8 Determining Site-Specific Loads

This chapter provides guidance on determining site-specific loads from high winds, flooding, and seismic events. The loads determined in accordance with this guidance are applied to the design of building elements described in Chapters 9 through 15.

The guidance is intended to illustrate important concepts and best practices in accordance with building codes and standards and does not represent an exhaustive collection of load calculation methods. Examples of problems are provided to illustrate the application of design load provisions of ASCE 7-10. For more detailed guidance, see the applicable building codes or standards.

Figure 8-1 shows the process of determining site-specific loads for three natural hazards (flood, wind, and seismic events). The process includes identifying the applicable building codes and standards for the selected site, identifying building characteristics that affect loads, and determining factored design loads using applicable load combinations. Model building codes and standards may not provide

CROSS REFERENCE

For resources that augment the guidance and other information in this Manual, see the Residential Coastal Construction Web site (http://www.fema.gov/rebuild/mat/fema55.shtm).

NOTE

All coastal residential buildings must be designed and constructed to prevent flotation, collapse, and lateral movement due to the effects of wind and water loads acting simultaneously.

load determination and design guidance for the hazards that are listed in the figure. In such instances, supplemental guidance should be sought.

The loads and load combinations used in this Manual are required by ASCE 7-10 unless otherwise noted. Although the design concepts that are presented in this Manual are applicable to both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), all calculations, analyses, and load combinations are based on ASD. Extension of the design concepts presented in this Manual to the LRFD format can be achieved by modifying the calculations to use strength-level loads and resistances.

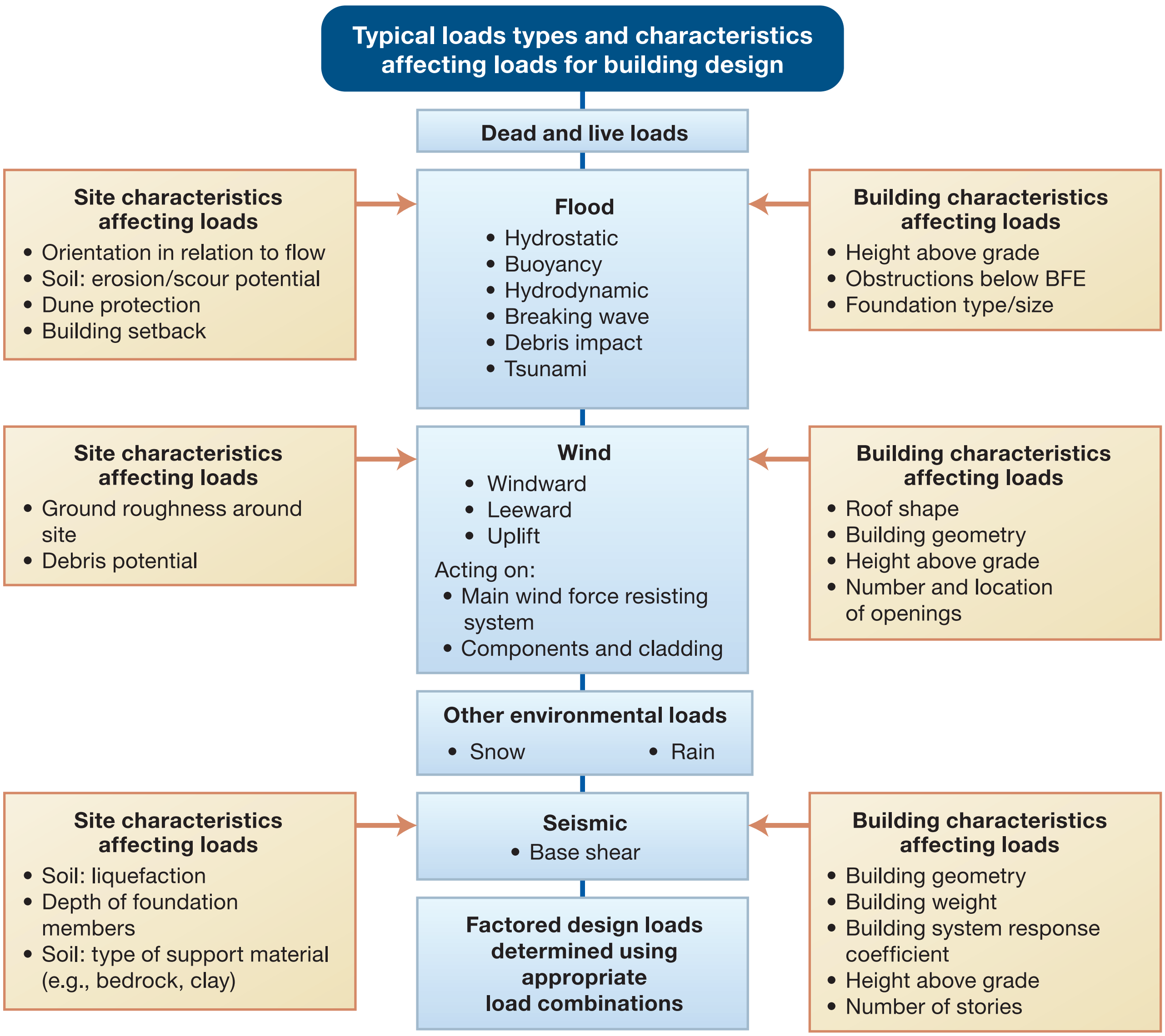


Figure 8-1.
Summary of typical loads and characteristics affecting determination of design load

8.1 Dead Loads

Dead load is defined in ASCE 7-10 as "… the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes." The sum of the dead loads of all the individual elements equals the unoccupied weight of a building.

The total weight of a building is usually determined by multiplying the unit weight of the various building materials—expressed in pounds per unit area—by the surface area of the materials. Unit weights of building elements, such as exterior walls, floors and roofs, are commonly used to simplify the calculation of building weight. Minimum design dead loads are contained in ASCE 7-10, *Commentary*. Additional information about material weights can be found in *Architectural Graphic Standards* (The American Institute of Architects 2007) and other similar texts.

Determining the dead load is important for several reasons:

- The dead load determines in part the required size of the foundation (e.g., footing width, pile embedment depth, number of piles).
- Dead load counterbalances uplift forces from buoyancy when materials are below the stillwater depth (see Section 8.5.7) and from wind (see Example 8.9).
- Dead load counterbalances wind and earthquake overturning moments.
- Dead load changes the response of a building to impacts from floodborne debris and seismic forces.
- Prescriptive design in the following code references and other code references is dependent on the dead load of the building. For example, wind uplift strap capacity, joist spans, and length of wall bracing required to resist seismic forces are dependent on dead load assumptions used to tabulate the prescriptive requirements in the following examples of codes and prescriptive standards:
  - 2012 IRC, *International Residential Code for One-and Two-Family Dwellings* (ICC 2011b)
  - 2012 IBC, *International Building Code* (ICC 2011a)
  - ICC 600-2008, *Standard for Residential Construction in High-Wind Regions* (ICC 2008)
  - WFCM-12, *Wood Frame Construction Manual for One- and Two-Family Dwellings* (AF&PA 2012)
  - AISI S230-07, *Standard for Cold-formed Steel Framing-prescriptive Method for One- and Two-family Dwellings* (AISI 2007)

8.2 Live Loads

Live loads are defined in ASCE 7-10 as "… loads produced by the use and occupancy of the building … and do not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load, or dead load." Live loads are usually taken as a uniform load spread across the surface being designed. For residential one- and two-family buildings, the uniformly distributed live load for habitable areas (except sleeping and attic areas) in ASCE 7-10 is 40 pounds/square foot. For balconies and decks on one- and two-family buildings, live load is 1.5 times the live load of the occupancy served but not to exceed 100 pounds/square foot. This requirement typically translates to a live load of 60 pounds/square foot for a deck or balcony accessed from a living room or den, or a live load of 45 pounds/square foot for a deck or balcony accessed from a bedroom. ASCE 7-10 contains no requirements for supporting a concentrated load in a residential building.

NOTE

The live loads in the 2012 IBC and 2012 IRC for balconies and decks attached to one- and two-family dwellings differ from those in ASCE 7-10. Under the 2012 IBC, the live load for balconies and decks is the same as the occupancy served. Under the 2012 IRC, a minimum 40 pounds/square foot live load is specified for balconies and decks. Strict adherence to the ASCE 7-10 live loads for a residential deck requires a complete engineering design and does not permit use of the prescriptive deck ledger table in the 2012 IRC or the prescriptive provisions in AWC DCA6, which are based on a 40 pounds/square foot live load.

8.3 Concept of Tributary or Effective Area and Application of Loads to a Building

The tributary area of an element is the area of the floor, wall, roof, or other surface that is supported by that element. The tributary area is generally a rectangle formed by one-half the distance to the adjacent element in each applicable direction.

The tributary area concept is used to distribute loads to various building elements. Figure 8-2 illustrates tributary areas for roof loads, lateral wall loads, and column or pile loads. The tributary area is a factor in calculating wind pressure coefficients, as described in Examples 8.7 and 8.8.

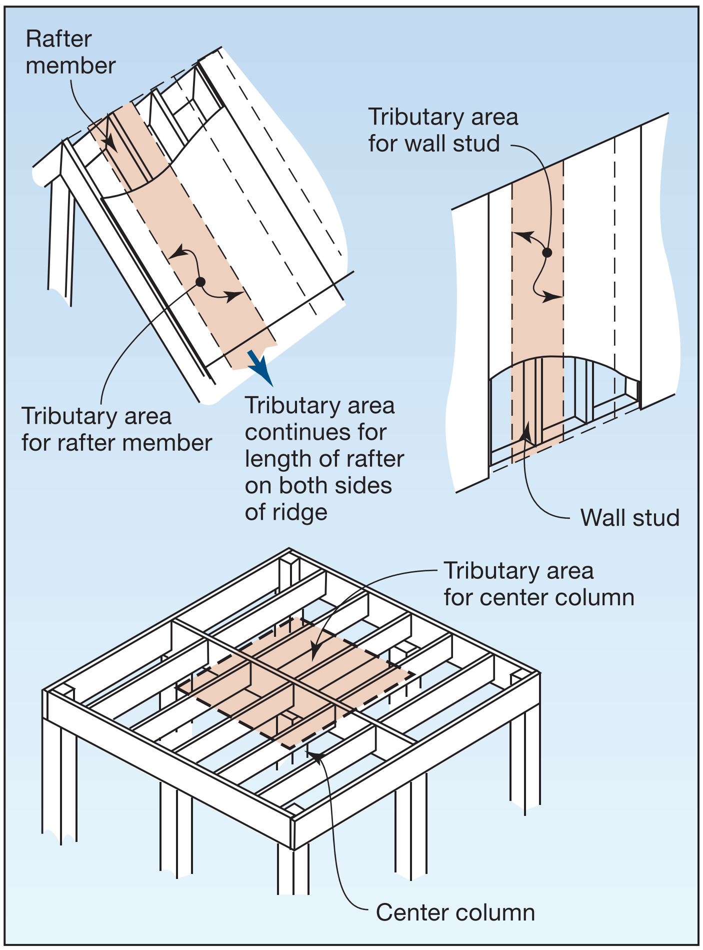


Figure 8-2. Examples of tributary areas for different structural elements

8.4 Snow Loads

Snow loads are applied as a vertical load on the roof or other exposed surfaces such as porches or decks. Ground snow loads are normally specified by the local building code or building official. In the absence of local snow load information, ASCE 7-10 contains recommended snow loads shown on a map of the United States.

When the flat roof snow load exceeds 30 pounds/square foot, a portion of the weight of snow is added to the building weight when the seismic force is determined.

8.5 Flood Loads

Floodwaters can exert a variety of load types on building elements. Both hydrostatic and depth-limited breaking wave loads depend on flood depth.

Flood loads that must be considered in design include:

- Hydrostatic load – buoyancy (flotation) effects, lateral loads from standing water, slowly moving water, and nonbreaking waves
- Breaking wave load
- Hydrodynamic load – from rapidly moving water, including broken waves
- Debris impact load – from waterborne objects

NOTE

- Flood load calculation procedures cited in this Manual are conservative, given the uncertain conditions of a severe coastal event.
- Background information and procedures for calculating coastal flood loads are presented in a number of publications, including ASCE 7-10 and the *Coastal Engineering Manual* (USACE 2008).

The effects of flood loads on buildings can be exacerbated by storm-induced erosion and localized scour and by long-term erosion, all of which can lower the ground surface around foundation elements and cause the loss of load-bearing capacity and loss of resistance to lateral and uplift loads. As discussed in Section 8.5.3, the lower the ground surface elevation, the deeper the water, and because the wave theory used in this Manual is based on depth-limited waves, deeper water creates larger waves and thus greater loads.

8.5.1 Design Flood

In this Manual, "design flood" refers to the locally adopted regulatory flood. If a community regulates to minimum NFIP requirements, the design flood is identical to the base flood (the 1-percent-annual-chance flood or 100-year flood). If a community has chosen to exceed minimum NFIP building elevation requirements, the design flood can exceed the base flood. The design flood is always equal to or greater than the base flood.

TERMINOLOGY: FREEBOARD

Freeboard is additional height incorporated into the DFE to account for uncertainties in determining flood elevations and to provide a greater level of flood protection. Freeboard may be required by State or local regulations or be desired by a property owner.

8.5.2 Design Flood Elevation

Many communities have chosen to exceed minimum NFIP building elevation requirements, usually by requiring freeboard above the base flood elevation (BFE) but sometimes by regulating to a more severe flood than the base flood. In this Manual, "design flood elevation" (DFE) refers to the locally adopted regulatory flood elevation.

In ASCE 24-05, the DFE is defined as the "elevation of the design flood, including wave height, relative to the datum specified on the community's flood hazard map." The design flood is the "greater of the following two flood events: (1) the base flood, affecting those areas identified as SFHAs on the community's FIRM or (2) the flood corresponding to the area designated as a flood hazard area on a community's flood hazard map or otherwise legally designated." The DFE is often taken as the BFE plus any freeboard required by a community, even if the community has not adopted a design flood more severe than the 100-year flood.

Coastal floods can and do exceed BFEs shown on FIRMs and minimum required DFEs established by local and State governments. When there are differences between the minimum required DFE and the recommended elevation based on consideration of other sources, the designer, in consultation with the owner, must decide whether elevating above the DFE provides benefits relative to the added costs of elevating higher than the minimum requirement. For example, substantially higher elevations require more stairs to access the main floor and may require revised designs to meet the community's height restriction. Benefits include reduced flood damage, reduced flood insurance premiums, and the ability to reoccupy homes faster than owners of homes constructed at the minimum allowable elevation. In both Hurricanes Katrina and Ike, high water marks after the storms indicated that if the building elevations had been set to the storm surge elevation, the buildings may have survived. See FEMA 549, *Hurricane Katrina in the Gulf Coast* (FEMA 2006), and FEMA P-757, *Hurricane Ike in Texas and Louisiana* (FEMA 2009), for more information.

In addition to considering the DFE per community regulations, designers should consider the following before deciding on an appropriate lowest floor elevation:

- **The 500-year flood elevation as specified in the Flood Insurance Study (FIS) or similar study.** The 500-year flood elevation (including wave effects) represents a larger but less frequent event than the typical basis for the DFE (e.g., the 100-year event). In order to compare the DFE to the 500-year flood elevation, the designer must obtain the 500-year wave crest elevation from the FIS or convert the 500-year stillwater level to a wave crest elevation if the latter is not included in the FIS report.

- **The elevation of the expected maximum storm surge as specified by hurricane evacuation maps.** Storm surge evacuation maps provide a maximum storm surge elevation for various hurricane categories. Depending on location, maps may include all hurricane categories (1 to 5), or elevations for selected storm categories only. Most storm surge evacuation maps are prepared by the U.S. Army Corps of Engineers (USACE) and are usually available from the USACE District Office or State/local emergency management agencies. Storm surge elevations are stillwater levels and do not include wave heights, so the designer must convert storm surge elevations to wave crest elevations.

  When storm surge evacuation maps are based on landmark boundaries (e.g., roads or other boundaries of convenience) rather than storm surge depths, the designer needs to obtain the surge elevations for a building site from the evacuation study (if available). The topographic map of the region may also provide information about the storm surge depths because the physical boundary elevation should establish the most landward extent of the storm surge.

- **Historical information and advisory flood elevations.** Historical information showing flood levels and flood conditions during past flood events, if available, is an important consideration for comparison to the DFE. For areas subject to a recent coastal flood event, advisory flood elevations may be available based on the most recent flooding information unique to the site.

Community FIRMs do not account for the effects of long-term erosion, subsidence, or sea level rise, all of which could be considered when establishing lowest floor elevations in excess of the DFE. Erosion can increase future flood hazards by removing dunes and lowering ground levels (allowing larger waves to reach a building site). Sea level rise can increase future flood hazards by allowing smaller and more frequently occurring storms to inundate coastal areas and by increasing storm surge elevations.

Section 3.6 discusses the process a designer could follow to determine whether a FIRM represents flood hazards associated with the site under present-day and future-based flood conditions.

This section provides more information on translating erosion and sea level rise data into d_s (design flood depth) calculations. Figure 8-3 illustrates a procedure that designers can follow to determine d_s under a variety of future conditions. In essence, designers should determine the lowest expected ground elevation at the base of a building during its life and the highest expected stillwater elevation at the building during its life.

Determine subsidence effects (if any) on the site

- Obtain published subsidence rates
- Multiply the subsidence rate by the building lifetime; lower ground elevations by this amount

↓

Determine the most landward expected shoreline location over the anticipated life of the building

- Use published or calculated long-term erosion rate (feet/year), increasing the rate to account for errors and uncertainty. It is recommended that a minimum rate of 1.0 feet/year be used unless durable shore protection or erosion-resistant soil is present
- Multiply the resulting erosion rate by the building lifetime (years) to compute the long-term erosion distance (feet). Use a minimum lifetime of 50 years
- Measure landward (from the most landward historical shoreline) a distance equal to the long-term erosion distance. This will define the most landward expected shoreline

↓

Determine the lowest expected ground elevation at the base of the building or structure

Beginning with the most landward expected shoreline location:

- calculate an eroded dune profile using a storm erosion model; or
- calculate a stable bluff profile using available guidance and data

↓

Determine the highest expected stillwater elevation at the building

- Obtain published sea level rise rates for the site
- Multiply sea level rise rate by the building lifetime; increase present SWEL by this amount

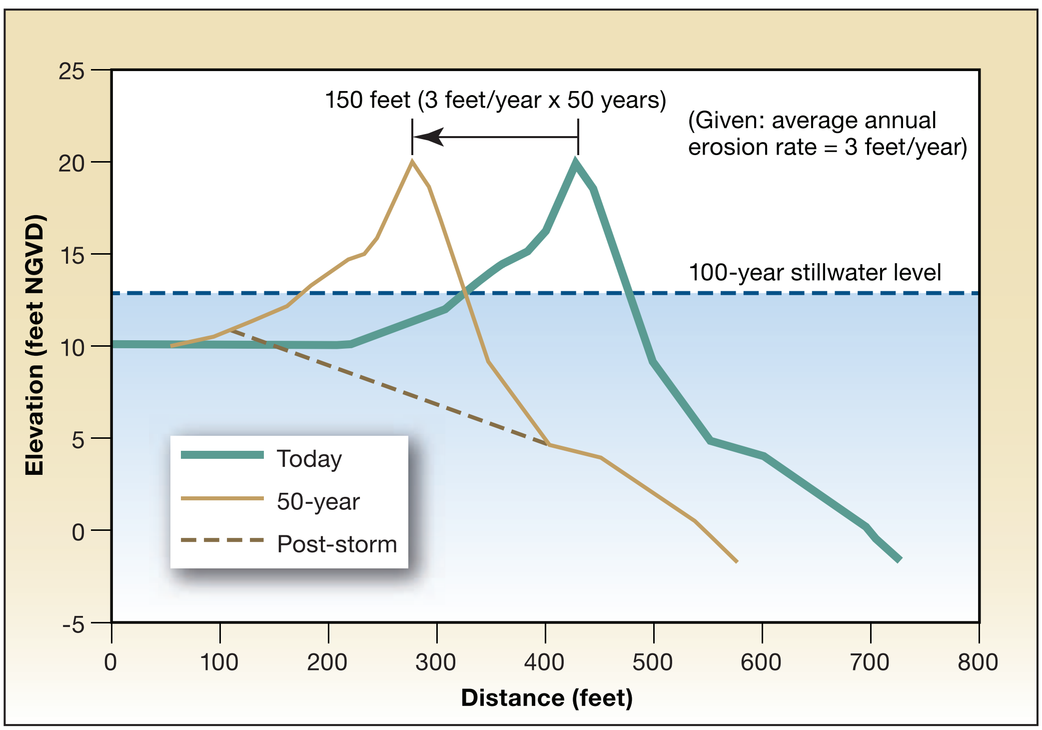
↓

Subtract future eroded ground elevation from future stillwater elevation to obtain design stillwater flood depth

Figure 8-3. Flowchart for estimating maximum likely design stillwater flood depth at the site

- The lowest expected ground elevation is determined by considering subsidence, long-term erosion, and erosion during the base flood.
  - Subsidence effects can be estimated by lowering all existing ground elevations at the site by the product of the subsidence rate and the building lifetime. For example, if subsidence occurs at a rate of 0.005 foot/year and the building lifetime is 50 years, the profile should be lowered 0.25 foot.
  - Figure 8-4 illustrates a simple way to estimate long-term effects on ground elevations at the building. Translate the beach and dune portion of the profile landward by an amount equal to the product of the long-term erosion rate and the building lifetime. If the erosion rate is 3 feet/year and the building lifetime is 50 years, shift the profile back 150 feet.
  - Figure 8-4 also shows the next step in the process, which is to assess dune erosion (see Section 3.5.1) to determine whether the dune will be removed during a base flood event.
  - The lowest expected grade will be evident once the subsidence, long-term erosion, and dune erosion calculations are made.
- The stillwater level is calculated by adding the expected sea level rise element to the base flood stillwater elevation. For example, if the FIS states the 100-year stillwater elevation is 12.2 feet NAVD, and if sea level is rising at 0.01 foot/year, and if the building lifetime is 50 years, the future conditions stillwater level will be 12.7 feet NAVD (12.2 + [(50)(0.01)]).
- The design stillwater flood depth, d_s, is then calculated by subtracting the future conditions eroded grade elevation from the future conditions stillwater elevation.

Figure 8-4. Erosion's effects on ground elevation



8.5.3 Design Stillwater Flood Depth

In a general sense, flood depth can refer to two different depths (see Figure 8-5):

- **Stillwater flood depth.** The vertical distance between the eroded ground elevation and the stillwater elevation associated with the design flood. This depth is referred to as the design stillwater flood depth (d_s).

NOTE

The design stillwater flood depth (d_s) (including wave setup; see Section 8.5.4) should be used for calculating wave heights and flood loads.

- **Design flood protection depth.** The vertical distance between the eroded ground elevation and the DFE. This depth is referred to as the design flood protection depth (d_{fp}) but is not used extensively in this Manual. This Manual emphasizes the use of the DFE as the minimum elevation to which flood-resistant design and construction efforts should be directed.

Determining the maximum design stillwater flood depth over the life of a building is the most important flood load calculation. Nearly every other coastal flood load parameter or calculation (e.g., hydrostatic load, design flood velocity, hydrodynamic load, design wave height, DFE, debris impact load, local scour depth) depends directly or indirectly on the design stillwater flood depth.

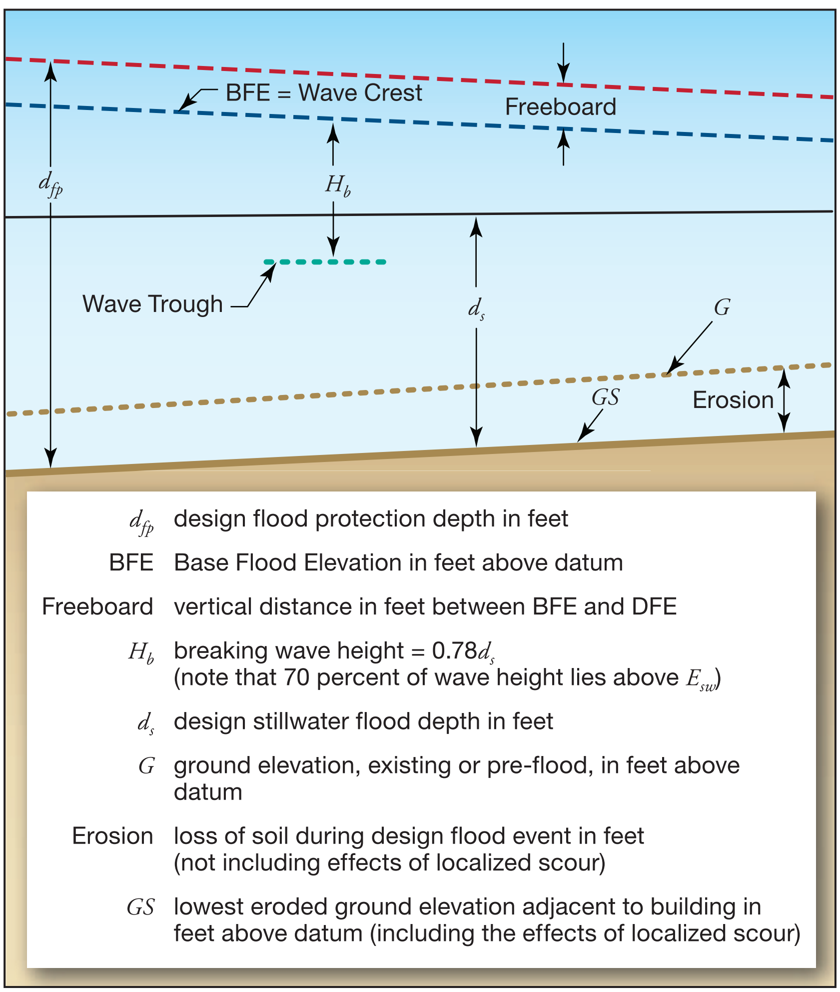


Figure 8-5. Parameters that are determined or affected by flood depth

In this Manual, the design stillwater flood depth (d_s) is defined as the difference between the design stillwater flood elevation (E_{sw}) and the lowest eroded ground surface elevation (GS) adjacent to the building (see Equation 8.1) where wave setup is included in the stillwater flood elevation.

EQUATION 8.1. DESIGN STILLWATER FLOOD DEPTH

$$d_s = E_{sw} - GS \qquad \text{(Eq. 8.1)}$$

where:

d_s = design stillwater flood depth (ft)

E_{sw} = design stillwater flood elevation in ft above datum (e.g., NGVD, NAVD)

GS = lowest eroded ground elevation, in ft above datum, adjacent to a building, excluding effects of localized scour around the foundation

Figure 8-5 illustrates the relationships among the various flood parameters that determine or are affected by flood depth. Note that in Figure 8-5 and Equation 8.1, GS is not the lowest existing pre-flood ground surface; it is the lowest ground surface that will result from long-term erosion and the amount of erosion expected to occur during a design flood, excluding local scour effects. The process for determining GS is described in Section 3.6.4.

CROSS REFERENCE

For a discussion of localized scour, see Section 8.5.10.

Values for E_{sw} are not shown on FEMA FIRMs, but they are given in the FISs, which are produced in conjunction with the FIRM for communities. FISs are usually available from community officials and NFIP State Coordinating Agencies. Some states have made FISs available on their Web sites. Many FISs are also available on the FEMA Web site for free or are available for download for a small fee. For more information, go to http://www.msc.fema.gov.

Design stillwater flood depth (d_s) is determined using Equation A in Example 8.1 for scenarios in which a non-100-year frequency-based DFE is specified by the Authority Having Jurisdiction (AHJ). Freeboard tied to the 100-year flood should not be used to increase d_s since load factors in ASCE 7 were developed for the 100-year nominal flood load.

Example 8.1 demonstrates the calculation of the design stillwater flood depth for five scenarios. All solutions to example problems are in bold text in this Manual.

EXAMPLE 8.1. DESIGN STILLWATER FLOOD DEPTH CALCULATIONS

Given:

- Oceanfront building site on landward side of a primary frontal dune (see Illustration A)
- Topography along transect perpendicular to shoreline is shown in Illustration B; existing ground elevation at seaward row of pilings = 7.0 ft NGVD
- Soil is dense sand; no terminating stratum above –25 ft NGVD
- Data from FIRM is as follows: flood hazard zone at site is Zone VE; BFE = 14.0 ft NGVD
- Data from FIS is as follows: 10-year stillwater elevation = 5.0 ft NGVD; 50-year stillwater elevation = 8.7 ft NGVD; 100-year stillwater elevation = 10.1 ft NGVD; 500-year stillwater elevation = 12.2 ft NGVD
- 500-year wave crest elevation (DFE) specified by AHJ = 18.0 ft NGVD
- Local government requires 1.0 ft freeboard; therefore DFE = 14.0 ft NGVD (BFE) + 1.0 ft = 15.0 ft NGVD
- Direction of wave and flow approach during design event is perpendicular to shoreline
- The eroded ground elevation (base flood conditions) at the seaward row of pilings = 5.5 ft NGVD
- Assume sea level rise is 0.01 ft/yr
- Assume long-term average annual erosion rate is 2.0 ft/yr, no beach nourishment or shoreline stabilization
- Assume building life = 50 years

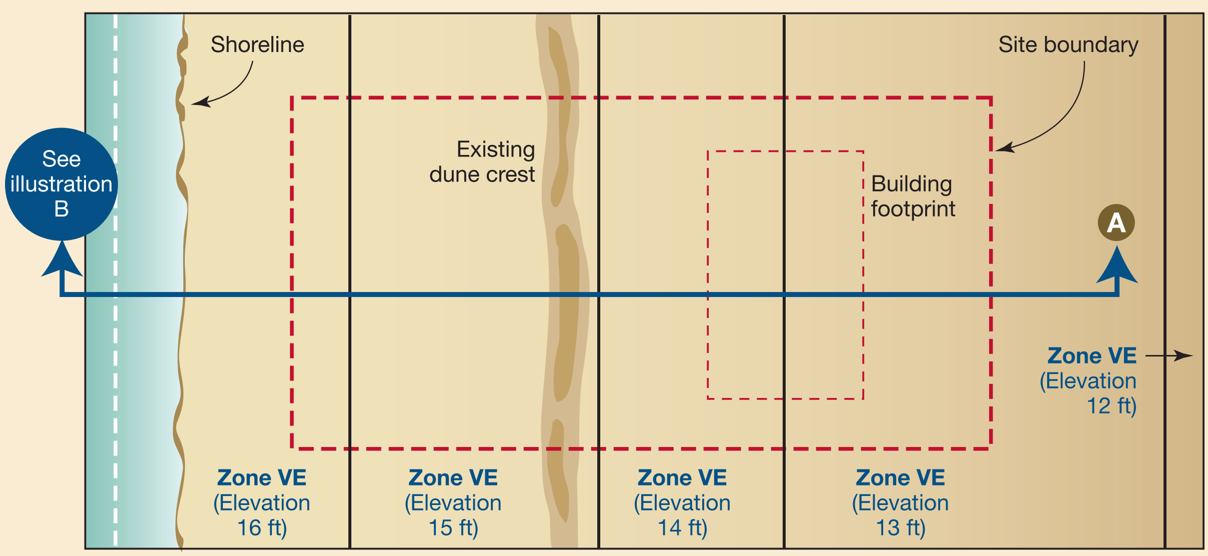


Illustration A. Plan view of site and building location with flood hazard zones

EXAMPLE 8.1. DESIGN STILLWATER FLOOD DEPTH CALCULATIONS (continued)

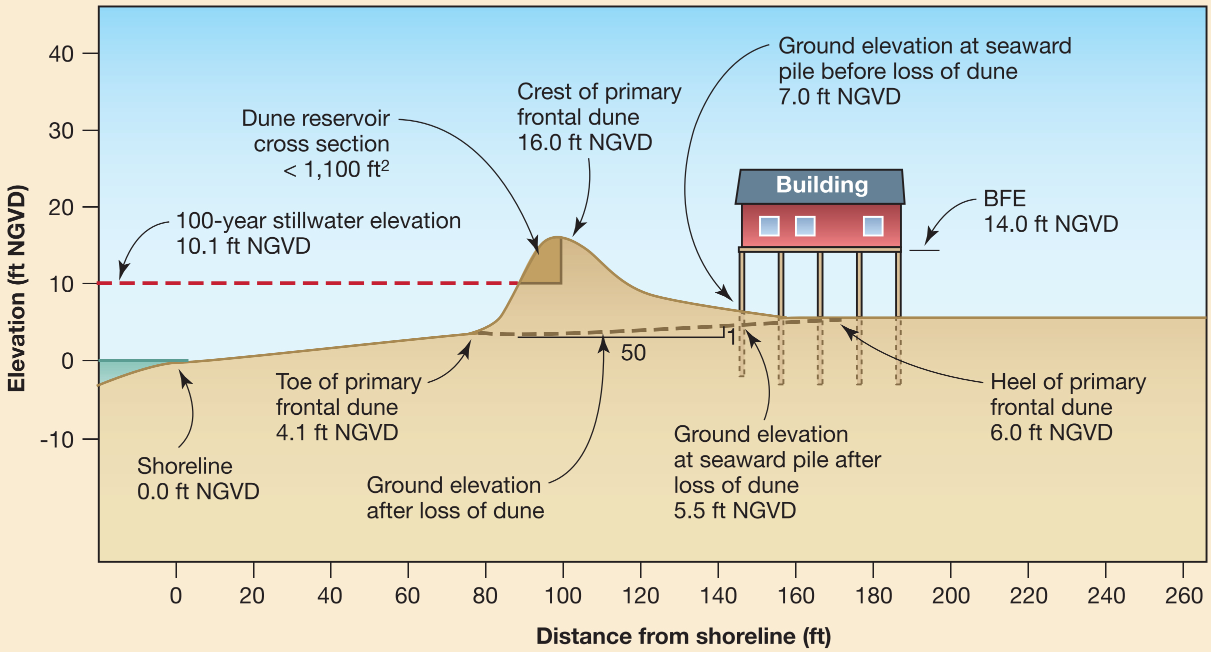


Illustration B. Primary frontal dune will be lost to erosion during a 100-year flood because dune reservoir is less than 1,100 ft² (Section A of Illustration A)

Find:

The design stillwater flood depth (d_s) at the seaward row of piles for varying values of stillwater elevation, presence of freeboard, and consideration of the effects of future conditions (e.g., sea-level rise and long-term erosion).

The basis of the design flood for four scenarios are as follows:

1. 100-year stillwater elevation (NGVD). Future conditions not considered.
2. 100-year stillwater elevation (NGVD) plus freeboard. Future conditions not considered.
3. 100-year stillwater elevation (NGVD). Future conditions (sea-level rise and long-term erosion) in 50 years considered.
4. 500-year wave crest elevation (NGVD). Future conditions not considered.

Note: *Design stillwater flood depth (d_s) is determined using Equation A for scenarios in which a non-100-year frequency-based DFE is specified by the AHJ. Freeboard tied to the 100-year flood should not be used to increase d_s since load factors in ASCE 7 were developed for the 100-year nominal flood load.*

EXAMPLE 8.1. DESIGN STILLWATER FLOOD DEPTH CALCULATIONS (continued)

EQUATION A

$$d_s = \left(\frac{DFE}{BFE}\right)(E_{SW}) - GS$$

where:

d_s = design stillwater flood depth

DFE = design flood elevation for a greater than 100-year flood event

BFE = base flood elevation

E_{SW} = design stillwater flood elevation in feet above datum (e.g. NGVD, NAVD)

GS = lowest eroded ground elevation, in feet above datum, adjacent to building, excluding effects of localized scour around foundations

Solution for Scenario #1: The design stillwater flood depth (d_s) at seaward row of pilings using the 100-year stillwater elevation can be calculated using Equation 8.1 as follows:

d_s = $E_{sw} - GS$

d_s = 10.1 ft NGVD – 5.5 ft NGVD

d_s = **4.6 ft**

Note: *This is the same solution that is calculated in Example 8.4, #3*

Solution for Scenario #2: The design stillwater flood depth (d_s) at seaward row of pilings using the 100-year stillwater elevation and freeboard will be calculated just as in Scenario #1–freeboard should not be included in the stillwater depth calculation but is used instead to raise the building to a higher-than-BFE level:

d_s = $E_{sw} - GS$

d_s = 10.1 ft NGVD – 5.5 ft NGVD

d_s = **4.6 ft**

Solution for Scenario #3: The design stillwater flood depth (d_s) at seaward row of pilings using the 100-year stillwater elevation and the future conditions of sea-level rise and long-term erosion can be calculated as follows:

Step 1: Increase 100-year stillwater elevation 50 years in the future to account for sea-level rise

E_{SW} = 10.1 ft NGVD + (0.01 ft/yr)(50 years) = 10.6 ft NGVD

Step 2: Calculate the lowest ground elevation in ft above the datum adjacent to the seaward row of pilings in 50 years

EXAMPLE 8.1. DESIGN STILLWATER FLOOD DEPTH CALCULATIONS (concluded)

- In 50 years, the front toe of the dune will translate horizontally toward the building by (50 yr)(2 ft/yr) = 100 ft landward
- Taking into account the 1:50 (*v:h*) slope of the eroded dune, the ground at the seaward row of piles will drop (100 ft)(1/50) = 2 ft over 50 years

GS = 5.5 ft – 2 ft = 3.5 ft NGVD in 50 years

Step 3: Combine the effects of sea-level rise and erosion to calculate d_s

$d_s = E_{sw} - GS$

d_s = 10.6 ft NGVD – 3.5 ft NGVD = **7.1 ft**

Solution for Scenario #4: The design stillwater flood depth (d_s) at seaward row of pilings using the AHJ's 500-year wave crest elevation (DFE) can be calculated using Equation A of Example 8.1 as follows:

$$d_s = \left(\frac{DFE}{BFE}\right)(E_{SW}) - GS$$

$$d_s = \left(\frac{18\text{ ft}}{14\text{ ft}}\right)(10.1\text{ ft}) - 5.5\text{ ft} = 13.0\text{ ft} - 5.5\text{ ft} = \mathbf{7.5\text{ ft}}$$

***Note**: Scenarios #1 through #4 show incremental increases in the design stillwater flood depth d_s, depending on how conservative the designer wishes to be in selecting the design scenario. As the design stillwater flood depth increases, the flood loads to which the building foundation must be designed also increase. The increase factor listed in Table A represents the square of the ratio of stillwater flood depth to the stillwater flood depth from Scenario #1(reference case).*

Table A. Stillwater Flood Depths for Various Design Scenarios and Approximate Load Increase Factor from Increased Values of d_s

Scenario #	Design Condition	d_s (ft)	Approximate Load Increase Factor
#1 (reference case)	100-year	4.6	1.0
#2	100-year + freeboard	4.6	1.0
#3	100-year + future conditions	7.1	2.4
#4	500-year	7.5	2.7

***Note:** In subsequent examples, the building in Illustrations A and B and d_s in Scenario #1 are used.*

8.5.4 Wave Setup Contribution to Flood Depth

Pre-1989 FIS reports and FIRMs do not usually include the effects of wave setup (d_{ws}), but some post-1989 FISs and FIRMs do. Because the calculation of design wave heights and flood loads depends on an accurate determination of the total stillwater flood depth, designers should review the effective FIS carefully, using the following procedure:

- Check the hydrologic analyses section of the FIS for mention of wave setup. Note the magnitude of the wave setup.
- Check the stillwater elevation table of the FIS for footnotes regarding wave setup. If wave setup is included in the listed BFEs but not in the 100-year stillwater elevation, add wave setup before calculating the design stillwater flood depth, the design wave height, the design flood velocity, flood loads, and localized scour. If wave setup is already included in the 100-year stillwater elevation, use the 100-year stillwater elevation to determine the design stillwater flood depth and other parameters. Wave setup should not be included in the 100-year stillwater elevation when calculating primary frontal dune erosion.

NOTE

Flood loads are applied to structures as follows:

- **Lateral hydrostatic loads –** at two-thirds depth point of stillwater elevation
- **Breaking wave loads –** at stillwater elevation
- **Hydrodynamic loads –** at mid-depth point of stillwater elevation
- **Debris impact loads –** at stillwater elevation

TERMINOLOGY: WAVE SETUP

Wave setup is an increase in the stillwater surface near the shoreline due to the presence of breaking waves. Wave setup typically adds 1.5 to 2.5 feet to the 100-year stillwater flood elevation and should be discussed in the FIS.

8.5.5 Design Breaking Wave Height

The design breaking wave height (H_b) at a coastal building site is one of the most important design parameters. Unless detailed analysis shows that natural or manmade obstructions will protect the site during a design event, wave heights at a site should be calculated as the heights of depth-limited breaking waves, which are equivalent to 0.78 times the design stillwater flood depth (see Figure 8-5). Note that 70 percent of the breaking wave height lies above the stillwater elevation. In some situations, such as steep ground slopes immediately seaward of a building, the breaking wave height can exceed 0.78 times the stillwater flood depth. In such instances, designers may wish to increase the breaking wave height used for design, with an upper limit for the breaking wave height equal to the stillwater flood depth.

8.5.6 Design Flood Velocity

Estimating design flood velocities (V) in coastal flood hazard areas is subject to considerable uncertainty. There is little reliable historical information concerning the velocity of floodwaters during coastal flood events. The direction and velocity of floodwaters can vary significantly throughout a coastal flood event. Floodwaters can approach a site from one direction during the beginning of a flood event and then shift

WARNING

This Manual does not provide guidance for estimating flood velocities during tsunamis. The issue is highly complex and site-specific. Designers should look for model results from tsunami inundation or evacuation studies.

to another direction (or several directions). Floodwaters can inundate low-lying coastal sites from both the front (e.g., ocean) and back (e.g., bay, sound, river). In a similar manner, flow velocities can vary from close to zero to high velocities during a single flood event. For these reasons, flood velocities should be estimated conservatively by assuming floodwaters can approach from the most critical direction relative to the site and by assuming flow velocities can be high (see Equation 8.2).

EQUATION 8.2. DESIGN FLOOD VELOCITY

Lower bound $$V = \frac{d_s}{t}$$ (Eq. 8.2a)

Upper bound $$V = (gd_s)^{0.5}$$ (Eq. 8.2b)

where:

V = design flood velocity (ft/sec)

d_s = design stillwater flood depth (ft)

t = 1 sec

g = gravitational constant (32.2 ft/sec^2)

For design purposes, flood velocities in coastal areas should be assumed to lie between $V = (gd_s)^{0.5}$ (the expected upper bound) and $V = d_s/t$ (the expected lower bound). It is recommended that designers consider the following factors before deciding whether to use the upper- or lower-bound flood velocity for design:

- Flood zone
- Topography and slope
- Distance from the source of flooding
- Proximity to other buildings or obstructions

The upper bound should be taken as the design flood velocity if the building site is near the flood source, in Zone V, in Zone AO adjacent to Zone V, in Zone A subject to velocity flow and wave action, on steeply sloping terrain, or adjacent to other large buildings or obstructions that will confine or redirect floodwaters and increase local flood velocities. The lower bound is a more appropriate design flood velocity if the site is distant from the flood source, in Zone A, on flat or gently sloping terrain, or unaffected by other buildings or obstructions.

Figure 8-6 shows the velocity versus design stillwater flood depth relationship for non-tsunami, upper- and lower-bound velocities. Equation 8.2 shows the equations for the lower-bound and upper-bound velocity conditions.

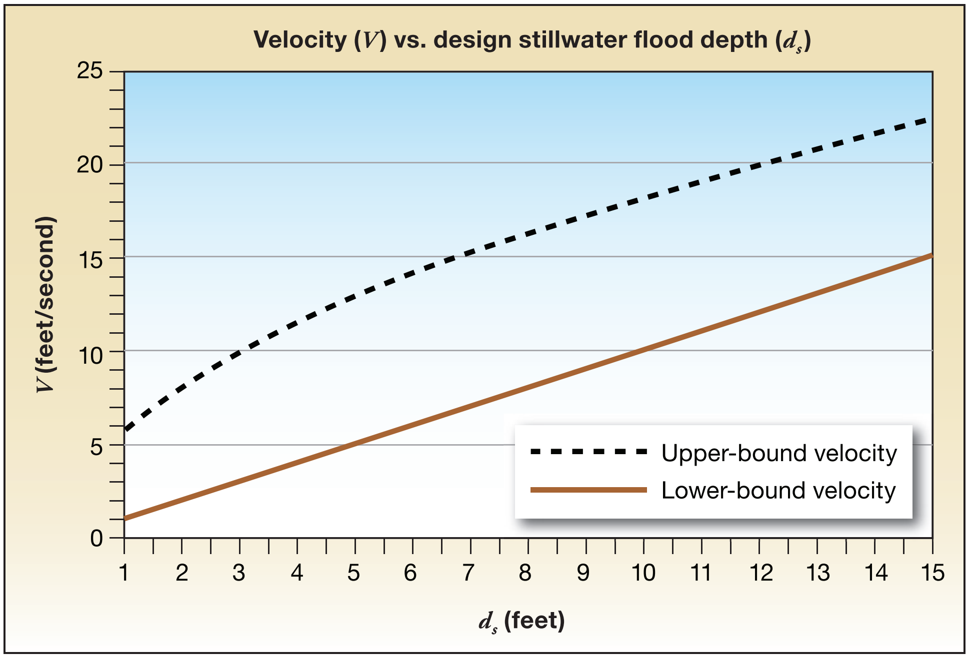


Figure 8-6.
Velocity versus design stillwater flood depth

8.5.7 Hydrostatic Loads

Hydrostatic loads occur whenever floodwaters come into contact with a foundation, building, or building element. Hydrostatic loads can act laterally or vertically.

Lateral hydrostatic forces are generally not sufficient to cause deflection or displacement of a building or building element unless there is a substantial difference in water elevation on opposite sides of the building or component. This is why the NFIP requires that openings be provided in vertical walls that form enclosures below the BFE for buildings constructed in Zone A (see Section 5.2.3.2).

Likewise, vertical hydrostatic forces (buoyancy or flotation) are not generally a concern for properly constructed and elevated coastal buildings founded on adequate foundations. However, buoyant forces can have a significant effect on inadequately elevated buildings on shallow foundations. Such buildings are vulnerable to uplift from flood and wind forces because the weight of a foundation or building element is much less when submerged than when not submerged. For example, one cubic foot of a footing constructed of normal weight concrete weighs approximately 150 pounds. But when submerged, each cubic foot of concrete displaces a cubic foot of saltwater, which weighs about 64 pounds/cubic foot. Thus, the foundation's submerged weight is only 86 pounds if submerged in saltwater (150 pounds/cubic foot – 64 pounds/cubic foot = 86 pounds/cubic foot), or 88 pounds if submerged in fresh water (150 pounds/cubic foot – 62 pounds/cubic foot = 88 pounds/cubic foot). A submerged footing contributes approximately 40 percent less uplift resistance during flood conditions.

Section 3.2.2 of ASCE 7-10 states that the full hydrostatic pressure of water must be applied to floors and foundations when applicable. Sections 2.3.3 and 2.4.2 of ASCE 7-10 require factored flood loads to be considered in the load combinations that model uplift and overturning design limit states. For ASD, flood loads are increased by a factor of 1.5 in Zone V and Coastal A Zones (and 0.75 in coastal flood zones with base flood wave heights less than 1.5 feet, and in non-coastal flood zones). These load factors are applied to account for uncertainty in establishing design flood intensity. As indicated in Equations 8-3 and 8-4 (per Figure 8-7), the design stillwater flood depth should be used when calculating hydrostatic loads.

Any buoyant force (F_{buoy}) on an object must be resisted by the weight of the object and any other opposing force (e.g., anchorage forces) resisting flotation. The contents of underground storage tanks and the live load on floors should not be counted on to resist buoyant forces because the tanks may be empty or the building may be vacant when the flood occurs. Buoyant or flotation forces on a building can be of concern if the actual stillwater flood depth exceeds the design stillwater flood depth. Buoyant forces are also of concern for empty or partially empty aboveground tanks, underground tanks, and swimming pools.

Lateral hydrostatic loads are given by Equation 8.3 and illustrated in Figure 8-7. Note that f_{sta} (in Equation 8.3) is equivalent to the area of the pressure triangle and acts at a point equal to 2/3 d_s below the water surface (see Figure 8-7). Figure 8-7 is presented here solely to illustrate the application of lateral hydrostatic force. In communities participating in the NFIP, local floodplain ordinances or laws require that buildings in Zone V be elevated above the BFE on an open foundation and that the foundation walls of buildings in Zone A be equipped with openings that allow floodwater to enter so that internal and external hydrostatic pressures will equalize (see Section 5.2) and not damage the structure.

Vertical hydrostatic forces are given by Equation 8.4 and are illustrated by Figure 8-8.

EQUATION 8.3. LATERAL HYDROSTATIC LOAD

$$f_{sta} = \frac{1}{2}\gamma_w d_s^{\,2} \qquad \text{(Eq. 8.3a)}$$

where:

f_{sta} = hydrostatic force per unit width (lb/ft) resulting from flooding against vertical element

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

d_s = design stillwater flood depth (ft)

$$F_{sta} = f_{sta}(w) \qquad \text{(Eq. 8.3b)}$$

where:

F_{sta} = total equivalent lateral hydrostatic force on a structure (lb)

f_{sta} = hydrostatic force per unit width (lb/ft) resulting from flooding against vertical element

w = width of vertical element (ft)

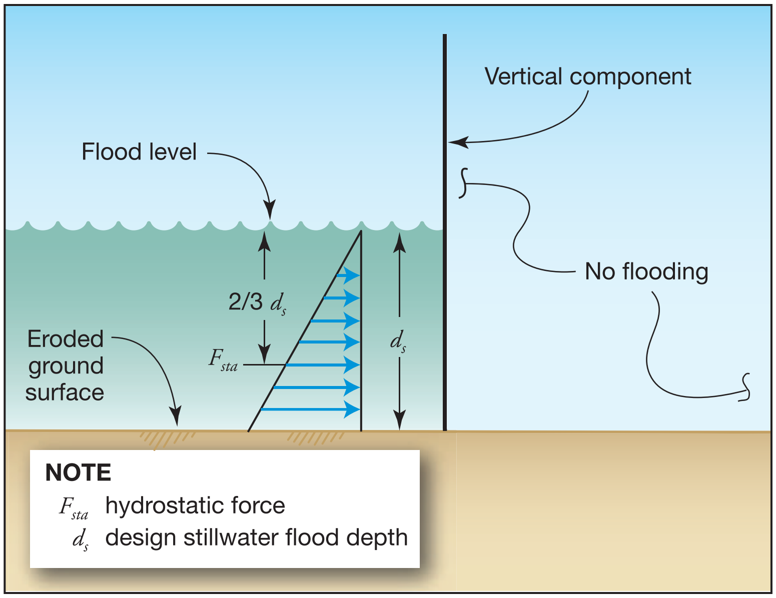


Figure 8-7.
Lateral flood force on a vertical component

EQUATION 8.4. VERTICAL (BUOYANT) HYDROSTATIC FORCE

$$F_{buoy} = \gamma_w (Vol) \quad \text{(Eq. 8.4)}$$

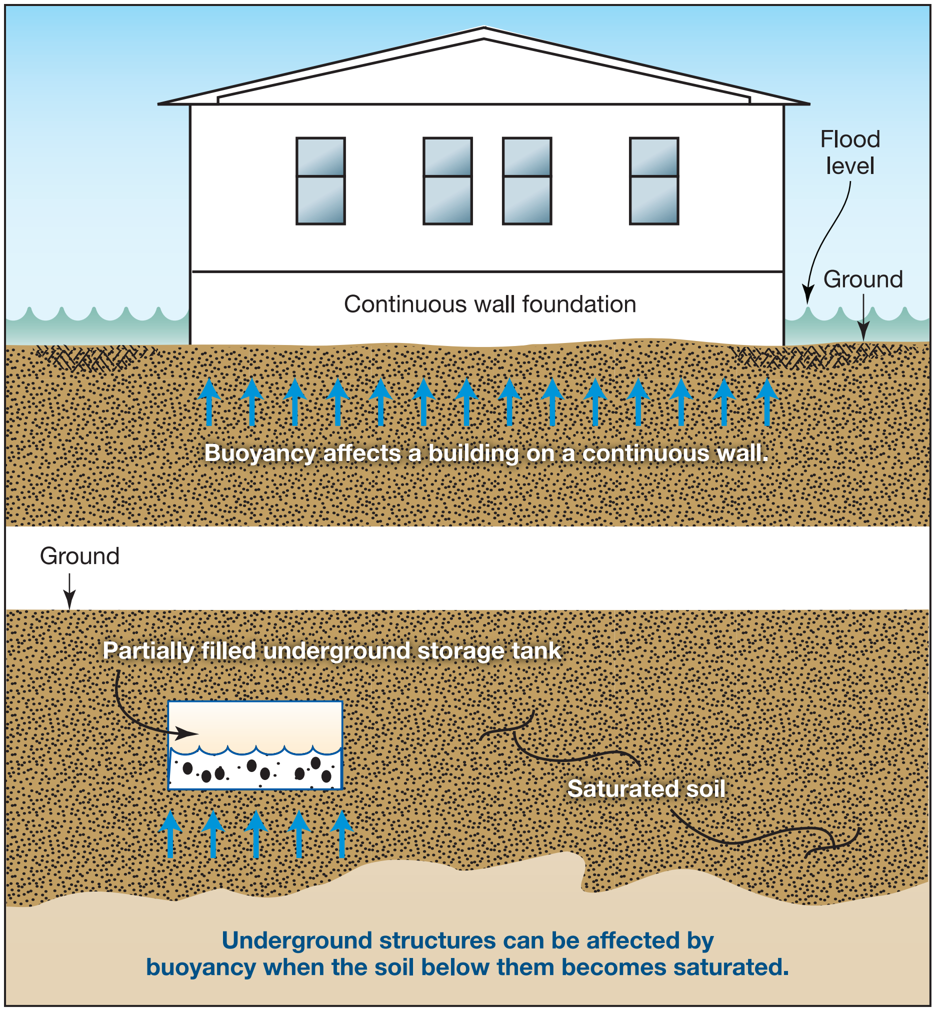
where:

F_{buoy} = vertical hydrostatic force (lb) resulting from the displacement of a given volume of floodwater

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

Vol = volume of floodwater displaced by a submerged object (ft^3)

Figure 8-8.
Vertical (buoyant) flood force; buoyancy forces are drastically reduced for open foundations (piles or piers)



8.5.8 Wave Loads

Calculating wave loads requires the designer to estimate expected wave heights, which, for the purposes of this Manual, are limited by water depths at the site of interest. These data can be estimated using a variety of models. FEMA uses its Wave Height Analysis for Flood Insurance Studies (WHAFIS) model to estimate wave heights and wave crest elevations, and results from this model can be used directly by designers to calculate wave loads.

CROSS REFERENCE

For additional guidance in calculating wave loads, see ASCE 7-10.

Wave forces can be separated into four categories:

- From nonbreaking waves – can usually be computed as hydrostatic forces against walls and hydrodynamic forces against piles
- From breaking waves – short duration but large magnitude
- From broken waves – similar to hydrodynamic forces caused by flowing or surging water

- Uplift – often caused by wave run-up, deflection, or peaking against the underside of horizontal surfaces

CROSS REFERENCE

For more information about FEMA's WHAFIS model, see http://www.fema.gov/plan/prevent/fhm/dl_wfis4.shtm.

The forces from breaking waves are the highest and produce the most severe loads. It is therefore strongly recommended that the breaking wave load be used as the design wave load.

The following three breaking wave loading conditions are of interest in residential design:

- Waves breaking on small-diameter vertical elements below the DFE (e.g., piles, columns in the foundation of a building in Zone V)
- Waves breaking against walls below the DFE (e.g., solid foundation walls in Zone A, breakaway walls in Zone V)
- Wave slam, where just the top of a wave strikes a vertical wall

8.5.8.1 Breaking Wave Loads on Vertical Piles

The breaking wave load on a pile can be assumed to act at the stillwater elevation and is calculated using Equation 8.5.

EQUATION 8.5. BREAKING WAVE LOAD ON VERTICAL PILES

$$F_{brkp} = \frac{1}{2} C_{db} \gamma_w D H_b^{\,2} \qquad \text{(Eq. 8.5)}$$

where:

F_{brkp} = drag force (lb) acting at the stillwater elevation

C_{db} = breaking wave drag coefficient (recommended values are 2.25 for square and rectangular piles and 1.75 for round piles)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

D = pile diameter (ft) for a round pile or 1.4 times the width of the pile or column for a square pile (ft)

H_b = breaking wave height (0.78 d_s), in ft, where d_s = design stillwater flood depth (ft)

Wave loads produced by breaking waves are greater than those produced by nonbreaking or broken waves. Example 8.3 shows the difference between the loads imposed on a vertical pile by nonbreaking waves and by breaking waves.

8.5.8.2 Breaking Wave Loads on Vertical Walls

Breaking wave loads on vertical walls are best calculated according to the procedure described in *Criteria for Evaluating Coastal Flood-Protection Structures* (Walton et al. 1989). The procedure is suitable for use in wave conditions typical during coastal flood and storm events. The relationship for breaking wave load per unit length of wall is shown in Equation 8.6.

NOTE

Equation 8.6 includes the hydrostatic component calculated using Equation 8.3. If Equation 8.6 is used, lateral hydrostatic force from Equation 8.3 should not be added to avoid double counting.

EQUATION 8.6. BREAKING WAVE LOAD ON VERTICAL WALLS

Case 1 (enclosed dry space behind wall):

$$f_{brkw} = 1.1C_p \gamma_w d_s^2 + 2.4\gamma_w d_s^2 \qquad \text{(Eq. 8.6a)}$$

Case 2 (equal stillwater elevation on both sides of wall):

$$f_{brkw} = 1.1C_p \gamma_w d_s^2 + 1.9\gamma_w d_s^2 \qquad \text{(Eq. 8.6b)}$$

where:

f_{brkw} = total breaking wave load per unit length of wall (lb/ft) acting at the stillwater elevation

C_p = dynamic pressure coefficient from Table 8-1

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

d_s = design stillwater flood depth (ft)

$$F_{brkw} = f_{brkw}(w) \qquad \text{(Eq. 8.6c)}$$

where:

F_{brkw} = total breaking wave load (lb) acting at the stillwater elevation

f_{brkw} = total breaking wave load per unit length of wall (lb/ft) acting at the stillwater elevation

w = width of wall (ft)

The procedure assumes that the vertical wall causes a reflected or standing wave to form against the seaward side of the wall and that the crest of the wave reaches a height of 1.2 d_s above the stillwater elevation. The resulting dynamic, static, and total pressure distributions against the wall and resulting loads are as shown in Figure 8-9.

Table 8-1. Value of Dynamic Pressure Coefficient (C_p) as a Function of Probability of Exceedance

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

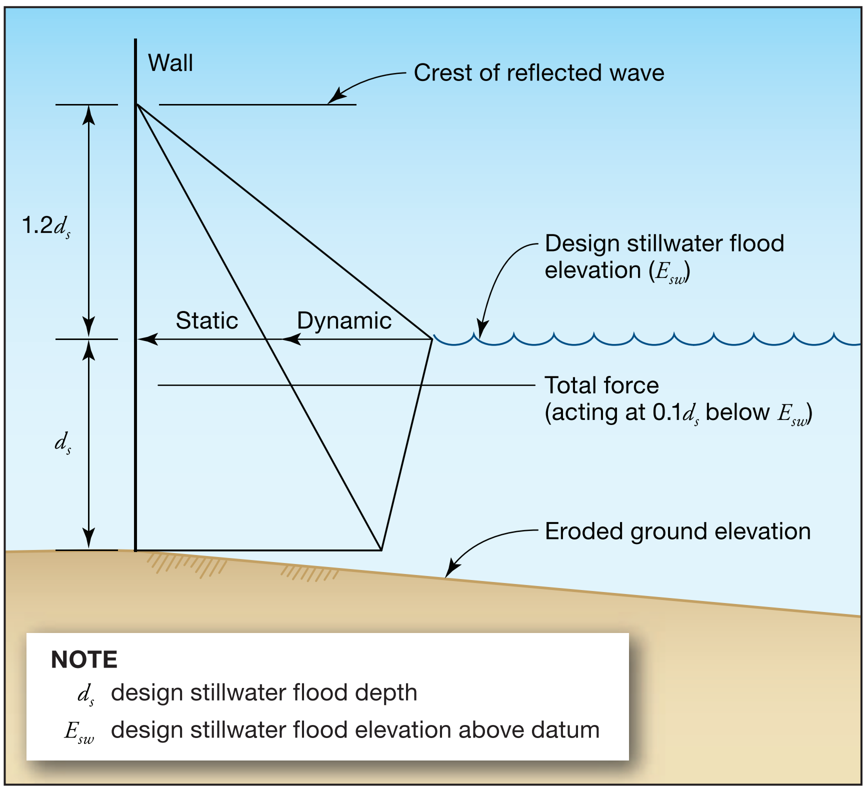


Figure 8-9. Breaking wave pressure distribution against a vertical wall

Equation 8.6 includes two cases: (1) a wave breaks against a vertical wall of an enclosed dry space, shown in Equation 8.6a, and (2) the stillwater elevation on both sides of the wall is equal, shown in Equation 8.6b. Case 1 is equivalent to a situation in which a wave breaks against an enclosure in which there is no floodwater inside the enclosure. Case 2 is equivalent to a situation in which a wave breaks against a breakaway wall or a wall equipped with openings that allow floodwaters to equalize on both sides of the wall. In both cases, waves are normally incident (i.e., wave crests are parallel to the wall). If breaking waves are obliquely incident (i.e., wave crests are not parallel to the wall; see Figure 8-10), the calculated loads would be lower.

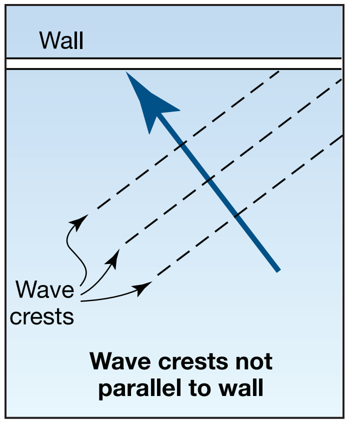


Figure 8-10.
Wave crests not parallel to wall

NOTE: BREAKAWAY WALLS

When designing breakaway versus solid foundation walls using Equation 8.6, the designer should use a C_p of 1.0 rather than the C_p of 1.6 shown in Table 8-1. For more information on breakaway walls, see Section 9.3.

WARNING

The likelihood of damage or loss can be reduced by installing louvered panels in solid walls or creating flood openings in breakaway walls for small flood depths, so that the panels do not break away under minor (nuisance) flood conditions.

Figure 8-11 shows the relationship between water depth and wave height, and between water depth and breaking wave force, for the 1 percent and 50 percent exceedance interval events (Case 2). The Case 1 breaking wave force for these two events is approximately 1.1 times those shown for Case 2.

The breaking wave forces shown in Figure 8-11 are much higher than the typical wind forces that act on a coastal building, even wind pressures that occur during a hurricane or typhoon. However, the duration of the wave pressures and loads is brief; peak pressures probably occur within 0.1 to 0.3 second after the wave breaks against the wall. See *Wave Forces on Inclined and Vertical Wall Surfaces* (ASCE 1995) for a discussion of breaking wave pressures and durations.

WARNING

Even waves less than 3 feet high can impose large loads on foundation walls. Buildings in Coastal A Zones should be designed and constructed to meet Zone V requirements (see Section 6.5.2 in Chapter 6).

Post-storm damage inspections show that breaking wave loads have destroyed virtually all types of wood-frame walls and unreinforced masonry walls below the wave crest elevation. Only highly engineered, massive structural elements are capable of withstanding breaking wave loads. Damaging wave pressures and loads can be generated by waves much lower than the 3-foot wave currently used by FEMA to distinguish Zone A from Zone V. This fact was confirmed by the results of FEMA-sponsored laboratory tests of breakaway wall failures in which measured pressures

WARNING

Under the NFIP, construction of solid foundation walls (such as those that the calculations of Figure 8-11 represent) is not permitted in Zone V for new, substantially damaged, and substantially improved buildings.

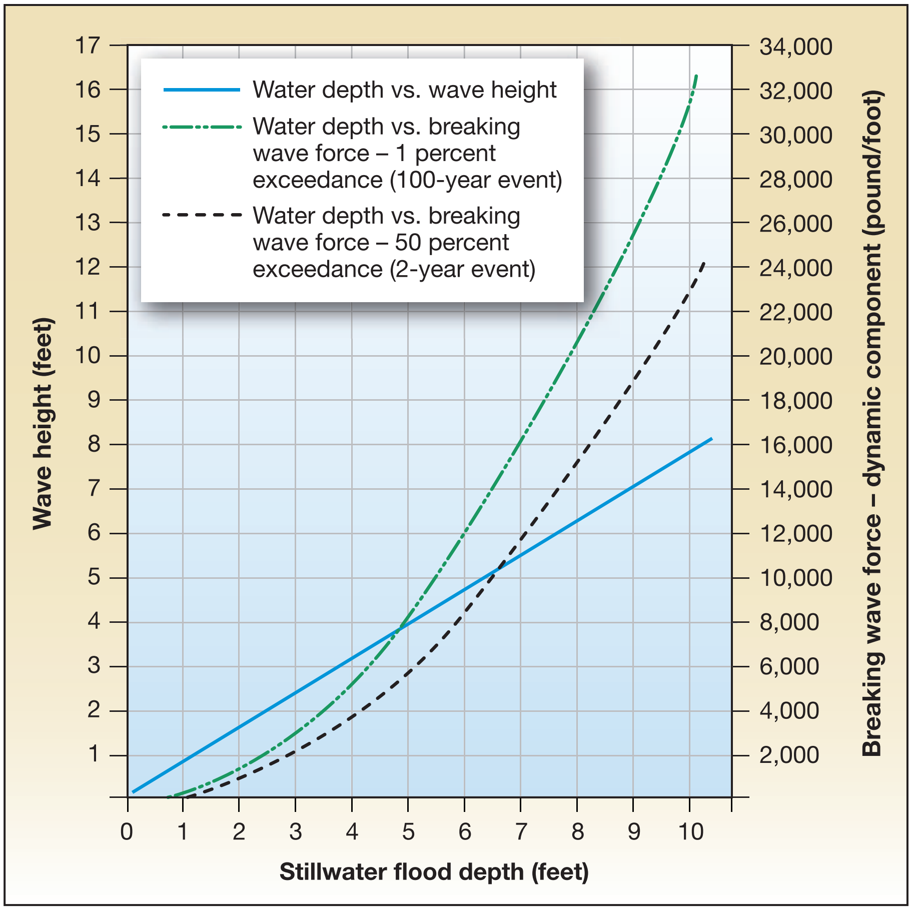


Figure 8-11.
Water depth versus wave height, and water depth versus breaking wave force against, a vertical wall

on the order of hundreds of pounds/ square foot were generated by waves that were only 12 to 18 inches high. See Appendix H for the results of the tests.

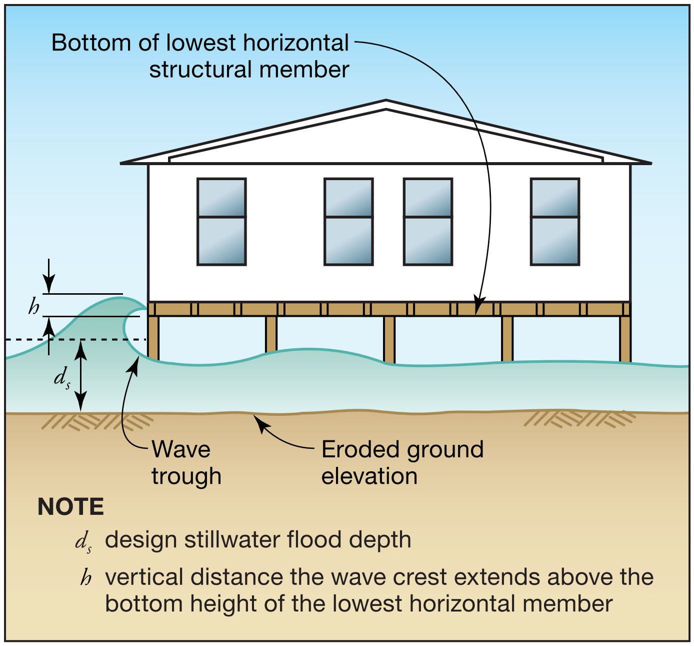
8.5.8.3 Wave Slam

The action of wave crests striking the elevated portion of a structure is known as "wave slam." Wave slam introduces lateral and vertical loads on the lower portions of the elevated structure (Figure 8-12). Wave slam force, which can be large, typically results in damaged floor systems (see Figure 3-26 in Chapter 3). This is one reason freeboard should be included in the design of coastal residential buildings. Lateral wave slam can be calculated using Equation 8.7, but vertical wave slam calculations are beyond the scope of this Manual.

Equation 8.7 is similar to Equation 8.8 (hydrodynamic load) with the wave crest velocity set at the wave celerity (upper-bound flow velocity, given by Equation 8.2b) and a wave slam coefficient instead of a drag coefficient. The wave slam coefficient used in Equation 8.7 is an effective slam coefficient, estimated using information contained in Bea et al. (1999) and McConnell et al. (2004).

Wave slam should not be computed for buildings that are elevated on solid foundation walls (the wave-load-on-wall calculation using Equation 8.6 includes wave slam) but should be computed for buildings that are elevated on piles or columns (wave loads on the piles or columns, and wave slam against the elevated building, can be computed separately and summed).

Figure 8-12.
Lateral wave slam against an elevated building



EQUATION 8.7. LATERAL WAVE SLAM

$$F_s = f_s w = \frac{1}{2}\gamma_w C_s d_s h w \qquad \text{(Eq. 8.7)}$$

where:

F_s = lateral wave slam (lb)

f_s = lateral wave slam (lb/ft)

C_s = slam coefficient incorporating effects of slam duration and structure stiffness for typical residential structure (recommended value is 2.0)

γ_w = unit weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

d_s = stillwater flood depth (ft)

h = vertical distance (ft) the wave crest extends above the bottom of the floor joist or floor beam

w = length (ft) of the floor joist or floor beam struck by wave crest

EXAMPLE 8.2. WAVE SLAM CALCULATION

Given:

- Zone V building elevated on pile foundation near saltwater
- Bottom of floor beam elevation = 15.0 ft NGVD
- Length of beam (parallel to wave crest) = 50 ft
- Design stillwater elevation = 12.0 ft NGVD
- Eroded ground elevation = 5.0 ft NGVD
- C_s (wave slam coefficient; see Equation 8.7) = 2.0
- γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)
- A = (8 ft)(0.833 ft) = 6.664 ft^2

Find:

1. Wave crest elevation
2. Vertical height of the beam subject to wave slam
3. Lateral wave slam acting on the elevated floor system

Solution for #1: The wave crest elevation can be calculated as 1.55 times the stillwater depth, above the eroded ground elevation

Wave crest elevation = 5.0 ft NGVD + 1.55 (12.0 ft NGVD – 5.0 ft NGVD) = **15.9 ft NGVD**

Solution for #2: The vertical height of the beam subject to wave slam can be found as follows:

Vertical height = wave crest elevation – bottom of beam elevation = 15.9 ft NGVD – 15.0 ft NGVD = **0.9 ft**

Solution for #3: Using Equation 8-7, the lateral wave slam acting on the elevated floor system can be found as follows:

$$F_s = f_s w = \frac{1}{2}\gamma C_s d_s h w = \left(\frac{1}{2}\right)(64\text{ lb/ft}^3)(2.0)(7.0\text{ ft})(0.9\text{ ft})(50\text{ ft}) = \mathbf{20{,}160\ lb}$$

8.5.9 Hydrodynamic Loads

As shown in Figure 8-13, water flowing around a building (or a structural element or other object) imposes loads on the building. In the figure, note that the lowest floor of the building is above the flood level and the loads imposed by flowing water affect only the foundation walls. However, open foundation systems, unlike that shown in Figure 8-13, can greatly reduce hydrodynamic loading. Hydrodynamic loads, which are a function of flow velocity and structural geometry, include frontal impact on the upstream face, drag along the sides, and suction on the downstream side. One of the most difficult steps in quantifying loads imposed by moving water is determining the expected flood velocity (see Section 8.5.6 for guidance on design flood velocities). In this Manual, the velocity of floodwater is assumed to be constant (i.e., steady-state flow). Hydrodynamic loads can be calculated using Equation 8.8.

Elevating above the DFE provides additional protection from hydrodynamic loads for elevated enclosed areas.

The drag coefficient used in Equation 8.8 is taken from the *Shore Protection Manual, Volume 2* (USACE 1984). Additional guidance is provided in Section 5.4.3 of ASCE 7-10 and in FEMA 259, *Engineering Principles and Practices for Retrofitting Floodprone Residential Buildings* (FEMA 2001). The drag coefficient is a function of the shape of the object around which flow is directed. When an object is something other than a round, square, or rectangular pile, the coefficient is determined by one of the following ratios (see Table 8-2):

1. The ratio of the width of the object (w) to the height of the object (h) if the object is completely immersed in water
2. The ratio of the width of the object (w) to the stillwater flood depth of the water (d_s) if the object is not fully immersed

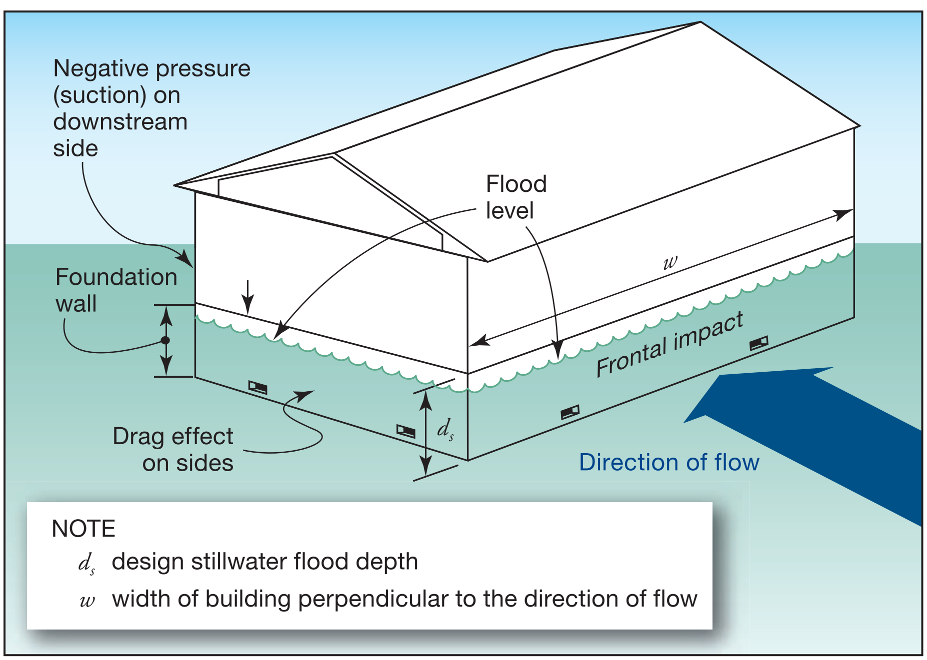


Figure 8-13. Hydrodynamic loads on a building

EQUATION 8.8. HYDRODYNAMIC LOAD (FOR ALL FLOW VELOCITIES)

$$F_{dyn} = \frac{1}{2} C_d \rho V^2 A \qquad \text{(Eq. 8.8)}$$

where:

F_{dyn} = horizontal drag force (lb) acting at the stillwater mid-depth (half way between the stillwater elevation and the eroded ground surface)

C_d = drag coefficient (recommended values are 2.0 for square or rectangular piles and 1.2 for round piles; for other obstructions, see Table 8-2)

ρ = mass density of fluid (1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for saltwater)

V = velocity of water (ft/sec); see Equation 8.2

A = surface area of obstruction normal to flow (ft^2) = $(w)(d_s)$(see Figure 8-13) or $(w)(h)$ if the object is completely immersed

Flow around a building or building element also creates flow-perpendicular forces (lift forces). When a building element is rigid, lift forces can be assumed to be small. When the element is not rigid, lift forces can be greater than drag forces. The equation for lift force is the same as that for hydrodynamic force except that the drag coefficient (C_d) is replaced with the lift coefficient (C_l). In this Manual, the foundations of coastal residential buildings are considered rigid, and hydrodynamic lift forces can therefore be ignored.

Equation 8.8 provides the total force against a building of a given surface area, A. Dividing the total force by either length or width yields a force per linear unit; dividing by surface area, A, yields a force per unit area. Example 8.3 shows the difference between the loads imposed on a vertical pile by nonbreaking and breaking waves. As noted in Section 8.5.8, nonbreaking wave forces on piles can be calculated as hydrodynamic forces.

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

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

NOTE

Lift coefficients (C_l) are provided in *Introduction to Fluid Mechanics* (Fox and McDonald 1985) and in many other fluid mechanics textbooks.

EXAMPLE 8.3. HYDRODYNAMIC LOAD ON PILES VERSUS BREAKING WAVE LOAD ON PILES

Given:

- Building elevated on round-pile foundation near saltwater
- C_d (drag coefficient for nonbreaking wave on round pile; see Equation 8.8) = 1.2
- C_{db} (drag coefficient for breaking wave on round pile; see Equation 8.5) = 1.75
- D = 10 in. or 0.833 ft
- d_s = 8 ft
- Velocity ranges from 8 ft/sec to 16 ft/sec
- ρ = mass density of fluid (1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for saltwater)
- γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)
- A = (8 ft)(0.833 ft) = 6.664 ft^2

Find:

1 The range of loads from hydrodynamic flow around a pile
2. Load from a breaking wave on a pile

Solution for #1: The hydrodynamic load from flow past a pile is calculated using Equation 8.8 as follows:

For a flood velocity of 8 ft/sec:

$$F_{nonbrkp} = \frac{1}{2} C_d \rho V^2 A$$

$$F_{nonbrkp} = \frac{1}{2}(1.2)\left(1.99 \text{ slugs/ft}^3\right)(8 \text{ ft/sec})^2 (6.664 \text{ ft}^2)$$

$$F_{nonbrkp} = 509 \text{ lb/pile}$$

For a flood velocity of 16 ft/sec:

$$F_{nonbrkp} = \frac{1}{2} C_d \rho V^2 A$$

$$F_{nonbrkp} = \left(\frac{1}{2}\right)(1.2)\left(1.99 \text{ slugs/ft}^3\right)(16 \text{ ft/sec})^2 (6.664 \text{ ft}^2)$$

$$F_{nonbrkp} = 2{,}037 \text{ lb/pile}$$

The range of loads from a nonbreaking wave: 509 lb/pile to 2,037 lb/pile

EXAMPLE 8.3. HYDRODYNAMIC LOAD ON PILES VERSUS BREAKING WAVE LOAD ON PILES (concluded)

Solution for #2: The load from a breaking wave on a pile is calculated with Equation 8.5 as follows:

$$F_{brkp} = \left(\frac{1}{2}\right)(1.75)\left(64.0 \text{ lb/ft}^3\right)(0.833 \text{ ft})(0.78)(8 \text{ ft}^2)$$

where:

H_b is the height of the breaking wave or $(0.78)d_s$

F_{brkp} = **1,816 lb/pile**

Note: *The load from the breaking wave is approximately 3.5 times the lower estimate of the hydrodynamic load. The upper estimate of the hydrodynamic load exceeds the breaking wave load only because of the very conservative nature of the upper flood velocity estimate.*

8.5.10 Debris Impact Loads

Debris impact loads are imposed on a building by objects carried by moving water. The magnitude of these loads is very difficult to predict, but some reasonable allowance must be made for them. The loads are influenced by where the building is located in the potential debris stream, specifically if it is:

- Immediately adjacent to or downstream from another building
- Downstream from large floatable objects (e.g., exposed or minimally covered storage tanks)
- Among closely spaced buildings

A familiar equation for calculating debris loads is given in ASCE 7-10, *Commentary*. This equation has been simplified into Equation 8.9 using C_{Str}, the values of which are based on assumptions appropriate for the typical coastal buildings that are covered in this Manual. The parameters in Equation 8.9 are discussed below. See Chapter C5 of ASCE 7-10 for a more detailed discussion of the parameters.

Equation 8.9 contains the following uncertainties, each of which must be quantified before the effect of debris loading can be calculated:

- Size, shape, and weight (W) of the waterborne object
- Design flood velocity (V)
- Velocity of the waterborne object compared to the flood velocity
- Duration of the impact (Δt) (assumed to be equal to 0.03 seconds in the case of residential buildings is incorporated in C_{Str}, which is explained in more detail below)
- Portion of the building to be struck

EQUATION 8.9. DEBRIS IMPACT LOAD

$$F_i = W\,V\,C_D C_B C_{Str} \qquad \text{(Eq. 8.9)}$$

where:

F_i = impact force acting at the stillwater elevation (lb)

W = weight of the object (lb)

V = velocity of water (ft/sec), approximated by $1/2(gd_s)^{1/2}$

C_D = depth coefficient (see Table 8-3)

C_B = blockage coefficient (taken as 1.0 for no upstream screening, flow path greater than 30 ft; see below for more information)

C_{Str} = Building structure coefficient (refer to the explanation of C_{Str} at the end of this section)

= 0.2 for timber pile and masonry column supported structures 3 stories or less in height above grade

= 0.4 for concrete pile or concrete or steel moment resisting frames 3 stories or less in height above grade

= 0.8 for reinforced concrete foundation walls (including insulated concrete forms)

Designers should consider locally adopted guidance because it may be based on more recent information than ASCE 7-10 or on information specific to the local hazards. Local guidance considerations may include the following:

- **Size, shape, and weight of waterborne debris.** The size, shape, and weight of waterborne debris may vary according to region. For example, the coasts of Washington, Oregon, and other areas may be subject to very large debris in the form of whole trees and logs along the shoreline. The southeastern coast of the United States may be more subject to debris impact from dune crossovers and destroyed buildings than other areas. In the absence of information about the nature of potential debris, a weight of 1,000 pounds is recommended as the value of *W*. Objects with this weight could include portions of damaged buildings, utility poles, portions of previously embedded piles, and empty storage tanks.

- **Debris velocity.** As noted in Section 8.5.6, flood velocity can be approximated within the range given by Equation 8.2. For calculating debris loads, the velocity of the waterborne object is assumed to be the same as the flood velocity. Although this assumption may be accurate for small objects, it may overstate debris velocities for large objects that drag on the bottom or that strike nearby structures.

- **Portion of the building to be struck.** The object is assumed to be at or near the water surface level when it strikes the building and is therefore assumed to strike the building at the stillwater elevation.

- **Depth coefficient.** The depth coefficient (C_D) accounts for reduced debris velocity as water depth decreases. For buildings in Zone A with stillwater flood depths greater than 5 feet or for buildings in Zone V, the depth coefficient = 1.0. For other conditions, the depth coefficient varies, as shown in Table 8-3.

Table 8-3. Depth Coefficient (C_D) by Flood Hazard Zone and Water Depth

Flood Hazard Zone and Water Depth	C_D
Floodway[(a)] or Zone V	1.0
Zone A, stillwater flood depth $\geq$ 5 ft	1.0
Zone A, stillwater flood depth = 4 ft	0.75
Zone A, stillwater flood depth = 2.5 ft	0.375
Zone A, stillwater flood depth $\leq$ 1 ft	0.00

(a) Per ASCE 24-05, a "floodway" is a "channel and that portion of the floodplain reserved to convey the base flood without cumulatively increasing the water surface elevation more than a designated height."

- **Blockage coefficient.** The blockage coefficient (C_B) is used to account for the reduction in debris velocity expected to occur because of the screening provided by trees and other structures upstream from the structure or building on which the impact load is being calculated. The blockage coefficient varies, as shown in Table 8-4.

Table 8-4. Values of Blockage Coefficient C_B

Degree of Screening or Sheltering within 100 Ft Upstream	C_B
No upstream screening, flow path wider than 30 ft	1.0
Limited upstream screening, flow path 20-ft wide	0.6
Moderate upstream screening, flow path 10-ft wide	0.2
Dense upstream screening, flow path less than 5-ft wide	0.0

- **Building structure coefficient.** The building structure coefficient, C_{str}, is derived from Equation C5-3, Chapter C5, ASCE 7-10. Coefficient values for C_{str}, (0.2, 0.4, and 0.8 as defined above for Equation 8.9) were generated by selecting input values recommended in ASCE 7-10, Chapter C5, with appropriate assumptions made to model typical coastal residential structures. The derived building structure coefficient formula with inputs is defined as follows:

$$C_{Str} = \frac{3.14 C_I C_O R_{\max}}{2 g \Delta t}$$

where:

C_I = importance coefficient = 1.0

C_O = orientation coefficient = 0.80

Δt = duration of impact = 0.03 sec

g = gravitational constant (32.2 ft/sec^2)

R_{max} = maximum response ratio assuming approximate natural period, T, of building types as follows: for timber pile and masonry column, T = 0.75 sec; for concrete pile or concrete or steel moment resisting frames, T = 0.35 sec; and for reinforced concrete foundation walls, T = 0.2 sec. The ratio of impact duration (0.03 sec) to approximate natural period (T) is entered into Table C5-4 of ASCE 7-10 to yield the R_{max} value.

8.5.11 Localized Scour

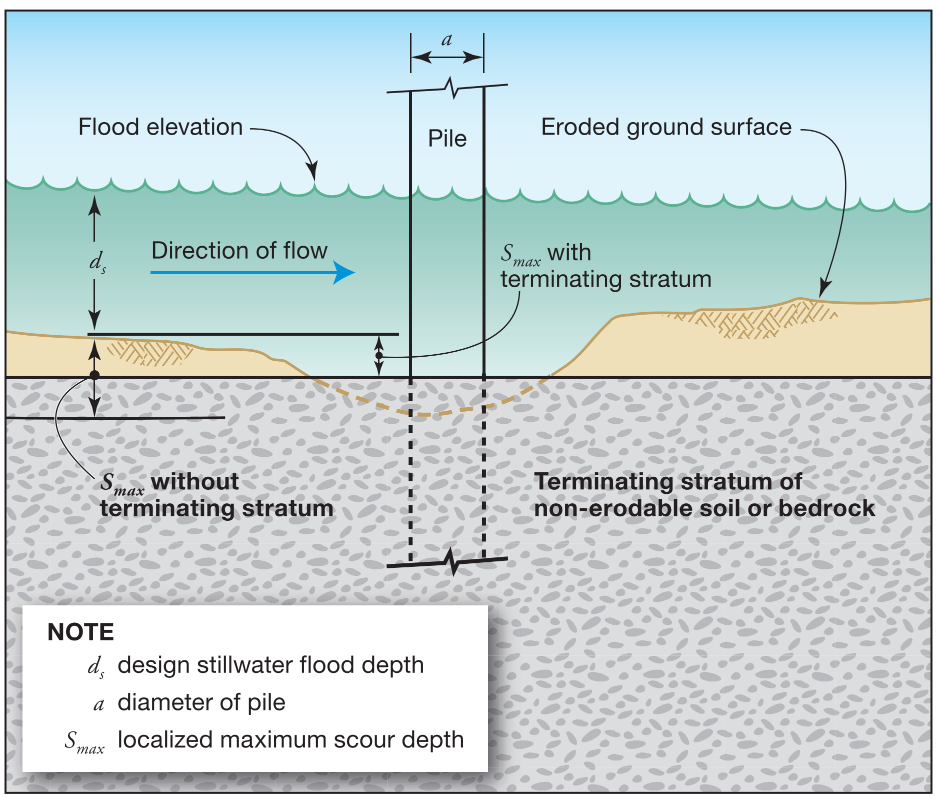
Waves and currents during coastal flood conditions create turbulence around foundation elements, causing localized scour around those elements. Determining potential scour is critical in designing coastal foundations to ensure that failure does not occur as a result of the loss in either bearing capacity or anchoring resistance around the posts, piles, piers, columns, footings, or walls. Localized scour determinations will require knowledge of the flood conditions, soil characteristics, and foundation type.

At some locations, soil at or below the ground surface can be resistant to localized scour, and the scour depths calculated as described below would be excessive. When the designer believes the soil at a site may be scour-resistant, the assistance of a geotechnical engineer should be sought before calculated scour depths are reduced.

- **Localized scour around vertical piles.** Generally, localized scour calculation methods in coastal areas are based largely on laboratory tests and empirical evidence gathered after storms.

  The evidence suggests that the localized scour depth around a single pile or column or other thin vertical members is equal to approximately 1.0 to 1.5 times the pile diameter. In this Manual, a ratio of 2.0 is recommended (see Equation 8.10), consistent with the rule of thumb given in the *Coastal Engineering Manual* (USACE 2008). Figure 8-14 illustrates localized scour at a pile, with and without a scour-resistant terminating stratum.

Figure 8-14.
Scour at single vertical foundation member, with and without underlying scour-resistant stratum



EQUATION 8.10. LOCALIZED SCOUR AROUND A SINGLE VERTICAL PILE

$$S_{\text{max}} = 2.0a \qquad \text{(Eq. 8.10)}$$

where:

S_{max} = maximum localized scour depth (ft)

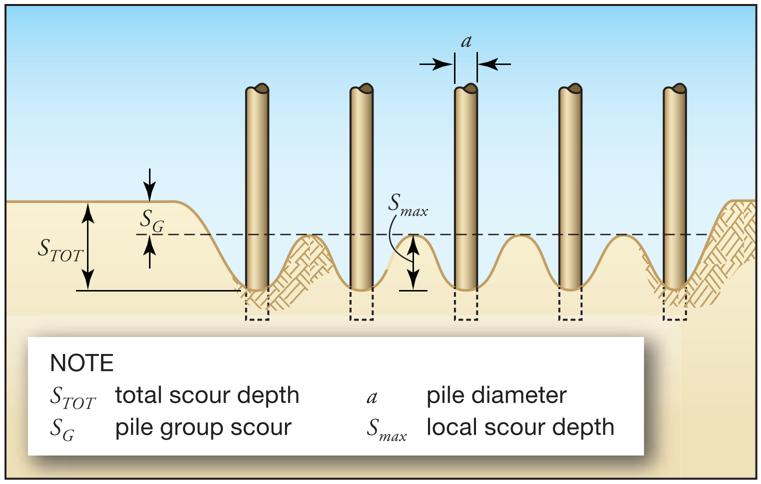
a = diameter of a round foundation element or the maximum diagonal cross-section dimension for a rectangular element

Observations after some hurricanes have shown cases in which localized scour around foundations far exceeded twice the diameter of any individual foundation pile. This was probably a result of flow and waves interacting with the group of foundation piles. In some cases, scour depressions were observed or reported to be 5 to 10 feet deep (see Figure 8-15). This phenomenon has been observed at foundations with or without slabs on grade but appears to be aggravated by the presence of the slabs.

Figure 8-15. Deep scour around foundation piles, Hurricane Ike (Bolivar Peninsula, TX, 2008)

Some research on the interaction of waves and currents on pile groups suggests that the interaction is highly complex and depends on flow characteristics (depth, velocity, and direction), wave conditions (wave height, period, and direction), structural characteristics (pile diameter and spacing) and soil characteristics (Sumer et al. 2001). Conceptually, the resulting scour at a pile group can be represented as shown in Figure 8-16. In this Manual, the total scour depth under a pile group is estimated to be 3 times the single pile scour depth, plus an allowance for the presence of a slab or grade beam, as shown in Equation 8.11. The factor of 3 is consistent with data reported in the literature and post-hurricane observations.

Figure 8-16.
Scour around a group of foundation piles
SOURCE: ADAPTED FROM SUMER ET AL. 2001



EQUATION 8.11. TOTAL LOCALIZED SCOUR AROUND VERTICAL PILES

$$S_{TOT} = 6a + 2\text{ ft} \text{ (if grade beam and/or slab-on-grade present)} \quad \text{(Eq. 8.11a)}$$

$$S_{TOT} = 6a \text{ (if no grade beam or slab-on-grade present)} \quad \text{(Eq. 8.11b)}$$

where:

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

a = 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

One difficulty for designers is determining whether local soils and coastal flood conditions will result in pile group scour according to Equation 8.11. Observations after Hurricanes Rita and Ike suggest that such scour is widespread along the Gulf of Mexico shoreline in eastern Texas and southwestern Louisiana, and observations after Hurricanes Opal and Ivan suggest that it occurs occasionally along the Gulf of Mexico shoreline in Alabama and Florida. Deep foundation scour has also been observed occasionally on North Carolina barrier islands (Hurricane Fran) and American Samoa (September 2009 tsunami).

These observations suggest that some geographic areas are more susceptible than others, but deep foundation scour can occur at any location where there is a confluence of critical soil, flow, and wave conditions. Although these critical conditions cannot be identified precisely, designers should (1) be aware of the phenomenon, (2) investigate historical records for evidence of deep foundation scour around pile groups, and (3) design for such scour when the building site is low-lying, the soil type is predominantly silty, and the site is within several hundred feet of a shoreline.

- **Localized scour around vertical walls and enclosures.** Localized scour around vertical walls and enclosed areas (e.g., typical Zone A construction) can be greater than that around single vertical piles, but it usually occurs at a corner or along one or two edges of the building (as opposed to under the entire building). See Figure 8-16.

Scour depths around vertical walls and enclosed areas should be calculated in accordance with Equation 8.12, which is derived from information in *Coastal Engineering Manual* (USACE 2006). The equation is based on physical model tests conducted on large-diameter vertical piles exposed to waves and currents ("large" means round and square objects with diameters/side lengths corresponding to several tens of feet in the real world, which is comparable to the coastal residential buildings considered in this Manual). Equation 8.12, like Equation 8.11, has no explicit consideration of soil type, so designers must consider whether soils are highly erodible and plan accordingly.

EQUATION 8.12. TOTAL SCOUR DEPTH AROUND VERTICAL WALLS AND ENCLOSURES

$$S_{TOT} = 0.15L \qquad \text{(Eq. 8.12)}$$

where:

S_{TOT} = total scour depth (ft), maximum value is 10 ft

L = horizontal length along the side of the building or obstruction exposed to flow and waves

8.5.12 Flood Load Combinations

Designers should be aware that not all of the flood loads described in Section 8.5 act at certain locations or against certain building types. Table 8-5 provides guidance for calculating appropriate flood loads in Zone V and Coastal A Zones (flood load combinations for the portion of Zone A landward of the Limit of Moderate Wave Action [LiMWA] are shown for comparison).

Table 8-5. Selection of Flood Loads for F_a in ASCE 7-10 Load Combinations for Global Forces

Description	Load Combination
Pile or open foundation in Zone V or Coastal A Zone	Greater of F_{brkp} or F_{dyn} (on front row of piles only) + F_{dyn} (on all other piles) + F_i (on one pile only)
Solid (perimeter wall) foundation	Greater of F_{brkw} or F_{dyn} + F_i (in one corner)

As discussed in Section 8.5.7, hydrostatic loads are included only when standing water will exert lateral or vertical loads on the building; these situations are usually limited to lateral forces being exerted on solid walls or buoyancy forces being exerted on floors and do not dominate in the Zone V or Coastal A Zone environment. Section 8.5.7 includes a discussion about how to include these hydrostatic flood loads in the ASCE 7-10 load combinations.

The guidance in ASCE 7-10, Sections 2.3 and 2.4 (Strength Design and Allowable Stress Design, respectively) also indicates which load combinations the flood load should be applied to. In the portion of Zone A landward of the LiMWA, the flood load F_a could either be hydrostatic or hydrodynamic loads. Both of these loads could be lateral loads; only hydrostatic will be a vertical load (buoyancy). When designing for global forces that will create overturning, sliding or uplift reactions, the designer should use F_a as the flood load that creates the most restrictive condition. In sliding and overturning, F_a should be determined by the type of expected flooding. Hydrostatic forces govern if the flooding is primarily standing water possibly saturating the ground surrounding a foundation; hydrodynamic forces govern if the flooding is primarily from moving water.

When designing a building element such as a foundation, the designer should use F_a as the greatest of the flood forces that affect that element (F_{sta} or F_{dyn}) + F_i (impact loads on that element acting at the stillwater level). The combination of these loads must be used to develop the required resistance that must be provided by the building element.

The designer should assume that breaking waves will affect foundation elements in both Zone V and Zone A. In determining total flood forces acting on the foundation at any given point during a flood event, it is generally unrealistic to assume that impact loads occur on all piles at the same time as breaking wave loads. Therefore, it is recommended that the load be calculated as a single wave impact load acting in combination with other sources of flood loads.

For the design of foundations in Zone V or Coastal A Zone, load combination cases considered should include breaking wave loads alone, hydrodynamic loads alone, and the greater of hydrodynamic loads and breaking wave loads acting in combination with debris impact loads. The value of flood load, F_a, used in ASCE 7-10 load combinations, should be based on the greater of F_{brk} or F_{dyn}, as applicable for global forces (see Table 8-5) or F_i + (F_{brk} or F_{dyn}), as applicable for an individual building element such as a pile.

Example 8-4 is a summary of the information regarding flood loads and the effects of flooding on an example building.

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM

Given:

- Oceanfront building site on landward side of a primary frontal dune (see Example 8.1, Illustration A)
- Topography along transect perpendicular to shoreline is shown in Example 8.1, Illustration B; existing ground elevation at seaward row of pilings = 7.0 ft NGVD
- Soil is dense sand; no terminating stratum above –25 ft NGVD
- Data from FIRM are as follows: flood hazard zone at site is Zone VE, BFE = 14.0 ft NGVD
- Data from FIS are as follows: 100-year stillwater elevation = 10.1 ft NGVD, 10-year stillwater elevation = 5.0 ft NGVD

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM (continued)

- Local government requires 1.0 ft freeboard; therefore DFE = 14.0 ft NGVD (BFE) + 1.0 ft = 15.0 ft NGVD
- Building to be supported on 8-in. × 8-in. square piles, as shown in Illustration A
- Direction of wave and flow approach during design event is perpendicular to shoreline (see Illustration A)
- The assumption is no grade beam or slab-on-grade present

Find:

1. Primary frontal dune reservoir: determine whether dune will be lost or provide protection during design event
2. Eroded ground elevation beneath building resulting from storm erosion
3. Design flood depth (d_s) at seaward row of piles
4. Probable range of design event flow velocities
5. Local scour depth (S) around seaward row of piles
6. Total localized scour (S_{TOT}) around piles
7. Design event breaking wave height (H_b) at seaward row of piles
8. Hydrodynamic (velocity flow) loads (F_{dyn}) on a pile (not in seaward row)
9. Breaking wave loads (F_{brk}) on the seaward row of piles
10. Debris impact load (F_i) from a 1,000-lb object acting on one pile

Solution for #1: Whether the dune will be lost or provides protection can be determined as follows:

- The cross-sectional area of the frontal dune reservoir is above the 100-year stillwater elevation and seaward of the dune crest.
- The area (see Example 8.1, Illustration B) can be approximated as a triangle with the following area:

$$A = \frac{1}{2}bh$$

Where b is the base dimension and h is the height dimension of the approximate triangle:

$$= \frac{1}{2}(16 \text{ ft NGVD dune crest elevation} - 10.1 \text{ ft NGVD 100-year stillwater elevation})(15 \text{ ft})$$

A = 44 ft^2 but the area shown is slightly larger than that of the triangular area, so assume A = 50 ft^2

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM (continued)

- According to this Manual, the cross-sectional area of the frontal dune reservoir must be at least 1,100 ft^2 to survive a 100-year flood event.
- 50 ft^2 <1,100 ft^2 and therefore, **the dune will be lost and provide no protection during the 100-year event**.

Solution for #2: The eroded ground elevation beneath building can be found as follows:

- Remove dune from transect by drawing an upward-sloping (1:50 *v:h*) line landward from the lower of the dune toe or the intersection of the 10-year stillwater elevation and the pre-storm profile.
- The dune toe is 4.1 ft NGVD. The intersection of the 10-year stillwater elevation and pre-storm profile is 5.0 ft NGVD.
- The dune toe is lower (4.1 ft NGVD < 5.0 ft NGVD).
- Draw a line from the dune toe (located 75 ft from the shoreline at an elevation of 4.1 ft NGVD) sloping upward at a 1:50 (*v:h*) slope and find where the seaward row of piles intersects this line.

$$\text{Elevation} = 4.1\text{ ft NGVD} + (145\text{ ft} - 75\text{ ft})\left(\frac{1}{50}\right) = 5.5\text{ ft NGVD}$$

Therefore, the eroded ground elevation at the seaward row of pilings = **5.5 ft NGVD**

Note: *This value does not include local scour around the piles.*

Solution for #3: The design stillwater flood depth (d_s) at seaward row of pilings can be calculated with Equation 8.1 as follows:

$$d_s = E_{sw} - GS$$

Using the 100-year stillwater elevation (NGVD):

$$d_s = 10.1\text{ ft NGVD} - 5.5\text{ ft NGVD}$$

$$d_s = \mathbf{4.6\text{ ft}}$$

Note: *This is the same solution as calculated in Example 8.1, Solution #1.*

Solution for #4: Use Equations 8.2a and 8.2b to determine the range of design flow velocities (V) as follows:

- Lower-bound velocity:

$$V = \frac{d_s}{t}$$

$$V = \frac{4.6\text{ ft}}{1\text{ sec}}$$

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM (continued)

Lower-bound $V = 4.6$ ft/sec

- Upper-bound velocity:

$$V = (gd_s)^{0.5}$$

$$\text{Upper-bound } V = \left(32.2 \text{ ft/sec}^2\right)(4.6 \text{ ft})^{0.5} = 12.2 \text{ ft/sec}$$

The range of velocities: **4.6 ft/sec to 12.2 ft/sec**

Note: t *is assumed to be equal to 1 sec, as given in Equation 8.2.*

Solution for #5: Local scour depth (S) around seaward row of pilings can be found using Equation 8.10 as follows:

$$S = 2.0a$$

where:

$$a = \frac{\sqrt{7.5^2 \text{ in.} + 7.5^2 \text{ in.}}}{12 \text{ in./ft}} = \frac{10.6 \text{ in.}}{12 \text{ in./ft}} = 0.88 \text{ ft}$$

$$S = (2.0)(0.88 \text{ ft}) = \mathbf{1.76\ ft}$$

Solution for #6: To find the total localized scour (S_{TOT}) around piles, use Equation 8.11b as follows:

$$S_{TOT} = 6a = 6(0.88 \text{ ft}) = \mathbf{5.28\ ft}$$

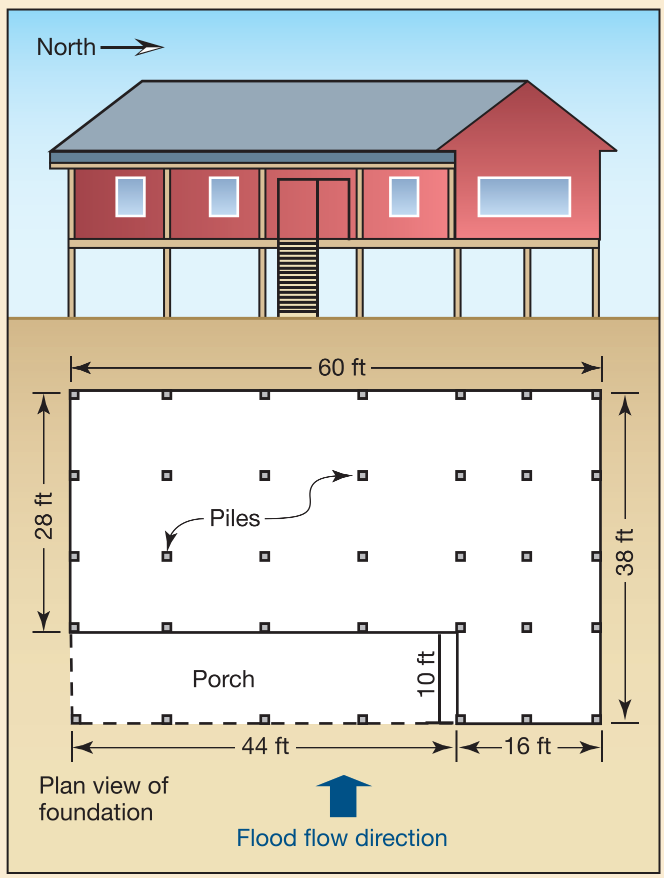


Illustration A. Building elevation and plan view of pile foundation

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM (continued)

Solution for #7: Breaking wave height (H_b) at seaward row of pilings can be found as follows:

At seaward row of pilings, H_b = (d_s)(0.78) where d_s = 4.6 ft from Solution #3

$H_b = (4.6 \text{ ft})(0.78) = \mathbf{3.6\ ft}$

Solution for #8: Hydrodynamic (velocity flow) loads (F_{dyn}) on a pile (not in seaward row) can be calculated using Equation 8.8 as follows:

On one pile: $F_{dyn} = \frac{1}{2} C_d \rho V^2 A$

where:

C_d = 2.0 for a square pile

ρ = 1.99 slugs/ft^3

$A = \frac{8 \text{ in.}}{12 \text{ in.}} (10.1 \text{ ft} - 5.5 \text{ ft}) = 3.07 \text{ ft}^2$

V = 12.2 ft/sec (because the building is on oceanfront, use the upper bound flow velocity for calculating loads)

$F_{dyn} = \frac{1}{2}(2.0)(1.99)(12.2)^2(3.07)$

F_{dyn} on one pile = **909 lb**

Solution for #9: Breaking wave loads (F_{brkp}) on seaward row of pilings can be found using Equation 8.5 as follows:

F_{brkp} on one pile $= \frac{1}{2} C_{db} \gamma_w D H_b^2$

where:

$C_{db} = 2.25$ for square piles

$\gamma_w = 64.0 \text{ lb/ft}^3$ for saltwater

$D = \frac{8 \text{ in.}}{12 \text{ in.}} (1.4) = 0.93 \text{ ft}$

$H_b = 3.6$ ft from Solution #7

$F_{brkp} = \frac{1}{2}(2.25)\left(64.0 \text{ lb/ft}^3\right)(0.93 \text{ ft})(3.6 \text{ ft})^2$

F_{brkp} on one pile = 868 lb

F_{brkp} on seaward row of piles (i.e., 7 piles) = (625 lb)(7) = **6,076 lb**

EXAMPLE 8.4. FLOOD LOAD EXAMPLE PROBLEM (concluded)

Solution for #10: Debris impact load (F_i) from a 1,000-lb object on one pile can be determined using Equation 8.9 as follows: $F_i = W\,V\,C_D C_B C_{Str}$

where:

W = 1,000 lb

C_D = 1.0

C_B = 1.0

C_{Str} = 0.2 (timber pile)

Debris impact load = $(1{,}000 \text{ lb})(12.2 \text{ ft/sec})(1.0)(1.0)(0.2)$

Debris impact load = **2,440 lb**

Note: *C_D and C_B are each assumed to be 1.0.*

The following worksheets will facilitate flood load computations.

Worksheet 1. Flood Load Computation Non-Tsunami Coastal A Zones (Solid Foundation)

Flood Load Computation Worksheet: Non-Tsunami Coastal A Zones (Solid Foundation)

OWNER'S NAME: ____________________ PREPARED BY: ____________________

ADDRESS: ____________________ DATE: ____________________

PROPERTY LOCATION: ____

Constants

γ_w = specific weight of water = 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater

ρ = mass density of fluid) = 1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for saltwater

g = gravitational constant = 32.2 ft/sec^2

Variables

d_s = design stillwater flood depth (ft) =

Vol = volume of floodwater displaced (ft^3) =

V = velocity (fps) =

C_{db} = breaking wave drag coefficient =

H_b = breaking wave height (ft) =

C_p = dynamic pressure coefficient =

C_s = slam coefficient =

C_d = drag coefficient =

w = width of element hit by water (ft) =

h = vertical distance (ft) wave crest extends above bottom of member =

A = area of structure face (ft^2) =

W = weight of object (lb) =

C_D = depth coefficient =

C_B = blockage coefficient =

C_{Str} = building structure coefficient =

a = diameter of round foundation element =

L = horizontal length alongside building exposed to waves (ft)

Summary of Loads

F_{sta} =

F_{buoy} =

F_{brkw} =

F_s =

F_{dyn} =

F_i =

S_{max} =

S_{TOT} =

Worksheet 1. Flood Load Computation Non-Tsunami Coastal A Zones (Solid Foundation) (concluded)

Equation 8.3 Lateral Hydrostatic Load (Flood load on one side only)
$F_{sta} = \frac{1}{2}\gamma_w d_s^2 w =$
Equation 8.4 Vertical (Buoyancy) Hydrostatic Load
$F_{buoy} = \gamma_w (Vol) =$
Equation 8.6 Breaking Wave Load on Vertical Walls
$F_{brkw} = \left(1.1C_p \gamma_w d_s^2 + 2.4\gamma_w d_s^2\right)w$ (if dry behind wall) =   or $F_{brkw} = \left(1.1C_p \gamma_w d_s^2 + 1.9\gamma_w d_s^2\right)w$ (if stillwater elevation is the same on both sides of wall) =
Equation 8.7 Wave Slam
$F_S = \frac{1}{2}\gamma_w C_S d_s h w =$
Equation 8.8 Hydrodynamic Load
$F_{dyn} = \frac{1}{2}C_d \rho V^2 A =$
Equation 8.9 Debris Load
$F_i = W\,V\,C_D C_B C_{Str} =$
Equation 8.10 Localized Scour Around Single Vertical Pile
$S_{max} = 2a =$
Equation 8.11 Total Localized Scour Around Vertical Piles
$S_{TOT} = 6a + 2$ ft (if grade beam and/or slab-on-grade present) = $S_{TOT} = 6a$ (if no grade beam or slab-on-grade present) =
Equation 8.12 Total Scour Depth Around Vertical Walls and Enclosures
$S_{MAX} = 0.15L =$

Worksheet 2. Flood Load Computation Non-Tsunamic Zone V and Coastal A Zone (Open Foundation)

Flood Load Computation Worksheet: Non-Tsunami Zones V and Coastal A Zone (Open Foundation)

OWNER'S NAME: ______________________ PREPARED BY: ______________________

ADDRESS: ______________________ DATE: ______________________

PROPERTY LOCATION: ______________________

Constants

γ_w = specific weight of water = 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater

ρ = mass density of fluid = 1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for saltwater

g = gravitational constant = 32.2 ft/sec^2

Variables

d_s = design stillwater flood depth (ft) =

V = velocity (fps) =

C_{db} = breaking wave drag coefficient =

a, D = pile diameter (ft) =

H_b = breaking wave height (ft) =

C_p = dynamic pressure coefficient =

C_s = slam coefficient =

C_d = drag coefficient for piles =

w = width of element hit by water (ft) =

h = vertical distance (ft) wave crest extends above bottom of member =

W = debris object weight (lb) =

C_D = depth coefficient =

C_B = blockage coefficient =

C_{Str} = building structure coefficient =

L = horizontal length alongside building exposed to waves (ft) =

Summary of Loads

F_{brkp} =

F_s =

F_{dyn} =

F_i =

S_{max} =

S_{TOT} =

Equation 8.5 Breaking Wave Load on Vertical Piles

$$F_{brkp} = \frac{1}{2} C_{db} \gamma_w D H_b^2 =$$

Worksheet 2. Flood Load Computation Non-Tsunamic Zone V and Coastal A Zone (Open Foundation) (concluded)

Equation 8.7 Wave Slam
$F_S = \frac{1}{2}\gamma_w C_S d_s h w =$
Equation 8.8 Hydrodynamic Load
$F_{dyn} = \frac{1}{2} C_{dr} V^2 A =$
Equation 8.9 Debris Load
$F_i = W V C_D C_B C_{Str} =$
Equation 8.10 Localized Scour around Single Vertical Pile
$S_{max} = 2a =$
Equation 8.11 Total Localized Scour Around Vertical Piles
$S_{TOT} = 6a + 2$ ft (if grade beam and/or slab-on-grade present) =
$S_{TOT} = 6a$ (if no grade beam or slab-on-grade present) =

8.6 Tsunami Loads

In general, tsunami loads on residential buildings may be calculated in the same fashion as other flood loads; the physical processes are the same, but the scale of the flood loads is substantially different in that the wavelengths and runup elevations of tsunamis are much greater than those of waves caused by tropical and extratropical cyclones (see Section 3.2). If the tsunami acts as a rapidly rising tide, most of the damage is the result of buoyant and hydrostatic forces (see *Tsunami Engineering* [Camfield 1980]). When the tsunami forms a bore-like wave, the effect is a surge of water to the shore and the expected flood velocities are substantially higher than in non-tsunami conditions.

The tsunami velocities are very high and if realized at the greater water depths, would cause substantial damage to buildings in the path of the tsunami. Additional guidance on designing for tsunami forces including flow velocity, buoyant forces, hydrostatic forces, debris impact, and impulsive forces is provided in FEMA P646, *Guidelines for Design of Structures for Vertical Evacuation from Tsunami* (FEMA 2008b). For debris impact loads under tsunami conditions, see Section 6.5.6 of FEMA P646, which recommends an alternative to Equation 8.6 in this Manual for calculating tsunami debris impact loads.

8.7 Wind Loads

ASCE 7-10 is the state-of-the-art wind load design standard. It contains a discussion of the effects of wind pressure on a variety of building types and building elements. Design for wind loads is essentially the same whether the winds are due to hurricanes, thunderstorms, or tornadoes.

Important factors that affect wind load design pressures include:

- Location of the building site on wind speed maps

- Topographic effects (hills and escarpments), which create a wind speedup effect
- Building risk category (one- and two-family dwellings are assigned to Risk Category II; accessory structures may be assigned to Risk Category I) (see Section 6.2.1.1)
- Building height and shape
- Building enclosure category: enclosed, partially enclosed or open
- Terrain conditions, which determine building exposure category

> **NOTE**
>
> Basic mapped wind speeds in ASCE 7-10 for Category II structures (residential buildings) are higher than those in ASCE 7-05 because they represent ultimate wind speeds or strength-based design wind speeds. Load factors for wind in ASCE 7-10 are also different from those in ASCE 7-05. In ASCE 7-10, the wind load factor in the load combinations for LRFD strength design (LRFD) is 1.0 (but ASCE 7-05 provides a load factor of 1.6), and the ASD wind load factor in the load combinations for allowable stress design (ASD) for wind is 0.6 (but ASCE 7-05 provides a load factor of 1.0).

The effects of wind on buildings can be summarized as follows:

- Windward walls and windward surfaces of steep-sloped roofs are acted on by inward-acting, or positive pressures. See Figure 8-17.
- Leeward walls and leeward surfaces of steep-sloped roofs and both windward and leeward surfaces of low-sloped roofs are acted on by outward-acting, or negative pressures. See Figure 8-17.
- Air flow separates at sharp edges and at locations where the building geometry changes.
- Localized suction, or negative, pressures at eaves, ridges, and the corners of roofs and walls are caused by turbulence and flow separation. These pressures affect loads on components and cladding (C&C) and elements of the main wind force resisting system (MWFRS).

The phenomena of localized high pressures occurring at locations where the building geometry changes is accounted for by the various pressure coefficients in the equations for both MWFRS and C&C. Internal pressures must be included in the determination of net wind pressures and are additive to (or subtractive from) the external pressures. Openings and the natural porosity of building elements contribute to internal

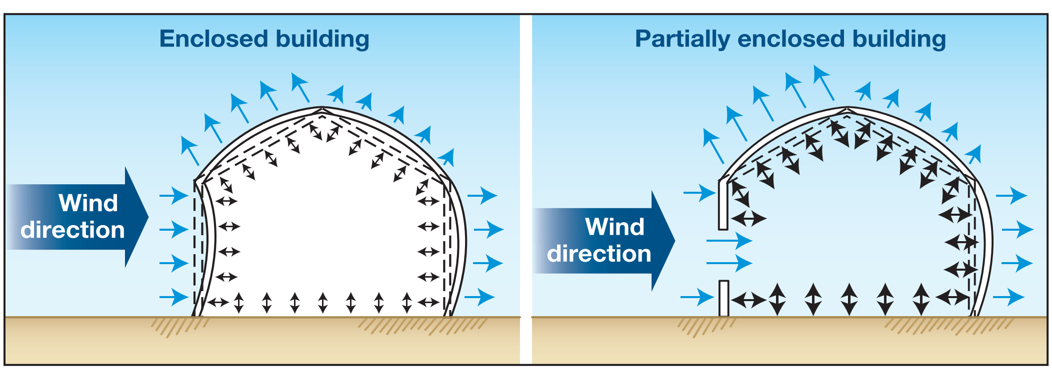


Figure 8-17.
Effect of wind on an enclosed building and a building with an opening

pressure. The magnitude of internal pressures depends on whether the building is enclosed, partially enclosed, or open, as defined in ASCE 7-10. Figure 8-17 shows the effect of wind on an enclosed and partially enclosed building.

In wind-borne debris regions (as defined in ASCE 7-10), in order for a building to be considered enclosed for design purposes, glazing must either be impact-resistant or protected with shutters or other devices that are impact-resistant. This requirement also applies to glazing in doors.

Methods of protecting glazed openings are described in ASCE 7-10 and in Chapter 11 of this Manual.

TERMINOLOGY: HURRICANE-PRONE REGIONS

In the United States and its territories, hurricane-prone areas are defined by ASCE 7-10 as (1) the U.S. Atlantic Ocean and Gulf of Mexico Coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph and (2) Hawaii, Puerto Rico, Guam, the Virgin Islands, and American Samoa.

8.7.1 Determining Wind Loads

In this Manual, design wind pressures for MWFRS are based on the results of the envelope procedure for low-rise buildings. A low-rise building is defined in ASCE 7-10. The envelope procedure in ASCE 7-10 is only one of several for determining MWFRS pressures in ASCE 7-10, but it is the procedure most commonly used for designing low-rise residential buildings. The envelope procedure for low-rise buildings is applicable for enclosed and partially enclosed buildings with a mean roof height (*h*) of less than or equal to 60 feet and where mean roof height (*h*) does not exceed the smallest horizontal building dimension.

Figure 8-18 depicts the distribution of external wall and roof pressures and internal pressures from wind. The figure also shows the mean roof height, which is defined in ASCE 7-10

FORMULA

The following formula converts ASCE 7-05 wind speeds to ASCE 7-10 Risk Category II wind speeds.

$$\text{ASCE 7-10} = (\text{ASCE 7-05})\left(\sqrt{1.6}\right)$$

For conversion from ASCE 7-10 to ASCE 7-05, use:

$$\text{ASCE 7-05} = \frac{\text{ASCE 7-10}}{\sqrt{1.6}}$$

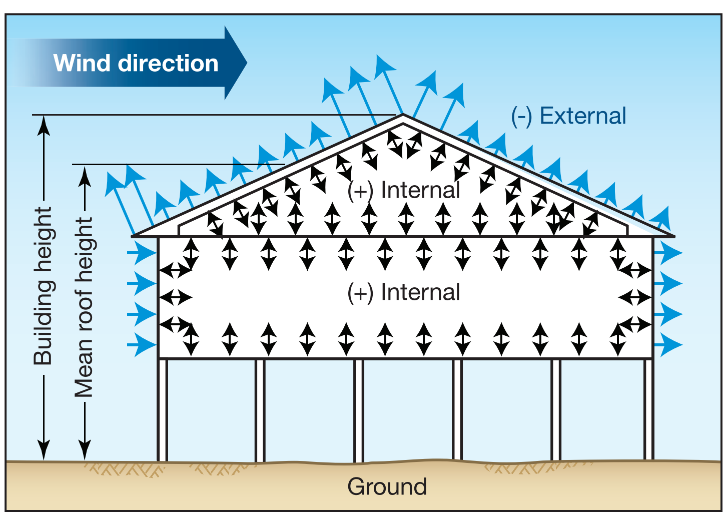


Figure 8-18. Distribution of roof, wall, and internal pressures on one-story, pile-supported building

as "the average of the roof eave height and the height to the highest point on the roof surface ..." Mean roof height is not the same as building height, which is the distance from the ground to the highest point.

For calculating both MWFRS and C&C pressures, velocity pressures (q) should be calculated in accordance with Equation 8.13. Velocity pressure varies depending on many factors including mapped wind speed at the site, height of the structure, local topographic effects, and surrounding terrain that affects the exposure coefficient.

NOTE

ASCE 7-10 *Commentary* states that where a single component, such as a roof truss, comprises an assemblage of structural elements, the elements of that component should be analyzed for loads based on C&C coefficients, and the single component should be analyzed for loads as part of the MWFRS.

EQUATION 8.13. VELOCITY PRESSURE

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(Eq. 8.13)}$$

where:

q_z = velocity pressure evaluated at height z (psf)

K_z = velocity pressure exposure coefficient evaluated at height z

K_{zt} = topographic factor

K_d = wind directionality factor

V = basic wind speed (mph) (3-sec gust speed at 33 ft above ground in Exposure Category C)

The design wind pressure is calculated from the combination of external and internal pressures acting on a building element. This combination of pressures for both MWFRS and C&C loads in accordance with provisions of ASCE 7-10 is represented by Equation 8.14.

EQUATION 8.14. DESIGN WIND PRESSURE FOR LOW-RISE BUILDINGS

$$p = q_h \left[GC_{pf} - GC_{pi} \right] \qquad \text{(Eq. 8.14)}$$

where:

p = design wind pressure

q_h = velocity pressure evaluated at mean roof height (h) (see Figure 8-18 for an illustration of mean roof height)

GC_{pf} = external pressure coefficient for C&C loads or MWFRS loads per the low-rise building provisions, as applicable

GC_{pi} = internal pressure coefficients based on exposure classification as applicable; GC_{pi} for enclosed buildings is +/- 0.18

Figure 8-19 depicts how net suction pressures can vary across different portions of the building. Central portions of the walls represent the location of the least suction, while wall corners, the roof ridge, and the roof perimeter areas have potential for suction pressures that are 1.3, 1.4, and 2 times the central wall areas, respectively. Wall areas and roof areas that experience the largest suction pressures are shown as edge zones in Figure 8-19. The variation of pressures for different portions of the building is based on an enclosed structure (e.g., GC_{pi} = +/- 0.18) and use of external pressure coefficients of the low-rise building provisions.

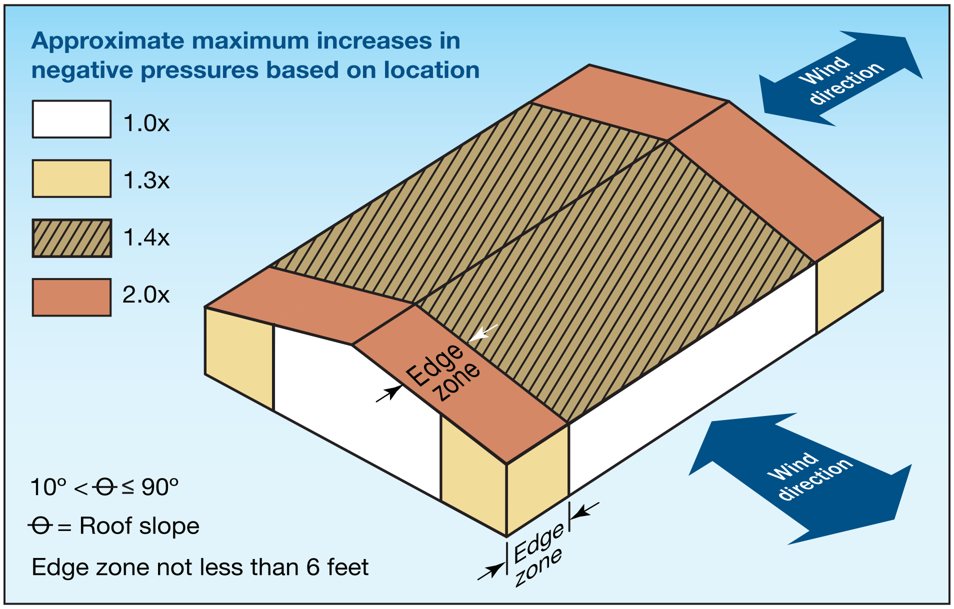


Figure 8-19.
Variation of maximum negative MWFRS pressures based on envelope procedures for low-rise buildings

To simplify design for wind, as well as establish consistency in the application of the wind design provisions of ASCE 7-10, several consensus standards with prescriptive designs tabulate maximum wind loads for the design of specific building elements based on wind pressures (both MWFRS and C&C are often referred to as "prescriptive" standards because they prescribe or tabulate load requirements for pressures) in accordance with ASCE 7-10. These standards, which are referenced in the 2012 IRC, are specific building applications based on factors such as wind speed, exposure, and height above grade. Examples of prescriptive standards for wind design that are referenced in the 2012 IRC are:

- ICC 600-2008, *Standard for Residential Construction in High-Wind Regions* (ICC 2008)
- ANSI/AF&PA, *Wood Frame Construction Manual* (WFCM) (AF&PA 2012)
- ANSI/AISI-S230, *Standard for Cold-Formed Steel Framing-Prescriptive Method for One and Two Family Dwellings* (AISI 2007)

Tabulated wind load requirements in these standards often use conservative assumptions for sizing members and connections. Therefore, load requirements are often more conservative than those developed by direct application of ASCE 7-10 pressures when design loads can be calculated for each element's unique characteristics.

8.7.2 Main Wind Force Resisting System

The MWFRS consists of the foundation; floor supports (e.g., joists, beams); columns; roof rafters or trusses; and bracing, walls, and diaphragms that assist in transferring loads. ASCE 7-10 defines the MWFRS as "… an assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind load from more than one surface." Individual MWFRS elements of shear walls and roof diaphragms (studs and cords) may also act as components and should also be analyzed under the loading requirements of C&C.

For a typical building configuration with a gable roof, the wind direction is perpendicular to the roof ridge for two cases and parallel to the ridge in the other two cases. A complete analysis of the MWFRS includes determining windward and leeward wall pressures, side wall pressures, and windward and leeward roof pressures for wind coming from each of four principal directions. Figure 8-18 depicts pressures acting on the building structure for wind in one direction only. The effect of the combination of pressures on the resulting member and connection forces is of primary interest to the designer. As a result, for each direction of wind loading, structural calculations are required to determine the maximum design forces for members and connections of the building structure.

Prescriptive standards can be used to simplify the calculation of MWFRS design loads. Examples of prescriptive MWFRS design load tables derived from the application of ASCE 7-10 wind load provisions are included in this Manual for the purpose of illustration, as follows:

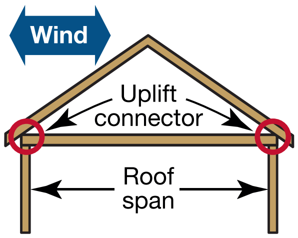
- **Roof uplift connector loads** (see Table 8-6). The application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.5. Equation 8.13 for velocity pressure and Equation 8.14 for determining design wind pressure are used to arrive at the design uplift connector load. The roof uplift connection size is based on moment balance of forces acting on both the windward and leeward side of the roof. The uplift load is used to size individual connectors and also provides the distributed wind uplift load acting at the buildings perimeter walls. Note that while wind speeds are based on 700-year Mean Recurrence Interval, the resulting uplift loads are based on ASD design.

- **Diaphragm loads due to wind acting perpendicular to the ridge** (see Table 8-7). Application of the ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.6. The diaphragm load is based on wind pressures simultaneously acting on both the windward and leeward side of the building. The diaphragm load is used to size the diaphragm for resistance to wind and is also used for estimating total lateral forces for a given wind direction based on combining diaphragm loads for the roof and wall(s) as applicable. Total lateral forces from wind for a given direction can be used for preliminary sizing of the foundation and for determining shear wall capacity requirements.

The example loads in Table 8-6 and Table 8-7, which are based on ASCE 7-10 envelope procedures for low-rise buildings, are used in Examples 8.7 and 8.8 to illustrate their application in the wind design of select load path elements. Tables 8-6 and 8-7 and Examples 8.5 and 8.6 are derived from wind load procedures in the WFCM (AF&PA 2012). Tables 8-6 and 8-7 are not intended to replace requirements of the building code or applicable reference standards for the actual design of a building to resist wind.

Table 8-6. Roof Uplift Connector Loads (Based on ASD Design) at Building Edge Zones, plf (33-ft mean roof height, Exposure C)

Roof Span (ft)	Wind Speed[(a)] (mph)								
	110	115	120	130	140	150	160	170	180
	Roof uplift connector load[(b)(c)(d)] (plf)								
24	189	215	241	298	358	424	494	568	647
32	237	269	303	374	451	534	622	716	816
40	285	324	364	450	544	643	750	864	985
48	333	379	426	527	636	753	879	1,012	1,154



(a) 700-year wind speed, 3-sec gust.

(b) Uplift connector loads are based on 33-ft mean roof height, Exposure C, roof dead load of 10 psf, and roof overhang length of 2 ft (see Example 8.5).

(c) Uplift connector loads are tabulated in pounds per linear ft of wall. Individual connector loads can be calculated for various spacing of connectors (e.g., for spacing of connectors at 2 ft o.c., the individual connector load would be 2 ft times the tabulated value).

(d) Tabulated uplift connector loads are conservatively based on a 20-degree roof slope. Reduced uplift forces may be calculated for greater roof slopes.

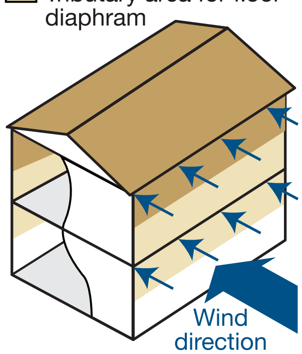
Table 8-7. Lateral Diaphragm Load from Wind Perpendicular to Ridge, plf (33-ft mean roof height, Exposure C)

Roof Span (ft)	Wind Speed[(a)] (mph)								
	110	115	120	130	140	150	160	170	180
	Roof diaphragm load[(b)(c)] for 7:12 roof slope (plf)								
24	138	151	164	192	223	256	291	329	369
32	161	176	191	224	260	299	340	384	430
40	186	203	221	259	301	345	393	443	497
48	210	230	250	294	341	391	445	503	563
	Floor diaphragm load (plf)								
Any	154	168	183	214	249	286	325	367	411

Legend

Tributary area for roof diaphram

Tributary area for floor diaphram



Same figure as Example 8.6, Illustration A

(a) 700-year wind speed, 3-sec gust.

(b) Lateral diaphragm loads are based on 33-ft mean roof height, Exposure C, and wall height of 8 ft (see Example 8.6). Tabulated roof diaphragm loads are for a 7:12 roof slope. Larger loads can be calculated for steeper roof slopes and smaller loads can be calculated for shallower roof slopes.

(c) Total shear load equals the tabulated unit lateral load by the building length perpendicular to the wind direction.

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS

Given:

- Roof span of 24 ft with 2-ft overhangs
- Roof/ceiling dead load of 10 psf
- Wind load based on 150 mph, Exposure C at 33-ft mean roof height
- Building is enclosed
- K_z = 1.0 (velocity pressure exposure coefficient evaluated at height of 33 ft)
- K_{zt} = 1.0 (topographic factor)
- K_d = 0.85 (wind directionality)

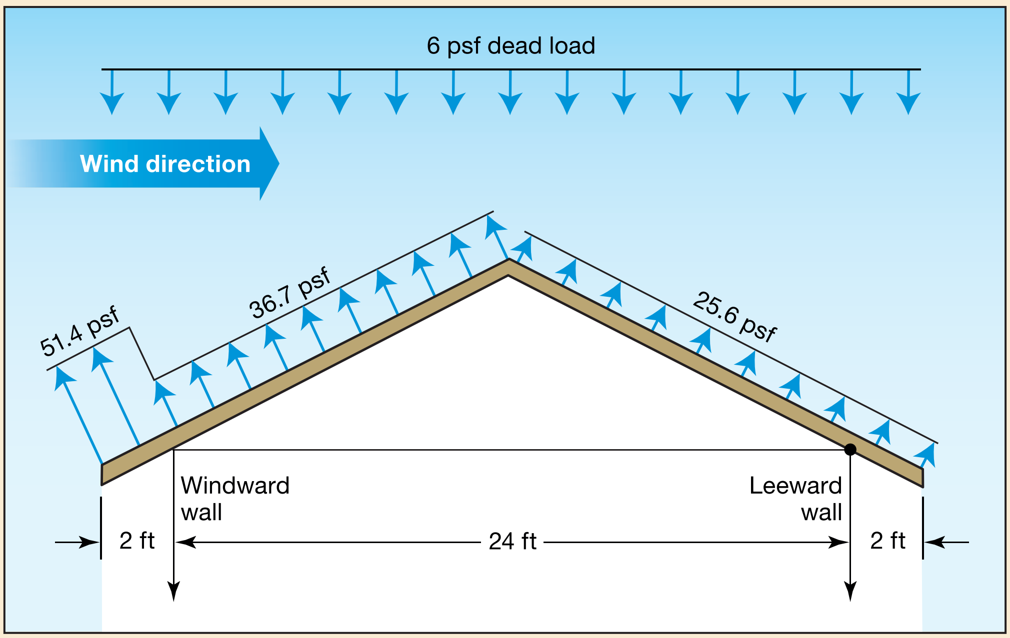


Illustration A. Roof-to-wall uplift connection loads from wind forces

Find: The roof-to-wall uplift connection load using the envelope procedure for low-rise buildings (see Figure 28.4-1 in ASCE 7-10).

Solution: The roof-to-wall uplift connection load can be found using the envelope procedure for low-rise buildings as follows:

- The velocity pressure (q) for the site conditions is determined from Equation 8.13 as follows:

$$q_h = 0.00256 K_z K_{zt} K_d V^2$$

$$q_h = 0.00256(1.0)(1.0)(0.85)(150\text{ mph})^2$$

$$q_h = 48.96\text{ psf}$$

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (continued)

- For ASD, multiply by the ASD wind load factor of 0.6, which comes from Load Combination 7 (See Section 8.10) 0.6D + 0.6W:

  $q_h = 48.96 \text{ psf}(0.6) = 29.38 \text{ psf}$

  The largest uplift forces occur for a roof slope of 20 degrees where wind is perpendicular to the ridge. The addition of an overhang also increases the roof-to-wall uplift connection load. For the windward overhang, a pressure coefficient of 0.68 is used based on the gust factor of 0.85 and pressure coefficient of 0.80 from ASCE 7-10. Otherwise, pressure coefficients for other elements of the roof are based on GC_{pi} = 0.18 and GC_{pf} from the edge zone coefficients shown in Figure 28.4-1 of ASCE 7-10.

Pressures and moments given below contain subscripts for their location:

- W = windward
- L = leeward
- O = overhang
- R = roof

The design wind pressure is determined from Equation 8.14 as follows:

$$p = q_h(GC_{pf} - GC_{pi})$$

$$p_{WO} = 29.38 \text{ psf}(-1.07 - 0.68) = -51.4 \text{ psf}$$

$$p_{WR} = 29.38 \text{ psf}(-1.07 - 0.18) = -36.7 \text{ psf}$$

$$p_{LR} = 29.38 \text{ psf}(-0.69 - 0.18) = -25.6 \text{ psf}$$

$$p_{LO} = 29.38 \text{ psf}(-0.69 - 0.18) = -25.6 \text{ psf}$$

- The roof/ceiling dead load is adjusted for the load case where dead load is used to resist uplift forces as follows:

  Dead load $= 10 \text{ psf}(0.6) = 6 \text{ psf}$ where 0.6 is the ASD load factor for dead load in the applicable load combination

- Wind loads on the roof have both a horizontal and vertical component. The uplift connector force, located at the windward wall, can be determined by summing moments about the leeward roof-to-wall connection and solving for the connector force that will maintain moment equilibrium. Clockwise moments are considered positive.

Moment (M) created by windward overhang pressures is solved as follows:

Vertical component, windward overhang (VWO):

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (continued)

$$M_{VWO} = [(51.4 \text{ psf} \cos(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(2 \text{ ft})](1 \text{ ft} + 24 \text{ ft}) = 2{,}270 \text{ ft-lb}$$

Horizontal component, windward overhang (*HWO*):

$$M_{HWO} = [-51.4 \text{ psf} \sin(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right)\left(-\frac{2\tan(20)}{2}\right) = 13.6 \text{ ft-lb}$$

Moment (*M*) created by windward roof pressures is solved as follows:

Vertical component, windward roof (*VWR*):

$$M_{VWR} = [(36.7 \text{ psf} \cos(20)]\left(\frac{12 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(12 \text{ ft})](18 \text{ ft}) = 6{,}631.2 \text{ ft-lb}$$

Horizontal component, windward roof (*HWR*):

$$M_{HWR} = [-36.7 \text{ psf} \sin(20)]\left(\frac{12}{\cos(20)}\right)\left(\frac{12\tan(20)}{2}\right) = -349.7 \text{ ft-lb}$$

Moment (*M*) created by leeward roof pressures is solved as follows:

Vertical component, leeward roof (*VLR*):

$$M_{VLR} = [(25.6 \text{ psf} \cos(20)]\left(\frac{12}{\cos(20)}\right) + (-6 \text{ psf})(12 \text{ ft})](6 \text{ ft}) = 1{,}411.2 \text{ ft-lb}$$

Horizontal component, leeward roof (*HLR*):

$$M_{HLR} = [25.6 \text{ psf} \sin(20)]\left(\frac{12}{\cos(20)}\right)\left(\frac{12\tan(20)}{2}\right) = 243.9 \text{ ft-lb}$$

Moment(*M*) created by leeward overhang pressures is solved as follows:

Vertical component, leeward overhang (*VLO*):

$$M_{VLO} = [(25.6 \text{ psf} \cos(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(2 \text{ ft})](-1 \text{ ft}) = -39.2 \text{ ft-lb}$$

Horizontal component, leeward overhang (*HLO*):

$$M_{HLO} = [25.6 \text{ psf} \sin(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right)\left(\frac{-2\tan(20)}{2}\right) = -6.8 \text{ ft-lb}$$

The total overturning moment per ft of roof width = 10,174.3 ft-lb

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (concluded)

Solving for uplift load: F_w = 10174.3 ft-lb/roof span ft = 10,174.3 ft-lb/24 ft = 424 lb

Assuming the uplift forces are calculated for a 1-ft-wide section of the roof, the unit uplift connector force can be expressed as f_w = **424 plf.**

Note: *This solution matches the information in Table 8-6.*

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE

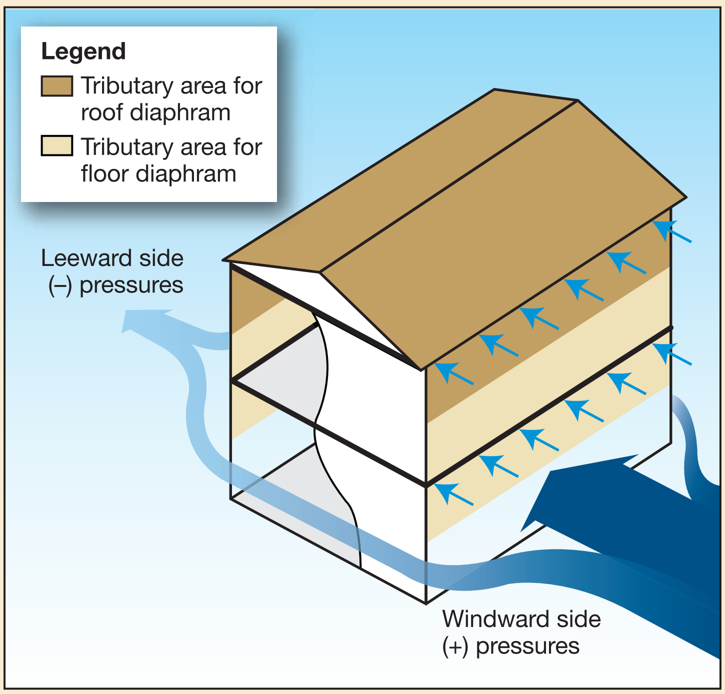


Illustration A. Lateral diaphragm loads from wind perpendicular to building ridge

Given:

- Roof span of 24 ft
- 7:12 roof pitch
- The wind load is based on 150 mph, Exposure C at 33-ft mean roof height
- The building is enclosed
- From Example 8.5, for the same site condition, the ASD velocity pressure q = 29.38 psf

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (continued)

Find: The roof diaphragm load using the envelope procedure for low-rise buildings (see Figure 28.4-1 in ASCE 7-10).

Solution: The roof diaphragm load using the envelope procedure for low-rise buildings can be found as follows:

- Lateral loads (see Illustration A) into the roof diaphragm are a function of roof slope and wall loads tributary to the roof diaphragm.
- Pressure coefficients for elements of the roof GC_{pi} and GC_{pf} are given in Table A.

Table A. Pressure Coefficients for Roof and Wall Zones

Diaphragm Zone			GC_{pi}	GC_{pf}
Roof diaphragm	Wall interior zone	Windward	0.18	0.56
		Leeward	0.18	–0.37
	Wall end zone	Windward	0.18	0.69
		Leeward	0.18	–0.48
	Roof interior zone	Windward	0.18	0.21
		Leeward	0.18	–0.43
	Roof end zone	Windward	0.18	–0.53
		Leeward	0.18	0.27
Floor diaphragm	Wall interior zone	Windward	0.18	0.53
		Leeward	0.18	–0.43
	Wall end zone	Windward	0.18	0.80
		Leeward	0.18	–0.64

- GC_{pi} is determined using the Enclosure Classification (enclosed building in this example) and Table 26.11-1 from ASCE 7-10
- GC_{pf} is determined using Figure 28.4-1 in ASCE 7-10
- Both interior zone and end zone coefficients are used to establish an average pressure on the wall and roof.

The design wind pressure is determined from Equation 8.14 ($q = q_h$ in this case) as follows:

$$p = q\,|\,GC_{pf} - GC_{pi}\,|$$

Step 1: Roof Diaphragm

- L = leeward
- W = windward

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (continued)

- w = wall

Wall interior zone

$$p_{Ww} = 29.38 \text{ psf}(0.56 - 0.18) = 11.16 \text{ psf}$$

$$p_{Lw} = 29.38 \text{ psf}(-0.37 - 0.18) = -16.16 \text{ psf}$$

$Sum = 11.16 \text{ psf} + \left|-16.16\text{psf}\right| = 27.3 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Wall end zone

$$p_{Ww} = 29.38 \text{ psf}(0.69 - 0.18) = 14.98 \text{ psf}$$

$$p_{Lw} = 29.38 \text{ psf}(-0.48 - 0.18) = -19.39 \text{ psf}$$

$Sum = 14.98 \text{ psf} + \left|-19.39 \text{ psf}\right| = 34.4 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Under the procedures and notes shown in Figure 28.4-1 of ASCE 7-10, end zones extend a minimum of 3 ft at each end of the wall. For long or tall walls, end zone lengths are based on 10 percent of the least horizontal dimension or 40 percent of the mean roof height, whichever is smaller, but not less than either 4 percent of the least horizontal dimension or 3 ft at each end of the wall. The end zone width where the pressures are applied is 3 ft.

The average pressure on the wall is:

$$P = \frac{[34.4 \text{ psf}(6 \text{ ft}) + 27.3 \text{ psf}(24 \text{ ft} - 6 \text{ ft})]}{24 \text{ ft}} = 29.1 \text{ psf}$$

where:

24 ft = building length assumed to be equal to the roof span for purposes of accounting for average effects of pressure differences at end zones and interior zones

Roof interior zone

$$p_{Ww} = 29.38 \text{ psf}(0.21 - 0.18) = 0.88 \text{ psf}$$

$$p_{Lw} = 29.38 \text{ psf}(-0.43 - 0.18) = -17.92 \text{ psf}$$

$Sum = 0.88 \text{ psf} + \left|-17.92 \text{ psf}\right| = 18.8 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Roof end zone

$$p_{Ww} = 29.38 \text{ psf}(0.27 - 0.18) = 2.64 \text{ psf}$$

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (concluded)

$p_{Lw} = 29.38\text{ psf}(-0.53 - 0.18) = -20.86\text{ psf}$

$Sum = 2.64\text{ psf} + |-20.86\text{ psf}| = 23.5\text{ psf}$ (note that leeward and windward forces are acting in the same direction)

The average pressure on the roof is:

$$P = \frac{23.5\text{ psf}(6\text{ ft}) + 18.8\text{ psf}(24\text{ ft} - 6\text{ ft})}{24\text{ ft}} = 19.98\text{ psf}$$

The roof diaphragm will take its load plus half the load of the 8-ft-tall wall below.

$$w_{roof} = \frac{1}{2}(29.1\text{ psf})(8\text{ ft}) + 19.98\text{ psf}(7\text{ ft}) = \mathbf{256.3\text{ plf}}$$

Step 2: Floor Diaphragm

- The floor diaphragm loads are based on the maximum MWRFS coefficients associated with a 20-degree roof slope. It is assumed that the floor diaphragm tributary area is the height of one 8-ft wall plus 1 ft to account for floor framing depth.

Wall interior zone

$p_{Ww} = 29.38\text{ psf}(0.53 - 0.18) = 10.28\text{ psf}$

$p_{Lw} = 29.38\text{ psf}(-0.43 - 0.18) = -17.92\text{ psf}$

$Sum = 10.28\text{ psf} + |-17.92\text{ psf}| = 28.2\text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Wall end zone

$p_{Ww} = 29.38\text{ psf}(0.80 - 0.18) = 18.22\text{ psf}$

$p_{Lw} = 29.38\text{ psf}(-0.64 - 0.18) = -24.09\text{ psf}$

$Sum = 18.22\, psf + |-24.09\, psf| = 42.3\, psf$ (note that leeward and windward forces are acting in the same direction)

The average pressure on the wall is:

$$P = \frac{42.3\text{ psf}(6\text{ ft}) + 28.2\text{ psf}(24\text{ ft} - 6\text{ ft})}{24\text{ ft}} = 31.73\text{ psf}$$

The floor diaphragm load is based on a 9-ft tributary height obtained from adding the height of one 8-ft wall plus 1 ft to account for the floor framing depth.

$$w_{floor} = 31.73\text{ psf}(9\text{ ft}) = \mathbf{286\text{ plf}}$$

***Note**: This solution matches the information in Table 8-7.*

8.7.3 Components and Cladding

ASCE 7-10 defines components and cladding (C&C) as "... elements of the building envelope that do not qualify as part of the MWFRS." These elements include roof sheathing, roof coverings, exterior siding, windows, doors, soffits, fascia, and chimneys. The design and installation of the roof sheathing attachment may be the most critical consideration because the attachment location for the sheathing is where the uplift load path begins (load path is described more fully in Chapter 9 of this Manual).

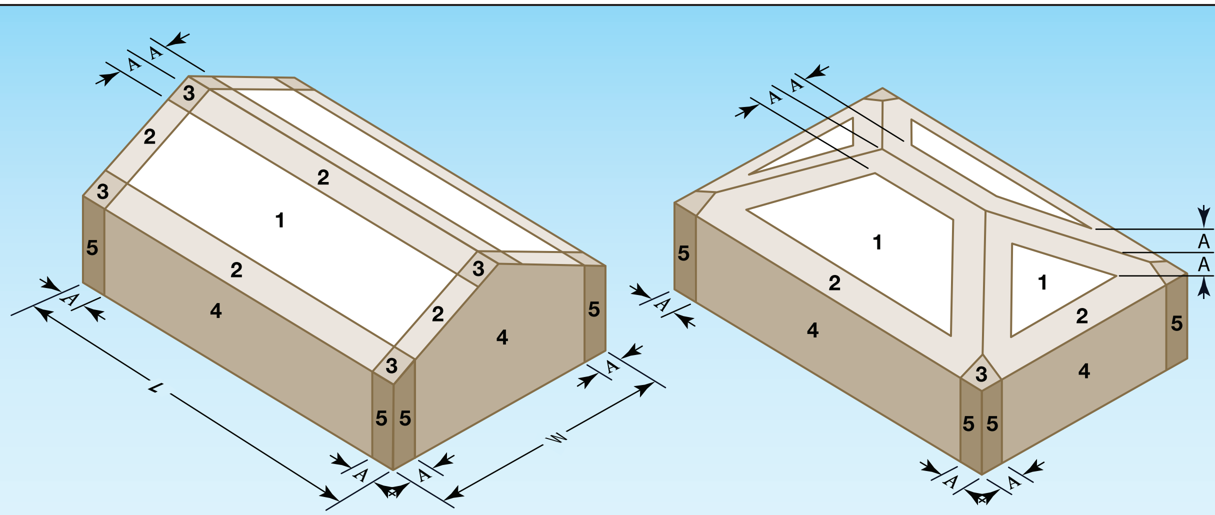
C&C pressures are determined for various "zones" of the building. ASCE 7-10 includes illustrations of those zones for both roofs and walls. Illustrations for gable, monoslope, and hip roof shapes are presented. The pressure coefficients vary according to roof pitch (from 0 degrees to 45 degrees) and effective wind area (defined in ASCE 7-10).

C&C loads act on all elements exposed to wind. These elements and their attachments must be designed to resist these forces to prevent failure that could lead to breach of the building envelope and create sources of flying debris. Examples of building elements and their connections subject to C&C loads include the following:

- Exterior siding
- Roof sheathing
- Roof framing
- Wall sheathing
- Wall framing (e.g., studs, headers)
- Wall framing connections (e.g., stud-to-plate, header-to-stud)
- Roof coverings
- Soffits and overhangs
- Windows and window frames
- Skylights
- Doors and door frames, including garage doors
- Wind-borne debris protection systems
- Any attachments to the building (e.g., antennas, chimneys, roof and ridge vents, roof turbines)

Furthermore, individual MWFRS elements of shear walls and roof diaphragms (studs and chords) may also act as components and should also be analyzed under the loading requirements of C&C.

Figure 8-20 shows the locations of varying localized pressures on wall and roof surfaces. The magnitude of roof uplift and wall suction pressures is based on the most conservative wind pressures in each location for given roof types and slopes in accordance with Figure 30.4 of ASCE 7-10. As noted previously, prescriptive



NOTE

Edge zone dimension, A, is measured as the horizontal projection on the building roof and walls.

A = the smaller of 10 percent of the least horizontal dimension of the building (i.e. either L or W) or 40 percent of the mean roof height (MRH), but not less than either 4 percent of the least horizontal dimension or 3 feet.

L = length

W = width

Figure 8-20.
Components and cladding wind pressures

standards can be used to simplify the calculation of C&C design loads. Examples of prescriptive C&C design load tables for purposes of illustration are included in this Manual as follows:

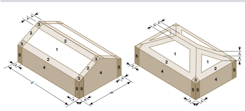
- **Roof and wall suction pressures** (see Table 8-8). Application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.7. Suction pressures are used to size connections between sheathing and framing and to size the sheathing material itself for wind induced bending. In ASCE 7-10, there is no adjustment for effective wind areas less than 10 square feet; therefore, sheathing suction loads are based on an effective wind area of 10 square feet.

- **Lateral connector loads from wind and building end zones** (see Table 8-9). Application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.8. Lateral connector loads from wind are used to size the connection from wall stud-to-plate, wall plate-to-floor, and wall plate-to-roof connections to ensure that higher C&C loads acting over smaller wall areas can be adequately resisted and transferred into the roof or floor diaphragm. In ASCE 7-10, the effective wind area for a member is calculated as the span length times an effective width of not less than one-third the span length. For example, the effective area for analysis is calculated as $h^2/3$ where h represents the span (or height) of the wall stud.

Example load tables and example problems are derived from more wind load procedures provided in the WFCM-2012 load Tables 8-8 and 8-9 are not intended to replace requirements of the building code or reference standard for the actual design of C&C attachments for a building.

Table 8-8. Roof and Wall Sheathing Suction Loads (based on ASD design), psf (33-ft mean roof height, Exposure C)

Sheathing Location	Wind Speed[(a)] (mph)								
	110	115	120	130	140	150	160	170	180
	Roof, suction pressure[(b)(c)] (psf)								
Zone 1	18.6	20.4	22.2	26.0	30.2	34.7	39.4	44.5	44.9
Zone 2	31.3	34.2	37.2	43.7	50.7	58.2	66.2	74.7	83.8
Zone 2 Overhang	34.8	38.0	41.4	48.5	56.3	64.6	73.5	83.0	93.1
Zone 3	47.1	51.5	56.0	65.8	76.3	87.5	99.6	112.4	126.1
Zone 3 Overhang	58.5	63.9	69.6	81.6	94.7	108.7	123.7	139.6	156.5
	Wall, suction pressure[(b)(c)] (psf)								
Zone 4	20.2	22.1	24.1	28.2	32.8	37.6	42.8	48.3	54.1
Zone 5	25.0	27.3	29.7	34.9	40.4	46.4	52.8	59.6	66.8



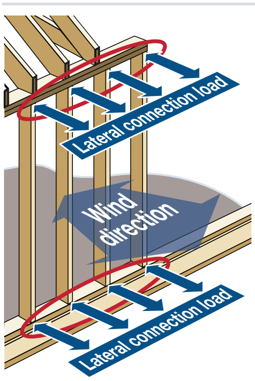
(a) 700-year wind speed, 3-sec gust.

(b) Roof and wall sheathing suction loads are based on 33-ft mean roof height and Exposure C (see Example 8.7).

(c) Loads based on minimum effective area of 10 ft^2

Table 8-9. Lateral Connector Loads from Wind at Building End Zones (Based on ASD Design), plf (33-ft mean roof height, Exposure C)

Wall Height (ft)	Wind Speed[(a)] (mph)								
	110	115	120	130	140	150	160	170	180
	Lateral connector loads[(b)(c)(d)] for wall zone 5 (plf)								
8	92	101	110	129	150	172	196	221	248
10	110	120	131	154	179	205	233	263	295
12	127	139	151	177	206	236	269	303	340
14	143	156	170	200	231	266	302	341	383
16	158	173	188	221	256	294	335	378	423



(a) 700-year wind speed, 3-sec gust.

(b) Lateral connector loads are based on 33-ft mean roof height and Exposure C (see Example 8.8).

(c) Lateral connector loads are tabulated in pounds per linear ft of wall. Individual connector loads can be calculated for various spacing of connectors (e.g., for spacing of connectors at 2 ft o.c., the individual connector load would be 2 ft times the tabulated value).

(d) Loads based on minimum area of (wall height)$^{2/3}$

EXAMPLE 8.7. ROOF SHEATHING SUCTION LOADS

Given:

- The wind load is based on 150 mph, Exposure C at 33-ft mean roof height
- The building is enclosed
- From Example 8.4, for the same site condition, the ASD velocity pressure q = 29.38 psf
- The internal pressure coefficient for roof and wall sheathing is GC_{pi}= 0.18

Find: Roof sheathing and wall sheathing suction loads using the C&C coefficients specified in Figure 30.4 of ASCE 7-10.

For cladding and fasteners, the effective wind area should not be greater than the area that is tributary to an individual fastener. In ASCE 7-10, there is no adjustment for wind areas less than 10 ft^2; therefore, sheathing suction loads are based on an effective wind area of 10 ft^2 for different zones on the roof.

Solution: The roof sheathing and wall sheathing suction loads can be determined using the C&C coefficients specified in Figure 30.4 of ASCE 7-10, as follows:

- The design wind pressure is determined from Equation 8.14 (where q = q_h in this case) as follows:

  $$p = q\,|\,GC_{pf} - GC_{pi}\,|$$

- Determine the roof sheathing suction load pressure coefficients using Figure 30.4 of ASCE 7-10 as follows:

Step 1: Roof sheathing suction loads pressure coefficients

Pressure coefficient equations developed from C&C, Figure 30.4, graphs of ASCE 7-10 coefficients:

$$\text{Zone 1: } GC_{pf} = 0.9 - 0.1\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -1.0$$

$$\text{Zone 2: } GC_{pf} = -1.1 - 0.7\left[\frac{\log\left(\frac{A}{100}\right)}{\text{og}\left(\frac{10}{100}\right)}\right] = -1.8$$

$$\text{Zone 2 Overhang: } GC_{pf} = -2.2$$

EXAMPLE 8.7. ROOF SHEATHING SUCTION LOADS (concluded)

$$\text{Zone 3: } GC_{pf} = -1.7 - 1.1\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -2.8$$

$$\text{Zone 3 Overhang: } GC_{pf} = 2.5 - 1.2\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -3.7$$

Step 2: Wall sheathing suction loads pressure coefficient

Pressure coefficient equations developed from C&C, Figure 30.4 graphs of ASCE 7-10 coefficients:

$$\text{Zone 4: } GC_{pf} = -0.8 - 0.3\left[\frac{\log\left(\frac{A}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.1$$

$$\text{Zone 5: } GC_{pf} = -0.8 - 0.6\left[\frac{\log\left(\frac{A}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.4$$

Step 3: For all zones – internal pressure coefficient

GC_{pi} = +/-0.18

Step 4: Calculate roof sheathing and wall sheathing suction pressures for all zones using Equation 8.14

Zone 1: $p = 29.38\text{ psf}(-1 - 0.18) = -\mathbf{34.7\ psf}$

Zone 2: $p = 29.38\text{ psf}(-1.8 - 0.18) = -\mathbf{58.2\ psf}$

Zone 2 Overhang: $p = 29.38\text{ psf}(-2.2) = -\mathbf{64.6\ psf}$

Zone 3: $p = 29.38\text{ psf}(-2.8 - 0.18) = -\mathbf{87.5\ psf}$

Zone 3 Overhang: $p = 29.38\text{ psf}(-3.7) = -\mathbf{108.7\ psf}$

Zone 4: $p = 29.38\text{ psf}(-1.1 - 0.18) = -\mathbf{37.6\ psf}$

Zone 5: $p = 29.38\text{ psf}(-1.4 - 0.18) = -\mathbf{46.4\ psf}$

***Note**: This solution matches the information in Table 8-8.*

EXAMPLE 8.8. LATERAL CONNECTION FRAMING LOADS FROM WIND

Given:

- Wind load is based on 150 mph, Exposure C at 33-ft mean roof height, and wall and diaphragm framing as shown in Illustration A
- Building is enclosed
- Wall height is 10 ft
- Stud spacing is 16 in. o.c.
- Sheathing effective area is 10 ft^2
- ASD velocity pressure q = 29.38 psf (from Example 8.5)
- Wall suction equations for Zone 4 and Zone 5 are provided in Example 8.7
- Internal pressure coefficient for wall sheathing is GC_{pi} = +/- 0.18

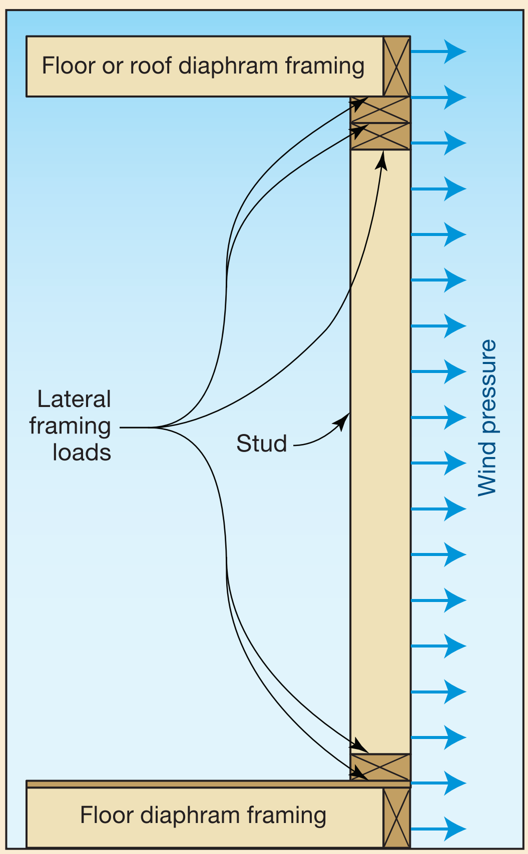


Illustration A. Lateral connector loads for wall-to-roof and wall-to-floor connections

Find: Framing connection requirements at the top and base of the wall.

Solution: The connector load can be determined as follows:

EXAMPLE 8.8. LATERAL CONNECTION FRAMING LOADS FROM WIND (concluded)

- C&C coefficients are used
- When determining C&C pressure coefficients, the effective wind area equals the tributary area of the framing members
- For long and narrow tributary areas, the area width may be increased to one-third the framing member span to account for actual load distributions
- The larger area results in lower average wind pressures
- The increase in width for long and narrow tributary areas applies only to calculation of wind pressure coefficients
- Determine the tributary area and pressure coefficient GC_{pf} for the wall sheathing: Stud effective wind area equals 13.33 ft². The minimum required area for analysis is $h^2/3$=33.3 ft², where h is 10 ft
- In accordance with ASCE 7-10, the pressure coefficient, GC_{pf}, for wall sheathing can be determined based on a minimum effective wind area of 33.3 ft² as follows:

$$\text{Zone 5: } GC_{pf} = -0.8 - 0.6\left[\frac{\log\left(\frac{33.3}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.22$$

The design wind pressure is determined as follows from Equation 8.14: $p = q(GC_{pf} - GC_{pi})$

$$\text{Zone 5: } p = 29.38\text{ psf}(-1.22 - 0.18) = -41.13\text{ psf}$$

The required capacity of connectors assuming load is based on half the wall height:

$$\text{Zone 5: } w = 41.13\text{ psf}\left(\frac{10\text{ ft}}{2}\right) = \mathbf{205\ plf}$$

Note: *This solution matches the information in Table 8-9.*

8.8 Tornado Loads

Tornadoes have wind speeds that vary based on the magnitude of the event; more severe tornadoes have wind speeds that are significantly greater than the minimum design wind speeds required by the building code. Designing an entire building to resist tornado-force winds of EF3 or greater based on the Enhanced Fujita tornado damage scale (in EF2 tornadoes, large trees are snapped or uprooted) may be beyond the realm

WARNING

Safe rooms should be located outside known flood-prone areas, including the 500-year floodplain, and away from any potential large debris sources. See Figure 5-2 of FEMA 320 for more direction regarding recommended siting for a safe room.

of practicality and cost-effectiveness, but this does not mean that solutions that provide life-safety protection cannot be achieved while maintaining cost-effectiveness.

A more practical approach is to construct an interior room or space that is "hardened" to resist not only tornado-force winds but also the impact of wind-borne missiles. FEMA guidance on safe rooms can be found in FEMA 320, *Taking Shelter from the Storm: Building a Safe Room for Your Home or Small Business* (FEMA 2008c), which provides prescriptive design solutions for safe rooms of up to 14 feet x 14 feet. These solutions can be incorporated into a structure or constructed as a nearby stand-alone safe room to provide occupants with a place of near-absolute protection. The designs in FEMA 320 are based on wind pressure calculations that are described in FEMA 361, *Design and Construction Guidance for Community Safe Rooms* (FEMA 2008a). FEMA 361 focuses on larger community safe rooms, but the process of design and many of the variables are the same for smaller residential safe rooms.

An additional reference, ANSI/ICC 500-2008 complements the information in FEMA 320 and FEMA 361 and is referenced in the 2012 IBC and 2012 IRC.

Safe rooms can be designed to resist both tornado and hurricane hazards, and though many residents of coastal areas are more concerned with hurricanes, tornadoes can be as prevalent in coastal areas as they are in inland areas such as Oklahoma, Kansas, and Missouri. Constructing to minimum requirements of the building code does not include the protection of life-safety or property of occupants from a direct hit of large tornado events. Safe rooms are not recommended in flood hazard areas.

8.9 Seismic Loads

This Manual uses the seismic provisions of ASCE 7-10 to illustrate a method for calculating the seismic base shear. To simplify design, the effect of dynamic seismic ground motion accelerations can be considered an equivalent static lateral force applied to the building. The magnitude of dynamic motion, and therefore the magnitude of the equivalent static design force, depends on the building characteristics, and the spectral response acceleration parameter at the specific site location.

The structural configuration in Figure 8-21 is called an "inverted pendulum" or "cantilevered column" system. This configuration occurs in elevated pile-supported buildings where almost all of the weight is at the top of the piles. For a timber frame cantilever column system, ASCE 7-10 assigns a response modification factor (R) equal to 1.5 (e.g., R = 1.5). For wood frame, wood structural panel shear walls, ASCE 7-10 assigns an R factor equal to 6.5. The R factor of 1.5 can be conservatively used to determine shear for the design of all elements and connections of the structure. An R factor of 1.5 is not permitted for use in Seismic Design Categories E and F per ASCE 7-10.

ASCE 7-10 contains procedures for the seismic design of structures with different structural systems stacked vertically within a single structure. Rules for vertical combinations can be applied to enable the base of the structure to be designed for shear forces associated with R=1.5 and the upper wood frame, wood structural panel shear wall structure to be designed for reduced shear forces associated with R=6.5.

ASCE 7-10 also provides R factors for cantilever column systems using steel and concrete columns. A small reduction in shear forces for steel piles or concrete columns could be obtained by using what ASCE 7-10 calls a "steel special cantilever column system" or a "special reinforced concrete moment frame," both of which have an R = 2.5. However, these systems call for additional calculations, connection design, and

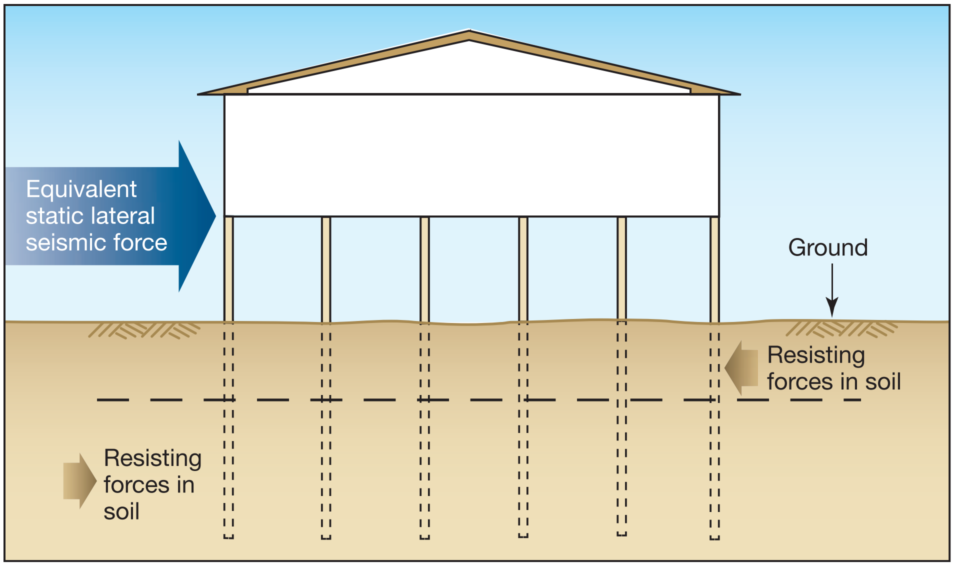


Figure 8-21.
Effect of seismic forces on supporting piles

detailing, which are not commonly done for low-rise residential buildings. An engineer experienced in seismic design should be retained for this work, and builders should expect larger pile and column sizes and more reinforcing than is normally be required in a low-seismic area.

Total seismic base shear can be calculated using the Equivalent Lateral Force (ELF) procedure of ASCE 7-10 in accordance with Equation 8.15.

EQUATION 8.15. SEISMIC BASE SHEAR BY EQUIVALENT LATERAL FORCE PROCEDURE

$$V = C_s W \quad \text{(Eq. 8.15a)}$$

$$C_s = \frac{S_{DS}}{(R/I)} \quad \text{(Eq. 8.15b)}$$

where:

V = seismic base shear

C_s = seismic response coefficient

S_{DS} = design spectral response acceleration parameter in the short period range

R = response modification factor

I = occupancy importance factor

W = effective seismic weight

Lateral seismic forces are distributed vertically through the structure in accordance with Equation 8.16, taken from ASCE 7-10.

EQUATION 8.16. VERTICAL DISTRIBUTION OF SEISMIC FORCES

$$F_x = C_{vx}V \qquad \text{(Eq. 8.16a)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(Eq. 8.16b)}$$

where:

F_x = lateral seismic force induced at any level

C_{vx} = vertical distribution factor

V = seismic base shear

w_i *and* w_x = portion of the total effective seismic weight of the structure (w) located or assigned to level i or x

h_i *and* h_x = height from the base to Level i or x

k = exponent related to the structure period; for structures having a period of 0.5 sec or less, k=1

The calculated seismic force at each story must be distributed into the building frame. The horizontal shear forces and related overturning moments are taken into the foundation through a load path of horizontal floor and roof diaphragms, shear walls, and their connections to supporting structural elements. A complete seismic analysis includes evaluating the structure for vertical and plan irregularities, designing elements and their connections in accordance with special seismic detailing, and considering structural system drift criteria. Example 8.9 illustrates the use of basic seismic calculations.

EXAMPLE 8.9. SEISMIC LOAD

Given:

- S_{DS} for the site = 2/3F_aS_s, which is determined to be 2/3(1.2)(0.50) = 0.4
- The building structure as shown in Illustration A. Dead load for the building is as follows:

Roof and ceiling = 10 lb/ft^2

Exterior walls = 10 lb/ft^2

Interior Walls = 8 lb/ft^2

Floor = 10 lb/ft^2

Piles = 409 lb each

EXAMPLE 8.9. SEISMIC LOAD (continued)

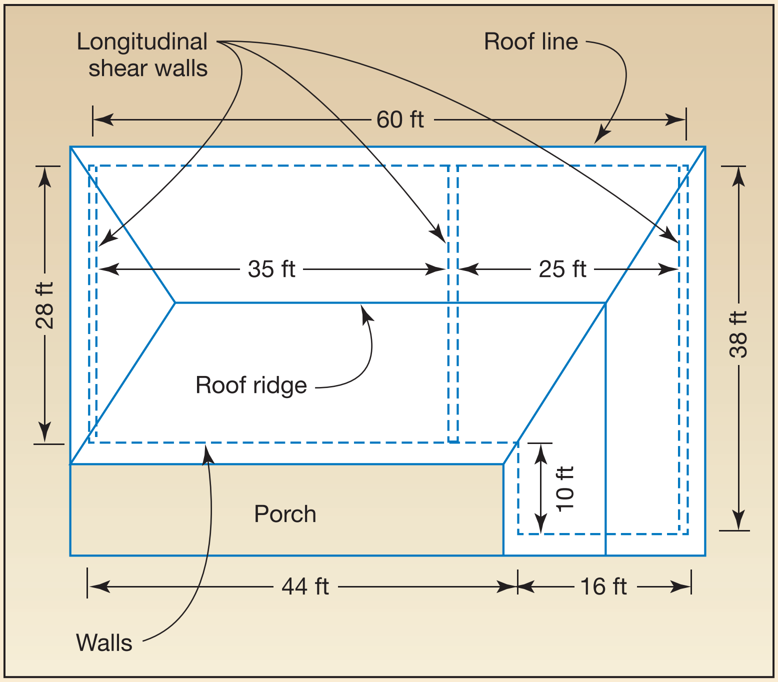


Illustration A. Building elevation and plan view of roof showing longitudinal shearwalls; dimensions are wall-to-wall and do not include the 2-ft roof overhang

Find (using ASCE 7-10 ELF procedure):

1. The total shear wall force
2. The shear force at the top of the pile foundation

Solution for #1: The total shear wall force using the ASCE 7-10 ELF procedure can be determined as follows:

- Calculate effective seismic weight:

  Roof/ceiling: (10 lb/ft^2)(2,390 ft^2) = 23,900 lb

  Exterior walls: (10 lb/ft^2)(1,960 ft^2) = 19,600 lb

  Interior partitions: (8 lb/ft^2)(2,000 ft^2) = 16,000 lb

  Floor = (10 lb/ft^2)(2,160 ft^2) = 21,600 lb

  Piles: (409 lb/pile)(31 piles) = 12,679 lb

  Total effective seismic weight:
  W = 23,900 + 19,600 lb + 16,000 lb + 21,600 lb + 12,679 lb = 93,454 lb

EXAMPLE 8.9. SEISMIC LOAD (continued)

- Seismic forces are distributed vertically as follows:

  Roof level:
  Effective seismic weight, $w_{x\ roof}$ = 23,900 lb + (0.5)(19,600) lb + (0.5)(16,000/2) lb = 41,700 lb

  Height from base, $h_{x\ roof}$ = 18 ft

  $w_{x\ roof}(h_{x\ roof})$ = 750,600 ft-lb

  Floor level:
  Effective seismic weight, $w_{x\ floor}$ = 19,600/2 lb + 16,000/2 lb + 21,600 lb + 12,679 lb = 52,079 lb

  Height from base: $h_{x\ floor}$ = 8 ft

  $w_{x\ floor}(h_{x\ floor})$ = (52,079 lb)(8 ft) = 416,632 ft-lb

  $$C_{vx,roof} = \frac{750{,}600 \text{ ft-lb}}{750{,}600 \text{ ft-lb} + 416{,}632 \text{ ft-lb}} = 0.64 \text{ from Equation 8.16}$$

  $$C_{vx,\,floor} = \frac{416{,}632 \text{ ft-lb}}{750{,}000 \text{ ft-lb} + 416{,}632 \text{ ft-lb}} = 0.36 \text{ from Equation 8.16}$$

  The force in the shear walls and at the top of the piles will vary by the *R* factor for the shear wall system and the pile system (e.g., cantilevered column system).

- Lateral seismic force at the roof level for design of wood-frame shear walls (*R* = 6.5):

  $$F_{x\ roof} = C_{vx\ roof}V = C_{vx\ roof}\frac{S_{DS}}{R/I}W \text{ using Equation 8.15 for } V \text{ substituted into Equation 8.16}$$

  $$F_{x,roof} = \left[\frac{(0.64)(0.4)}{\frac{6.5}{1.0}}\right](93{,}454 \text{ lb}) = 3{,}681 \text{ lb}$$

where:

R = 6.5 for light-frame walls with plywood

I = 1.0 for residential structure

The design shear force for the shear walls is based on the lateral seismic force at the roof level. Total seismic force for shear wall design is **3,681 lb**

Solution for #2: The shear force to the top of the pile foundation (i.e., cantilevered column system, R = 1.5) can be determined as follows:

- Roof level

  $$F_{x\ roof} = C_{vx\ roof}V = C_{vx\ roof}\frac{S_{DS}}{R/I}W \text{ using Equation 8.15 for } V \text{ substituted into Equation 8.16}$$

EXAMPLE 8.9. SEISMIC LOAD (concluded)

$$F_{x,roof} = \left[\frac{(0.64)(0.4)}{\frac{1.5}{1.0}}\right](93{,}454 \text{ lb}) = 15{,}949 \text{ lb}$$

- Floor level

$$F_{x\ floor} = C_{vx\ floor}V = C_{vx\ floor}\frac{S_{DS}}{R/I}W$$ using Equation 8.15 for V substituted into Equation 8.16

$$F_{x,\ floor} = \left[\frac{(0.36)(0.4)}{\frac{1.5}{1.0}}\right](93{,}454 \text{ lb}) = 8{,}972 \text{ lb}$$

where:

R = 1.5 for cantilevered column system. For vertically mixed seismic-force-resisting systems, ASCE 7-10 allows a lower R to be used below a higher R value.

I = 1.0 for a residential structure

Total shear at the floor is based on the sum of the force at the roof level and the floor level:

F_{floor} = 15,949 lb + 8,972 lb = **24,921 lb**

8.10 Load Combinations

It is possible for more than one type of natural hazard to occur at the same time. Floods can occur at the same time as a high-wind event, which happens during most hurricanes. Heavy rain, high winds, and flooding conditions can occur simultaneously. ASCE 7-10 addresses the various load combination possibilities.

The following symbols are used in the definitions of the load combinations:

D = dead load

L = live load

E = earthquake load

F = load due to fluids with well-defined pressures and maximum heights (e.g., fluid load in tank)

F_a = flood load

H = loads due to weight and lateral pressures of soil and water in soil

L_r = roof live load

S = snow load

R = rain load

T = self-straining force

W = wind load

Loads combined using the ASD method are considered to act in the following combinations for buildings in Zone V and Coastal A Zone (Section 2.4.1 of ASCE 7-10), whichever produces the most unfavorable effect on the building or building element:

Combination No. 1: D

Combination No. 2: $D + L$

Combination No. 3: $D + (Lr \text{ or } S \text{ or } R)$

Combination No. 4: $D + 0.75L + 0.75(Lr \text{ or } S \text{ or } R)$

Combination No. 5: $D + (0.6W \text{ or } 0.7E)$

Combination No. 6a: $D + 0.75L + 0.75(0.6W) + 0.75(Lr \text{ or } S \text{ or } R)$

Combination No. 6b: $D + 0.75L + 0.75(0.7E) + 0.75S$

Combination No. 7: $0.6D + 0.6W$

Combination No. 8: $0.6D + 0.7E$

When a structure is located in a flood zone, the following load combinations should be considered in addition to the basic combinations in Section 2.4.1 of ASCE 7-10:

- In Zone V or Coastal A Zone, $1.5F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6
- In the portion of Zone A landward of the LiMWA, $0.75F_a$ should be added to combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6.

The ASCE 7-10 *Commentary* states "Wind and earthquake loads need not be assumed to act simultaneously. However, the most unfavorable effects of each should be considered in design, where appropriate."

The designer is cautioned that F is intended for fluid loads in tanks, not hydrostatic loads. F_a should be used for all flood loads, including hydrostatic loads, and should include the various components of flood loads as recommended in Section 8.5.11 in this chapter. It is important to note that wind and seismic loads acting on a building produce effects in both the vertical and horizontal directions. The load combinations discussed in this section must be evaluated carefully, with consideration given to whether a component of the wind or seismic load acts in the same vertical or horizontal direction as other loads in the combination. In some cases, gravity loads (dead and live loads) may counteract the effect of the wind or seismic load, either vertically or horizontally. Building elements submerged in water have a reduced effective weight due to buoyancy. Example 8.10 illustrates the use of load combinations for determining design loads.

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM

Given:

Use the flood loads from Example 8.3:

- $F_{sta} = 0$
- $F_{dyn} = 909$ lb
- $F_{brkp} = 625$ lb
- $F_i = 2{,}440$ lb
- $d_s = 4.6$ ft

Use for wind loads:

- Roof span = 28 ft
- Roof pitch = 7:12
- Wall height = 10 ft
- Wind uplift load = 33,913 lb (pre-factored with 0.6)
- Exposure Category D (multiply Exposure C wind loads by 1.18 at 33 ft mean roof height)
  - 1.18 is a conservative value because while the higher Exposure Category D has been factored, the lower roof height (24 ft versus 33 ft) has not. Refer to ASCE 7-10, Figure 28.6-1 for guidance.

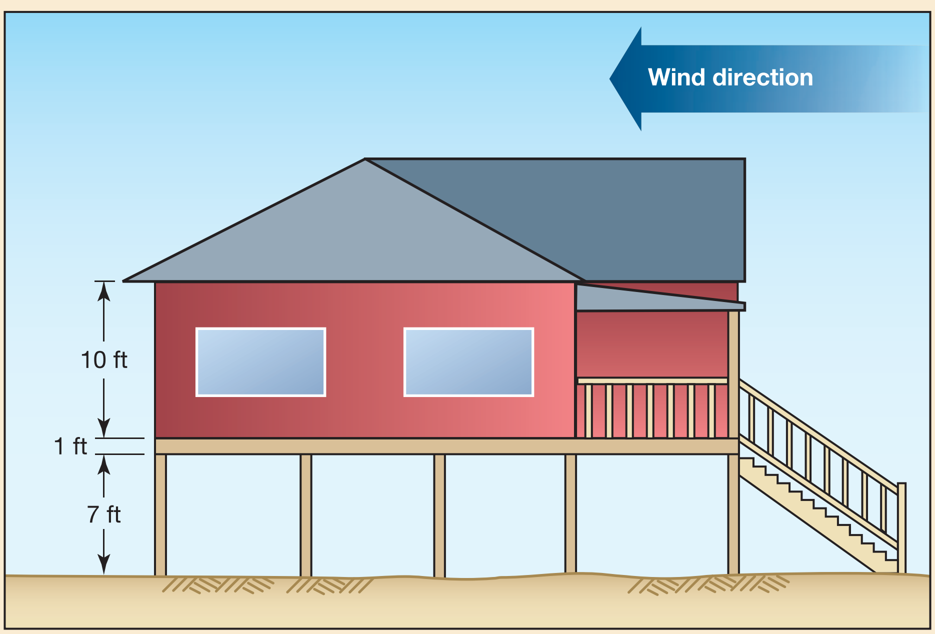


Illustration A. Side view of building

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

Use for dead load:

- 95,090 lb for house and piles

Use for buoyancy load:

- 9,663 lb

The locations given in Illustration B for the forces.

Find:

1. Calculate maximum horizontal wind load that occurs perpendicular to the ridge and the floor for the example building
2. Find the horizontal load required for foundation design
3. Calculate global overturning moment due to horizontal loads and wind uplift (see Illustration B)

Solution for #1: To determine the horizontal wind load perpendicular to the ridge, use the projected area method as follows:

- For wind perpendicular to the ridge of a roof with a span of 28 ft (using Table 8-7), 7:12 roof pitch and wall height of 10 ft, Category D as shown in Illustration A, the lateral roof diaphragm load, w_{roof}, can be found by interpolation between the 24 ft and 32 ft roof span w_{roof} values in Table 8.7:

  w_{roof} = (256 plf + 299 plf)(0.5) = 278 plf

To adjust w_{roof} for Exposure Category D due to the fact Table 8-7 assumes Exposure Category C:

$$w_{roof} = 1.18(278 \text{ plf}) = 328 \text{ plf}$$

where

1.18 = Exposure D adjustment factor (33 ft mean roof height)

To adjust w_{roof} for a wall height of 10 ft due to the fact Table 8.7 assumes a wall height of 8 ft:

$$w_{roof} = (328 \text{ plf})\left(\frac{10 \text{ ft}}{8 \text{ ft}}\right) = 410 \text{ plf}$$

- Determine lateral floor diaphragm load, w_{floor} from Table 8-7. Once more, this value needs to be adjusted for Exposure Category D from the assumed Exposure Category C:

  $$w_{floor} = 1.18(286 \text{ plf}) = 338 \text{ plf}$$

where

1.18 = Exposure D adjustment factor (33 ft mean roof height)

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

To adjust w_{roof} for a wall height of 10 ft due to the fact Table 8.7 assumes a wall height of 8 ft:

$$w_{floor} = (338\text{ plf})\left(\frac{10\text{ ft}}{8\text{ ft}}\right) = 423\text{ plf}$$

Finally, adjust this value to account for the reference case in Table 8-7 assuming the lateral floor diaphragm load is from wind pressures on the lower half of the wall above and the upper half of the wall below the floor diaphragm. Because the structure is open below the floor diaphragm level, adjust w_{floor} to account for the presence of only half of the wall area used in the reference case for Table 8-7 (e.g., structure is open below first floor diaphragm):

$$w_{floor} = 0.5(423\text{ plf}) = 212\text{ plf}$$

- For building length = 60 ft, total horizontal shear at the top of the foundation is:

  $W_{foundation}$ = (410 plf + 212 plf)(60 ft) = **37,320 lb**

Solution for #2: The horizontal load required for foundation design, can be determined using the following calculations of the load combination equations given in Section 8.10:

- Zone V and Coastal A Zone

  5. $D + 0.6W + 1.5F_a$

  6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) + 1.5F_a$

  6b. $D + 0.75L + 0.75S + 1.5F_a$

  7. $0.6D + 0.6W + 1.5F_a$

  Load combination No. 5 produces the maximum shear at the foundation for the loads considered. This load combination includes a wind load factor adjustment of 0.6. Because the value of $W_{foundation}$ from Solution #1 has already been adjusted by 0.6 for ASD, it will not be further adjusted in the calculations that follow.

  For flood load, the value of F_a is determined in accordance with Table 8-5. The hydrodynamic load is greater than breaking wave load, therefore, F_a for an individual pile and the foundation as a whole (i.e., global) is calculated as:

  $F_{a(individual)} = F_i + F_{dyn}$ = 2,440 lb + 909 lb = 3,349 lb

  $F_{a(global)}$ = (1 pile)F_i + (35 piles)F_{dyn} = 34,255 lb

  5. Total shear: $37,320\text{ lb} + 1.5(34,255\text{ lb}) = \mathbf{88,703\text{ lb}}$

- Portion of Zone A landward of the LiMWA

  5. $D + 0.6W + 0.75F_a$

  6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) + 0.75F_a$

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

6b. $D + 0.75L + 0.75S + 0.75F_a$

7. $0.6D + 0.6W + 0.75F_a$

Load combination 5 produces the maximum shear at the foundation for the loads considered.

5. Total shear: $37{,}320 + 0.75(34{,}255\text{ lb}) = \mathbf{63{,}011\ lb}$

Note: *Considering seismic force from Example 8.8, ASD shear force at the foundation is determined by load combination No. 8:*

8. Total seismic base shear = 0.7(24,921 lb) = **17,444 lb**

Solution for #3: To determine the factored global overturning moment due to the factored loads on the building, take the moments about the pivot point in Illustration B.

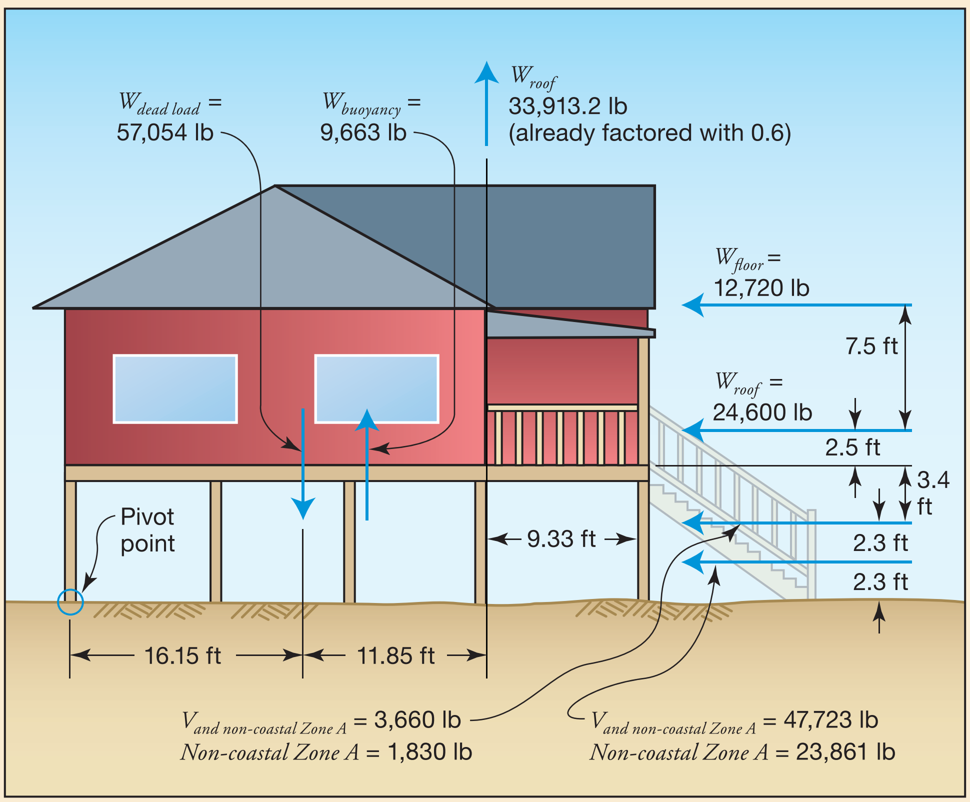


Illustration B. Loads on building for global overturning moment calculation

Load Combination 7 produces the maximum overturning at the foundation for the loads considered. Factored global overturning moment can be calculated from the factored loads and their location of application as shown in Illustration B.

- Zone V and Coastal A Zone

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (concluded)

7. $0.6D + 0.6W + 1.5F_a$ gives the appropriate factors to be used in calculating the factored global overturning moment

From Illustration B:

$$M_{global} = (0.6)w_{roof}(18\text{ ft}) + (0.6)w_{floor}(10.5\text{ ft}) + (1.5)F_i(4.6\text{ ft}) + (1.5)F_{dyn}(2.3\text{ ft}) + W_{uplift}(28\text{ ft}) - (0.6)DL(16.15\text{ ft}) + (1.5)F_b(19\text{ ft})$$

$$M_{global} = (0.6)(24{,}600\text{ lb})(18\text{ ft}) + (0.6)(12{,}720\text{ lb})(10.5\text{ ft}) + (1.5)(2{,}440\text{ lb})(4.6\text{ ft}) + (1.5)(31{,}815\text{ lb})(2.3\text{ ft}) + (33{,}913\text{ lb})(28\text{ ft}) - (0.6)(95{,}090\text{ lb})(16.15\text{ ft}) + (1.5)(9{,}663\text{ lb})(19\text{ ft})$$
$$= \mathbf{776{,}000\text{ ft-lb}}$$

- The portion of Zone A landward of the LiMWA

7. $0.6D + 0.6W + 0.75F_a$ gives the appropriate factors to be used in calculating the factored global overturning moment

From Illustration B:

$$M_{global} = (0.6)w_{roof}(18\text{ ft}) + (0.6)w_{floor}(10.5\text{ ft}) + (0.75)F_i(4.6\text{ ft}) + (0.75)F_{dyn}(2.3\text{ ft}) + W_{uplift}(28\text{ ft}) - (0.6)DL(16.15\text{ ft}) + (.75)F_b(19\text{ ft})$$

$$M_{global} = (0.6)(24{,}600\text{ lb})(18\text{ ft}) + (0.6)(12{,}720\text{ lb})(10.5\text{ ft}) + (0.75)(2{,}440\text{ lb})(4.6\text{ ft}) + (0.75)(31{,}815\text{ lb})(2.3\text{ ft}) + (33{,}913\text{ lb})(28\text{ ft}) - (0.6)(95{,}090\text{ lb})(16.15\text{ ft}) + (0.75)(9{,}663\text{ lb})(19\text{ ft}) = \mathbf{575{,}000\text{ ft-lb}}$$

***Note**: In this example, the required uplift capacity to resist overturning is estimated by evaluating the skin friction capacity of the piles. The total pile uplift capacity is approximately 908,000 ft-lb. which exceeds both calculated overturning moments and is based on the horizontal distance to each row of piles from the pivot point and the following assumptions:*

- Pile embedment: 19.33 ft
- Pile size: 10 in.
- Coefficient of friction: 0.4 for wood piles
- Density of sand: 128 lb/ft^3
- Coefficient of lateral pressure: 09.5
- Critical depth fir sand: 15 ft
- Angle of internal friction: 38°
- Scour depth: 5 ft
- Factor of safety: 2

The following worksheet can be used to facilitate load combination computations.

Worksheet 3. Load Combination Computation

Load Combination Computation Worksheet	
OWNER'S NAME: ________________	PREPARED BY: ________________
ADDRESS: ________________	DATE: ________________
PROPERTY LOCATION: ________________	

Variables

D (dead load)	=
E (earthquake load)	=
L (live load)	=
F (fluid load)	=
F_a (flood load)	=
H (lateral soil and water in soil load)	=
L_r (roof live load)	=
S (snow load)	=
R (rain load)	=
T (self-straining force)	=
W (wind load)	=

Summary of Load Combinations:

1.

2.

3.

4.

5.

6a.

6b.

7.

8.

Combination No. 1

D =

Combination No. 2

$D + L$ =

Combination No. 3

$D + (L_r$ or S or $R)$ =

Worksheet 3. Load Combination Computation (concluded)

Combination No. 4
$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) =$
Combination No. 5
$D + (0.6W \text{ or } 0.7E) =$
Combination No. 6a
$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) =$
Combination No. 6b
$D + 0.75L + 0.75(0.7E) + 0.75S =$
Combination No. 7
$0.6D + 0.6W =$
Combination No. 8
$0.6D + 0.7E =$
When a structure is located in a flood zone, the following load combinations should be considered in addition to the basic combinations: ■ In Zone V or Coastal A Zone, $1.5F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6. ■ In the portion of Zone A landward of the LiMWA, $0.75F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6.

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