Building on Strong and

Safe Foundations

3. Foundation Design Loads

This chapter provides guidance on how to determine the magnitude of the loads placed on a building by a particular natural hazard event or a combination of events. The methods presented are intended to serve as the basis of a methodology for applying the calculated loads to the building during the design process.

The process for determining site-specific loads from natural hazards begins with identifying the building codes or engineering standards in place for the selected site (e.g., the International Building Code 2003 (IBC 2003) or ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures)*, if model building codes and other building standards do not provide load determination and design guidance for each of the hazards identified. In such instances, supplemental guidance such as FEMA 55 should be sought, the loads imposed by each of the identified hazards should be calculated, and the load combinations appropriate for the building site should be determined. The load combinations used in this manual are those specified by ASCE 7-02, the standard referenced by the IBC 2003. Either Allowable Stress Design (ASD) or Strength

Design methods can be used to design a building. For this manual, all of the calculations, analyses, and load combinations presented are based on ASD. The use of Strength Design methods will require the designer to modify the design values presented in this manual to accommodate Strength Design concepts. Assumptions utilized in this document can be found in Appendix C.

3.1 Wind Loads

Wind loads on a building structure are calculated using the methodology presented in ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures.* This document is the wind standard referenced by the 2003 editions of the IBC and IRC. Equations used to calculate wind loads are presented in Appendix D.

The most important variable in calculating wind load is the design wind speed. Design wind speed can be obtained from the local building official or the ASCE 7-02 wind speed map (see Figure 3-1). The speeds shown in this figure are 3-second gust speeds for Exposure Category C at a 33-foot (10-meter) height. ASCE 7-02 includes scaling factors for other exposures and heights.

ASCE 7-02 specifies wind loads for structural components known as a Main Wind Force Resisting System (MWFRS). The foundation designs developed for this manual are based on MWFRS pressures calculated for Exposure Category C, the category with the highest anticipated wind loads for land-based structures.

ASCE 7-02 also specifies wind loads for components and cladding (C&C). Components and cladding are considered part of the building envelope, and ASCE 7-02 requires C&C to be designed to resist higher wind pressures than MWFRS.
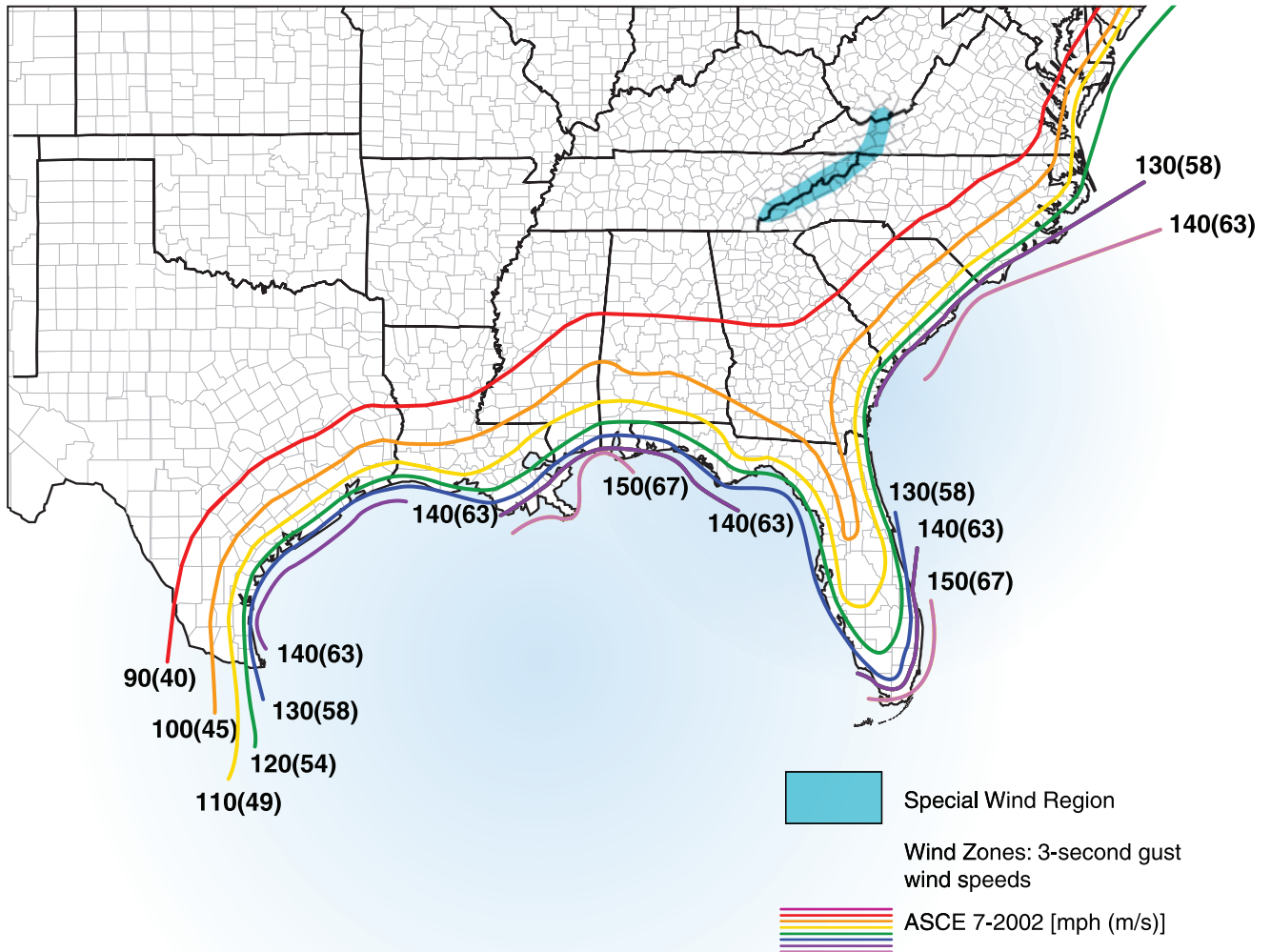
130(58)

140(63)

150(67)

140(63)

130(58)

140(63)

140(63)

150(67)

140(63)

90(40)

130(58)

100(45)

120(54)

110(49)

Special Wind Region

Wind Zones: 3-second gust wind speeds

ASCE 7-2002 [mph (m/s)]

Figure 3-1.   Wind speed (in mph) in the U.S. Gulf Coast area

SOURCE: ASCE 7-02

3.2   Flood Loads

This manual develops in more detail flood load calculations and incorporates the methodology presented in ASCE 7-02. Although wind loads can directly affect a structure and dictate the actual foundation design, the foundation is more affected by flood loads. ASCE 24 discusses floodproof construction. Loads developed in ASCE 24 come directly from ASCE 7-02, which is what the designs presented herein are based upon.

The effects of flood loads on buildings can be exacerbated by storm-induced erosion and localized scour, and by long-term erosion. Erosion and scour lower the ground surface around foundation members and can cause the loss of load-bearing capacity and resistance to lateral and uplift loads. Erosion and scour also increase flood depths and, therefore, increase depth dependent flood loads.
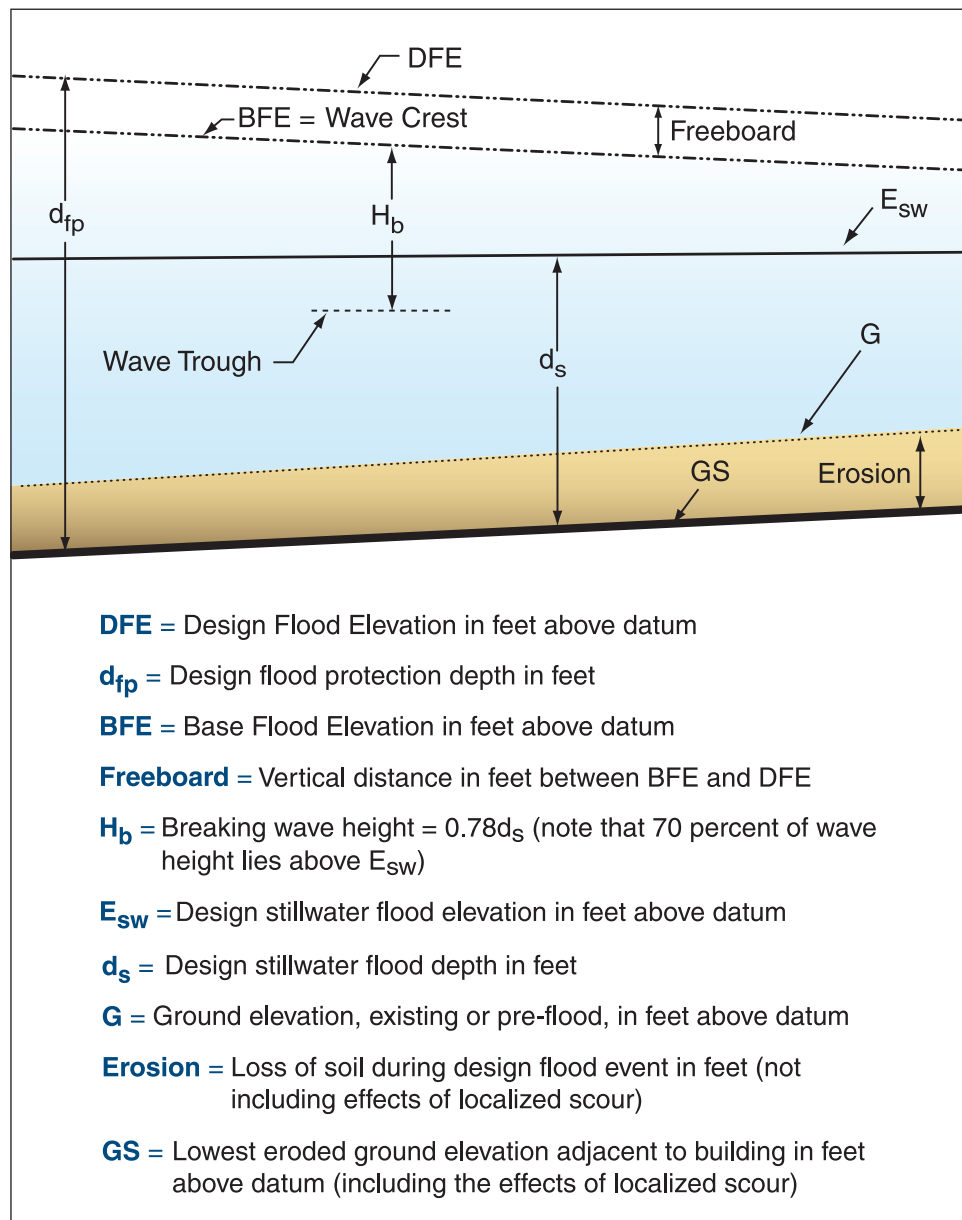
3.2.1 Design Flood and Design Flood Elevation (DFE)

The design flood is defined by ASCE 7-02 as the greater of the following two flood events:

1. 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 defined as the elevation of the design flood, including wave height and freeboard, relative to the datum specified on a community's flood hazard map. Figure 3-2 shows the parameters that determine or are affected by flood depth.

**Figure 3-2.
Parameters that
determine or are
affected by flood depth**

SOURCE: *COASTAL
CONSTRUCTION MANUAL*
(FEMA 55)

DFE = Design Flood Elevation in feet above datum

d_{fp} = Design flood protection depth in feet

BFE = Base Flood Elevation in feet above datum

Freeboard = Vertical distance in feet between BFE and DFE

H_b = Breaking wave height = $0.78d_s$ (note that 70 percent of wave
height lies above E_{sw})

E_{sw} = Design stillwater flood elevation in feet above datum

d_s = Design stillwater flood depth in feet

G = Ground elevation, existing or pre-flood, in feet above datum

Erosion = Loss of soil during design flood event in feet (not
including effects of localized scour)

GS = Lowest eroded ground elevation adjacent to building in feet
above datum (including the effects of localized scour)

3.2.2 Design Stillwater Flood Depth (d_S)

Design stillwater flood depth (d_s) is the vertical distance between the eroded ground elevation and the stillwater flood elevation associated with the design flood. Determining the maximum design stillwater flood depth over the life of a building is the single most important flood load calculation that will be made; nearly all other coastal flood load parameters or calculations (e.g., hydrostatic load, design flood velocity, hydrodynamic load, design wave height, DFE, debris impact load, local scour depth) depend directly or indirectly on the design stillwater flood depth. The design stillwater flood depth (d_s) is defined as

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

Where

d_s = Design stillwater flood depth (ft)

E_{sw} = Design stillwater flood elevation (ft) above the datum (e.g., National Geodetic Vertical Datum [NGVD], North American Vertical Datum [NAVD]), including wave setup effects

GS = Lowest eroded ground elevation above datum (ft), adjacent to building, including the effects of localized sour around pilings

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 Chapter 7 of FEMA 55.

Values for E_{sw} are not shown on a FIRM, but they are given in the Flood Insurance Study (FIS) report, which is produced in conjunction with the FIRM for a community. FIS reports are usually available from community officials, from NFIP State Coordinating Agencies, and on the web at the FEMA Map Service Center (http://store.msc.fema.gov). Some states have FIS reports available on their individual web sites.

3.2.3 Design Wave Height (H_b)

The design wave height at a coastal building site will be one of the most important design parameters. Therefore, unless detailed analysis shows that natural or manmade obstructions will protect the site during a design event, wave heights at a site will be calculated from Equation 5-2 of ASCE 7-02 as the heights of depth-limited breaking waves (H_b), which are equivalent to 0.78 times the design stillwater flood depth:

$$H_b = 0.78d_s$$

Note: 70 percent of the breaking wave height ($0.7H_b$) lies above the stillwater flood level.

3.2.4 Design Flood Velocity (V)

Estimating design flood velocities in coastal flood hazard areas is subject to considerable uncertainty. Little reliable historical information exists concerning the velocity of floodwaters during coastal flood events. The direction and velocity of floodwaters can vary significantly throughout a coastal flood event, approaching a site from one direction during the beginning of the flood event before shifting to another (or several directions). Floodwaters can inundate some low-lying coastal sites from both the front (e.g., ocean) and the 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 that floodwaters can approach from the most critical direction and that flow velocities can be high.

For design purposes, the *Commentary* of ASCE7-02 suggested a range of flood velocities from:

$$V = d_s \div t \text{ (expected lower bound)}$$

to

$$V = (g d_s)^{0.5} \text{ (expected upper bound)}$$

Where

d_s = Design stillwater flood depth

t = Time (1 second)

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

Factors that should be considered before selecting the upper- or lower-bound flood velocity for design include:

- 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 a V Zone, in an AO Zone adjacent to a V Zone, in an A Zone subject to velocity flow and wave action, steeply sloping, or adjacent to other buildings or obstructions that will confine floodwaters and accelerate flood velocities. The lower bound is a more appropriate design flood velocity if the site is distant from the flood source, in an A Zone, flat or gently sloping, or unaffected by other buildings or obstructions.

3.3 Hydrostatic Loads

Hydrostatic loads occur when standing or slowly moving water comes into contact with a building or building component. These loads can act laterally (pressure) or vertically (buoyancy).

Lateral hydrostatic forces are generally not sufficient to cause deflection or displacement of a building or building component unless there is a substantial difference in water elevation on opposite sides of the building or component; therefore, the NFIP requires that floodwater openings be provided in vertical walls that form an enclosed space below the BFE for a building in an A Zone.

Lateral hydrostatic force is calculated by the following:

$$f_{stat} = \tfrac{1}{2}\, \gamma\, d_s^2$$

Where

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

γ = Specific weight of water (62.4 lb/ft^3 for freshwater and 64 lb/ft^3 for saltwater)

Vertical hydrostatic forces during design flood conditions are not generally a concern for properly constructed and elevated coastal buildings. Buoyant or flotation forces on a building can be of concern if the actual stillwater flood depth exceeds the design stillwater flood depth.

Vertical (buoyancy) hydrostatic force is calculated by the following:

$$F_{Buoy} = \gamma\, (Vol)$$

Where

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

Vol = volume of floodwater displaced by a submerged object (ft^3) = displaced area x depth of flooding

Buoyant force acting on an object must be resisted by the weight of the object and any other opposing force (e.g., anchorage forces) resisting flotation. In the case of a building, the live load on floors should not be counted on to resist buoyant forces.

3.4 Wave Loads

Calculating wave loads requires information about expected wave heights. For the purposes of this manual, the calculations will be limited by water depths at the site of interest. Wave forces can be separated into four categories:

- Non-breaking waves (can usually be computed as hydrostatic forces against walls and hydrodynamic forces against piles)

- Breaking waves (short duration but large magnitude forces against walls and piles)

- Broken waves (similar to hydrodynamic forces caused by flowing or surging water)

- Uplift (often caused by wave runup, deflection, or peaking against the underside of horizontal surfaces)

Of these four categories, the forces from breaking waves are the largest and produce the most severe loads. Therefore, it is strongly recommended that the breaking wave load be used as the design wave load.

Two breaking wave loading conditions are of interest in residential construction: waves breaking on small-diameter vertical elements below the DFE (e.g., piles, columns in the foundation of a building in a V Zone) and waves breaking against vertical walls below the DFE (e.g., solid foundation walls in A Zones, breakaway walls in V Zones).

3.4.1 Breaking Wave Loads on Vertical Piles

The breaking wave load (F_{brkp}) on a pile can be assumed to act at the stillwater flood level and is calculated by Equation 5-4 from ASCE 7-02:

$$F_{brkp} = (1/2)CD\gamma DH_b^2$$

Where

F_{brkp} = Net wave force (lb)

CD = Coefficient of drag for breaking waves = 1.75 for round piles or column, and 2.25 for square piles or columns

γ = Specific weight of water (lb/ft^3)

D   = Pile or column diameter (ft) for circular section. For a square pile or column, 1.4 times the width of the pile or column (ft).

H_b = Breaking wave height (ft)

3.4.2 Breaking Wave Loads on Vertical Walls

The net force resulting from a normally incident breaking wave (depth limited in size, with $H_b = 0.78d_s$) acting on a rigid vertical wall, can be calculated by Equation 5-6 from ASCE 7-02:

$$F_{brkw} = 1.1C_p\gamma d_s^2 + 2.4\gamma d_s^2$$

Where

F_{brkw} = net breaking wave force per unit length of structure (lb/ft) acting near the stillwater flood elevation

C_p = Dynamic pressure coefficient ($1.6 < C_p < 3.5$) (see Table 3-1)

Table 3-1. Building Category and Corresponding Dynamic Pressure Coefficient (C_p)

Building Category	C_p
I – Buildings and other structures that represent a low hazard to human life in the event of a failure	1.6
II – Buildings not in Category I, III, and IV	2.8
III – Buildings and other structures that represent a substantial hazard to human life in the event of a failure	3.2
IV – Buildings and other structures designated as essential facilities	3.5

SOURCE: ASCE 7-02

γ = Specific weight of water (lb/ft^3)

d_s = Design stillwater flood depth (ft) at base of building where the wave breaks

This formula assumes the following:

■ The vertical wall causes a reflected or standing wave against the seaward side of the wall with the crest of the wave, reaching a height of $1.2d_s$ above the design stillwater flood elevation, and

■ The space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall (see Figure 3-3).

If free-standing water exists behind the wall (see Figure 3-4), a portion of the hydrostatic component of the wave pressure and force disappears and the net force can be computed using Equation 5-7 from ASCE 7-02:

$$F_{brkw} = 1.1C_p\gamma d_s^2 + 1.9\gamma d_s^2$$

Post-storm damage inspections show that breaking wave loads have destroyed virtually all wood-frame or 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 between A Zones and V Zones.

Figure 3-3.
Normally incident breaking wave pressures against a vertical wall (space behind vertical wall is dry)
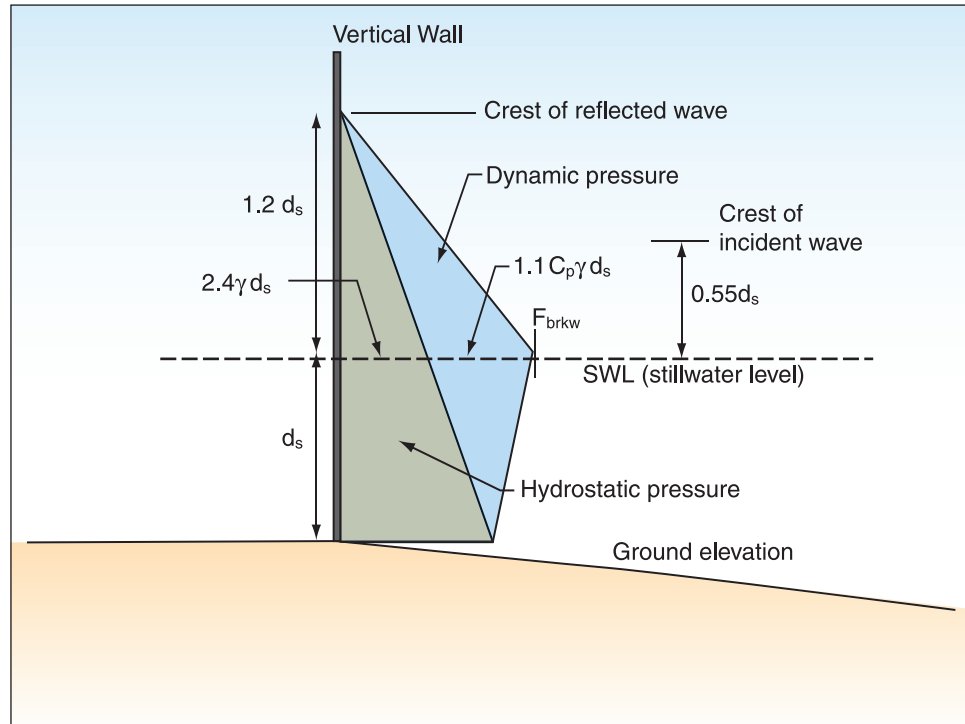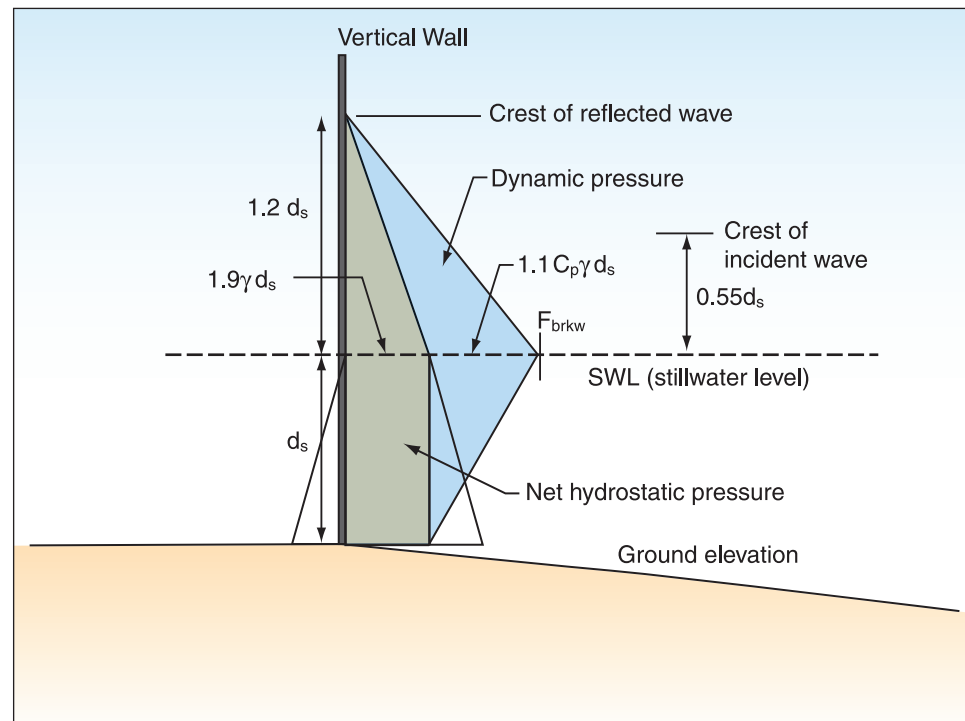
SOURCE: ASCE 7-02

Figure 3-4.
Normally incident breaking wave pressures against a vertical wall (stillwater level equal on both sides of wall)

SOURCE: ASCE 7-02

3.5   Hydrodynamic Loads

Water flowing around a building (or a structural element or other object) imposes additional loads on the building. The 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. This manual assumes that the velocity of the floodwaters is constant (i.e., steady state flow).

One of the most difficult steps in quantifying loads imposed by moving water is determining the expected flood velocity. Refer to Section 3.2.4 for guidance concerning design flood velocities.

The following equation from FEMA 55 can be used to calculate the hydrodynamic load from flows with velocity greater than 10 ft/sec:

$$F_{dyn} = \tfrac{1}{2}C_d \, \rho V^2 A$$

Where

F_{dyn} = Hydrodynamic force (lb) acting at the stillwater mid-depth (halfway 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)

ρ = Mass density of fluid (1.94 slugs/ft^3 for freshwater and 1.99 slugs/ft^3 for saltwater)

V = Velocity of water (ft/sec)

A = Surface area of obstruction normal to flow (ft^2)

Note that the use of this formula will provide the total force against a building of a given impacted surface area (A). Dividing the total force by either length or width would yield a force per unit length; dividing by "A" would yield a force per unit area.

The drag coefficient used in the previously stated equations is a function of the shape of the object around which flow is directed. If the object is something other than a round, square, or rectangular pile, the drag coefficient can be determined using Table 3-2.

Table 3-2. Drag Coefficient Based on Width to Depth Ratio

Width to Depth Ratio (w/d$_s$ or w/h)	Drag Coefficient (C$_d$)
1 to 12	1.25
13 to 20	1.30
21 to 32	1.40
33 to 40	1.50
41 to 80	1.75
81 to 120	1.80
>120	2.00

Note: "h" refers to the height of an object completely immersed in water.

SOURCE: FEMA 55

Flow around a building or building component will also create flow-perpendicular forces (lift forces). If the building component is rigid, lift forces can be assumed to be small. But if the building component is not rigid, lift forces can be greater than drag forces. The formula for lift force is similar to the formula for hydrodynamic force except that the drag coefficient (C_d) is replaced with the lift coefficient (C_l). For the purposes of this manual, the foundations of coastal residential buildings can be considered rigid, and hydrodynamic lift forces can therefore be ignored.

3.6 Debris Impact Loads

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

■ Immediately adjacent to or downstream from another building

■ Downstream from large flotable objects (e.g., exposed or minimally covered storage tanks)

■ Among closely spaced buildings

The following equation to calculate the magnitude of impact load is provided in the *Commentary* of ASCE 7-02:

$$F_i = (\pi WVC_iC_oC_DC_BR_{max}) \div (2g\Delta t)$$

Where

F_i = Impact force acting at the stillwater level (lb)

π = 3.14

W = Weight of debris (lb), suggest using 1,000 if no site-specific information is available

V = Velocity of object (assume equal to velocity of water) (ft/sec)

C_i = Importance coefficient (see Table C5.3 of ASCE 7-02)

C_o = Orientation coefficient = 0.8

C_D = Depth coefficient (see Table C5.5 and Figure C5-3 of ASCE 7-02)

C_B = Blockage coefficient (see Table C5.5 and Figure 5-4 of ASCE 7-02)

R_{max}= Maximum response ratio for impulsive load (see Table C5.6 of ASCE 7-02)

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

Δt = Duration of impact (sec)

When the C coefficients and R_{max} are set to 1.0, the above equation reduces to

$$F_i = (\pi WV) \div (2g\Delta t)$$

This equation is very similar to the equation provided in ASCE 7-98 and FEMA 55. The only difference is the $\pi/2$ term, which results from the half-sine form of the impulse load.

The following uncertainties must be quantified before the impact of debris loading on the building can be determined using the above equation:

■ Size, shape, and weight (W) of the waterborne object

■ Flood velocity (V)

■ Velocity of the object compared to the flood velocity

■ Portion of the building that will be struck and most vulnerable to collapsing

■ Duration of the impact (t)

Once floodborne debris impact loads have been quantified, decisions must be made on how to apply them to the foundation and how to design foundation elements to resist them. For open

foundations, the *Coastal Construction Manual* (FEMA 55) advises applying impact loading to a corner or critical column or piling concurrently with other flood loads (see *Coastal Construction Manual*, Table 11-6). For closed foundations (which are not recommended in Coastal A Zones and are not allowed in V Zones), the *Coastal Construction Manual* advises that the designer assume that one corner of the foundation will be destroyed by debris and recommends the foundation and the structure above be designed to contain redundancy to allow load redistribution to prevent collapse or localized failure. The following should be considered in determining debris impact loads:

Size, shape, and weight of the debris. It is recommended that, in the absence of information about the nature of the potential debris, a weight of 1,000 pounds be used for the debris weight (W). Objects of this weight could include portions of damaged buildings, utility poles, portions of previously embedded piles, and empty storage tanks.

Debris velocity. Flood velocity can be approximated by one of the equations discussed in Section 3.2.4. For the calculation of debris loads, the velocity of the waterborne object is assumed to be the same as the flood velocity. Note that, although this assumption may be accurate for small objects, it will overstate debris velocities for large objects (e.g., trees, logs, pier pilings). The *Commentary* of ASCE 7-02 provides guidance on estimating debris velocities for large debris.

Portion of building to be struck. The object is assumed to be at or near the water surface level when it strikes the building. Therefore, the object is assumed to strike the building at the stillwater flood level.

Duration of impact. Uncertainty about the duration of impact (Δt) (the time from initial impact, through the maximum deflection caused by the impact, to the time the object leaves) is the most likely cause of error in the calculation of debris impact loads. ASCE 7-02 showed that measured impact duration (from initial impact to time of maximum force) from laboratory tests varied from 0.01 to 0.05 second. The ASCE 7-02 recommended value for Δt is 0.03 second.

> **NOTE:** The method for determining debris impact loads in ASCE 7-02 was developed for riverine impact loads and has not been evaluated for coastal debris that may impact a building over several wave cycles. Although these impact loads are very large but of short duration, a structural engineer should be consulted to determine the structural response to the short load duration (0.03 second recommended).

3.7 Localized Scour

Waves and currents during coastal flood conditions are capable of creating turbulence around foundation elements and causing localized scour. Determining potential scour is critical in designing coastal foundations to ensure that failure during and after flooding 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 depth, flow conditions, soil characteristics, and foundation type.

In some locations, soil at or below the ground surface can be resistant to localized scour, and scour depths calculated below will be excessive. In instances where the designer believes the soil at a site will be scour-resistant, a geotechnical engineer should be consulted before calculated scour depths are reduced.

3.7.1   Localized Scour Around Vertical Piles

The methods for calculating localized scour (S_{max}) in coastal areas have been largely based on empirical evidence gathered after storms. Much of the evidence gathered suggests that localized scour depths around piles and other thin vertical members are approximately equal to 1.0 to 1.5 times the pile diameter. Figure 3-5 illustrates localized scour at a pile, with and without a scour-resistant terminating stratum. Currently, there is no design guidance in ASCE 7-02 on how to calculate scour. Localized scour around a vertical pile or similar foundation element should be calculated with the following formula as given in FEMA 55:

$$S_{max} = 2.0a$$

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 (ft)
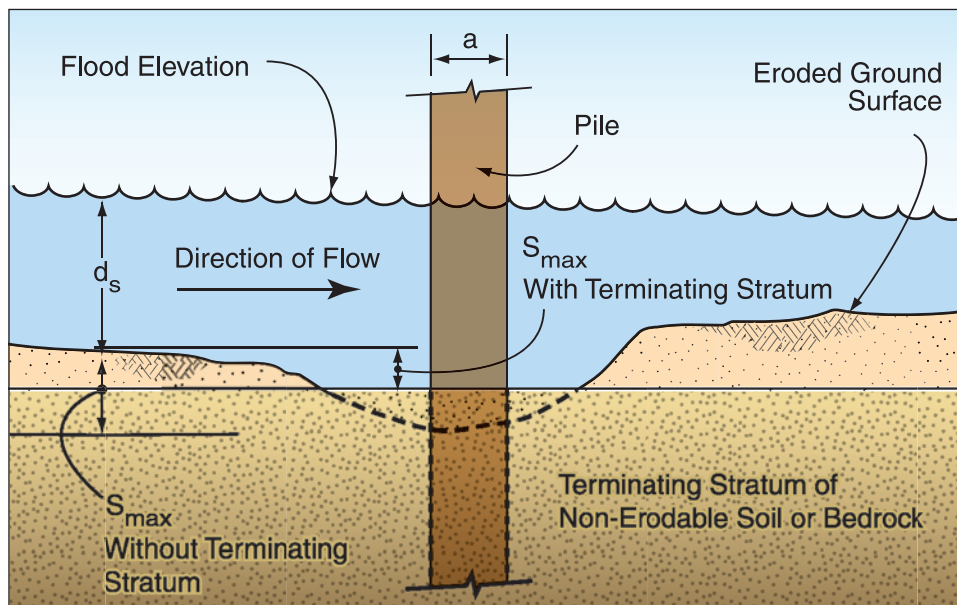

Figure 3-5.
Scour at vertical foundation member stopped by underlying scour-resistant stratum

SOURCE: *COASTAL CONSTRUCTION MANUAL* (FEMA 55)

3.7.2   Localized Scour Around Vertical Walls and Enclosures

Localized scour around vertical walls and enclosed areas (e.g., typical A Zone construction) can be greater than that around vertical piles, and should be estimated using Table 3-3.

Table 3-3. Local Scour Depth as a Function of Soil Type

Soil Type	Expected Depth (% of d_s)
Loose sand	80
Dense sand	50
Soft silt	50
Stiff silt	25
Soft clay	25
Stiff clay	10

SOURCE: FEMA 55

3.8 Flood Load Combinations

Load combinations (including those for flood loads) are given in ASCE 7-02, Sections 2.3.2 and 2.3.3 for strength design and Sections 2.4.1 and 2.4.2 for allowable stress design.

The basic load combinations are:

Allowable Stress Design

(1) D + F

(2) D + H + F + L + T

(3) D + H + F + (L_r or S or R)

(4) D + H + F + 0.75 (L + T) + 0.75 (L_r or S or R)

(5) D + H + F + (W or 0.7E)

(6) D + H + F + 0.75 (W or 0.7E) + 0.75L + 1.5F_a + 0.75 (L_r or S or R)

(7) 0.6D + W + H

(8) 0.6D + 0.7 E + H

Strength Design

(1) 1.4 (D + F)

(2) 1.2 (D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)

(3) 1.2D + 1.6(L_r or S or R) + (L or 0.8W)

(4) 1.2D + 1.6W + L + 0.5(L_r or S or R)

(5) 1.2D + 1.0E + L + 0.2S

(6) $0.9D + 1.6W + 1.6H$

(7) $0.9D + 1.0E + 1.6H$

For structures located in V or Coastal A Zones:

Allowable Stress Design

Load combinations 5, 6, and 7 shall be replaced with the following:

(5) $D + H + F + 1.5F_a + W$

(6) $D + H + F + 0.75W + 0.75L + 1.5F_a + 0.75(L_r$ or S or R$)$

(7) $0.6D + W + H + 1.5F_a$

Strength Design

Load combinations 4 and 6 given in ASCE 7-02 Section 2.3.1 shall be replaced with the following:

(4) $1.2D + 1.6W + 2.0F_a + L + 0.5(L_r$ or S or R$)$

(6) $0.9D + 1.6W + 2.0 F_a + 1.6H$

Where

D   = Dead Load

W   = Wind Load

E   = Earthquake Load

F_a = Flood Load

F   = Load due to fluids with well defined pressures and maximum heights

L   = Live Load

L_r = Roof Live Load

S   = Snow Load

R   = Rain Load

H   = Lateral Earth Pressure

Flood loads were included in the load combinations to account for the strong correlation between flood and winds in hurricane-prone regions that run along the Gulf of Mexico and the Atlantic Coast.

In non-Coastal A Zones, for Allowable Stress Design, replace the $1.5F_a$ with $0.75F_a$ in load combinations 5, 6, and 7 given above. For Strength Design, replace coefficients W and F_a in equations 4 and 6 above with 0.8 and 1.0, respectively.

Designers should be aware that not all of the flood loads will act at certain locations or against certain building types. Table 3-4 provides guidance to designers for the calculation of appropriate flood loads in V Zones and Coastal A Zones (non-Coastal A Zone flood load combinations are shown for comparison).

The floodplain management regulations enacted by communities that participate in the NFIP prohibit the construction of solid perimeter wall foundations in V Zones, but allow such foundations in A Zones. Therefore, the designer should assume that breaking waves will impact piles in V Zones and walls in A Zones. It is generally unrealistic to assume that impact loads will occur on all piles at the same time as breaking wave loads; therefore, this manual recommends that impact loads be evaluated for strategic locations such as a building corner.

Table 3-4. Selection of Flood Load Combinations for Design

Case 1	Pile or Open Foundation in V Zone (Required)
F_{brkp} (on all piles) + F_i (on one corner or critical pile only) or F_{brkp} (on front row of piles only) + F_{dyn} (on all piles but front row) + F_i (on one corner or critical pile only)	
Case 2	**Pile or Open Foundation in Coastal Z Zone (Recommended)**
F_{brkp} (on all piles) + F_i (on one corner or critical pile only) or F_{brkp} (on front row of piles only) + F_{dyn} (on all piles but front row) + F_i (on one corner or critical pile only)	
Case 3	**Solid (Wall) Foundation in Coastal A Zone (NOT Recommended)**
F_{brkp} (on walls facing shoreline, including hydrostatic component) + F_{dyn}; assume one corner is destroyed by debris, and design in redundancy	
Case 4	**Solid (Wall) Foundation in Non-Coastal A Zone (Shown for Comparison)**
F_{sta} + F_{dyn}	

SOURCE: *COASTAL CONSTRUCTION MANUAL* (FEMA 55)