Chapter C5
FLOOD LOADS

C5.1 GENERAL

This section presents information for the design of buildings and other structures in areas prone to flooding. Design professionals should be aware that there are important differences between flood characteristics, flood loads, and flood effects in riverine and coastal areas (e.g., the potential for wave effects is much greater in coastal areas; the depth and duration of flooding can be much greater in riverine areas; the direction of flow in riverine areas tends to be more predictable; and the nature and amount of flood-borne debris varies between riverine and coastal areas).

Much of the impetus for flood-resistant design has come about from the federal government sponsored initiatives of flood-damage mitigation and flood insurance, both through the work of the U.S. Army Corps of Engineers and the National Flood Insurance Program (NFIP). The NFIP is based on an agreement between the federal government and participating communities that have been identified as being flood-prone. The Federal Emergency Management Agency (FEMA), through the Federal Insurance and Mitigation Administration (FIMA), makes flood insurance available to the residents of communities provided that the community adopts and enforces adequate floodplain management regulations that meet the minimum requirements. Included in the NFIP requirements, found under Title 44 of the U.S. Code of Federal Regulations [Ref. C5-1], are minimum building design and construction standards for buildings and other structures located in Special Flood Hazard Areas (SFHA).

Special Flood Hazards Areas are those identified by FEMA as being subject to inundation during the 100-year flood. SFHA are shown on Flood Insurance Rate Maps (FIRM), which are produced for flood-prone communities. SFHA are identified on FIRMs as zones A, A1-30, AE, AR, AO, and AH, and in coastal high hazard areas as V1-30, V, and VE. The SFHA is the area in which communities must enforce NFIP-complaint, flood damage-resistant design and construction practices.

Prior to designing a structure in a flood-prone area, design professionals should contact the local building official to determine if the site in question is located in a SFHA or other flood-prone area that is regulated under the community's floodplain management regulations. If the proposed structure is located within the regulatory floodplain, local building officials can explain the regulatory requirements.

Answers to specific questions on flood-resistant design and construction practices may be directed to the Mitigation Division of each of FEMA's regional offices. FEMA has regional offices that are available to assist design professionals.

C5.2 DEFINITIONS

Three new concepts were added with ASCE 7-98. First, the concept of the design flood was introduced. The design flood will, at a minimum, be equivalent to the flood having a 1 percent chance of being equaled or exceeded in any given year (i.e., the base flood or 100-year flood, which served as the load basis in ASCE 7-95). In some instances, the design flood may exceed the base flood in elevation or spatial extent this will occur where a community has designated a greater flood (lower frequency, higher return period) as the flood to which the community will regulate new construction.

Many communities have elected to regulate to a flood standard higher than the minimum requirements of the NFIP. Those communities may do so in a number of ways. For example, a community may require new construction to be elevated a specific vertical distance above the base flood elevation (this is referred to as "freeboard"); a community may select a lower frequency flood as its regulatory flood; a community may conduct hydrologic and hydraulic studies, upon which flood hazard maps are based, in a manner different from the Flood Insurance Study prepared by the NFIP (e.g., the community may complete flood hazard studies based upon development conditions at build-out, rather than following the NFIP procedure, which uses conditions in existence at the time the studies are completed; the community may include watersheds smaller than 1 mi^2 (2.6 km^2) in size in its analysis, rather than following the NFIP procedure, which neglects watersheds smaller than 1 mi^2).

Use of the design flood concept will ensure that the requirements of this standard are not less restrictive than a community's requirements where that community has elected to exceed minimum NFIP requirements. In instances where a community has adopted the NFIP minimum requirements, the design flood described in this standard will default to the base flood.

Second, this standard also uses the terms, "flood hazard area" and "flood hazard map," to correspond to and show the areas affected by the design flood. Again, in instances where a community has adopted the minimum requirements of the NFIP, the flood hazard area defaults to the NFIP's SFHA and the flood hazard map defaults to the FIRM.

Third, the concept of a Coastal A Zone is used to facilitate application of load combinations contained in Section 2. Coastal A zones lie landward of V zones, or landward of an open coast shoreline where V zones have not been mapped (e.g., the shorelines of the Great Lakes), Coastal A Zones are subject to the effects of waves, high-velocity flows, and erosion, although not to the extent that V Zones are. Like V zones, flood forces in Coastal A Zones will be highly correlated with coastal winds or coastal seismic activity.

Coastal A Zones are not delineated on flood hazard maps prepared by FEMA, but are zones where wave forces and erosion potential should be taken into consideration by designers. The following guidance is offered to designers as help in determining whether or not an A zone in a coastal area can be considered a Coastal A Zone.

In order for a Coastal A Zone to be present, two conditions are required: (1) a stillwater flood depth greater than or equal to 2.0 ft (0.61 m); and (2) breaking wave heights greater than or equal to 1.5 ft (0.46 m). Note that the stillwater depth requirement is necessary, but is not sufficient by itself, to render an area a Coastal

A Zone. Many A Zones will have stillwater flood depths in excess of 2.0 ft (0.61 m), but will not experience breaking wave heights greater than or equal to 1.5 ft (0.46 m), and therefore should not be considered Coastal A Zones. Wave heights at a given site can be determined using procedures outlined in [Ref. C5-2] or similar references.

The 1.5 ft (0.46 m) breaking wave height criterion was developed from post-flood damage inspections, which show that wave damage and erosion often occur in mapped A zones in coastal areas, and from laboratory tests on breakaway walls that show that breaking waves 1.5 ft (0.46 m) in height are capable of causing structural failures in wood-frame walls [Ref. C5-3].

C5.3 DESIGN REQUIREMENTS

Sections 5.3.4 (dealing with A-Zone design and construction) and 5.3.5 (dealing with V-zone design and construction) of ASCE 7-98 were deleted in preparation of the 2002 edition of this standard. These sections summarized basic principles of flood-resistant design and construction (building elevation, anchorage, foundation, below Design Flood Elevation (DFE) enclosures, breakaway walls, etc.). Some of the information contained in these deleted sections was included in Section 5.3, beginning with ASCE 7-02, and the design professional is also referred to ASCE/SEI Standard 24 (*Flood Resistant Design and Construction*) for specific guidance.

C5.3.1 Design Loads. Wind loads and flood loads may act simultaneously at coastlines, particularly during hurricanes and coastal storms. This may also be true during severe storms at the shorelines of large lakes and during riverine flooding of long duration.

C5.3.2 Erosion and Scour. The term “erosion” indicates a lowering of the ground surface in response to a flood event, or in response to the gradual recession of a shoreline. The term “scour” indicates a localized lowering of the ground surface during a flood, due to the interaction of currents and/or waves with a structural element. Erosion and scour can affect the stability of foundations, and can increase the local flood depth and flood loads acting on buildings and other structures. For these reasons, erosion and scour should be considered during load calculations and the design process. Design professionals often increase the depth of foundation embedment to mitigate the effects of erosion and scour, and often site buildings away from receding shorelines (building setbacks).

C5.3.3 Loads on Breakaway Walls. Floodplain management regulations require buildings in coastal high hazard areas to be elevated to or above the design flood elevation by a pile or column foundation. Space below the DFE must be free of obstructions in order to allow the free passage of waves and high velocity waters beneath the building [Ref. C5-4]. Floodplain management regulations typically allow space below the DFE to be enclosed by insect screening, open lattice, or breakaway walls. Local exceptions are made in certain instances for shearwalls, firewalls, elevator shafts, and stairwells. Check with the authority having jurisdiction for specific requirements related to obstructions, enclosures, and breakaway walls.

Where breakaway walls are used, they must meet the prescriptive requirements of NFIP regulations or be certified by a registered professional engineer or architect as having been designed to meet the NFIP performance requirements. The prescriptive requirements call for breakaway wall designs that are intended to collapse at loads not less than 10 psf (0.48 kN/m^2) and not more than 20 psf (0.96 kN/m^3). Inasmuch as wind or earthquake loads often exceed 20 psf (0.96 kN/m^2), breakaway walls may be designed for a higher loads, provided the designer certifies that the walls have been designed to break away before base flood conditions are reached, without damaging the elevated building or its foundation. A recent reference [Ref. C5-5] provides guidance on how to meet the performance requirements for certification.

C5.4.1 Load Basis. Water loads are the loads or pressures on surfaces of buildings and structures caused and induced by the presence of floodwaters. These loads are of two basic types: hydrostatic and hydrodynamic. Impact loads result from objects transported by floodwaters striking against buildings and structures or part thereof. Wave loads can be considered a special type of hydrodynamic load.

C5.4.2 Hydrostatic Loads. Hydrostatic loads are those caused by water either above or below the ground surface, free or confined, which is either stagnant or moves at velocities less than 5 ft/s (1.52 m/s). These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts.

Hydrostatic pressure at any point is equal in all directions and always acts perpendicular to the surface on which it is applied. Hydrostatic loads can be subdivided into vertical downward loads, lateral loads, and vertical upward loads (uplift or buoyancy). Hydrostatic loads acting on inclined, rounded, or irregular surfaces may be resolved into vertical downward or upward loads and lateral loads based on the geometry of the surfaces and the distribution of hydrostatic pressure.

C5.4.3 Hydrodynamic Loads. Hydrodynamic loads are those loads induced by the flow of water moving at moderate to high velocity above the ground level. They are usually lateral loads caused by the impact of the moving mass of water and the drag forces as the water flows around the obstruction. Hydrodynamic loads are computed by recognized engineering methods. In the coastal high-hazard area the loads from high-velocity currents due to storm surge and overtopping are of particular importance. Reference [C5-2] is one source of design information regarding hydrodynamic loadings.

Note that accurate estimates of flow velocities during flood conditions are very difficult to make, both in riverine and coastal flood events. Potential sources of information regarding velocities of floodwaters include local, state, and federal government agencies and consulting engineers specializing in coastal engineering, stream hydrology, or hydraulics.

As interim guidance for coastal areas, [Ref. C5-3] gives a likely range of flood velocities as

$$V = d_s/(1\ \mathrm{s}) \tag{C5-1}$$

to

$$V = (gd_s)^{0.5} \tag{C5-2}$$

where

V = average velocity of water in ft/s (m/s)
d_s = local stillwater depth in ft (m)
g = acceleration due to gravity, 32.2 ft/s/s (9.81 m/s^2)

Selection of the correct value of “a” in Eq. 5-1 will depend upon the shape and roughness of the object exposed to flood flow, as well as the flow condition. As a general rule, the smoother and more streamlined the object, the lower the drag coefficient (shape factor). Drag coefficients for elements common in buildings and structures (round or square piles, columns, and rectangular shapes) will range from approximately 1.0 to 2.0, depending upon flow conditions. However, given the uncertainty surrounding flow conditions at a particular site, ASCE 7-05 recommends a minimum value of 1.25 be used. Fluid mechanics texts should be consulted for more information on when to apply drag coefficients above 1.25.

C5.4.4 Wave Loads. The magnitude of wave forces (lb/ft^2) (kN/m^2) acting against buildings or other structures can be 10 or more times higher than wind forces and other forces under design conditions. Thus, it should be readily apparent that elevating above the wave crest elevation is crucial to the survival of buildings and other structures. Even elevated structures, however, must be designed for large wave forces that can act over a relatively small surface area of the foundation and supporting structure.

Wave load calculation procedures in Section 5.3.3.4 are taken from Refs. [C5-2] and [C5-7]. The analytical procedures described by Eqs. 5-2 through 5-9 should be used to calculate wave heights and wave loads unless more advanced numerical or laboratory procedures permitted by this standard are used.

Wave load calculations using the analytical procedures described in this standard all depend upon the initial computation of the wave height, which is determined using Eqs. 5-2 and 5-3. These equations result from the assumptions that the waves are depth-limited, and that waves propagating into shallow water break when the wave height equals 78 percent of the local stillwater depth and that 70 percent of the wave height lies above the local stillwater level. These assumptions are identical to those used by FEMA in its mapping of coastal flood hazard areas on FIRMs.

Designers should be aware that wave heights at a particular site can be less than depth-limited values in some cases (e.g., when the wind speed, wind duration, or fetch is insufficient to generate waves large enough to be limited in size by water depth, or when nearby objects dissipate wave energy and reduce wave heights). If conditions during the design flood yield wave heights at a site less than depth-limited heights, Eq. 5-2 may overestimate the wave height and Eq. 5-3 may underestimate the stillwater depth. Also, Eqs. 5-4 through 5-7 may overstate wave pressures and forces when wave heights are less than depth-limited heights. More advanced numerical or laboratory procedures permitted by this section may be used in such cases, in lieu of Eqs. 5-2 through 5-7.

It should be pointed out that present NFIP mapping procedures distinguish between A Zones and V Zones by the wave heights expected in each zone. Generally speaking, A Zones are designated where wave heights less than 3 ft (0.91 m) in height are expected. V Zones are designated where wave heights equal to or greater than 3 ft (0.91 m) are expected. Designers should proceed cautiously, however. Large wave forces can be generated in some A Zones, and wave force calculations should not be restricted to V Zones. Present NFIP mapping procedures do not designate V Zones in all areas where wave heights greater than 3 ft (0.91 m) can occur during base flood conditions. Rather than rely exclusively on flood hazard maps, designers should investigate historical flood damages near a site to determine whether or not wave forces can be significant.

C5.4.4.2 Breaking Wave Loads on Vertical Walls. Equations used to calculate breaking wave loads on vertical walls contain a coefficient, C_p. Reference [C5-7] provides recommended values of the coefficient as a function of probability of exceedance. The probabilities given by [Ref. C5-7] are not annual probabilities of exceedance, but probabilities associated with a distribution of breaking wave pressures measured during laboratory wave tank tests. Note that the distribution is independent of water depth. Thus, for any water depth, breaking wave pressures can be expected to follow the distribution described by the probabilities of exceedance in Table 5-2.

This standard assigns values for C_p according to building category, with the most important buildings having the largest values of C_p. Category II buildings are assigned a value of C_p corresponding to a 1 percent probability of exceedance, which is consistent with wave analysis procedures used by FEMA in mapping coastal flood hazard areas and in establishing minimum floor elevations. Category I buildings are assigned a value of C_p corresponding to a 50 percent probability of exceedance, but designers may wish to choose a higher value of C_p. Category III buildings are assigned a value of C_p corresponding to a 0.2 percent probability of exceedance, while Category IV buildings are assigned a value of C_p corresponding to a 0.1 percent probability of exceedance.

Breaking wave loads on vertical walls reach a maximum when the waves are normally incident (direction of wave approach perpendicular to the face of the wall; wave crests are parallel to the face of the wall). As guidance for designers of coastal buildings or other structures on normally dry land (i.e., flooded only during coastal storm or flood events), it can be assumed that the direction of wave approach will be approximately perpendicular to the shoreline. Therefore, the direction of wave approach relative to a vertical wall will depend upon the orientation of the wall relative to the shoreline. Section 5.4.4.4 provides a method for reducing breaking wave loads on vertical walls for waves not normally incident.

C5.4.5 Impact Loads. Impact loads are those that result from logs, ice floes, and other objects striking buildings, structures, or parts thereof. Reference [C5-8] divides impact loads into three categories: (1) normal impact loads, which result from the isolated impacts of normally encountered objects, (2) special impact loads, which result from large objects, such as broken up ice floats and accumulations of debris, either striking or resting against a building, structure, or parts thereof, and (3) extreme impact loads, which result from very large objects, such as boats, barges, or collapsed buildings, striking the building, structure, or component under consideration. Design for extreme impact loads is not practical for most buildings and structures. However, in cases where there is a high probability that a Category III or IV structure (see Table 1-1) will be exposed to extreme impact loads during the design flood, and where the resulting damages will be very severe, consideration of extreme impact loads may be justified. Unlike extreme impact loads, design for special and normal impact loads is practical for most buildings and structures.

The recommended method for calculating normal impact loads has been modified beginning with ASCE 7-02. Previous editions of ASCE 7 used a procedure contained in [Ref. C5-8] (the procedure, which had been unchanged since at least 1972, relied on an impulse-momentum approach with a 1,000 lb (4.5 kN) object striking the structure at the velocity of the floodwater and coming to rest in 1.0 s). Recent work [Refs. C5-6 and C5-9] has been conducted to evaluate this procedure, through a literature review and laboratory tests. The literature review considered riverine and coastal debris, ice floes and impacts, ship berthing and impact forces, and various methods for calculating debris loads (e.g., impulse-momentum, work-energy). The laboratory tests included log sizes ranging from 380 lb (1.7 kN) to 730 lb (3.3 kN) traveling at up to 4 ft/s (1.2 m/s).

References [C5-6 and C5-9] conclude: (1) an impulse-momentum approach is appropriate; (2) the 1,000 lb (4.5 kN) object is reasonable, although geographic and local conditions may affect the debris object size and weight; (3) the 1.0-s impact duration is not supported by the literature or by laboratory tests—a duration of impact of 0.03 s should be used instead; (4) a half-sine curve represents the applied load and resulting displacement well; and (5) setting the debris velocity equivalent to the flood velocity is reasonable for all but the largest objects in shallow water or obstructed conditions.

Given the short-duration, impulsive loads generated by flood-borne debris, a dynamic analysis of the affected building or structure may be appropriate. In some cases (e.g., when the natural period of the building is much greater than 0.03 s), design professionals may wish to treat the impact load as a static load applied to the building or structure (this approach is similar to that used by some following the procedure contained in Section C5.3.3.5 of ASCE 7-98).

In either type of analysis—dynamic or static—Eq. C5-3 provides a rational approach for calculating the magnitude of the impact load.

$$F = \frac{\pi W V_b C_I C_O C_D C_B R_{max}}{2g\Delta g} \qquad \text{(C5-3)}$$

where

$F =$ impact force, in lb (N)
$W =$ debris weight in lb (N)
$V_b =$ velocity of object (assume equal to velocity of water, V) in ft/s (m/s)
$g =$ acceleration due to gravity, $= 32.2$ ft/s^2 (9.81 m/s^2)
$\Delta t =$ impact duration (time to reduce object velocity to zero), in s
$C_I =$ importance coefficient (see Table C5-1)
$C_O =$ orientation coefficient, $= 0.8$
$C_D =$ depth coefficient (see Table C5-2, Fig. C5-1)
$C_B =$ blockage coefficient (see Table C5-3, Fig. C5-2)
$R_{max} =$ maximum response ratio for impulsive load (see Table C5-4)

The form of Eq. C5-3 and the parameters and coefficients are discussed in the following text:

Basic Equation. The equation is similar to the equation used in ASCE 7-98, except for the $\pi/2$ factor (which results from the half-sine form of the applied impulse load) and the coefficients C_I, C_O, C_D, C_B, and R_{max}. With the coefficients set equal to 1.0 the equation reduces to $F = \pi\ WV_b/2g\Delta t$, and calculates the maximum static load from a head-on impact of a debris object. The coefficients have been added to allow design professionals to "calibrate" the resulting force to local flood, debris and building characteristics. The approach is similar to that employed by ASCE 7 in calculating wind, seismic, and other loads. A scientifically based equation is used to match the physics, and the results are modified by coefficients to calculate realistic load magnitudes. However, unlike wind, seismic, and other loads, the body of work associated with flood-borne debris impact loads does not yet account for the probability of impact.

Debris Object Weight. A 1,000 lb object can be considered a reasonable average for flood-borne debris (no change from ASCE 7-98). This represents a reasonable weight for trees, logs, and other large woody debris that is the most common form of damaging debris nationwide. This weight corresponds to a log approximately 30 ft (9.1 m) long and just under 1 ft (0.3 m) in diameter. The 1,000 lb object also represents a reasonable weight for other types of debris ranging from small ice floes, to boulders, to man-made objects.

However, design professionals may wish to consider regional or local conditions before the final debris weight is selected. The following text provides additional guidance. In riverine floodplains, large woody debris (trees and logs) predominates, with weights typically ranging from 1,000 lb (4.5 kN) to 2,000 lb (9.0 kN). In the Pacific Northwest, larger tree and log sizes suggest a typical 4,000 lb (18.0 kN) debris weight. Debris weights in riverine areas subject to floating ice typically range from 1,000 lb (4.5 kN) to 4,000 lb (18.0 kN). In arid or semiarid regions, typical woody debris may be less than 1,000 lb (4.5 kN). In alluvial fan areas, nonwoody debris (stones and boulders) may present a much greater debris hazard. Debris weights in coastal areas generally fall into three classes: in the Pacific Northwest, a 4,000 lb (18.0 kn) debris weight due to large trees and logs can be considered typical; in other coastal areas where piers and large pilings are available locally, debris weights may range from 1,000 lb (4.5 kN) to 2,000 lb (9.0 kN); and in other coastal areas where large logs and pilings are not expected, debris will likely be derived from failed decks, steps and building components, and will likely average less than 500 lb (2.3 kN) in weight.

Debris Velocity. The velocity with which a piece of debris strikes a building or structure will depend upon the nature of the debris and the velocity of the floodwaters. Small pieces of floating debris, which are unlikely to cause damage to buildings or other structures, will typically travel at the velocity of the floodwaters, in both riverine and coastal flood situations. However, large debris, such as trees, logs, pier pilings, and other large debris capable of causing damage, will likely travel at something less than the velocity of the floodwaters. This reduced velocity of large debris objects is due in large part to debris dragging along the bottom and/or being slowed by prior collisions. Large riverine debris traveling along the floodway (the deepest part of the channel that conducts the majority of the flood flow) is most likely to travel at speeds approaching that of the floodwaters. Large riverine debris traveling in the floodplain (the shallower area outside the floodway) is more likely to be traveling at speeds less than that of the floodwaters, for those reasons stated in the preceding text. Large coastal debris is also likely to be traveling at speeds less than that of the floodwaters. Eq. C5-2 should be used with the debris velocity equal to the flow velocity because the equation allows for reductions in debris velocities through application of a depth coefficient, C_D, and an upstream blockage coefficient, C_B.

Duration of Impact. A detailed review of the available literature [Ref. C5-6], supplemented by laboratory testing, concluded the previously suggested 1.0 s duration of impact is much too long and is not realistic. Laboratory tests showed that measured impact durations (from initial impact to time of maximum force Δt) varied from 0.01 s to 0.05 s [Ref. C5-6]. Results for one test, for example, produced a maximum impact load of 8,300 lb (37,000 N) for a log weighing 730 lb (3,250 N), moving at 4 ft/s, and impacting with a duration of 0.016 s. Over all the test conditions, the impact duration averaged about 0.026 s. The recommended value for use in Eq. C5-2 is therefore 0.03 s.

Coefficients C_I, C_O, C_D, and C_B. The coefficients are based in part on the results of laboratory testing and in part on engineering judgment. The values of the coefficients should be considered interim, until more experience is gained with them.

The *importance coefficient, C_I*, is generally used to adjust design loads for the structure category and hazard to human life following ASCE 7-98 convention in Table 1-1. Recommended values given in Table C5-1 are based on a probability distribution of impact loads obtained from laboratory tests in [Ref. C5-9].

The *Orientation Coefficient, C_O*, is used to reduce the load calculated by Eq. C5-3 for impacts that are oblique, not head-on. During laboratory tests [Ref. C5-9] it was observed that, while some debris impacts occurred as direct or head-on impacts that produced maximum impact loads, most impacts occurred as eccentric or oblique impacts with reduced values of the impact force. Based on these measurements, an orientation coefficient of $C_O = 0.8$ has been adopted to reflect the general load reduction observed due to oblique impacts.

The *depth coefficient,* C_D, is used to account for reduced debris velocity in shallow water due to debris dragging along the bottom. Recommended values of this coefficient are based on typical diameters of logs and trees, or on the anticipated diameter of the root mass from drifting trees that are likely to be encountered in a flood hazard zone. Reference [C5-6] suggests that trees with typical root mass diameters will drag the bottom in depths of less than 5 ft, while most logs of concern will drag the bottom in depths of less than 1 ft. The recommended values for the depth coefficient are given in Table C5-2 and Fig. C5-1. No test data are available to fully validate the recommended values of this coefficient. When better data are available, designers should use them in lieu of the values contained in Table C5-2 and Fig. C5-1.

The *blockage coefficient,* C_B, is used to account for the reductions in debris velocities expected due to screening and sheltering provided by trees or other structures within about 10 log-lengths (300 ft) upstream from the building or structure of interest. Reference [C5-6] quotes other studies in which dense trees have been shown to act as a screen to remove debris and shelter downstream structures. The effectiveness of the screening depends primarily on the spacing of the upstream obstructions relative to the design log length of interest. For a 1,000 lb log, having a length of about 30 ft, it is therefore assumed that any blockage narrower than 30 ft would trap some or all of the transported debris. Likewise, typical root mass diameters are on the order of 3 to 5 ft, and it is therefore assumed that blockages of this width would fully trap any trees or long logs. Recommended values for the blockage coefficient are given in Table C5-3 and Fig. C5-2 based on interpolation between these limits. No test data are available to fully validate the recommended values of this coefficient.

The *maximum response ratio,* R_{max}, is used to increase or decrease the computed load, depending on the degree of compliance of the building or building component being struck by debris. Impact loads are impulsive in nature, with the force rapidly increasing from zero to the maximum value in time Δt, then decreasing to zero as debris rebounds from structure. The actual load experienced by the structure or component will depend on the ratio of the impact duration Δt relative to the natural period of the structure or component, T_n. Stiff or rigid buildings and structures with natural periods similar to the impact duration will see an amplification of the impact load. More flexible buildings and structures with natural periods greater than approximately four times the impact duration will see a reduction of the impact load. Likewise, stiff or rigid components will see an amplification of the impact load; more flexible components will see a reduction of the impact load. Successful use of Eq. C5-3, then, depends on estimation of the natural period of the building or component being struck by flood-borne debris. Calculating the natural period can be carried out using established methods that take building mass, stiffness, and configuration into account. One useful reference is Appendix C of ANSI/ACI 349 [Ref. C5-10]. Design professionals are also referred to Chapter 9 of ASCE 7 for additional information.

Natural periods of buildings generally vary from approximately 0.05 s to several seconds (for high-rise, moment frame structures). For flood-borne debris impact loads with a duration of 0.03 s, the critical period (above which loads are reduced) is approximately 0.11 s (see Table C5-4). Buildings and structures with natural periods above approximately 0.11 s will see a reduction in the debris impact load, while those with natural periods below approximately 0.11 s will see an increase.

Recent shake table tests of conventional, one- to two-story wood-frame buildings have shown natural periods of ranging from approximately 0.14 s (7 Hz) to 0.33 s (3 Hz), averaging approximately 0.20 s (5 Hz). Elevating these types of structures for flood-resistant design purposes will act to increase these natural periods. For the purposes of flood-borne debris impact load calculations, a natural period of 0.5 to 1.0 s is recommended for one- to three-story buildings elevated on timber piles. For one- to three-story buildings elevated on masonry columns, a similar range of natural periods is recommended. For one- to three-story buildings elevated on concrete piles or columns, a natural period of 0.2 to 0.5 s is recommended. Finally, design professionals are referred to Section 12.8.2 of this standard where an approximate natural period for one- to 12-story buildings (story height equal to or greater than 10 ft [3 m]), with concrete and steel moment-resisting frames, can be approximated as 0.1 times the number of stories.

Special Impact Loads. Reference [C5-8] states that, absent a detailed analysis, special impact loads can be estimated as a uniform load of 100 lb per ft (1.48 kN/m), acting over a 1 ft (0.31 m) high horizontal strip at the design flood elevation or lower. However, [Ref. C5-6] suggests that this load may be too small for some large accumulations of debris, and suggests an alternative approach involving application of the standard drag force expression

$$F = (1/2)C_D \rho A V^2 \qquad \text{(C5-4)}$$

where

F = drag force due to debris accumulation, in lb (N)
V = flow velocity upstream of debris accumulation, in ft/s (m/s)
A = projected area of the debris accumulation into the flow, approximated by depth of accumulation times width of accumulation perpendicular to flow, in ft^2 (m^2)
ρ = density of water in slugs/ft^3 (kg/m^3)
C_D = drag coefficient = 1

This expression produces loads similar to the 100 lb/ft guidance from [Ref. C5-8] when the debris depth is assumed to be 1 ft and when the velocity of the floodwater is 10 ft/s. Other guidance from Refs. [C5-6] and [C5-9] suggest that the depth of debris accumulation is often much greater than 1 ft, and is only limited by the water depth at the structure. Observations of debris accumulations at bridge piers listed in these references show typical depths of 5 to 10 ft, with horizontal widths spanning between adjacent bridge piers whenever the spacing of the piers is less than the typical log length. If debris accumulation is of concern, the design professional should specify the projected area of the debris accumulation based on local observations and experience, and apply the preceding equation to predict the debris load on buildings or other structures.

REFERENCES

[Ref. C5-1] Federal Emergency Management Agency. (Oct. 1999 Ed.). National Flood Insurance Program, 44 CFR, ch. 1, parts 59 and 60.

[Ref. C5-2] U.S. Army Corps of Engineers. (2002). *Coastal Engineering Manual*, Coastal Hydraulics Laboratory, Waterways Experiment Station.

[Ref. C5-3] Federal Emergency Management Agency. (2000). *Revised Coastal Construction Manual*, FEMA-55. Mitigation Directorate.

[Ref. C5-4] Federal Emergency Management Agency. (1993). "Free-of-Obstruction Requirements for Buildings Located in Coastal High Hazard Areas in Accordance with the National Flood Insurance Program." *Technical Bulletin 5-93*. Mitigation Directorate.

[Ref. C5-5] Federal Emergency Management Agency. (1999). "Design and Construction Guidance for Breakaway Walls Below Elevated Coastal Buildings in Accordance with the National Flood Insurance Program." *Technical Bulletin 9-99*. Mitigation Directorate.

[Ref. C5-6] Kriebel, D.L., Buss, L., and Rogers, S. (2000). "Impact Loads from Flood-Borne Debris." Report to the American Society of Civil Engineers.

[Ref. C5-7] Walton, T.L., Jr., Ahrens, J.P., Truitt, C.L., and Dean, R.G. (1989). "Criteria for Evaluating Coastal Flood Protection Structures," *Technical Rep. CERC 89-15*, U.S. Army Corps of Engineers, Waterways Experiment Station.

[Ref. C5-8] U.S. Army Corps of Engineers. (Dec. 1995). Office of the Chief of Engineers, Flood Proofing Regulations, *EP 1165-2-314*.

[Ref. C5-9] Haehnel, R., and Daly, S. (2001). "Debris Impact Tests." Report prepared for the American Society of Civil Engineers by the U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH.

[Ref. C5-10] American Concrete Institute. (1985). Code Requirements for Nuclear Safety Related Concrete Structures, *ANSI/ACI 349*.

[Ref. C5-11] Clough, R.W., and Penzien, J. (1993). *Dynamics of Structures*, 2nd Ed., McGraw-Hill, New York.

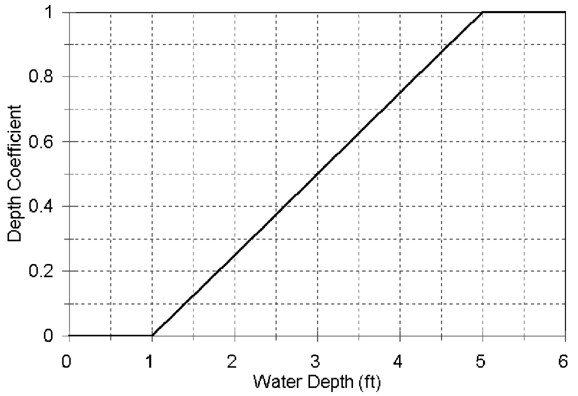

FIGURE C5-1 DEPTH COEFFICIENT, C_D

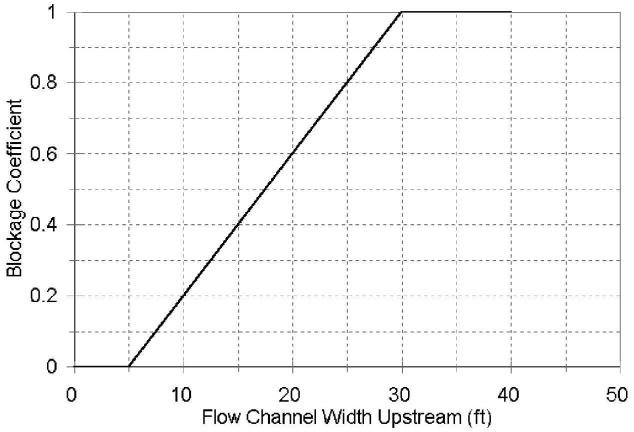

FIGURE C5-2 BLOCKAGE COEFfiCIENT, C_B

TABLE C5-1 VALUES OF IMPORTANCE COEFFICIENT, C_I

Building Category	C_I
I	0.6
II	1.0
III	1.2
IV	1.3

TABLE C5-2 VALUES OF DEPTH COEFFICIENT, C_D

Building Location in Flood Hazard Zone and Water Depth	C_D
Floodway or V-Zone	1.0
A-Zone, Stillwater Depth > 5 ft	1.0
A-Zone, Stillwater Depth = 4 ft	0.75
A-Zone, Stillwater Depth = 3 ft	0.5
A-Zone, Stillwater Depth = 2 ft	0.25
Any flood zone, Stillwater Depth < 1 ft	0.0

TABLE C5-3 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

TABLE C5-4 VALUES OF RESPONSE RATIO FOR IMPULSIVE LOADS, R_{MAX}, ADAPTED FROM [REF. C5-11]

Ratio of Impact Duration to Natural Period of Structure	R_{max} (Response Ratio for Half Sine Wave Impulsive Load)
0.00	0.0
0.10	0.4
0.20	0.8
0.30	1.1
0.40	1.4
0.50	1.5
0.60	1.7
0.70	1.8
0.80	1.8
0.90	1.8
1.00	1.7
1.10	1.7
1.20	1.6
1.30	1.6
≥1.40	1.5