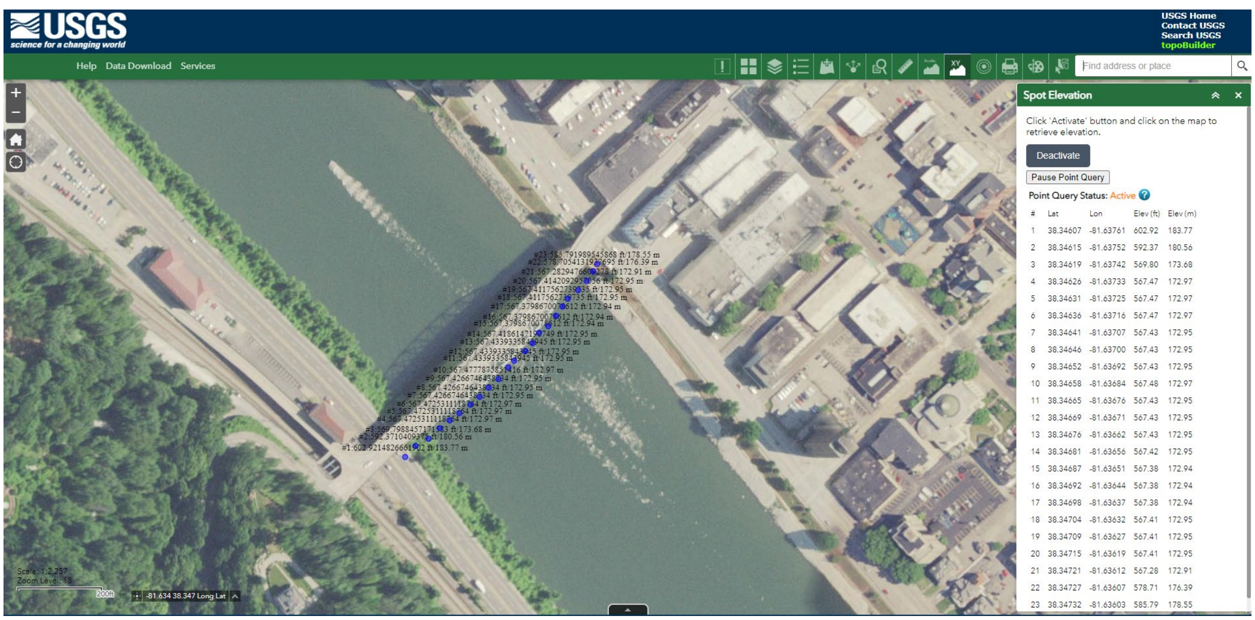


Figure 52. The National Map Viewer page depicting example spot elevations

2C. Input the point data from the "Spot Elevation" tool to Microsoft® Excel®. The data may be copy/pasted from the spot elevation tool into Excel. If the data appears in a single cell when entered into Excel copy/paste the data to a .txt file (e.g., Notepad) and then copy/paste from the .txt file into Excel to enable the data to be properly formatted in individual cells.

2D. Once the data is input into Excel, convert latitude and longitude[7] to feet. Then, calculate the distance between each point. The delta between points can be calculated by:

- Determining the delta between each neighboring latitude and longitude, then
- Squaring and summing both the delta latitude and the delta longitude, then
- Taking the square-root of the summation.

The station relative to the first point is then calculated by adding the distance between points progressively. Plotting the station relative to the first point vs. elevation provides a visual check for the data processing and will be useful in future steps. See Figure 53 for an example.

[7] One degree of latitude equals approximately 364,000 feet. One degree of longitude equals 288,200 feet. (https://www.usgs.gov/faqs/how-much-distance-does-a-degree-minute-and-second-cover-your-maps).

	A	B	C	D	E	F	G	H	I	J	K	L
2	Point	Lat	Lon	Elev (ft)	Elev (m)	Lat Converted (feet)	Lon Converted (feet)	Lat Delta (feet)	Lon Delta (feet)	Distance between Points (feet)	Station Relative to first point	Elev (ft)
3	1	38.34607	-81.63761	602.92	183.77	13957969.48	-23527959.2			0	0	602.92
4	2	38.34615	-81.63752	592.37	180.56	13957998.6	-23527933.26	29.12	25.938	39.0	39.0	592.37
5	3	38.34619	-81.63742	569.8	173.68	13958013.16	-23527904.44	14.56	28.82	32.3	71.3	569.8
6	4	38.34626	-81.63733	567.47	172.97	13958038.64	-23527878.51	25.48	25.938	36.4	107.6	567.47
7	5	38.34631	-81.63725	567.47	172.97	13958056.84	-23527855.45	18.2	23.056	29.4	137.0	567.47
8	6	38.34636	-81.63716	567.47	172.97	13958075.04	-23527829.51	18.2	25.938	31.7	168.7	567.47
9	7	38.34641	-81.63707	567.43	172.95	13958093.24	-23527803.57	18.2	25.938	31.7	200.4	567.43
10	8	38.34646	-81.637	567.43	172.95	13958111.44	-23527783.4	18.2	20.174	27.2	227.6	567.43
11	9	38.34652	-81.63692	567.43	172.95	13958133.28	-23527760.34	21.84	23.056	31.8	259.3	567.43
12	10	38.34658	-81.63684	567.48	172.97	13958155.12	-23527737.29	21.84	23.056	31.8	291.1	567.48
13	11	38.34665	-81.63676	567.43	172.95	13958180.6	-23527714.23	25.48	23.056	34.4	325.4	567.43
14	12	38.34669	-81.63671	567.43	172.95	13958195.16	-23527699.82	14.56	14.41	20.5	345.9	567.43
15	13	38.34676	-81.63662	567.43	172.95	13958220.64	-23527673.88	25.48	25.938	36.4	382.3	567.43
16	14	38.34681	-81.63656	567.42	172.95	13958238.84	-23527656.59	18.2	17.292	25.1	407.4	567.42
17	15	38.34687	-81.63651	567.38	172.94	13958260.68	-23527642.18	21.84	14.41	26.2	433.6	567.38
18	16	38.34692	-81.63644	567.38	172.94	13958278.88	-23527622.01	18.2	20.174	27.2	460.7	567.38
19	17	38.34698	-81.63637	567.38	172.94	13958300.72	-23527601.83	21.84	20.174	29.7	490.5	567.38
20	18	38.34704	-81.63632	567.41	172.95	13958322.56	-23527587.42	21.84	14.41	26.2	516.6	567.41
21	19	38.34709	-81.63627	567.41	172.95	13958340.76	-23527573.01	18.2	14.41	23.2	539.8	567.41
22	20	38.34715	-81.63619	567.41	172.95	13958362.6	-23527549.96	21.84	23.056	31.8	571.6	567.41
23	21	38.34721	-81.63612	567.28	172.91	13958384.44	-23527529.78	21.84	20.174	29.7	601.3	567.28
24	22	38.34727	-81.63607	578.71	176.39	13958406.28	-23527515.37	21.84	14.41	26.2	627.5	578.71
25	23	38.34732	-81.63603	585.79	178.55	13958424.48	-23527503.85	18.2	11.528	21.5	649.0	585.79
26	From USGS National Map					= B#*364000	= C#*288200	= F# - F(#-1)	= I# - I(#-1)	= SQRT((H#^2)+(I#^2))	= K(#-1) + J(#)	= D#

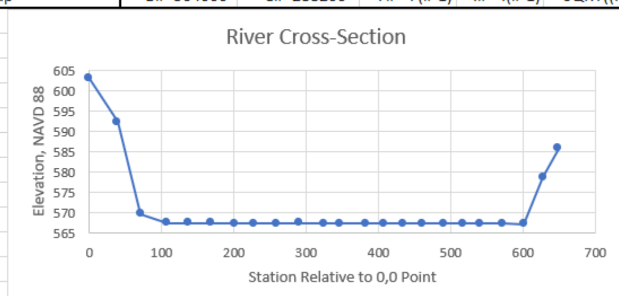
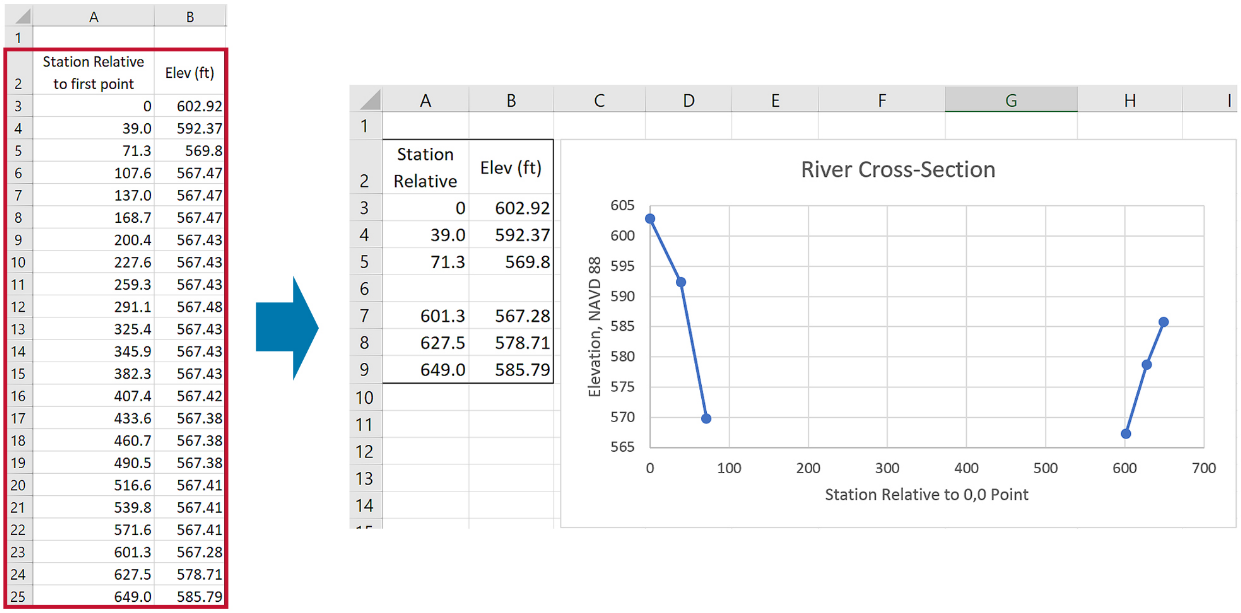


Figure 53. Example of process for converting data points from The National Map Viewer

Step 3) Create a generalized bathymetric cross-section

Step 3 is to create a generalized bathymetric cross-section. The 3DEP dataset provided by the USGS does not accurately represent the channel bottom. To approximate the bathymetric cross-section, the elevation of the channel bottom from the FIS is used. The side slopes of the channel cross-section will be extrapolated downwards to connect the edges of the hydro-flattened surface to the channel bottom elevation.

3A. Make a copy the river cross-section sheet. Copy/paste all values in-place in the new sheet. The values should be pasted as “Values” to remove all formulas. Delete Columns A through J. Delete the rows that contain the hydro-flattened surface values and do not influence the side-slope profiles. See Figure 54 for an example; the values in the box on the left represent the hydro-flattened surface values that were removed for this step.



	A	B
1		
2	Station Relative to first point	Elev (ft)
3	0	602.92
4	39.0	592.37
5	71.3	569.8
6	107.6	567.47
7	137.0	567.47
8	168.7	567.47
9	200.4	567.43
10	227.6	567.43
11	259.3	567.43
12	291.1	567.48
13	325.4	567.43
14	345.9	567.43
15	382.3	567.43
16	407.4	567.42
17	433.6	567.38
18	460.7	567.38
19	490.5	567.38
20	516.6	567.41
21	539.8	567.41
22	571.6	567.41
23	601.3	567.28
24	627.5	578.71
25	649.0	585.79

	A	B
1		
2	Station Relative	Elev (ft)
3	0	602.92
4	39.0	592.37
5	71.3	569.8
6		
7	601.3	567.28
8	627.5	578.71
9	649.0	585.79

Figure 54. Example of process for removing hydro-flattened surface values

3B. Obtain the riverbed elevation from the FIS. The riverbed elevation can be found in the FIS by using the Flood Profile associated with the project location. Flood Profiles reference the riverbed as the stream bed, which is noted on the Flood Profile legend. To identify the elevation, draw a line on the Flood Insurance Rate Map (FIRM) from the project site to the channel centerline; the line must be perpendicular to the channel centerline. Using the FIRM, measure the distance along the channel centerline from where the project site line intersects the channel centerline to a cross-section (indicated by letters surrounded by a hexagon) or a major feature, such as a road crossing the channel. On the associated Flood Profile in the FIS, use this distance to identify the site location and the stream bed (riverbed) elevation. Step 4A shows the Flood Profile for the example project location and indicates the site's stream bed on the Flood Profile. Input this elevation into two elevation cells where the hydro-flattened (water) surface values were just removed, see Figure 55.

	A	B
1		
2	Station Relative	Elev (ft)
3	0	602.92
4	39.0	592.37
5	71.3	569.8
6		543
7		543
8	601.3	567.28
9	627.5	578.71
10	649.0	585.79

Figure 55. Example of adding riverbed elevations

3C. Extend the riverbank to the riverbed by continuing the existing riverbank slope. See Figure 56 for an example of how this can be accomplished in Excel®.

	A	B	C
1			
2	Station Relative	Elev (ft)	
3	0	602.92	
4	39.0	592.37	
5	71.3	569.8	
6	=((A5-A4)/(B5-B4))*(B6-B5)+A5		
7	545.7	543	
8	601.3	567.28	
9	627.5	578.71	
10	649.0	585.79	

	A	B	C
1			
2	Station Relative	Elev (ft)	
3	0	602.92	
4	39.0	592.37	
5	71.3	569.8	
6	109.6	543	
7	=A8-(((A8-A9)/(B8-B9))*(B8-B7))		
8	601.3	567.28	
9	627.5	578.71	
10	649.0	585.79	

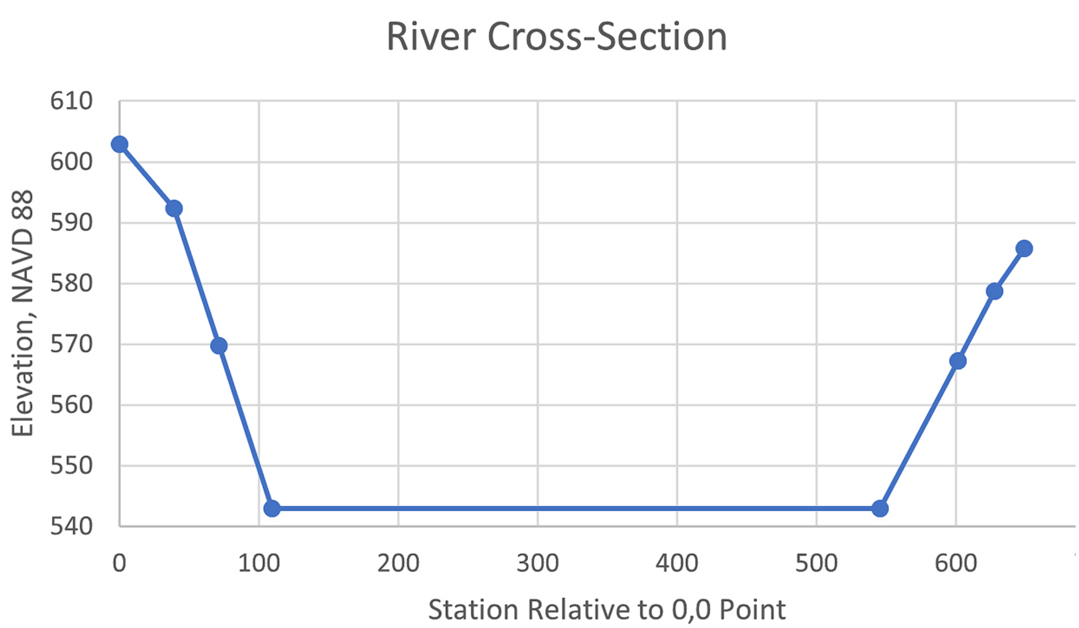


Figure 56. Example of extending the riverbank to the riverbed

Step 4) Find the water surface elevation for a specified AEP

Step 4 uses the data from the previous steps and Manning's equation to find the water surface elevation associated with the discharge for a specific AEP from StreamStats.

4A. Identify the slope of the stream bed along the channel centerline. Channel centerline slopes should be determined from the stream bed identified on the Flood Profile in the FIS. To obtain the channel centerline slope, find the corresponding station on the FIS, and calculate the slope using a point upstream and downstream. It is best to avoid starting or ending on any large anomalies in the profile. Trying to begin and end on a similar bed feature (i.e., a peak to a peak or a valley to a valley) is also recommended. See Figure 57 for an example of slope determination. The star symbol depicts where the cross-section associated with the project site intersects the riverbed, and points A and B denote the selected downstream and upstream points, respectively. The slope can be calculated as (547-547)/(315400-307600) = 0.00026.

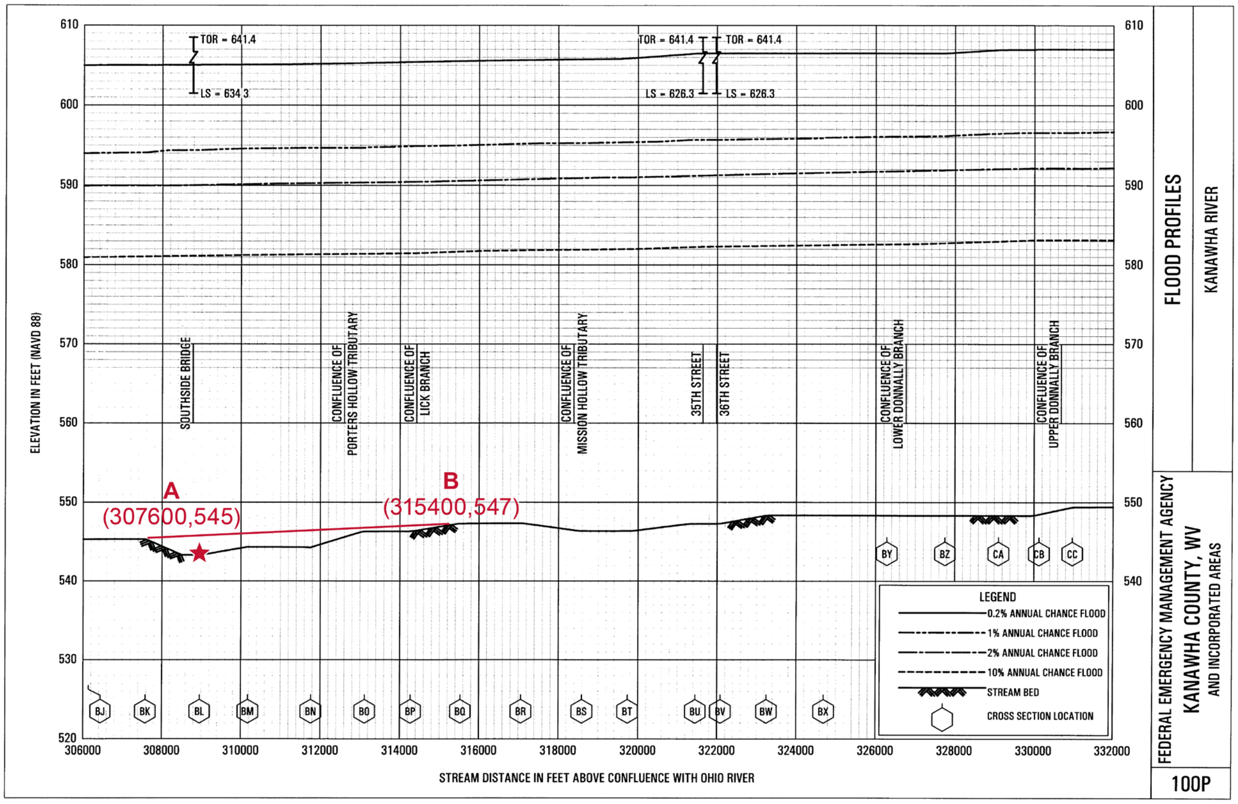


Figure 57. Example of slope determination by selecting points upstream and downstream

4B. Engineering judgement is required to select a Manning's n value. This can be done with the help of the USGS *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains* (USGS 1989). The Manning's n value may also be specified in the FIS under a table containing the words "Manning's "n" value." The Manning's n value from the FIS is sometimes specified as a large range; in those cases, both the FIS data and the USGS guide should be used to determine an appropriate value. Selecting a lower Manning's n value will result in a lower water surface elevation, which produces a conservative $SWEL_{MRI}$.

4C. The Normal Depth Calculator tool by the National Weather Service can be used to solve for the depth of flow (https://www.weather.gov/aprfc/NormalDepthCalc). Select "Water Surface (normal depth)" from the "Solve For:" dropdown. Enter the following items into the calculator tool:

- The localized slope at the cross-section from Step 4A,
- The Manning's n value from Step 4B,

- The flow for the 50% (2-year event) or 66.7% (1.5-year event) AEP flood from Step 1H, and
- The cross-section data entered in feet, which can be entered as x,y (the columns from a csv or xlsx file can be copy/pasted into the "Load CSV XS Data Below" data box) from Step 3C.

NOTE: Do not edit the value for the WSE (water surface elevation).

This will output the water surface elevation (WSE) that can be used as the Z_{datum} value. See Figure 58 for an example of The Normal Depth Calculator tool inputs and outputs. The user should check the WSE output against other available flood elevations in the FIS. For example, the WSE in Figure 58 is 573 feet and represents the 1.5-year flood elevation because the 66.6% AEP was used in the example. The 10-year flood elevation from the FIS is 581 feet. Since the calculated 1.5-year flood elevation is less than the 10-year flood elevation and no MRI flood elevations below the 10-year flood elevation are known for comparison, the example solution can be accepted. However, if the calculated 1.5-year flood elevation were at or above the 10-year flood elevation, the user should re-check their work and, if required, consider alternate methods.

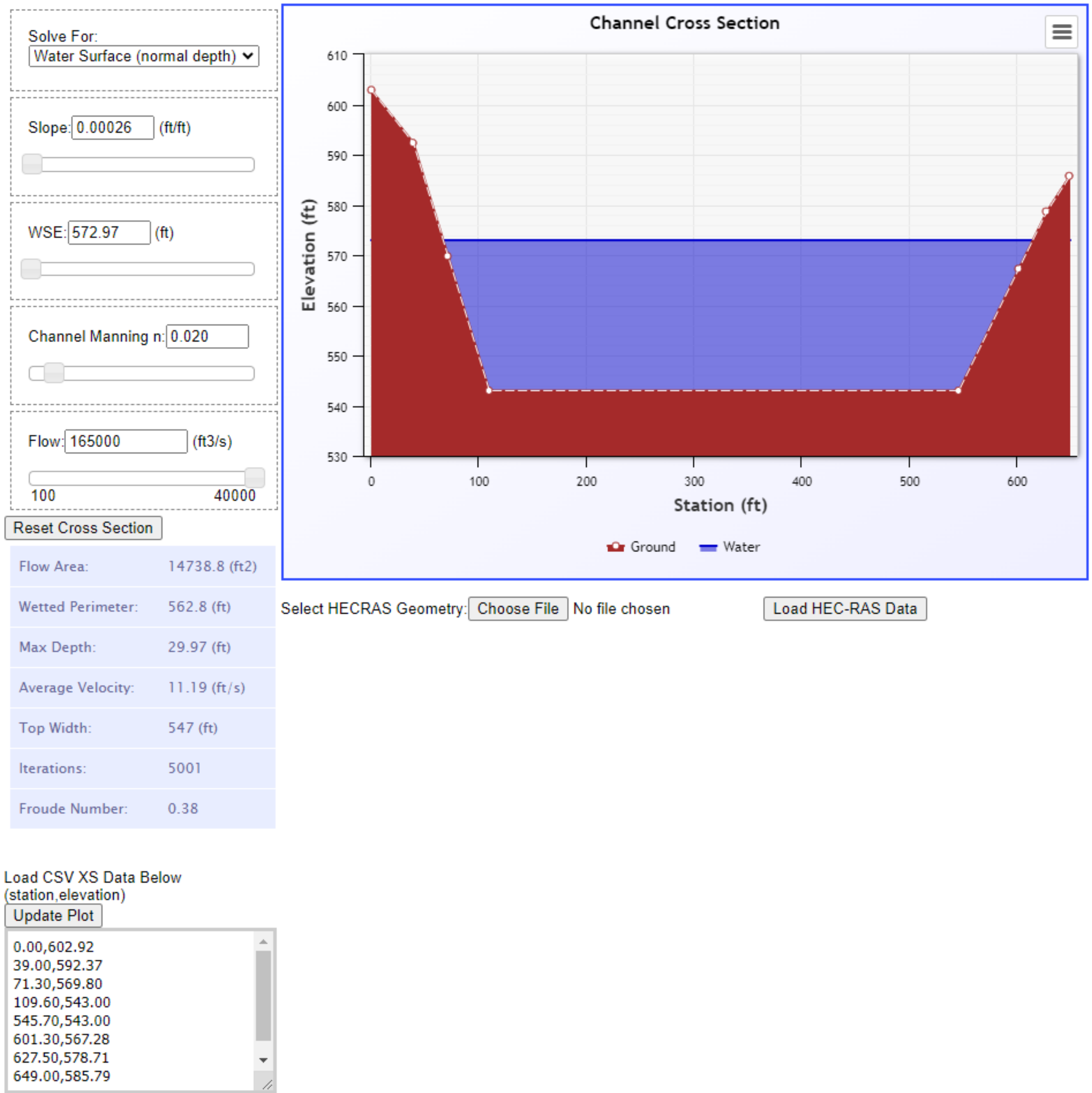


Figure 58. Example of the Normal Depth Calculator tool inputs and outputs

Approach 2: Qualitative methodology to determine the annual high water level, ordinary high water mark

Designers should be cautioned that this method is the least recommended option for approximation of the Z_{datum}. This method relies on field indicators and could result in a less conservative Z_{datum} value than using other methods.

In order to determine the Z_{datum} using this method, assumptions must be made to provide a generalized approached for building designers. The first and most important assumption is that the annual high water level can equate to the ordinary high water mark (OHWM). The OHWM is required for establishing the lateral extents of US Army Corps of Engineers jurisdiction in non-tidal streams. The ordinary high water mark is not tied to an annual flood frequency, but rather relies on physical indicators to identify the OHWM (Curtis et al. 2011). The OHWM does not consider extreme flows, defined as relatively rare events that deeply submerge the floodplain, but is instead driven by small, frequent flood flows (USACE 2022).

The OHWM is often associated with the bankfull stage. Per NOAA, the bankfull stage is "an established gage height at a given location along a river or stream, above which a rise in water surface will cause the river or stream to overflow the lowest natural stream bank somewhere in the corresponding reach" (NWS n.d.). The bankfull stage is often associated with the 1.5-year recurrence interval (Rosgen 1996). The bankfull stage and the OHWM are both broad descriptions of vertical elevation thresholds at which physical and vegetative transitions will occur. Field indicators of both determinations often overlap, although the terms are not interchangeable. However, in order to make a determination of the Z_{datum}, or the annual high water level, an assumption must be made to tie the elevation to a recurrence interval. Therefore, given the assumption that the OHWM is equivalent to the 1.5-year recurrence interval, the OHWM can be used to approximate the Z_{datum}.

However, there is uncertainty with the assumption that the OHWM is equivalent to the 1.5-year recurrence interval. The uncertainty is due to the OHWM's visibility and discernability being masked or affected by anthropogenic influences and large flood events, as well as other natural disasters such as fires and landslides. Therefore, the OHWM is not consistently connected to a specific recurrence interval and any frequency of flow connected to the OHWM may vary greatly across different climatic and landscape settings. While the OHWM may occur at the 1.01 event in one stream, it could be the 15-year event in another (Curtis et al. 2011). Therefore, when using the OHWM, designers should be aware that there is a large degree of uncertainty in flood elevations and their associated recurrence intervals.

With consideration of the noted caveats, the Z_{datum} may be approximated as being equal to the OHWM elevation. The OHWM can be obtained by performing a survey for OHWM elevations. As previously stated, this is a generalized approach and OHWM values obtained should be understood to be a broad estimate of Z_{datum}. There are several key indicators of the OHWM defined by the USACE. Ideally, the selected professional surveyor will have experience in assessing OHWMs and may have prior experience performing jurisdictional determinations for USACE. The survey will look for indicators such as shelving, wracking, water staining, etc. These indicators will be individually described for clarity. The indicator descriptions are from a USACE report titled *National Ordinary High Water Mark Field Delineation Manual for Rivers and Streams* (USACE 2022). The USACE report is a nearly 400-page manual with more in-depth descriptions of the identified indicators as well as figures, images, and examples.

OHWM indicators include:

Shelving

Shelving is the formation of natural berms, levees, and terraces that result from depositional and erosional processes. These features will be adjacent and parallel to the flow along the channel banks. These features are clear breaks in slope along the channel banks.

Changes in the Character of Soil

Refer to USACE report titled *National Ordinary High Water Mark Field Delineation Manual for Rivers and Streams* (USACE 2022).

Wracking

Wrack or wrack lines are made by organic litter that is deposited on the channel margins after high flows. It also includes inorganic material, such as plastics and other trash. This litter can accumulate in woody vegetation surrounding the channel. This should be used as a corroborating indicator, not a primary indicator.

Sediment Sorting

Sediment sorting will generally be identifiable on channel bars. The sediment is sorted because the velocities near the center of the channel are higher and can, therefore, carry larger material. So, as the flow gets further away from the channel, larger debris is no longer able to be transported. Therefore, the larger material will be closer to the channel center, while the finer material will be located higher on the channel bar.

Leaf Litter Disturbed or Washed Away

Leaf litter accumulation is likely to redistributed or washed away more frequently in areas that are frequently inundated (Riedl et al. 2013). Therefore, a line or trend could emerge at the edge of the leaf litter layer. This should be used as a corroborating indicator, not a primary indicator.

Water Staining

Water staining is the discoloration of rock at the air-water interface. It occurs in areas frequently inundated, so a staining line can be an indicator of the OHWM. However, staining can also occur via suspended microbial or mineral material. This should be used as a corroborating indicator, not a primary indicator.

Change in Plant Community

There is a transitional zone where the dominant vegetation changes from wetland vegetation to upland vegetation. This transitional zone gives a good idea of the OHWM because it is the zone where water is no longer frequently inundating the soil. Below the OHWM, the soil is frequently inundated and is therefore a much better habitat for wetland vegetation. It is important to be familiar with local and regional vegetation before trying to make this distinction.

Appendix D. Calculating Riverine Velocity

Approach 1: Scaling Riverine Floodway Velocity by MRI Based on Manning's Equation

The guidance outlined in this appendix provides a method to scale a riverine floodway velocity given by a Flood Insurance Study (FIS) or flood hazard study based on the given and desired mean recurrence interval (MRI).

Equation 30 and **Equation 31** are a set of equations derived from Manning's Equation for open channel flow. **Equation 30** determines a site-specific constant, C, which includes the roughness coefficient and the streambed (or channel) slope parameters. **Equation 31** incorporates the site-specific constant, C, and the design flood depth in the floodway to determine the design flood velocity.

$$C = \frac{V_{100}}{\left(\frac{d_{f100}}{w + 2d_{f100}}\right)^{2/3}}$$ **Equation 30**

where,

C = site-specific constant in ft/s (m/s)
V_{100} = flood velocity for the 100-year MRI taken at the center of the floodway, in ft/s (m/s)
d_{f100} = the 100-year flood depth taken at the center of the floodway, in ft (m). Note: *This value can usually be obtained from the flood profiles in the FIS by subtracting the riverbed (ground) elevation from the 1% annual-chance flood elevation.*
w = width of the floodway, in ft (m)

$$V_{MRI} = C\left(\frac{d_{fMRI}}{w + 2d_{fMRI}}\right)^{2/3}$$ **Equation 31**

where,

C = site-specific constant as determined by **Equation 30**, in ft/s (m/s)
V_{MRI} = design flood velocity for a specified MRI, in ft/s (m/s)
d_{fMRI} = the MRI design flood depth taken at the center of the floodway, in ft (m). This value may be obtainable from the flood profiles in the FIS by subtracting the riverbed (ground) elevation from the MRI flood elevation or it may be calculated with **Equation 32.**
w = width of the floodway, in ft (m)

$$d_{fMRI} = SWEL_{MRI} - G_r$$ **Equation 32**

where,

$SWEL_{MRI}$ = stillwater elevation corresponding to the specified risk category and MRI, in ft (m). See Section 3.2
G_r = the riverbed (ground) elevation taken at the center of the floodway. This value may be obtainable from the flood profiles in the FIS.

Equation 30 may be applied to MRIs other than the 100-year MRI. If an alternate MRI is used to calculate "C," the velocity (V) and the flood depth (d_f) inputs to **Equation 30** must have the same MRI.

While the velocity in the floodway is representative of the velocity in the stream channel and adjacent areas, it does not represent flood velocities within the remainder of the floodplain. Professional judgment is required to access whether the floodway velocity is sufficiently representative, and this may require a site visit to the floodplain to better understand whether the floodway velocity will provide a representative velocity value. The only way to obtain a more comprehensive understanding of the velocity specific to the project site is to have a two-dimensional hydraulic model conducted for the project site. See Approach 2 for information on a method that uses the principles of two-dimensional hydraulic modeling to determine the flood velocity both inside of and outside of the floodway.

DERIVATION OF APPROACH 1

Approach 1: Scaling the riverine floodway velocity by MRI based on Manning's Equation as presented in this section is based on the Manning's Equation for open channel flow. A rectangular profile was assumed for the equation generation.

Manning's Equation for open channel flow:

$$V = (R)^{2/3} * \frac{1.49}{n} * s^{1/2}$$

where,
n = Manning's Roughness Coefficient
s = Channel Slope, ft/ft
R = Hydraulic Radius, ft

Hydraulic radius for a rectangular profile is:

$$R = \frac{A}{P} = \frac{yw}{w + 2y}$$

where,
w = floodway width, ft
y = flood depth in the floodway, ft

Thus, Manning's Equation for open channel flow for a rectangular profile is:

$$V = \left(\frac{yw}{w + 2y}\right)^{2/3} * \frac{1.49}{n} * s^{1/2}$$

Manning's Equation for open channel flow for a rectangular profile was reorganized to isolate variables/portions of the equation that remain constant for a specific location with variable water depths and can be easily isolated. This portion of the equation was then set to equal a constant, *C*. The use of the constant *C* removes the need to define the slope or roughness value of the floodway. The *w* in the numerator was included in *C* to further simply the equation.

$$\frac{V}{\left(\frac{y}{w + 2y}\right)^{2/3}} = w^{2/3} * \frac{1.49}{n} * s^{1/2} = C$$

Manning's Equation for open channel flow for a rectangular profile could then be re-written as:

$$V = \left(\frac{y}{w + 2y}\right)^{2/3} * C$$

A set of equations, a and b, were generated with the previously presented derivation to enable scaling of a known velocity from a specified MRI event to the unknown velocity for another MRI event given the same location.

Equation a:

$$C = \frac{V_{100}}{\left(\frac{y_{100}}{w + 2y_{100}}\right)^{2/3}}$$

C = site-specific constant in ft/s (m/s)
V_{100} = flood velocity for the 100-year MRI taken at the center of the floodway, in ft/s (m/s)
y_{100} = the 100-year flood depth taken at the center of the floodway, in ft (m). Note: *This value can usually be obtained from the flood profiles in the FIS by subtracting the ground elevation from the 1% annual-chance flood elevation.*
w = width of the floodway, in ft (m)

Note: Equation a may be applied to MRIs other than the 100-year MRI. If an alternate MRI is used to calculate "C," the velocity (V) and the flood depth (d_f) inputs to Equation a must have the same MRI. A 100-year MRI was built into the equation to avoid confusion as the alternate option would be for Equations a and b to both use the "MRI" designation, but there needs to be some differentiation between starting MRI and ending MRI.

Equation b:

$$V_{MRI} = \left(\frac{y_{MRI}}{w + 2y_{MRI}}\right)^{2/3} * C$$

C = site-specific constant, in ft/s (m/s)
V_{MRI} = design flood velocity for a specified MRI, in ft/s (m/s)
y_{MRI} = the MRI design flood depth taken at the center of the floodway, in ft (m). This value may be obtainable from the flood profiles in the FIS by subtracting the ground elevation from the MRI flood elevation or it may be calculated with **Equation 32**.
w = width of the floodway, in ft (m)

y_{100} and y_{MRI} are converted to d_{f100} and d_{fMRI}, respectively, in Approach 1 to remain consistent with ASCE 7-22-S2 nomenclature.

Table 33 provides an overview of the scaling factors obtained with the set of equations presented in this section. Table 33 assumes a Risk Category II structure that incorporates a 500-year MRI with a C_{MRI} depth scaling of 1.35. In this table, $V_{MRI} = V_{500}$.

Table 33. Risk Category II, Riverine Velocity Floodway Scaling, V_{500}/V_{100} Ratio Comparison

Variable	*Site A*	*Site B*	*Site C*	*Site A*	*Site B*	*Site C*	*Site A*	*Site B*	*Site C*
Width (ft)	1000	1000	1000	100	100	100	10	10	10
V_{100} (ft/s)	5	5	5	5	5	5	5	5	5
C_{MRI_500} (for 500- year MRI)	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35	1.35
y_{100}	20	10	5	20	10	5	20	10	5
y_{500}	23.5	12.1	6.4	23.5	12.1	6.4	23.5	12.1	6.4
Coefficient, C	69.7	109.2	172.1	18.3	26.2	39.3	9.2	10.4	12.6
V_{500}	5.54	5.66	5.88	5.39	5.55	5.80	5.10	5.20	5.40
Ratio V_{500}/V_{100}	**1.11**	**1.13**	**1.18**	**1.08**	**1.11**	**1.16**	**1.02**	**1.04**	**1.08**

Based on the scenarios compared in Table 33, the ratios of V_{500}/V_{100} range from 1.04 to 1.18.

COMPARISON STUDY OF APPROACH 1

In order to assess the applicability of **Equation 30** and **Equation 31** for scaling velocities, a study was conducted using two-dimensional (2D) Base Level Engineering (BLE) model data at 13 sites. The study compiled data for the 100-year (1% annual-chance) flood and 500-year (0.2% annual-chance) flood velocities, which were resolved into scaling factors between the 100-year and 500-year velocities at each location. Table 34 provides a list of each of the 13 locations and the calculated scaling factors. The majority of the sites focused on entire watersheds as opposed to individual riverine cross-sections.

A review of the V_{500}/V_{100} scaling factors provided in Table 34, and a comparison with the **Equation 30** and **Equation 31** outputs shown in Table 33, concluded that there is sufficient correlation to support that Approach 1 will produce a reasonable scaling of the velocity increase in riverine floodways.

The scaling factors derived in Table 34 may vary from those in Table 33 for numerous reasons, one being that Table 33 derives the 500-year flood depth using a scaling factor of 1.35, while the results in Table 34 utilize 2D BLE modeling data to derive the 500-year flood depth.

Table 34. Summary Table of 2D BLE Study Outputs

State	*Watershed / River XS*	*V_{500}/V_{100} Average Ratio from 2D BLE Analysis*
Kentucky	Big Sandy Blaine Creek Upper Fork	1.23
Kentucky	Little Sandy Middle Fork	1.24
Kentucky	Bayou de Chen Lower Fork	1.22
Kentucky	Little Scioto-Tygarts Upper Fork	1.16
Kentucky	Tradewater River Middle Fork*	1.15
North Dakota	Downtown Minot	1.35
North Dakota	Souris River downstream of Minot	1.37
North Dakota	Tributaries west of Minot	1.20
North Dakota	Livingston Creek, NE of Minot	1.28
North Dakota	Mouse River, XS AF	1.16
North Dakota	Mouse River, XS AH	1.42
Louisiana	Whiskey Chitto	1.24
Vermont	Mettawee	1.12 – 1.15

* While all other watershed's values are those contained within the floodway, Tradewater River's values are within the extent of the 10-year event and tributaries west of Minot are not within a designated floodway.

Future studies would aid in validating the method provided by Approach 1. These additional studies could evaluate 2D BLE model velocity data at individual riverine cross-sections and extract the inputs for **Equation 30** and **Equation 31** from the 2D BLE model data at these locations, which would provide a more accurate comparative analysis.

Approach 2: Determining Velocity in Riverine Areas Using Simplified Approach from HEC-RAS

National Research Council (NRC) of Canada published a flood design guide, *Guide for Design of Flood-Resistant Buildings* (NRC 2021), which provides a method to estimate riverine velocity from mapped flood elevations. Refer to Section 2.11.1 and Appendix B of the document for guidance.

Document access: https://nrc-publications.canada.ca/eng/view/object/?id=96b3275c-b731-4fa6-847e-e2a9a0f080d8