# Chapter 11: Determining Site-Specific Loads

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# Determining Site-Specific Loads

## 11.1 Introduction

Chapters 4 and 5 of this manual describe how a design professional would begin to assess the risk of a particular hazard event occurring at a given location. Regulatory requirements that affect coastal construction are discussed in Chapter 6. Chapter 7 presents information about how to identify site-specific hazards and discusses methodologies for delineating hazard zones. Chapter 8 provides guidance on how the siting of a building on a particular lot or parcel influences the magnitude of the hazard effects on the building.

**NOTE**

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

This chapter provides the design professional and others with guidance on how to determine—by calculation or graphical interpretation—the magnitude of the loads placed on a building by a particular natural hazard event or a combination of events. The calculation methods presented in this chapter are intended to serve as the basis of a methodology for applying the calculated loads to the building during the design process. This methodology will be presented in Chapter 12, *Designing the Building*.

The flowchart in Figure 11-1 shows that 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. Be aware, however, that model building codes and other building standards may not provide load determination and design guidance for each of the hazards identified. In such instances, supplemental guidance should be sought.

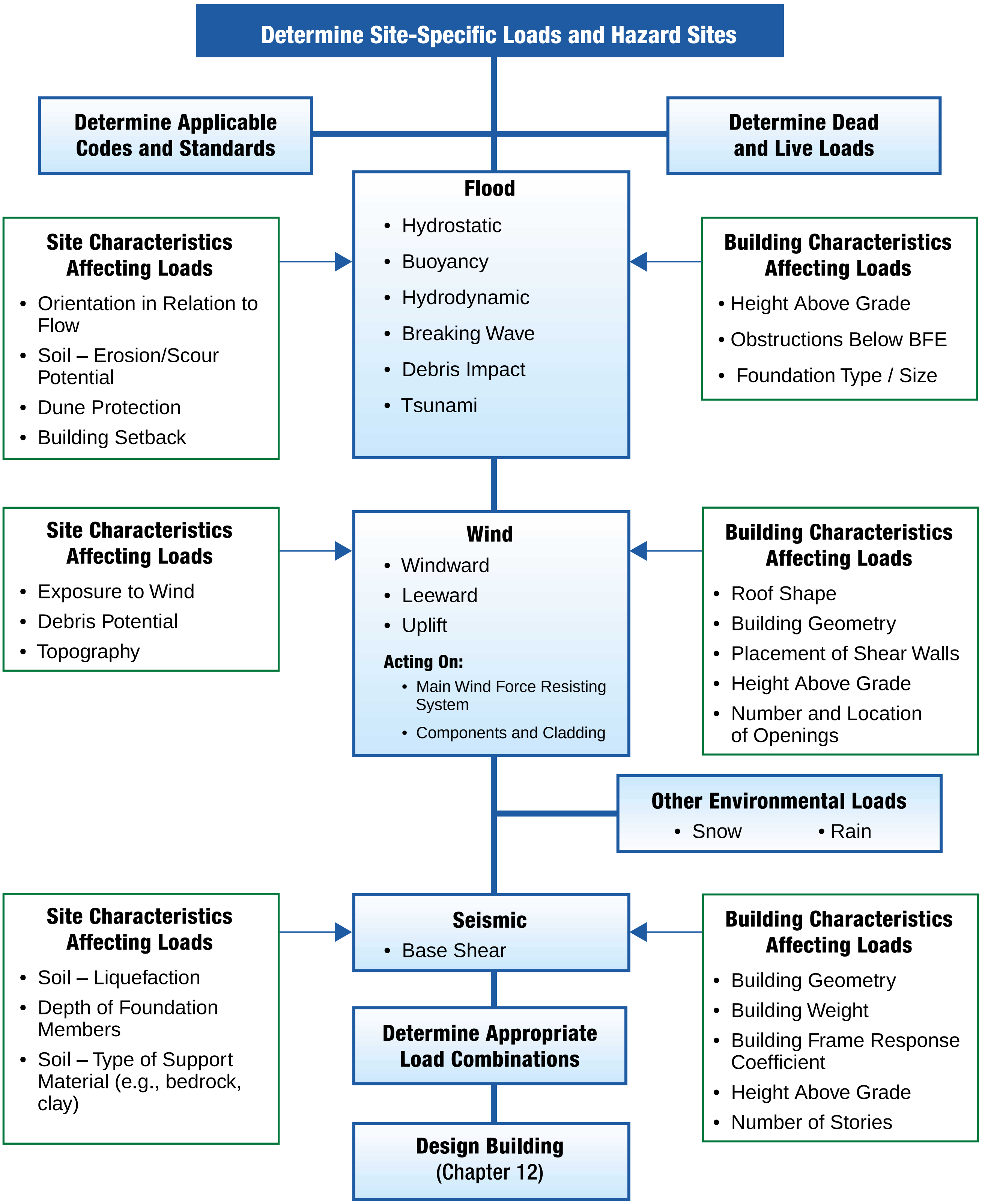
**NOTE**

Throughout this manual, the recommendations of the engineering standards ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures* (ASCE 1998b) will be followed unless otherwise noted. ASCE 7-98 includes procedures for calculating dead and live loads; loads due to soil pressure, fluids, wind, snow, atmospheric ice, and earthquake; and load combinations.

The procedure continues with the **calculation of the loads** imposed by each of the identified hazards. The final step is to determine the **load combinations** appropriate for the building site. It is possible that some loads will be highly correlated and can be assumed to occur simultaneously (such as during a hurricane, when high winds and flooding are closely related). Other loads, however, are weakly correlated, and their simultaneous occurrence is unlikely (e.g., seismic and flood loads).

The load combinations used in this manual are those recommended in ASCE 7-98 (ASCE 1998b). All of the calculations, analyses, and load combinations presented in this manual are based on **Allowable Stress Design** (ASD). The use of factored loads and Strength Design methods will require the designer to modify the approaches presented in this manual to accommodate ultimate strength concepts.

**Figure 11-1** Load determination flowchart.



## 11.2 Dead Loads

The first step in determining the loads placed on a building is to determine the weight of the building and its appurtenances (i.e., dead load). The definition of dead load in ASCE 7-98 is "…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." The sum of the dead loads of all the individual components will equal the unoccupied weight of the building.

The total weight of a building is usually determined by multiplying the unit weight of the various building materials—expressed in pounds (lb) per unit area—by the surface area of those materials. This approach requires that the designer develop a complete list of all of the materials and determine their representative unit weights. Minimum design dead loads are included in ASCE 7-98, *Commentary*. Additional information about material weights can be found in *Architectural Graphic Standards* (Ramsey and Sleeper 1996) and numerous other texts. A simpler, alternative technique is to determine the surface area of building elements such as exterior walls, floors, and roofs and then develop an average unit weight for each. The total weight is equal to the unit weight of the element multiplied by the area of the element.

Determining dead loads is important for several reasons:

- Foundation size (e.g., footing width, pile embedment depth, number of piles) depends partly on dead load.
- Dead load counterbalances uplift forces due to buoyancy and wind.
- Dead load counterbalances wind and earthquake overturning moments.
- Dead load changes the response of the building to both seismic forces and impact forces generated by floating objects.

## 11.3 Live Loads

ASCE 7-98 defines live loads as "… those 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." The flood, wind, and earthquake loads referred to here are the natural hazard loads discussed in detail later in this chapter. For residential one- and two-family buildings, the uniformly distributed live load for habitable areas (except sleeping and attic areas) recommended by ASCE 7-98 is 40 lb/ft$^2$. For balconies and decks on one- and two-family buildings not exceeding 100 ft$^2$, the recommended uniformly distributed live load is 60 lb/ft$^2$. ASCE 7-98 contains no requirements for supporting a concentrated load in a residential building.

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

All loads (e.g., dead, live, snow, flood, wind, seismic) affect a building by acting on some area of the building and being transferred to the structural member(s) that supports that area. Loads are usually applied to a "tributary" area, or the smallest area of the building supported by a structural member. Seismic loads, however, are usually distributed through larger building areas such as an entire roof or floor area. For example, when taken as part of the structural frame, a roof truss spaced 24 inches on center (o.c.) and spanning 30 feet has a tributary area of 2 feet x 30 feet or 60 $ft^2$. In this example, one half of the applied load is carried on each supporting wall. Figure 11-2 illustrates tributary areas for roof loads, lateral wall loads, and column or pile loads. The concept of loads being carried by a tributary area is important to the concept of "continuous load path," which will be fully developed in Chapter 12.

ASCE 7-98 uses effective wind area to define the area of a building component or cladding element that will be affected by wind. Component and cladding elements include items such fasteners, panels, studs, trusses, and window and door mullions. The effective wind area may be the same as the tributary area defined above or, for areas supported by long, slender members (e.g., studs, trusses), may be taken to be at least one third the length of the area (span of the member). Thus, effective area is used only in the determination of the gust coefficient $GC_p$.

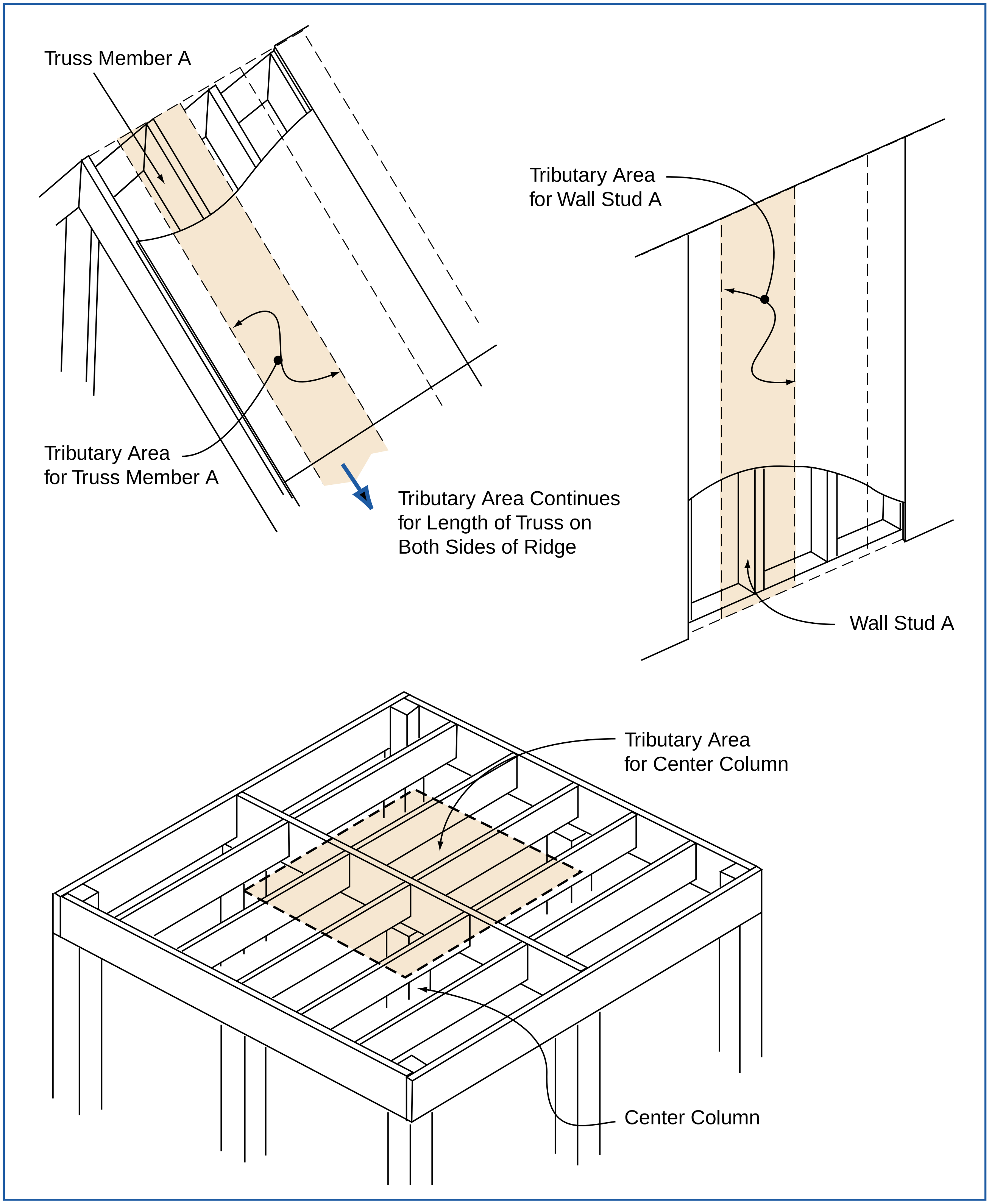
**CROSS-REFERENCE**

See ASCE 7-98 and Section 11.8.2 of this manual for additional information regarding effective wind area.

## 11.5 Snow Loads

Snow loads are applied as a vertical load on the roof or other flat, exposed surfaces such as porches or decks. Recommended ground snow loads are normally specified by the local building code or building official; however, ASCE 7-98 (ASCE 1998b) includes a map of the United States with recommended snow loads that can be used in the absence of local snow load information. The weight of snow is added to the building weight when the seismic force is determined. Chapter 16 of the *International Building Code* 2000, hereafter referred to as the IBC 2000 (ICC 2000a) contains information about how to apply snow loads for this purpose.

**Figure 11-2** Examples of tributary areas for different structural members.



## 11.6 Flood Loads

Flood waters can create a variety of loads on building components. Both hydrostatic and breaking wave loads depend explicitly on flood depth. Coastal engineers also assume, as a first approximation, that hydrodynamic loads will be a function of flood depth. This assumption results from the fact that many coastal flood currents are generated by waves—the assumption will not hold in riverine floods. Flood loads include the following:

- hydrostatic, including buoyancy or flotation effects (from standing water, slowly moving water, and non-breaking waves)
- breaking wave
- hydrodynamic (from rapidly moving water, including broken waves and tsunami runup)
- debris impact (from waterborne objects)

**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 members and cause the loss of load-bearing capacity and the loss of resistance to lateral and uplift loads.**

NOTE

1. Flood load calculation procedures cited in this manual are **very conservative**, given the uncertain conditions that will exist during a severe coastal event.

2. Background information and calculation procedures for determining coastal flood loads are presented in a number of publications, including ASCE 7-98 (ASCE 1998b), ASCE 24-98 (ASCE 1998a), and the U. S. Army Corps of Engineers (USACE) *Coastal Engineering Manual* (scheduled for release in 2000 and available at the USACE website at http://bigfoot.wes.army.mil/cem001.html).

### 11.6.1 Design Flood

For the purposes of this manual, the term 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 flood that has a 1-percent probability of being equaled or exceeded in any given year). If a community chooses to exceed minimum NFIP requirements, the design flood can exceed the base flood. **The design flood will always be greater than or equal to the base flood.**

### 11.6.2 Design Flood Elevation (DFE)

Many communities have chosen to exceed minimum NFIP building elevation requirements, usually by requiring **freeboard** above the BFE (see Figure 11-3), but sometimes by regulating to a more severe flood than the base flood. This manual uses the term design flood elevation (DFE) to refer to the locally adopted regulatory flood elevation. If a community regulates to minimum NFIP requirements, the design flood elevation is identical to the **base flood elevation** (BFE). If a community chooses to exceed minimum NFIP elevation requirements, the design flood elevation will exceed the base flood elevation. **The DFE will always be greater than or equal to the BFE.**

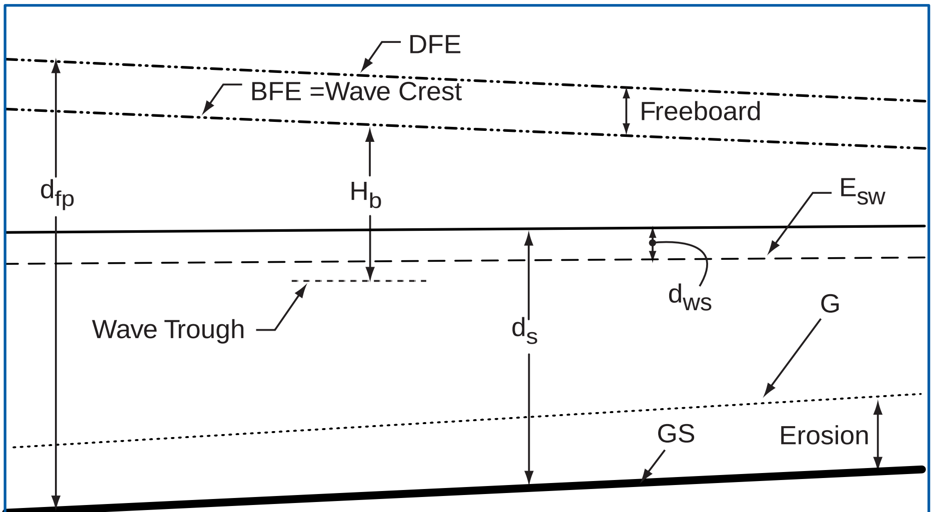
DEFINITION

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

### 11.6.3 Design Flood Depth ($d_s$)

In the general sense, flood depth can refer to two depths (see Figure 11-3):

1. The vertical distance between the eroded ground elevation and the stillwater elevation associated with the design flood—this depth will be referred to as the design stillwater flood depth, $d_s$.
2. The vertical distance between the eroded ground elevation and the DFE—this depth will be referred to as the design flood protection depth, $d_{fp}$, but will not be used extensively by this manual. This manual will emphasize use of the DFE as the minimum elevation to which flood-resistant design and construction efforts should be directed.




**DFE** = Design Flood Elevation in feet above datum

$\mathbf{d_{fp}}$ = design flood protection depth in feet

**BFE** = Base Flood Elevation in feet above datum

**Freeboard** = vertical distance in feet between BFE and DFE

$\mathbf{H_b}$ = breaking wave height = $0.78d_S$ (note that 70 percent of wave height lies above $E_{SW}$)

$\mathbf{E_{SW}}$ = design stillwater flood elevation in feet above datum

$\mathbf{d_{WS}}$ = wave setup in feet

$\mathbf{d_S}$ = 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)

**NOTE**

The design stillwater flood depth ($d_s$) (including wave setup—see Section 11.6.4) should be used for calculating wave heights and flood loads. The design flood protection depth ($d_{fp}$) should be used for building elevation purposes, but **not** for calculating wave heights or flood loads.

**Figure 11-3**
Parameters that determine or are affected by flood depth.

**DEFINITION**

**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 –2.5 feet to the 100-year stillwater flood elevation.

**CROSS-REFERENCE**

See Section 11.6.11 for a discussion of localized scour.

**Determining the maximum design stillwater depth over the life of a building is the single most important flood load calculation that will be made**—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.

For the purposes of this manual, the design stillwater flood depth ($d_s$) is defined as the difference between the total stillwater flood elevation ($E_{sw}$ + wave setup, if not already included in the 100-year stillwater elevation) and the lowest eroded ground surface elevation (GS) adjacent to the building (see Formula 11.1).

formula

Design Stillwater Flood Depth

**Formula 11.1 Design Stillwater Flood Depth**

$$d_s = E_{sw} + d_{ws} - GS$$

where: $d_s$ = design stillwater flood depth (feet)

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

$d_{ws}$ = wave setup in feet

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

**NOTE**

Flood loads are applied to structures as follows:

**Hydostatic Loads:** at 2/3 depth point

**Breaking Wave Loads:** at stillwater level

**Hydrodynamic Loads:** at mid-depth point

**Debris Impact Loads:** at stillwater level

Figure 11-3 illustrates the relationships among the various flood parameters that determine or are affected by flood depth. Note that in Figure 11-3 and in Formula 11.1, 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 detail in Chapter 7.

Values for $E_{sw}$ are not shown on a FEMA Flood Insurance Rate Map (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 and from NFIP State Coordinating Agencies (see Appendix D). Some states have placed FIS reports on their World Wide Web sites.

### 11.6.4 Wave Setup ($d_{ws}$) Contribution to Flood Depth

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

1. Check the *Hydrologic Analyses* section of the FIS for mention of wave setup. Note the magnitude of the wave setup.
2. 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, etc. Do not add wave setup to the 100-year stillwater elevation when calculating Primary Frontal Dune erosion.

**NOTE**

Wave setup effects decrease as one moves inland from the shoreline. Consult *Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping* (FEMA 1995b) and a qualified coastal professional if you have questions about how to account for wave setup in flood depth, wave height, and flood load calculations.

### 11.6.5 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 as the heights of **depth-limited breaking waves**, which are equivalent to 0.78 times the design stillwater flood depth (see Figure 11-3). Note that 70 percent of the breaking wave height lies above the stillwater flood level.

### 11.6.6 Design Flood Velocity (V)

**The estimation of design flood velocities 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 flood waters can vary significantly throughout a coastal flood event. Flood waters can approach a site from one direction during the beginning of the flood event, then shift to another direction (or several directions) during the remainder of the flood event. Flood waters 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 flood waters can approach from the most critical direction and by assuming flow velocities can be high (see Formula 11.2).

Formula

Design Flood Velocity

**Formula 11.2 Design Flood Velocity**

**Lower Bound:** $V = d_s / t$

**Upper Bound:** $V = (gd_s)^{0.5}$

**Extreme (tsunami):** $V = 2(gd_s)^{0.5}$

where: **V** = design flood velocity in ft/sec

$\mathbf{d_s}$ = design stillwater flood depth in feet

**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), where g is the gravitational constant (32.2 ft/sec), $d_s$ is the design stillwater flood depth, and t = time = 1 sec. It is recommended that designers consider the following factors before selecting 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

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 flood waters and accelerate flood velocities, the upper bound should be taken as the 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, the lower bound is a more appropriate design flood velocity.

**CROSS-REFERENCE**

For information about tsunami forces, see Section 11.7 of this chapter.

In some extreme circumstances (e.g., near the shoreline in a tsunami inundation zone) flood velocities should be estimated as high as $V = 2(gd_s)^{0.5}$.

Figure 11-4 shows the velocity/design stillwater depth relationship for the upper- and lower-bound velocities. Formula 11.2 shows the equations for the lower-bound, upper-bound, and extreme velocity conditions.

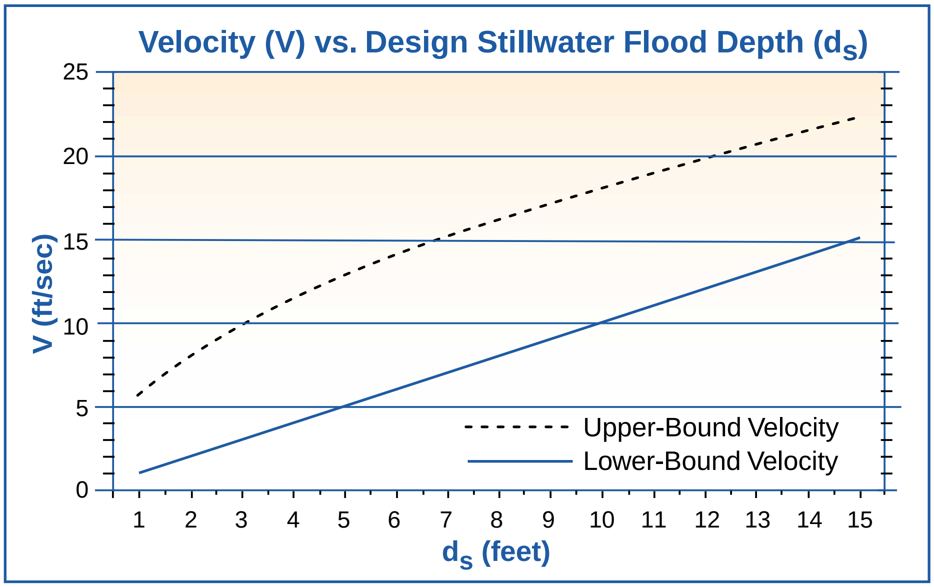



**Figure 11-4**
Velocity vs. design stillwater flood depth in areas not subject to tsunamis. (For a comparison of non-tsunami and tsunami velocities vs. design stillwater depth, see Figure 11-16.)

## 11.6.7 Hydrostatic Loads

Hydrostatic loads occur when standing or slowly moving water comes into contact with a building or building component. Hydrostatic loads can act laterally or vertically, and the forces they exert include buoyant or flotation forces.

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 —hence, the National Flood Insurance Program (NFIP) requirement that flood water openings be provided in vertical walls that form an enclosed space below the BFE in an A zone building (see Chapter 6, Section 6.4.3.2).

Likewise, vertical hydrostatic forces are not generally a concern for properly constructed and elevated coastal buildings during design flood conditions. 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 aboveground and belowground tanks and for swimming pools.

Lateral hydrostatic forces are given by Formula 11.3 and are illustrated in Figure 11-5. Vertical hydrostatic forces are given by Formula 11.4 and are illustrated by Figure 11-6. Note that $F_{sta}$ (in Formula 11.3) 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 11-5).

Formula

Lateral Hydrostatic Load

**Formula 11.3 Lateral Hydrostatic Load**

$$f_{sta} = (^1/_2)\gamma d_s^2$$

where:

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

$\gamma$ = specific weight of water (62.4 lb/ft$^3$ for fresh water and 64.0 lb/ft$^3$ for salt water)

$d_s$ = design stillwater flood depth in feet

and

$$F_{sta} = f_{sta}(w)$$

where: $w$ = width of vertical element in feet

$F_{sta}$ = total equivalent lateral hydrostatic force on structure in lb

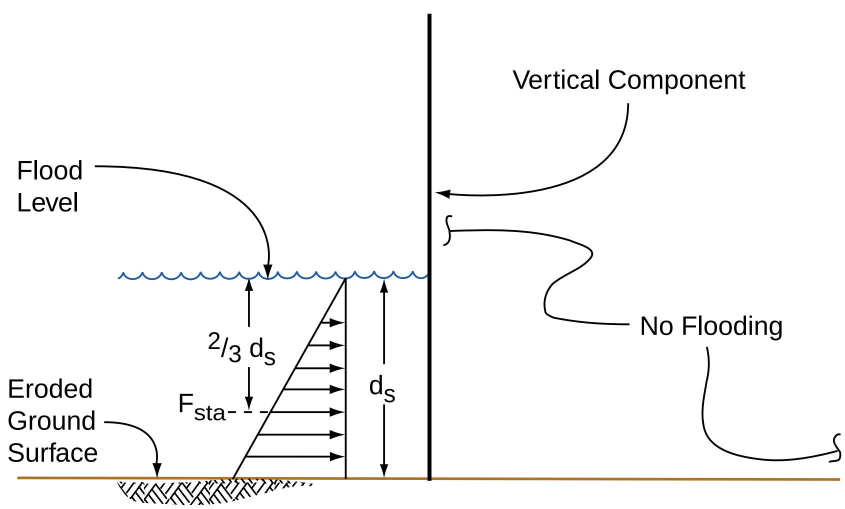



**Figure 11-5**
Lateral flood force on a vertical component.

Figure 11-5 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 V zones be elevated above the BFE on an open foundation and that the foundation walls of buildings in A zones be equipped with openings that allow flood water to enter so that internal and external hydrostatic pressures will equalize (see Chapter 6, Section 6.4.3.2).

**Formula 11.4 Vertical (Buoyant) Hydrostatic Force**

$$F_{buoy} = \gamma(Vol)$$

where: $F_{buoy}$ = vertical hydrostatic force in lb resulting from the displacement of a given volume of flood water

$\gamma$ = specific weight of water (62.4 lb/ft$^3$ for fresh water and 64.0 lb/ft$^3$ for salt water)

**Vol** = volume of flood water displaced by a submerged object in ft$^3$

Formula

Vertical (Buoyant) Hydrostatic Force

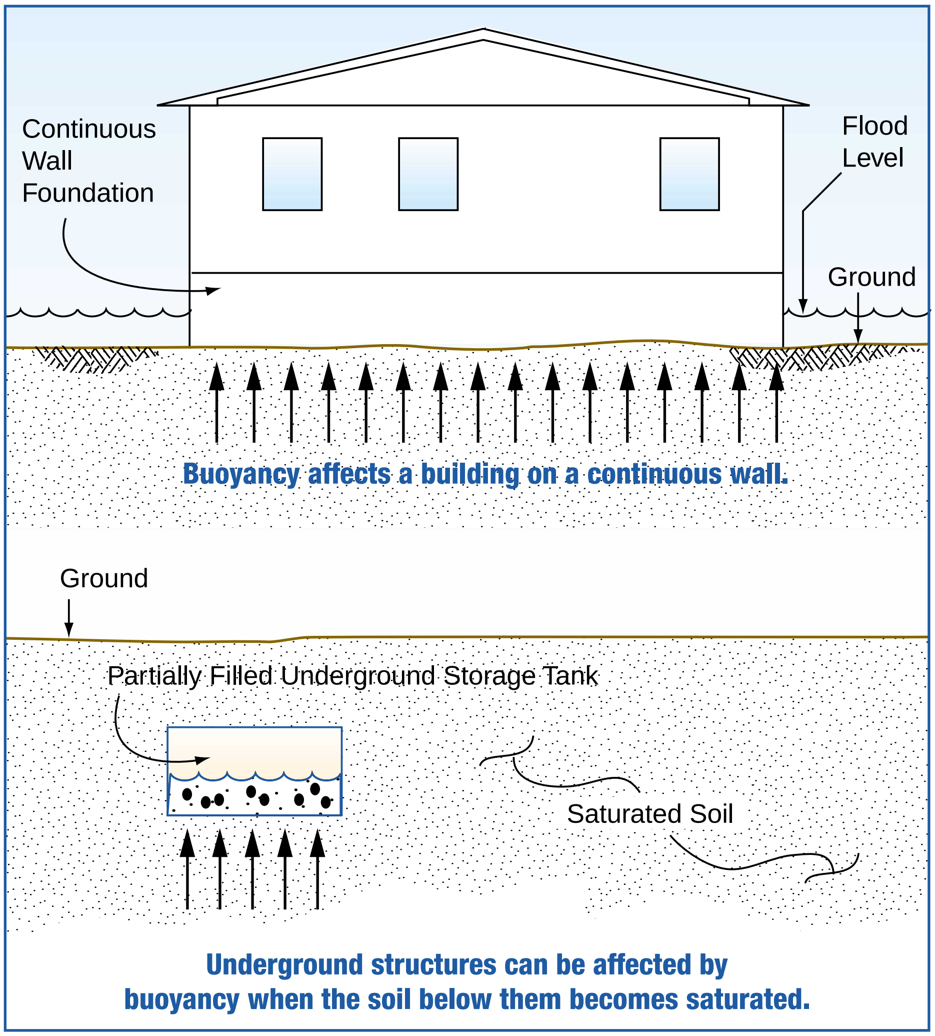



**Figure 11-6**
Vertical (buoyant) flood force.

**NOTE**

Additional guidance for calculating wave loads is presented in ASCE 7-98 (ASCE 1998b).

Moving water exerts hydrodynamic loads (see Section 11.6.9), but where flow velocities do not exceed 10 ft/sec, the hydrodynamic loads can be converted to an equivalent hydrostatic force (see Formula 11.7 in Section 11.6.9).

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 since the tank may be empty or the house may be vacant when the flood occurs. Empty or partially empty tanks or pools are particularly vulnerable.

### 11.6.8 Wave Loads

Calculation of wave loads requires information about expected wave heights, which, for the purposes of this manual, will be limited by water depths at the site of interest. These data can be estimated with 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.

Wave forces can be separated into four categories:

- those from non-breaking waves (these forces can usually be computed as hydrostatic forces against walls and hydrodynamic forces against piles)
- those from breaking waves (these forces will be of short duration, but large magnitude)
- those from broken waves (these forces are similar to hydrodynamic forces caused by flowing or surging water)
- uplift (these forces are often caused by wave runup, deflection, or peaking against the underside of horizontal surfaces)

Of these, the forces from breaking waves are the highest and produce the most severe loads. **Therefore, this manual strongly recommends 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 walls below the DFE (e.g., solid foundation walls in A zones, breakaway walls in V zones). For information and comparative purposes, both loading conditions will be discussed.

### 11.6.8.1 Breaking Wave Loads on Vertical Piles

The breaking wave load on a pile can be assumed to act at the stillwater level and is calculated with Formula 11.5.

As noted previously, the wave loads produced by breaking waves are greater than those produced by non-breaking or broken waves. The following example shows the difference between the loads imposed on a vertical pile by non-breaking waves and breaking waves.

**Formula 11.5 Breaking Wave Load on Vertical Piles**

$$F_{brkp} = (1/2)\, C_{db} \gamma D H_b^2$$

where: $F_{brkp}$ = drag force in lb acting at the stillwater level

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

$\gamma$ = specific weight of water (62.4 lb/ft$^3$ for fresh water and 64.0 lb/ft$^3$ for salt water)

$D$ = pile diameter in feet

$H_b$ = breaking wave height in feet ($0.78d_s$)

where: $d_s$ = design stillwater flood depth in feet

Formula

Breaking Wave Load on Vertical Piles

### 11.6.8.2 Breaking Wave Loads on Vertical Walls

Breaking wave loads on vertical walls are best calculated according to the procedure outlined in *Criteria for Evaluating Coastal Flood-Protection Structures* (Walton, et. al 1989). This procedure is suitable for use in wave conditions typical during coastal flood and storm events. The relationship developed for breaking wave load per unit length of wall is shown in Formula 11.6.

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 $1.2d_s$ above the stillwater elevation. The resulting dynamic, static, and total pressure distributions against the wall, and the resulting loads, are as shown in Figure 11-7.

Formula

Breaking Wave Load on Vertical Walls

## Formula 11.6 Breaking Wave Load on Vertical Walls

**Case 1 (enclosed dry space behind wall):**

$$f_{brkw} = 1.1C_p \gamma d_s^2 + 2.41 \gamma d_s^2$$

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

$$f_{brkw} = 1.1C_p \gamma d_s^2 + 1.91 \gamma d_s^2$$

where:

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

$F_{brkw}$ = total breaking wave load (lb) acting at the stillwater level = $f_{brkw}$ w, where **w** = width of wall in feet

$C_p$ = dynamic pressure coefficient from Table 11.1

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

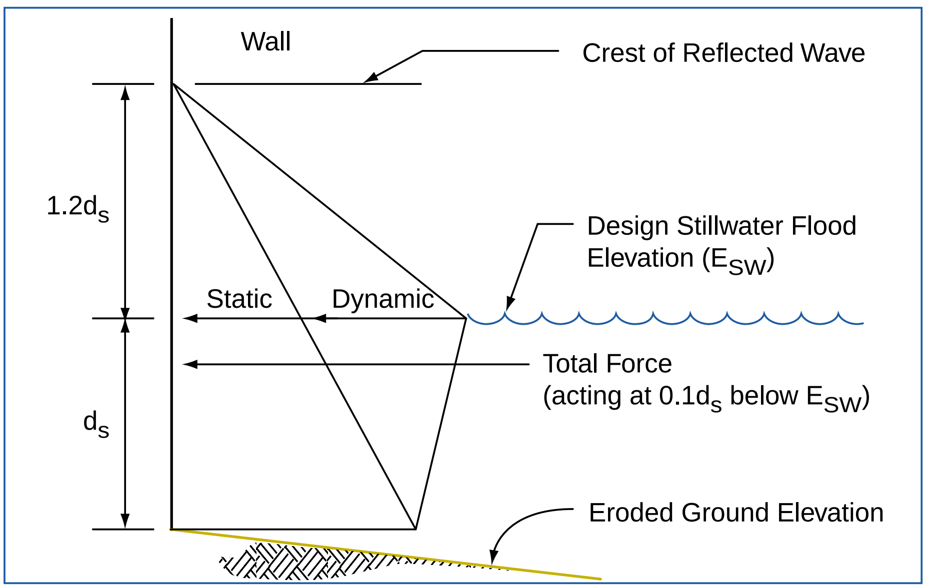
$d_s$ = design stillwater flood depth in feet

**Note:** Formula 11.6 includes the hydrostatic component calculated by Formula 11.3
If Formula 11.6 is used, **do not** add a lateral hydrostatic force from Formula 11.3.

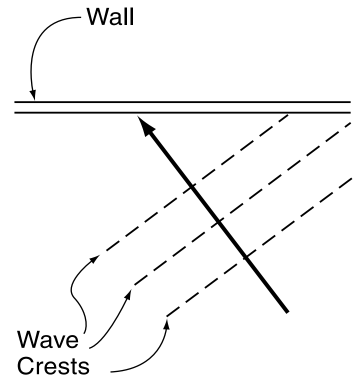
**Table 11.1**
Value of Dynamic Pressure Coefficient, Cp, as a Function of Probability of Exceedance (from Walton, et al. 1989)

| $C_p$ | Building Type | Probability of Exceedance |
|---|---|---|
| 1.6 | accessory structure, low hazard to human life or property in the event of failure | 0.5 |
| 2.8 | coastal residential building | 0.01 |
| 3.2 | high-occupancy building or critical facility | 0.001 |

**Figure 11-7**
Dynamic, static, and total pressure distributions against a vertical wall.



This procedure allows two cases to be considered: (1) where a wave breaks against a vertical wall of an enclosed dry space and (2) where the stillwater level on both sides of the wall is equal. Case 1 is equivalent to a situation where a wave breaks against an enclosure in which there is no floodwater below the stillwater level; Case 2 is equivalent to a situation in which a wave breaks against a breakaway wall or a wall equipped with openings that allow flood waters to equalize on both sides of the wall. In both cases, waves are normally incident (i.e., wave crests parallel to the wall). If breaking waves are obliquely incident (i.e., wave crests **not** parallel to the wall—see illustration at right), the calculated loads would be lower.




**Wave Crests Not Parallel to Wall**

Figure 11-8 shows, for Case 2, 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. By comparison, the Case 1 breaking wave forces would be approximately 1.1 times those shown for these two events.

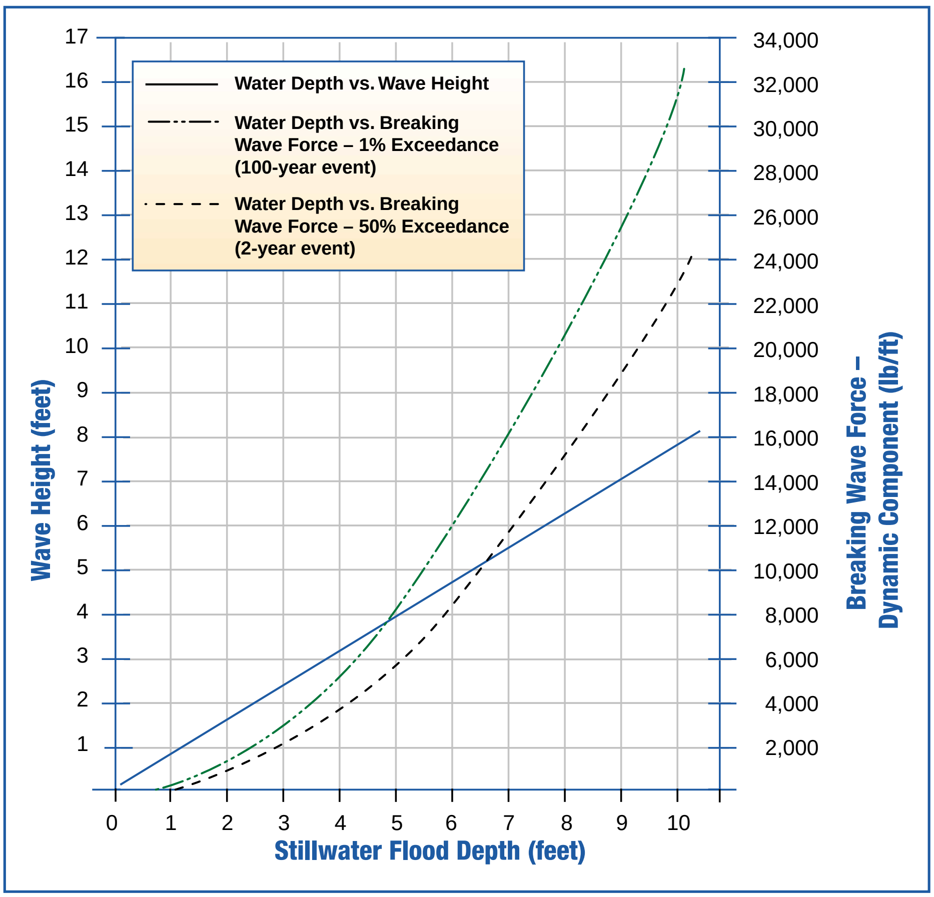



**Figure 11-8**
Water depth vs. wave height, and water depth vs. breaking wave force against a vertical wall (for Case 2, with stillwater behind the wall) for the 1-percent and 50-percent exceedance interval events.

It is important to note that the wave pressures shown in Figure 11-8 are much higher than typical wind pressures 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 papers contained in *Wave Forces on Inclined and Vertical Wall Surfaces* [ASCE 1995] for a detailed discussions on breaking wave pressures and durations).

**WARNING**

**Even waves less than 3 feet high can impose large loads on foundation walls. This manual recommends that buildings in coastal A zones be designed and constructed to meet V-zone requirements (see Section 6.5.2 in Chapter 6).**

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. It is important to note that 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. This fact was confirmed by the results of recent FEMA-sponsored laboratory tests of breakaway wall failures, in which measured pressures on the order of hundreds of lb/ft$^2$ were generated by waves 12–18 inches high. The test results are presented in FEMA NFIP Technical Bulletin 9 (see Appendix H).

**WARNING**

**Under the NFIP, construction of solid foundation walls (such as those shown in Figure 11-9) for new, substantially damaged, and substantially improved buildings is not permitted in V zones.**

### 11.6.9 Hydrodynamic Loads

Water flowing around a building (or a structural element or other object) imposes additional loads on the building as shown in Figure 11-9. The loads (which are a function of flow velocity and structure 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 flood waters is constant (i.e., steady state flow) and, as noted previously, that the hydrodynamic load imposed by flood waters moving at less than 10 ft/sec can be converted to an equivalent hydrostatic load (USACE 1992, ASCE 1998b).

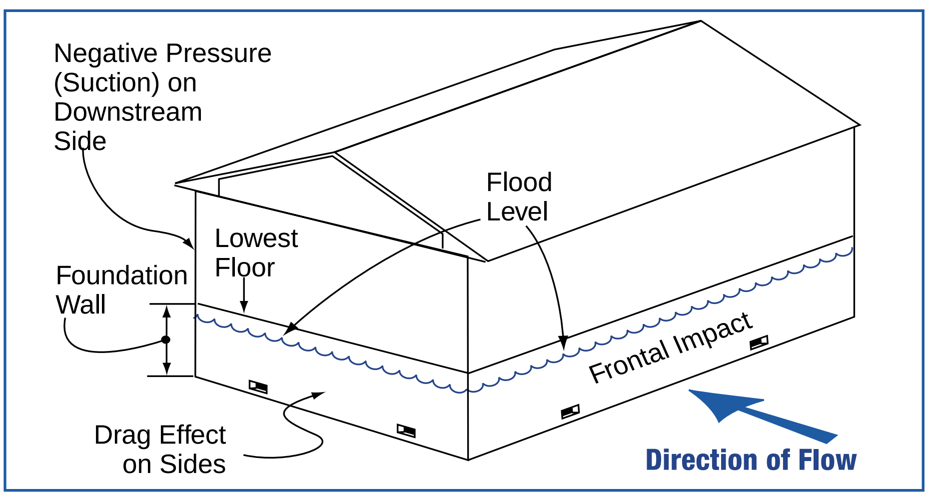



**Figure 11-9**
Hydrodynamic loads on a building. Note that the lowest floor of the building shown here is above the flood level and that the loads imposed by flowing water affect only the foundation walls.

One of the most difficult steps in quantifying loads imposed by moving water is determining the expected flood velocity. Refer to Formula 11.2 in Section 11.6.6 for guidance concerning design flood velocities.

The equivalent hydrostatic load imposed by water moving at less than 10 ft/sec is calculated with Formula 11.7

**Formula 11.7 Hydrodynamic Load**

**(flow velocity less than 10ft/sec)**

$$d_{dyn} = (1/2)\, C_d V^2/g$$

where:

**$d_{dyn}$** = equivalent additional flood depth to be applied to the upstream side of the affected structure, in feet

**V** = velocity of water in ft/sec (see Formula 11.2)

**g** = acceleration due to gravity (32.2 ft/sec$^2$ )

**$C_d$** = drag coefficient (recommended values are 2.0 for square or rectangular piles and 1.2 for round piles, or from Table 11.2 for larger obstructions)

and

$$f_{dyn} = \gamma d_s d_{dyn}$$

where:

**$f_{dyn}$** = equivalent hydrostatic force per unit width (lb/ft) due to low-velocity flow acting at the point 2/3 below the stillwater surface of the water

**$\gamma$** = specific weight of water (62.4 lb/ft$^3$ for fresh water and 64.0 lb/ft$^3$ for saltwater)

and

$$F_{dyn} = f_{dyn}(w)$$

where:

**$F_{dyn}$** = total equivalent lateral hydrostatic force in lb acting at the point 2/3 below the stillwater surface of the water

**w** = width of structure in feet

Formula

Hydrodynamic Load From Flood Flows Moving at Less Than 10 ft/sec

**CROSS-REFERENCE**

For guidance regarding drag coefficients ($C_d$), refer to Volume II of the U.S. Army Corps of Engineers *Shore Protection Manual* (USACE 1984), Section 5.3.3 of ASCE 7-98 (ASCE 1998b), and FEMA 259 (FEMA 1995a).

The drag coefficient used in the above equation is taken from the U.S. Army Corps of Engineers *Shore Protection Manual*, Volume II (USACE 1984). Additional guidance is provided in Section 5.3.3 of ASCE 7-98 (ASCE 1998b) and in FEMA 259 (FEMA 1995a).

The drag coefficient is a function of the shape of the object around which flow is directed. When the object is something other than a round, square, or rectangular pile, the coefficient is determined by one of the following ratios:

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

**NOTE**

Lift coefficients ($C_l$) can be found in *Introduction to Fluid Mechanics* (Fox and McDonald 1985) and in many other fluid mechanics textbooks.

2. the ratio of the width of the object (w) to the stillwater depth of the water ($d_s$), if the object is not fully immersed (see Figure 11-10 and Table 11.2)

**Figure 11-10**
Determining drag coefficient from width-to-depth ratio. Note that the lowest floor of the building shown here is above the flood level.

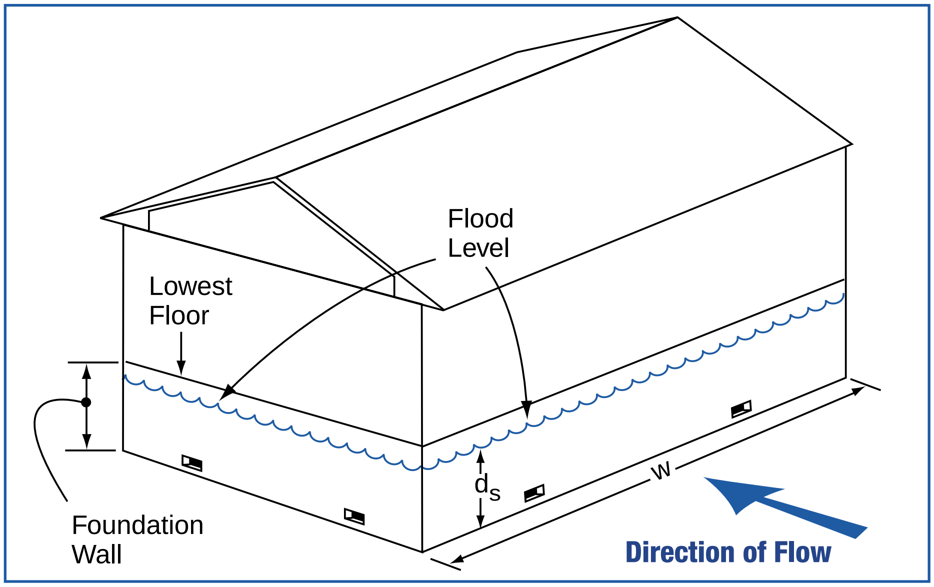



**Table 11.2**
Drag Coefficients for Ratios of Width to Depth ($w/d_s$) and Width to Height (w/h)

| Width to Depth Ratio ($w/d_s$ or w/h) | Drag Coefficient $C_d$ |
|---|---|
| From 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 |

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

The hydrodynamic loads imposed by flood waters with velocities greater than 10 ft/sec cannot be converted to equivalent hydrostatic loads. Instead, they must be determined according to the principles of fluid mechanics or hydraulic models (see Formula 11.8).

**Formula 11.8 Hydrodynamic Load**

**(flow velocity greater than 10ft/sec)**

$$F_{dyn} = (^1/_2)\, C_d \rho\, V^2 A$$

where:

$F_{dyn}$ = horizontal drag force in 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, or from Table 11.2 for larger obstructions)

$\rho$ = mass density of fluid (1.94 slugs/ft$^3$ for fresh water and 1.99 slugs/ft$^3$ for salt water)

$V$ = velocity of water in ft/sec (see Formula 11.2)

$A$ = surface area of obstruction normal to flow in ft$^2$
= $wd_s$ (see Figure 11-10) or $wh$

**ƒormula**

**Hydrodynamic Load From Flood Flows Moving at Greater Than 10 ft/sec**

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 linear unit; dividing by A would yield a force per unit area. Also, note that the drag coefficients for square, rectangular, and round piles in Formula 11.8 ($C_d$) are lower than those in Formula 11.5 ($C_{db}$).

Example

Non-Breaking Wave Load on Piles vs. Breaking Wave Load on Piles

NOTE

Treat non-breaking wave loads as hydrodynamic loads.

**Example: Non-Breaking Wave Load on Piles vs. Breaking Wave Load on Piles**

The following conditions are assumed:

- **house elevated on round-pile foundation near saltwater**
- **$C_d$ (drag coefficient for non-breaking wave on round pile – see Formula 11.7) = 1.2**
- **$C_{db}$ (drag coefficient for breaking wave on round pile – see Formula 11.5)= 1.75**
- **D = 10 in or 0.833 foot**
- **$d_s$ = 8 feet**
- **Velocity ranges from 8 ft/sec to 16 ft/sec**
- **$\rho$ = mass density of water (1.94 slugs/ft$^3$ for fresh water and 1.99 slugs/ft$^3$ for salt water)**
- **A = (8 ft)(0.833 ft)**

The load from a non-breaking wave on a pile is calculated as follows:
$F_{nonbrkp} = (^1/_2)C_d \rho V^2 A$ so
**$F_{nonbrkp}$ = 509 lb /pile to 2,038 lb/pile (depending on flood velocity)**

The load from a breaking wave on a pile is calculated with Formula 11.5:
$F_{brkp} = (^1/_2)C_{db} \gamma D H_b{}^2$ where $H_b$ is the height of the breaking wave or $(0.78)(d_s)$ so
**$F_{brkp}$ = 1,810 lb /pile**

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

## 11.6.10 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 in the potential debris stream:

- 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

The equation normally used for the calculation of debris loads is an expression for momentum and is given by Formula 11.9.

**Formula 11.9 Debris Impact Load**

$$F_i = wV/gt$$

where: $F_i$ = impact force in lb acting at the stillwater level

$w$ = weight of the object in lb

$V$ = velocity of water in ft/sec or approximated by $^1/_2(gd_s)^{1/2}$

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

$t$ = duration of impact in seconds

Formula

Debris Impact Load

This equation contains several uncertainties, each of which must be quantified before the impact of debris loading on the building can be determined:

- 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 the most vulnerable portion of the building where failure could mean collapse
- duration of the impact (t)

**Size, shape, and weight of the debris**

Although difficult to generalize, there may be regional differences in debris types. For example, the coasts of Washington, Oregon, and selected other areas may be subject to very large debris in the form of logs present along the shoreline. Other areas, such as the southeast coast of the United States, may be more subject to debris impact from dune crossovers, destroyed buildings, and the like. It is recommended that in the absence of information about the nature of the potential debris, a weight of 1,000 lb be used for the value of w. Objects of 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 11.6.6, flood velocity can be approximated by one of the equations in Formula 11.2; refer to that section for a discussion of how to choose the most appropriate equation. 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.

**NOTE**

**The assumption that debris velocity is equal to flood velocity may overstate the velocities of large debris objects; therefore, engineering judgment may be required in some instances. Designers may wish to reduce debris velocity for larger objects.**

**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 level.

**Duration of impact**

Uncertainty about the duration of impact (t)—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. According to physics and dynamics texts such as Chopra (1995), the duration of impact is influenced primarily by the natural frequency of the building, which is a function of the building's "stiffness." This stiffness is determined by the properties of the material being struck by the object, the number of supporting members (columns or piles), the height of the building above the ground, and the height at which the material is struck.

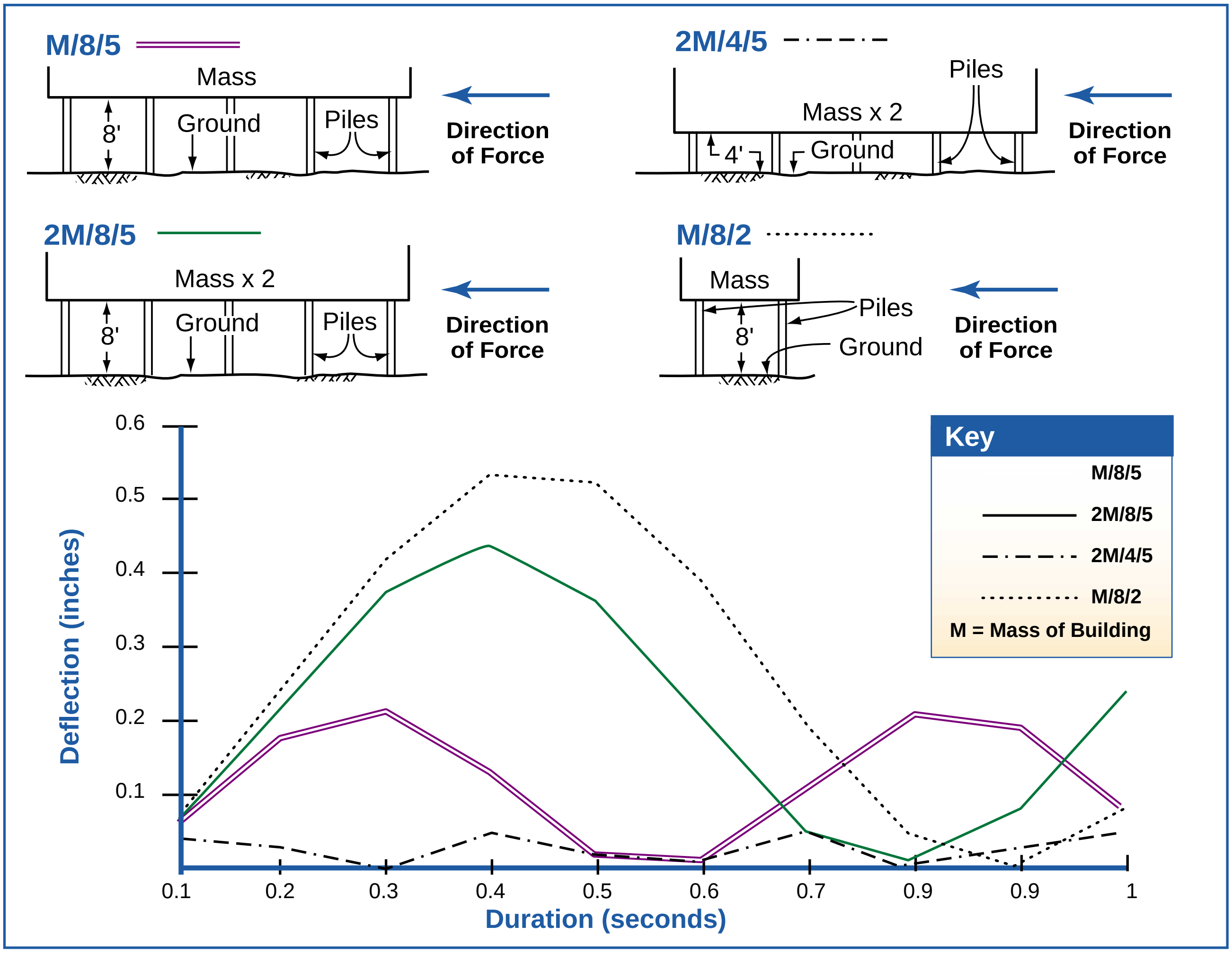
Although little guidance on duration of impact exists, the City of Honolulu Building Code recommends the following durations based on the type of construction being struck:

- wood: 1 second
- steel: 0.5 second
- reinforced concrete: 0.1 second

The graphs in Figure 11-11 show deflection vs. duration of impact (t) for several scenarios involving debris impact on one wood pile in each of four pile foundation examples. The largest deflection is for a mass (M) supported by only two piles 8 feet above grade (scenario M/8/2 in the figure). For five piles supporting twice that mass 4 feet above grade (scenario 2M/4/5 in the figure), the deflection is very small. The other deflection scenarios shown in Figure 11-11 are a mass (M) supported by five piles 8 feet above grade (M/8/5) and five piles supporting twice the mass 8 feet above grade (2M/8/5).

A complete mathematical analysis of this problem is beyond the scope of this manual; however, Table 11.3 suggests durations (t) to use in Formula 11.9. These durations were developed with a mathematical model from dynamic theory. They are of approximately the same order of magnitude as those provided in the City of Honolulu Building Code.

**Figure 11-11** Pile deflection vs. duration of impact (t) for alternative wood pile foundations.




| Type of Construction | Duration (t) of Impact (sec) | |
|---|---|---|
| | Wall | Pile |
| Wood | 0.7 – 1.1 | 0.5 – 1.0 |
| Steel | NA | 0.2 – 0.4 |
| Reinforced Concrete | 0.2 – 0.4 | 0.3 – 0.6 |
| Concrete Masonry | 0.3 – 0.6 | 0.3 – 0.6 |

NA - Not Applicable

**Table 11.3**
Impact Durations (t) for Use in Formula 11.9

### 11.6.11 Localized Scour

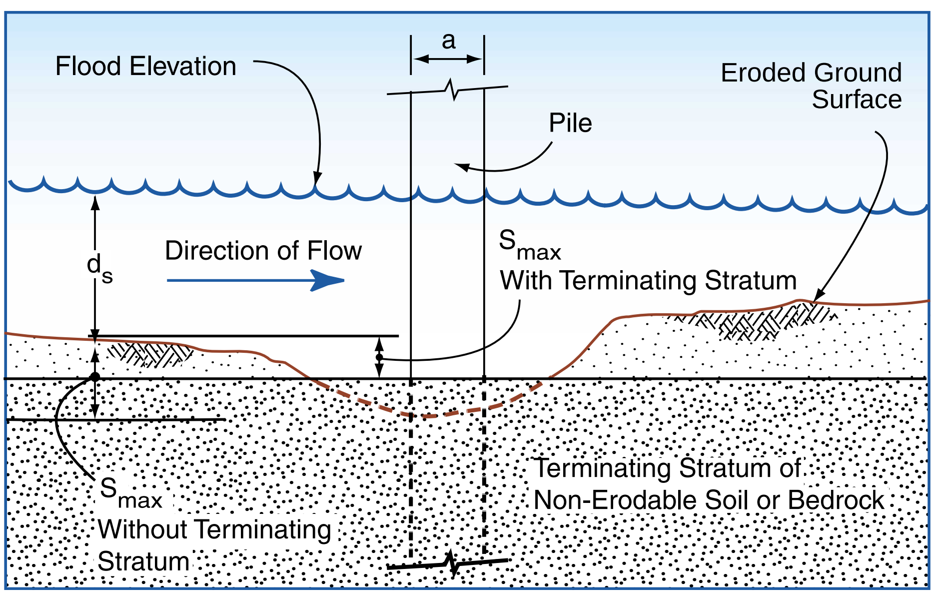
Waves and currents during coastal flood conditions are capable of creating turbulence around foundation elements, and causing localized scour around those elements. Determining potential scour is critical in designing coastal foundations to ensure that failure during and after flooding does not occur 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, the assistance of a geotechnical engineer should be sought before calculated scour depths are reduced.

#### 11.6.11.1 Localized Scour Around Vertical Piles (Non-Tsunami Condition)

Localized scour calculation methods in coastal areas have been largely based on empirical evidence gathered after storms. This evidence 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 11-12 illustrates localized scour at a pile, with and without a scour-resistant terminating stratum. Until such time as better design guidance is obtained, localized scour around a vertical pile or similar foundation element should be calculated with Formula 11.10a.

**Figure 11-12**
Scour at vertical foundation member stopped by underlying scour-resistant stratum.



**Formula 11.10a Localized Scour Around Vertical Pile**

**(Non-Tsunami Condition)**

$$S_{max} = 2.0a$$

where: $S_{max}$ = maximum localized scour depth in feet

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

Formula
Localized Scour Around Vertical Pile (Non-Tsunami Condition)

**NOTE**

Formula 11.10a can also be used to approximate local scour beneath grade beams—set "a" equal to the depth (vertical thickness) of the grade beam.

### 11.6.11.2 Localized Scour Around Vertical Walls and Enclosures (Non-Tsunami Condition)

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 calculated with Formula 11.10b.

**Formula 11.10b**
**Localized Scour Around Vertical Enclosure**

**(Non-Tsunami Condition)**

$$S_{max} = d_s \{2.2(a/d_s)^{0.65}[V/(gd_s)^{0.50}]^{0.43}\}K$$

where: $S_{max}$ = maximum localized scour depth in feet

$d_s$ = design stillwater flood depth in feet (upstream of the structure)

$a$ = half the width of the solid foundation perpendicular to the flood flow

$V$ = average velocity of water in ft/sec (see Formula 11.2)

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

$K$ = factor applied for Flow Angle of Attack (see Table 11.4)

Formula
Localized Scour Around Vertical Enclosure (Non-Tsunami Condition)

**WARNING**

Formula 11.10b is not applicable to new, substantially damaged, and substantially improved buildings in V zones, because the NFIP-compliant floodplain management laws or ordinances enacted by communities require the use of open foundations for such buildings.

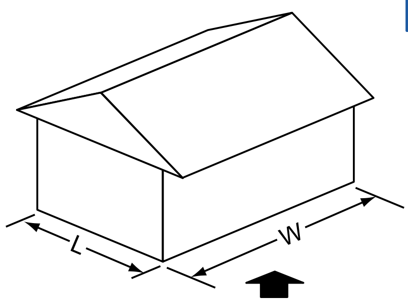
**WARNING**

Formula 11.10b was developed by hydraulic engineers to estimate local scour around bridge piers in rivers. Its use in coastal areas is suggested as an interim method until a better method is developed. **Scour depths estimated with Formula 11.10b can be unrealistically high for coastal areas and should be capped at 10 feet of localized scour.**

The magnitude of local scour at thin vertical members is not affected by the direction of flow, because the cross-sectional dimensions of such members are uniform or nearly uniform. However, the magnitude of scour at a wall of a building will vary with the angle at which the water strikes the wall. If the wall is not perpendicular to the direction of flow, a multiplying factor K should be applied to Formula 11.10b to account for the resulting increase in scour (see Table 11.4). As shown in the table, the value of K varies with not only the angle of attack, but also the width/length ratio of the building (see figure at left of Table 11.4).

**Table 11.4**
Scour Factor for Flow Angle of Attack, K (Angle = 0 corresponds to flow perpendicular to building face.)

| Angle of Attack (degrees) | Width/Length Ratio of Building in Flow | | | |
|---|---|---|---|---|
| | 4 | 8 | 12 | 16 |
| 0 | 1 | 1 | 1 | 1 |
| 15 | 1.15 | 2 | 2.5 | 3 |
| 30 | 2 | 2.5 | 3.5 | 4.5 |
| 45 | 2.5 | 3.5 | 3.5 | 5 |
| 60 | 2.5 | 3.5 | 4.5 | 6 |




### 11.6.11.3 Localized Scour (Tsunami Conditions)

Dames and Moore, in *Design and Construction Standards for Residential Construction in Tsunami-Prone Areas of Hawaii* (1980), suggest that scour depth depends on soil type and that scour depths in areas up to 300 feet from the shoreline can be determined as a percentage of the stillwater depth $d_s$, as shown in Table 11.5.

**Table 11.5**
Localized Scour Depth vs. Soil Type (From Dames & Moore 1980), Tsunami Conditions

| 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% |

### 11.6.12 Flood Load Combinations

Designers should be aware that not all of the flood loads described in Section 11.6 will act at certain locations or against certain building types. Therefore, Table 11.6 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).

**Table 11.6**
Selection of Flood Load Combinations for Design

| | |
|---|---|
| **Case 1:** | **Pile or Open Foundation in V Zone (Required)** |
| | $F_{brkp}$ (on all piles, Formula 11.5) + $F_i$ (on one corner or critical pile only, Formula 11.9)<br>or<br>$F_{brkp}$ (on front row of piles only, Formula 11.5) + $F_{dyn}$ (on all piles but front row, Formula 11.7 or 11.8) + $F_i$ (on one corner or critical pile only, Formula 11.9) |
| **Case 2:** | **Pile or Open Foundation in Coastal A Zone (Recommended)** |
| | $F_{brkp}$ (on all piles, Formula 11.5) + $F_i$ (on one corner or critical pile only, Formula 11.9)<br>or<br>$F_{brkp}$ (on front row of piles only, Formula 11.5) + $F_{dyn}$ (on all piles but front row, Formula 11.7 or 11.8) + $F_i$ (on one corner or critical pile only, Formula 11.9) |
| **Case 3:** | **Solid (Wall) Foundation in Coastal A Zone (NOT Recommended)** |
| | $F_{brkw}$ (on walls facing shoreline, Formula 11.6, including hydrostatic component) + $F_{dyn}$ (Formula 11.7 or 11.8); assume one corner is destroyed by debris, and design in redundancy |
| **Case 4:** | **Solid (Wall) Foundation in Non-Coastal A Zone (shown for comparison)** |
| | $F_{sta}$ (Formula 11.3 and 11.4) + $F_{dyn}$ (Formula 11.7 or 11.8) |

**NOTE**

$F_{sta}$ = hydrostatic load

$F_{dyn}$ = hydrodynamic load

$F_i$ = debris impact load on pile

$F_{brkp}$ = breaking wave load on pile

$F_{brkw}$ = breaking wave load on wall

As noted in Chapter 6, 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.

# Example Flood Load Example Problem

**Given:**

1. Oceanfront building site on landward side of a primary frontal dune (see Figure 11-13).

2. Topography along transect perpendicular to shoreline is shown inch Figure 11-14; existing ground elevation at seaward row of pilings = 7.0 feet NGVD.

3. Soil is dense sand; no terminating stratum above –25 feet NGVD.

4. Data from FIRM is as follows: flood hazard zone at site is VE, BFE = 14.0 feet NGVD.

5. Data from FIS is as follows: 100-year stillwater elevation = 10.1 feet. NGVD, 10-year stillwater elevation = 5.0 feet NGVD.

6. Local government requires 1.0 feet freeboard; therefore DFE = 14.0 feet NGVD (BFE) + 1.0 foot = 15.0 feet NGVD.

7. Building to be supported on 8-inch x 8-inch square piles as shown in Figure 11-15.

8. Direction of wave and flow approach during design event is perpendicular to shoreline (as shown in Figure 11-15).

**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. design event breaking wave height ($H_b$) at seaward row of piles

7. hydrodynamic (velocity flow) loads ($F_{dyn}$) on a pile (not in seaward row)

8. breaking wave loads ($F_{brk}$) on the seaward row of piles

9. debris impact load ($F_i$) from a 1,000-lb object acting on one pile

# Example Flood Load Example Problem (continued)

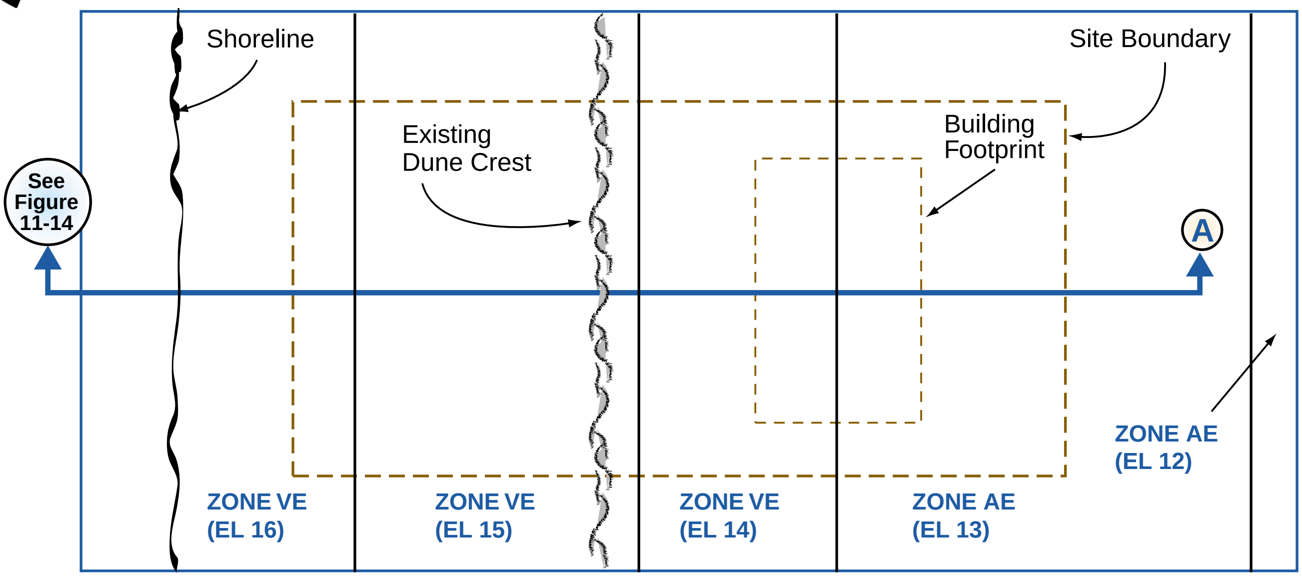



**Figure 11-13**
Plan view of site and building location, with flood hazard zones.

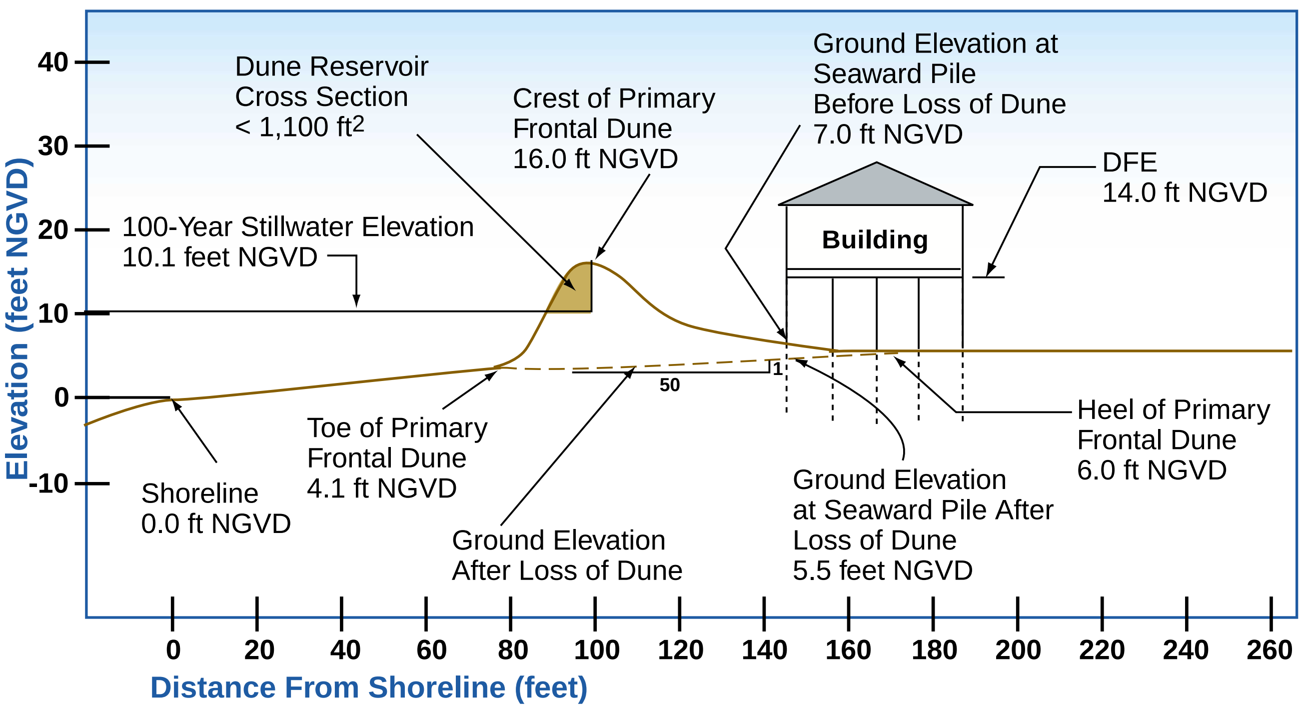



**Figure 11-14**
Section A: Primary frontal dune will be lost to erosion during 100-year flood because dune reservoir is less than 1,100 ft$^2$.

# Example Flood Load Example Problem (continued)

**1 Primary frontal dune reservoir:**

The cross-sectional area of the frontal dune reservoir is the area above 100-year stillwater elevation and seaward of dune crest. This area (see Figure 11-14) is approximately triangular, 5.9 feet high (16 feet NGVD dune crest elevation – 10.1 feet NGVD 100-year stillwater elevation), and approximately 15 feet wide at the base. This area will be slightly greater than the triangular area (1/2 x 15 feet x 5.9 feet = 44 $ft^2$, **say 50 $ft^2$**. This is far less than the 1,100 $ft^2$ required by this manual for dune to survive the 100-year event. **Therefore, assume the dune will be lost and provide no protection during 100-year event.**

**2 Eroded ground elevation beneath building:**

Remove dune from transect (see Section 7.8.1.4 and Figure 7-63, in Chapter 7) 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. In this instance, the dune toe is lower (4.1 feet NGVD vs. 5.0 feet NGVD). Therefore, draw a line from the dune toe (located 75 feet from the shoreline, at the elevation 4.1 feet NGVD) sloping upward at a 1:50 (v:h) slope. The seaward row of piles (located at a point 145 feet from the shoreline) intersects this line at an elevation of 5.5 feet NGVD (4.1 feet NGVD + [145-75][1/50]). **The eroded ground elevation at the seaward row of pilings = 5.5 feet NGVD (neglecting local scour around the piles).**

**3 Design stillwater flood depth ($d_s$) at seaward row of pilings:**

$d_s$ = 100-year stillwater elevation – eroded ground elevation beneath building

So $d_s$ = 10.1 feet NGVD – 5.5 feet NGVD

**$d_s$ = 4.6 ft**

**4 Range of design flow velocities (V):**

Lower V = design stillwater flood depth (in feet)/t

Where: t= 1sec

**Lower V = 4.6 ft/sec**

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

Where: g = 32.2 ft/sec$^2$

$d_s$ = 4.6 ft

So upper V = $(32.2 \times 4.6)^{0.5}$

**Upper V = 12.2 ft/sec**

**5 Local scour depth ( S) around seaward row of pilings:**

S = 2.0a (from reduction of Formula 11.10a, see Section 11.6.11.1)

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

So S = (2.0)(0.88)

**S = 1.76 ft**

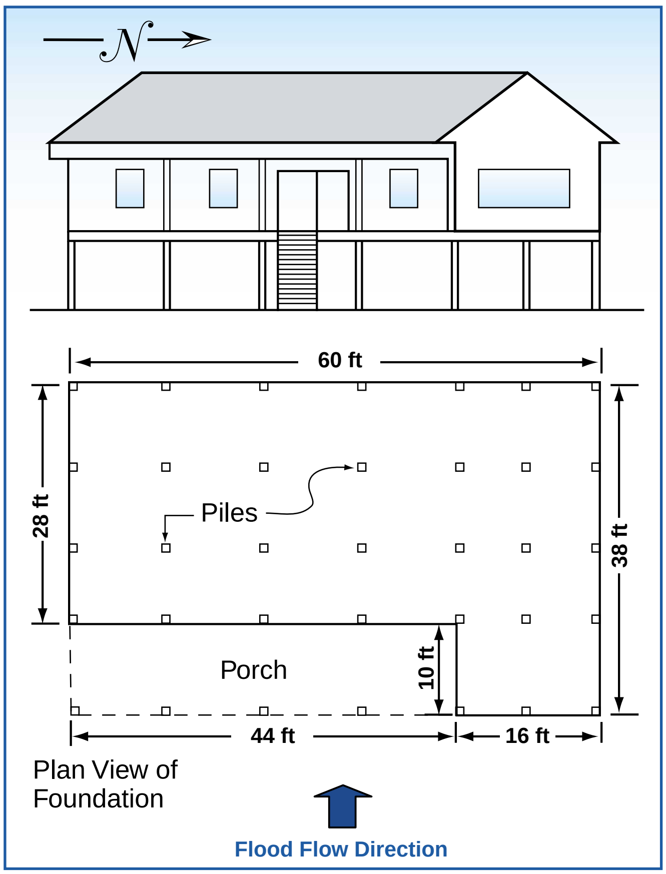



**Figure 11-15**
Building elevation and plan view of pile foundation.

# Example Flood Load Example Problem (continued)

**6 Breaking wave height ($H_b$) at seaward row of pilings:**

At seaward row of pilings, $H_b$ = (stillwater elevation - eroded ground elevation)(0.78)
So $H_b$ = (10.1 - 5.5) (0.78)
**$H_b$ = 3.6 ft**

**7 Hydrodynamic (velocity flow) loads $F_{dyn}$ on a pile (not in seaward row):**

$F_{dyn}$ on one pile = $(^1/_2)C_d\rho V^2A$

Where: $C_d$ = 2.0 for a square pile

$\rho$ = 1.99 slugs/ft$^3$

A = (8 in / 12 in)(10.1 - 5.5) = 3.07 ft$^2$

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

So $F_{dyn}$ = $(^1/_2)(2.0)(1.99)(12.2)^2(3.07)$

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

Number of piles in seaward row = 7

$F_{dyn}$ on all but seaward row of piles = (909)(24) = 21,816 lb

**NOTE**

Because the four piles under the seaward edge of the porch do not support the house, they are not included in the calculation in Step 7. Therefore, the seven piles in the seaward row referred to in Step 7 are those at the seaward edge of the house, and the piles not in the seaward row are the remaining 24 piles under the house.

**8 Breaking wave loads ($F_{brkp}$) on seaward row of pilings**:

$F_{brkp}$ on one pile = $(^1/_2)C_{db}\gamma DH_b^2$

Where: $C_{db}$ = 2.25 for square piles

$\gamma$ = 64.0 lb/ft$^3$ for salt water

D = 8 in/12 = 0.67 ft

$H_b = 0.78d_s$ = (0.78)(4.6) = 3.6 ft

So $F_{brkp}$ = $(^1/_2)(2.25)(64.0)(0.67)(3.6)^2$

$F_{brkp}$ on one pile = 625 lb

Number of piles in seaward row = 7

**$F_{brkp}$ on seaward row of piles = (625)(7) = 4,375 lb**

**9 Debris impact load ($F_i$) from a 1,000-lb object on one pile:**

$F_i$ = wV/gt

Where: w = 1,000 lb

g = 32.2 feet/sec$^2$

t is from Table 11.3 and is approximately 0.5 sec

So debris impact load = (1,000)(12.2)/(32.2)(0.5)

**Debris impact load = 758 lb**

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

Owner Name: ____________________ Prepared by: ____________________

Address: ____________________ Date: ____________________

Property Location: ____________________

### Constants

$\gamma$ (specific weight of water) = 62.4 lbs/ft$^3$ for fresh water and 64.0 lbs/ft$^3$ for salt water

$\rho$ (mass density of fluid) = 1.94 slugs/ft$^3$ for fresh water and 1.99 slugs/ft$^3$ for salt water

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

### Variables

$d_s$ (design stillwater flood depth (ft)) =

Vol (volume of flood water displaced (ft$^3$)) =

V (velocity (fps)) =

$C_{db}$ (breaking wave drag coefficient) =

$H_b$ (breaking wave height (ft)) =

$C_p$ (dynamic pressure coefficient) =

$C_d$ (drag coefficient) =

a,w (width of structure (ft)) =

w (debris object weight (lb)) =

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

### Summary of Loads:

$F_{sta}$ =

$F_{buoy}$=

$F_{brkw}$ =

$F_{dyn}$ =

$F_i$ =

$S_{max}$ =

### Formula 11.3 Lateral Hydrostatic Load

$F_{sta} = (^1/_2)\gamma d_s^2(w)$

### Formula 11.4 Vertical (Buoyancy) Hydrostatic Load

$F_{buoy} = \gamma(Vol)$

### Formula 11.6 Breaking Wave Load on Vertical Walls

$F_{brkw} = (1.1C_p \gamma d_s^2 + 2.41\gamma d_s^2)w$ (if dry behind wall)

or $F_{brkw} = (1.1C_p \gamma d_s^2 + 1.91\gamma d_s^2)w$ (if stillwater level is the same on both sides of wall)

### Formula 11.7 or 11.8 Hydrodynamic Load

$F_{dyn} = \gamma d_s\{(^1/_2)C_d V^2/g\}(w)$ (if flow velocity $\leq$ 10 fps)

or $F_{dyn} = (^1/_2) C_d\rho V^2A$ (if flow velocity > 10 fps)

### Formula 11.9 Debris Impact Load

$F_i = wV/gt$

### Formula 11.10b Local Scour

$S_{max} = d_s \{2.2(a/d_s)^{0.65}[V/(gd_s)^{0.50}]^{0.43}\}K$

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

Owner Name: ______________________ Prepared by: ______________________

Address: ______________________ Date: ______________________

Property Location: ______________________

### Constants

$\gamma$ (specific weight of water) = 62.4 lbs/ft$^3$ for fresh water and 64.0 lbs/ft$^3$ for salt water

$\rho$ (mass density of fluid) = 1.94 slugs/ft$^3$ for fresh water and 1.99 slugs/ft$^3$ for salt water

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) =

$C_d$ (drag coefficient for piles) =

a,w (width of structure (ft)) =

w (debris object weight (lb)) =

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

### Summary of Loads:

$F_{brkp}$=

$F_{dyn}$ =

$F_i$ =

$S_{max}$ =

### Formula 11.5 Breaking Wave Load on Vertical Piles

$F_{brkp} = (^1/_2) C_{db} \gamma D H_b^2$

### Formula 11.7 or 11.8 Hydrodynamic Load

$F_{dyn} = \gamma d_s \{(^1/_2) C_d V^2/g\}(w)$ (if flow velocity $\leq$ 10 fps)

or

$F_{dyn} = (^1/_2) C_d \rho V^2 A$ (if flow velocity > 10 fps)

### Formula 11.9 Debris Impact Load

$F_i = wV/gt$

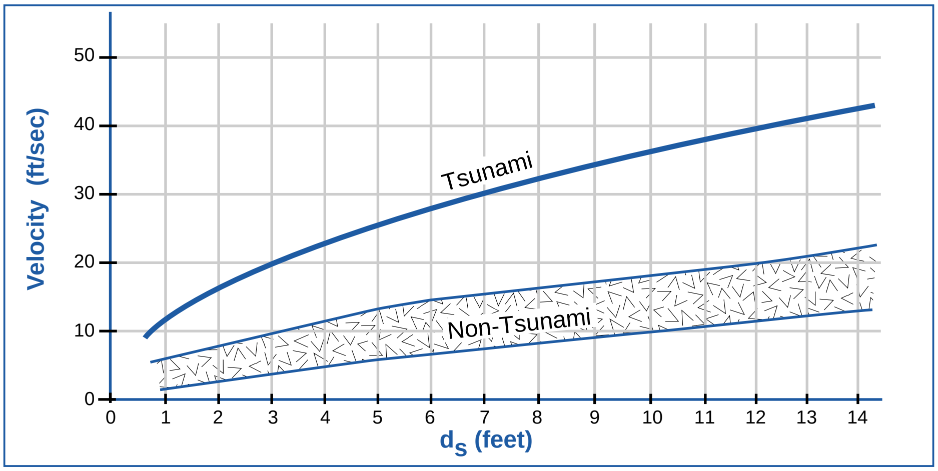
### Formula 11.10a Local Scour

$S_{max} = 2.0a$

## 11.7 Tsunami Loads

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 or extratropical cyclones. If the tsunami acts as a rapidly rising tide, most damage will be caused by buoyant and hydrostatic forces (see *Tsunami Engineering* [Camfield 1980]). When the tsunami forms a borelike wave, the effect is a surge of water to the shore. When this occurs, the expected flood velocities are substantially higher. Both Camfield and Dames & Moore (1980) suggest that this velocity should be $V = 2(gd)^{0.5}$. Figure 11-16 shows the relationship between design stillwater depth and expected velocity for tsunami and non-tsunami conditions.

**Figure 11-16**
Tsunami velocity vs. design stillwater depth. Non-tsunami velocities are shown for comparison.




The tsunami velocities shown in Figure 11-16 are very large and if realized at the greater water depths, would cause substantial damage to all buildings in the path of the tsunami. Designers should collect as much data as possible about expected tsunami depths to more accurately calculate tsunami flood forces.

**NOTE**

It is not the intent of this manual to replace or change provisions of ASCE 7-98. The determination of important wind load criteria is highlighted in this manual. For complete coverage of this subject, the designer should consult ASCE 7-98.

## 11.8 Wind Loads

The ASCE standard ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures* (ASCE 1998b), was considered the state-of-the-art in wind load design technology and was a consensus standard at the time this manual went to print. It comprehensively covers the effects of wind pressures on a variety of building types and building elements. The design for wind loads is essentially the same whether the winds are due to hurricanes, thunderstorms, or tornadoes. Two additional references are recommended as design aides: the *AIA Wind Design Primer* available from the American Institute of Architects (AIA 1994) and the *Guide to the Use of the Wind Load Provisions* (Mehta and Marshall 1995) available from ASCE.

Designers may notice several differences between the wind load provisions of ASCE 7-98 and those of familiar model building codes. As a result, in some circumstances, design wind loads determined with ASCE 7-98 may be higher or lower than those from local or model codes.

It is important to calculate wind pressures for both the structural frame (called the Main Wind Force Resisting System, or MWFRS, in ASCE 7-98) and for building components and cladding. Components and cladding include elements such as roof sheathing, roof coverings, exterior siding, windows, doors, soffits, facia, and chimneys. Investigations of wind-damaged buildings after disasters have shown that many building failures start because a component or piece of cladding is blown off the building, allowing wind and rain to enter the building. The uncontrolled entry of wind into the building creates internal pressure that, in conjunction with negative external pressures, can "blow the building apart. "

The most important factors that affect wind load design are as follows:

- ASCE uses the 3-sec peak gust wind speed as the criterion for velocity averaging times instead of the fastest-mile speed used in most codes that were available prior to 2000.
- Topographic effects (hills and escarpments) create a wind speedup effect.
- The wind speed maps in ASCE 7-98, IBC2000 (ICC 2000a), and the IRC (ICC 2000b) are based on an approximately 50-year – 100-year wind. Increasing the recurrence interval of the design wind to provide protection from a more infrequent event is accomplished in ASCE 7-98 by increasing the building importance factor (I).
- Building height and shape affect wind loads.
- Terrain conditions affect the exposure of the building to wind.

The following short discussion of wind/building interaction theory helps explain how wind flows over and around buildings. Methods for calculating wind pressure are presented after this discussion.

The effects of wind on buildings can be summarized as follows (see Figure 11-17):

- Windward walls and steep-sloped roofs are acted on by inward-acting, or positive, pressures.
- Leeward walls and steep- and low-sloped roofs are acted on by outward-acting, or negative, pressures.
- Air flow separates at sharp edges and at points where the building geometry changes.

**NOTE**

Using ASCE 7-98 effectively requires some practice in the application of many of its provisions. The following particularly require judgment in the use of the specific design guidelines, coefficients, and requirements:

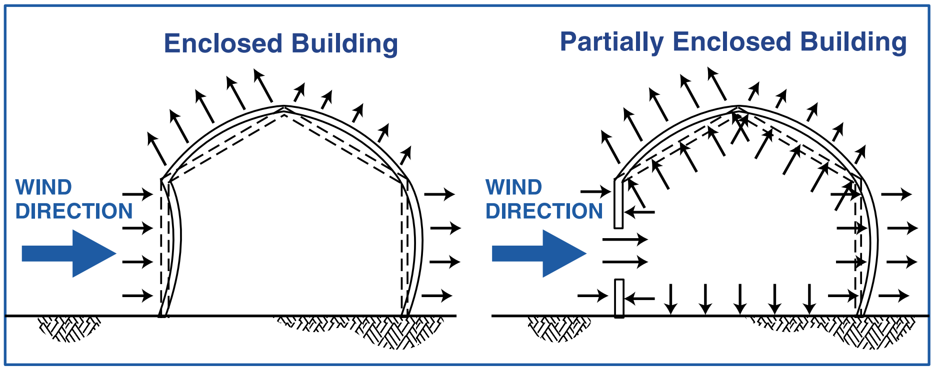
- determining exposure categories
- deciding whether to use Figure 6-3 or 6-4 for external pressure coefficients
- interpolating graphs for components and cladding external pressure coefficients
- determining the effective wind area for components and cladding pressure coefficients

**NOTE**

Basic wind speeds given by ASCE 7-98, shown in Figure 7-23, correspond to (1) a wind with a recurrence interval between 50 and 100 years in hurricane-prone regions (Atlantic and Gulf of Mexico coasts with a basic wind speed greater than 90 mph, and Hawaii, Puerto Rico, Guam, the U.S. Virgin Islands, and American Samoa), and (2) a recurrence interval of 50 years in non-hurricane-prone areas.

- 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.

**Figure 11-17**
Effect of wind on an enclosed building and a building with an opening.




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

It is important to understand the following about openings and the protection of openings:

- The magnitude of internal pressures depends on whether the building is "enclosed," "partially enclosed," or "open" as defined by ASCE 7-98.
- In hurricane-prone regions as defined in ASCE 7-98, 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. It should be noted that this requirement also applies to glazing in doors.
- In hurricane-prone regions as defined by ASCE 7-98, glazing that is not impact-resistant or is not protected by shutters or other devices that are impact-resistant is permitted, provided the building is considered to be a "partially enclosed building." This will require that the building be designed for higher internal pressures than required for enclosed buildings. Because of the high potential for glass breakage in hurricane-prone regions, which will result in damage from wind and water intrusion, this approach is not recommended.
- The test standard referenced in Section 12.7.4.2, in Chapter 12 of this manual, should be used in determining whether glazing or protection for glazing will withstand the impact of windborne debris.

**CROSS-REFERENCE**
Section 12.7.4.2, in Chapter 12, discusses test criteria for determining the resistance of glazing and glazing protection to the impact of windborne missiles.

Wind forces must be evaluated not only for inward- and outward-acting pressures, but also pressure normal to and parallel to the main roof ridge. Ultimately, the wind load case that results in the greatest pressures, either positive or negative, should be used as the basis for wind-resistant design. This procedure requires that the designer determine how various combinations of building characteristics such as size, shape, and height will affect the flow of the wind over and around the building and the resulting pressures on the building. Consequently, for a designer who is trying to minimize wind pressures by altering these characteristics, wind design will be a trial-and-error process.

**NOTE**

ASCE 7-98, Chapter 6, presents a simple procedure for determining velocity pressures for buildings defined as regular shaped that have a simple diaphragm, a mean roof height less than or equal to 30 feet, and a roof slope less than 10 degrees (a pitch of approximately 2:12). When these conditions are met, the ASCE 7-98 procedure may be used instead of the formulas that appear in the Wind Load Example Problem on pages 11-45 through 11-47.

### 11.8.1 Main Wind Force Resisting System (MWFRS)

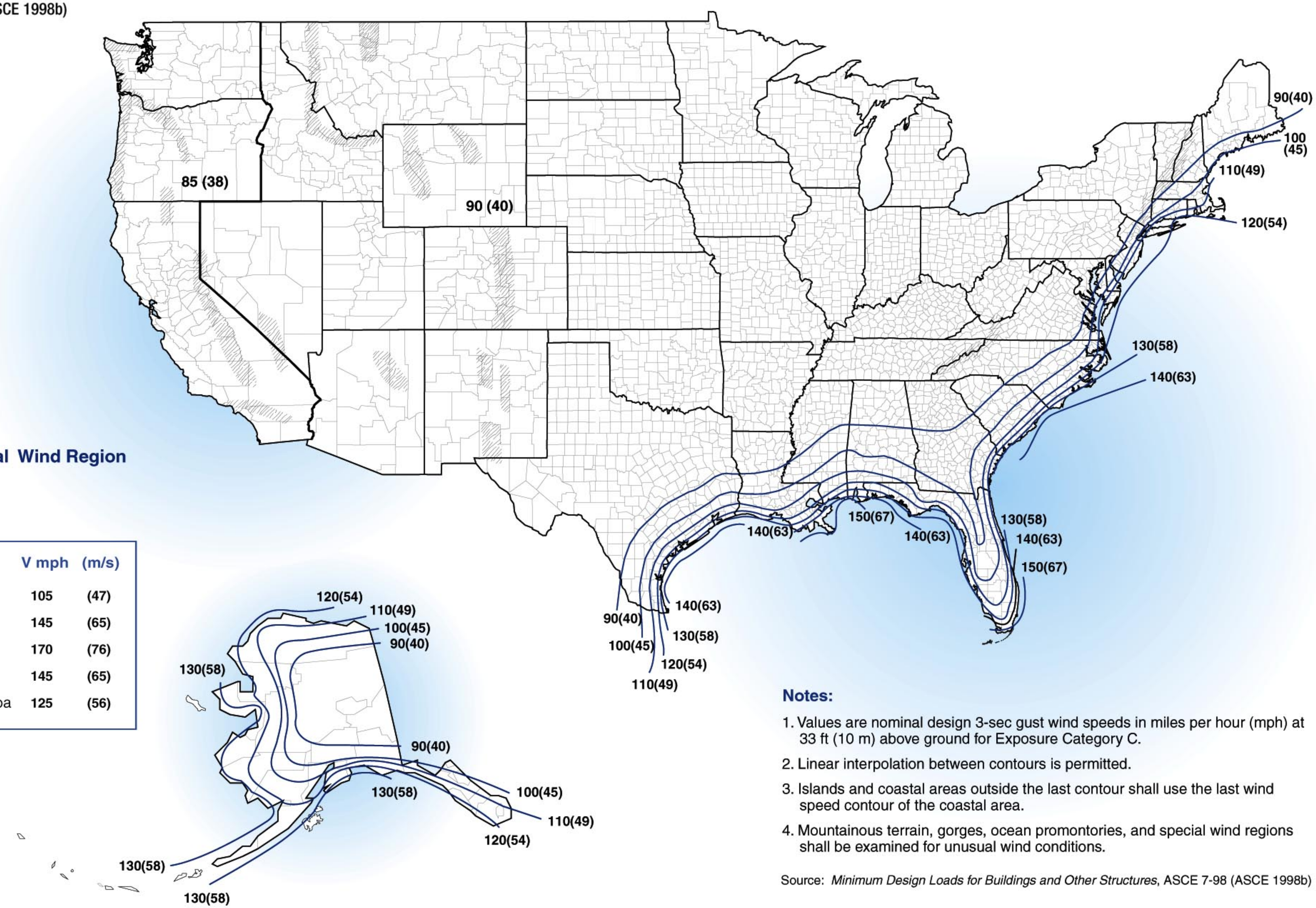
The MWFRS consists of the foundation, floor supports (e.g., joists, beams), columns, roof rafters or trusses, and bracing, walls, and diaphragms that assist in transferring loads. ASCE 7-98 (ASCE 1998b) defines the MWFRS as "… an assemblage of structural elements assigned to provide support and stability for the overall structure." The commentary in ASCE 7-98 suggests that the components of roof trusses be analyzed for loads based on components and cladding coefficients and that the truss, as a single element, be analyzed for loads as part of the MWFRS. The designer needs to consider appropriate loadings based on the type of building component used in the MWFRS.

The following procedure is suggested as a means of determining the net wind pressures that must be considered. It will help the designer use ASCE 7-98 effectively.

**Wind Load Determination Procedure**

| | |
|---|---|
| STEP 1 | Determine the wind speed from the map shown in Figure 11-18, on pages 11-38 and 11-39. (More detailed maps for Atlantic and Gulf of Mexico coasts are included in Figures 6.1a through 6.1c of ASCE 7-98.) |
| STEP 2 | Define the building as either open, partially enclosed, or enclosed. |
| STEP 3 | Determine the Exposure Category: A, B, C, or D (see ASCE 7-98). |
| STEP 4 | Determine the Importance Factor $I$ and the topographical influence factor $K_{zt.}$ |
| STEP 5 | Determine the velocity pressure at the appropriate mean roof height. |
| STEP 6 | Select appropriate internal and external pressure coefficients. |
| STEP 7 | Determine the design pressures (all pressures should be net pressure; use + to indicate inward-acting pressure and – to indicate outward-acting pressure). |
| STEP 8 | Apply the design pressure to the appropriate tributary area for the element or assembly under consideration. |

**Figure 11-18**
Wind speed map (ASCE 1998b)




| Location | V mph | (m/s) |
|---|---|---|
| Hawaii | 105 | (47) |
| Puerto Rico | 145 | (65) |
| Guam | 170 | (76) |
| Virgin Islands | 145 | (65) |
| American Samoa | 125 | (56) |

**Notes:**

1. Values are nominal design 3-sec gust wind speeds in miles per hour (mph) at 33 ft (10 m) above ground for Exposure Category C.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Source: *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98 (ASCE 1998b)

A complete analysis of the MWFRS requires that the building be exposed to wind coming from each of four principal directions. For a typical building configuration, this means that in two cases the wind direction will be perpendicular to the roof ridge, and in the other two cases it will be parallel to the ridge. A complete analysis includes a determination of the following:

- windward and leeward wall pressures
- side wall pressures
- windward and leeward roof pressures
- pressures on roof overhangs and porches

Figure 11-19 shows the distribution of these pressures on the building that appears in the Wind Load Example Problem on pages 11-45 through 11-47.

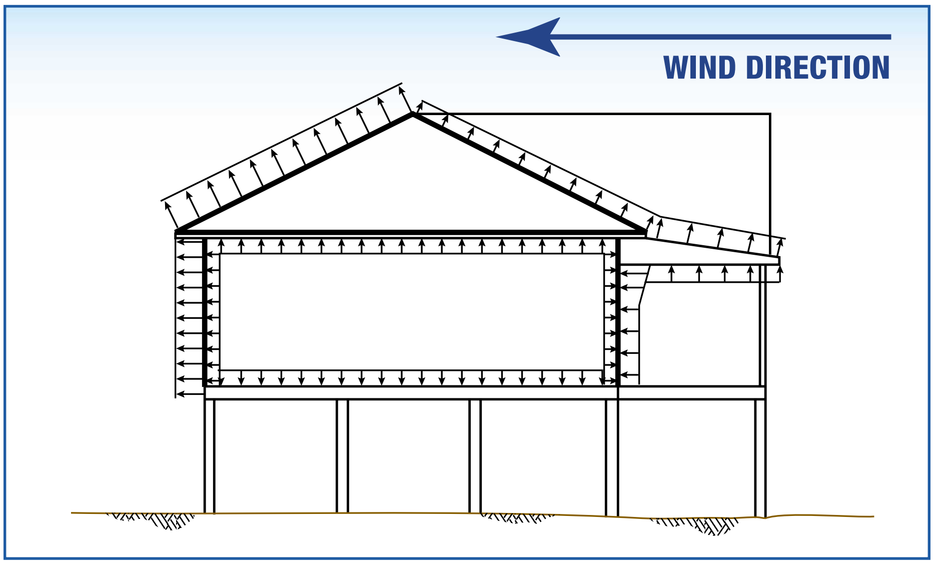



**Figure 11-19**
Wind pressures on the MWFRS on a cross section of the case study building.

**NOTE**

The windward side of the roof may receive positive (inward) pressure depending on the roof slope.

## 11.8.2 Components and Cladding

ASCE 7-98 defines components and cladding 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, facia, and chimneys. **The design and installation of the roof sheathing attachment may be the most critical consideration,** because the attachment point is where the uplift load path begins (see Chapter 12 for more information on load paths).

Component and cladding pressures are determined for various "zones" of the building. ASCE 7-98 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 7-98). Pressures at building corners are based on the effective wind area at the corner expressed as a percentage of the building length or width or as a minimum dimension.

Figure 11-20 shows the locations and relative magnitudes of localized pressures on wall and roof surfaces. These pressures are shown in Figure 6-5 of ASCE 7-98.

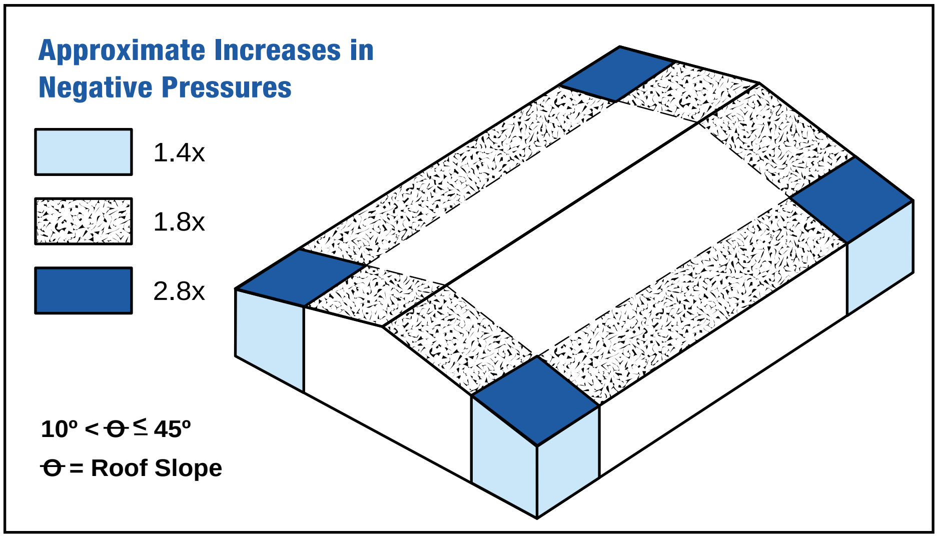



**Figure 11-20**
Wind pressures on the MWFRS.

It is important to note that the effective wind area for cladding fasteners is the area of the cladding attached by one fastener, so the area is very small. Pressure coefficients are highest for small areas as shown by the figures in ASCE 7-98.

The determination of wind pressures on components and cladding is illustrated in the Wind Load Example Problem on pages 11-45 through 11-47.

Residential buildings, particularly wood-frame and steel-frame buildings, are constructed with many small pieces, all of which are joined with some type of fastener such as a nail, screw, or mechanical connector. A failure of any one fastener increases the load on adjacent fasteners. The cyclic nature of wind forces can cause fatigue failure. For the designer, this failure mode suggests that design loads should be stipulated for the following:

- exterior siding, including the expected shear and withdrawal loads on fasteners
- roof sheathing
- wall sheathing
- roof coverings
- soffits and overhangs
- windows and window frames
- doors and door frames, including garage doors
- any attachments to the building (e.g., antennas, chimneys)

Figure 11-21 shows some of the many pieces that must be connected in a small section of a wood-frame residential building. A complete design would include load requirements for all of the connections necessary for the construction of this and all other wall sections in the building.

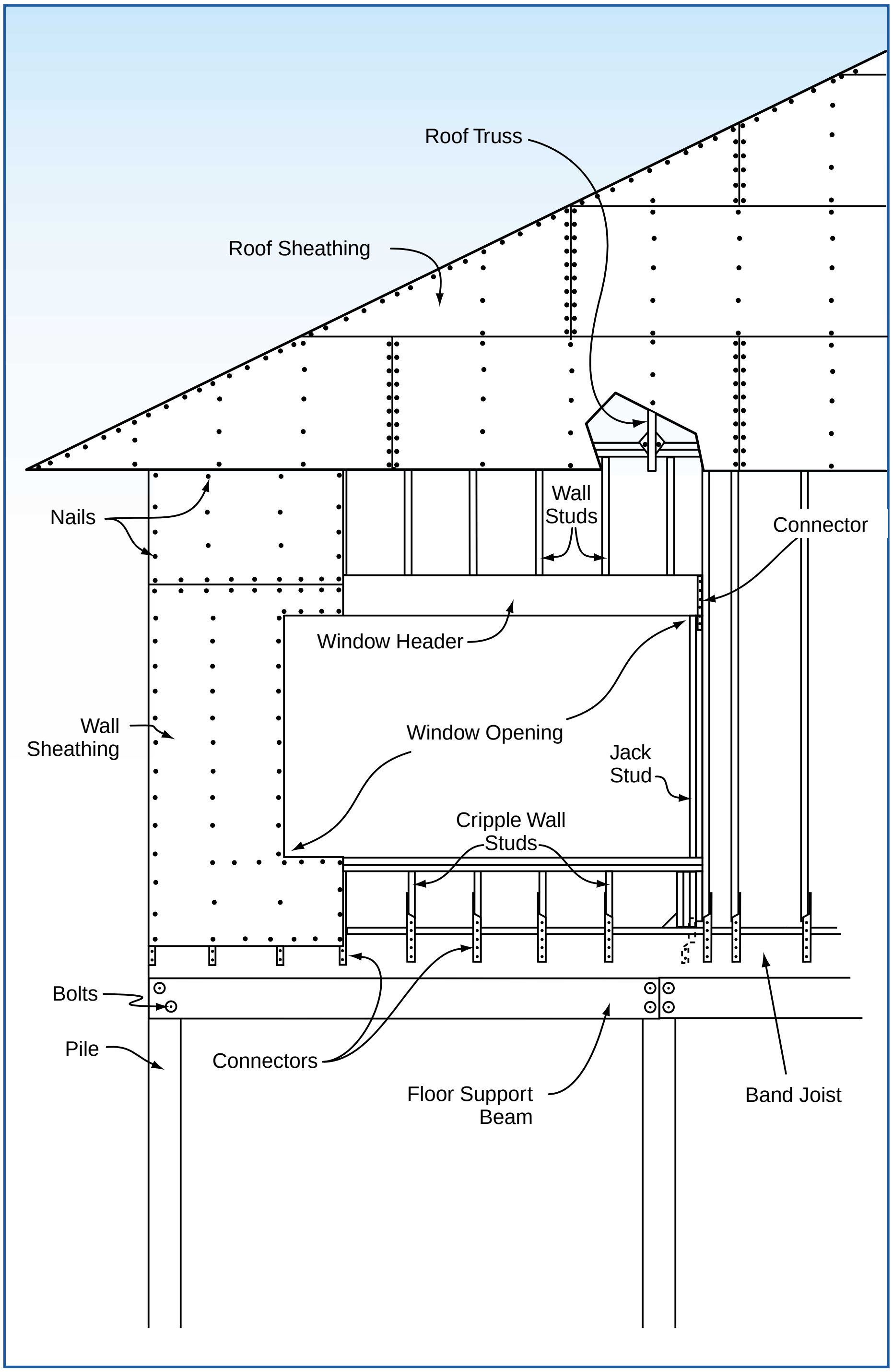



**Figure 11-21**
Typical building connections.

# Example Wind Load Example Problem

## Given:

1. Wind direction is perpendicular to the roof ridge (see Figure 11-22).
2. Wind speed is 120 mph (3-sec peak gust).
3. DFE = 14 feet NGVD
4. Building is one-story; wall height = 10 feet.
5. Main roof pitch is 6:12.
6. Exposure is Category C (refer to ASCE 7-98).
7. The following values are determined from ASCE 7-98:

   $K_d$ = 0.85

   $K_z$ = 0.93 (from interpolation of Table 6-5 in ASCE 7-98)

   $K_{zt}$ = 1.0

   G = 0.878 (calculated using Equation 6-2 in ASCE 7-98)

   Mean roof height (h) =24.0 feet above ground

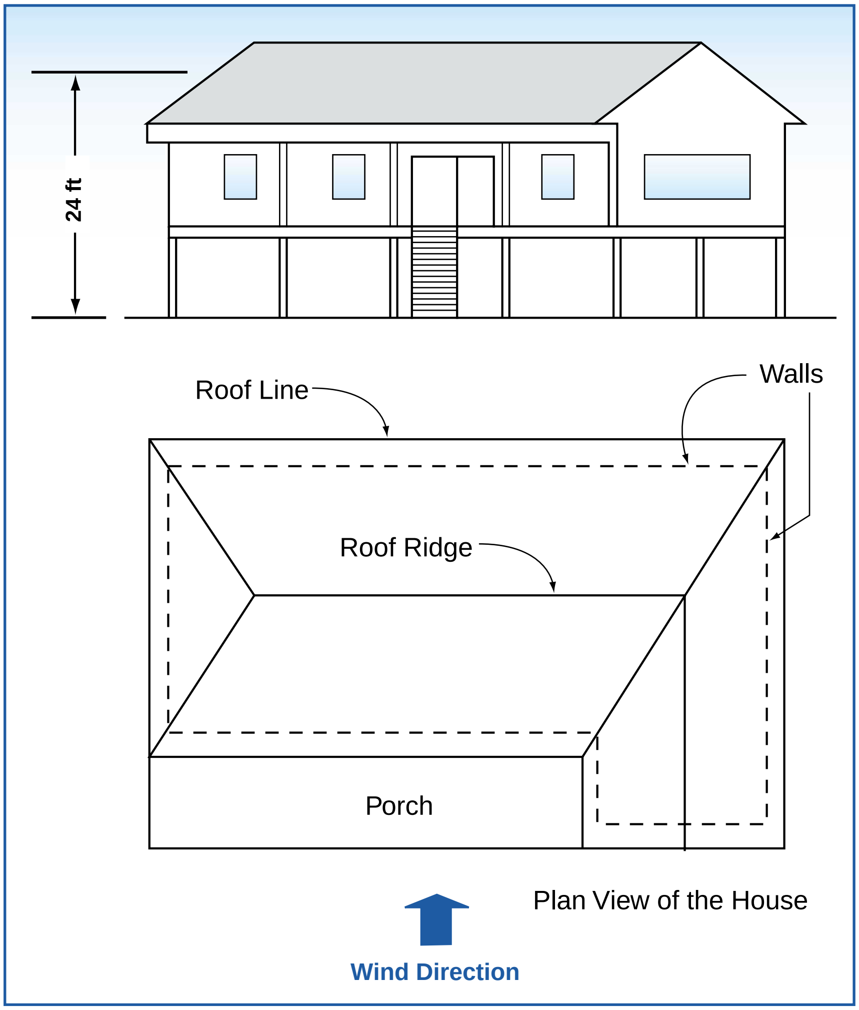



**Figure 11-22** Building elevation and plan view of roof.

8. The building is "enclosed."
9. The building is in a V zone within 1,600 feet of the shoreline.

## Find:

1. design pressures for the MWFRS (including the porch roof):
   a) for wind perpendicular to the main roof ridge (east-west)
   b) for wind parallel to the main roof ridge (north-south)
2. design pressures for the components and cladding for an assumed effective area of 20 $ft^2$

**Note:** Different elements may have different effective wind areas (e.g., large windows vs. siding fasteners). Calculations should be based on appropriate effective wind areas.

# Example Wind Load Example Problem (continued)

## 1 Wind pressure for the MWFRS:

$p = q_z(GC_p) - q_h(GC_{pi})$ [See ASCE 7-98 for a description of these terms.]

The wind pressure zones from Figure 6-3 in ASCE 7-98 are shown in Figure 11-23.
The computed pressures are listed in Table 11.7

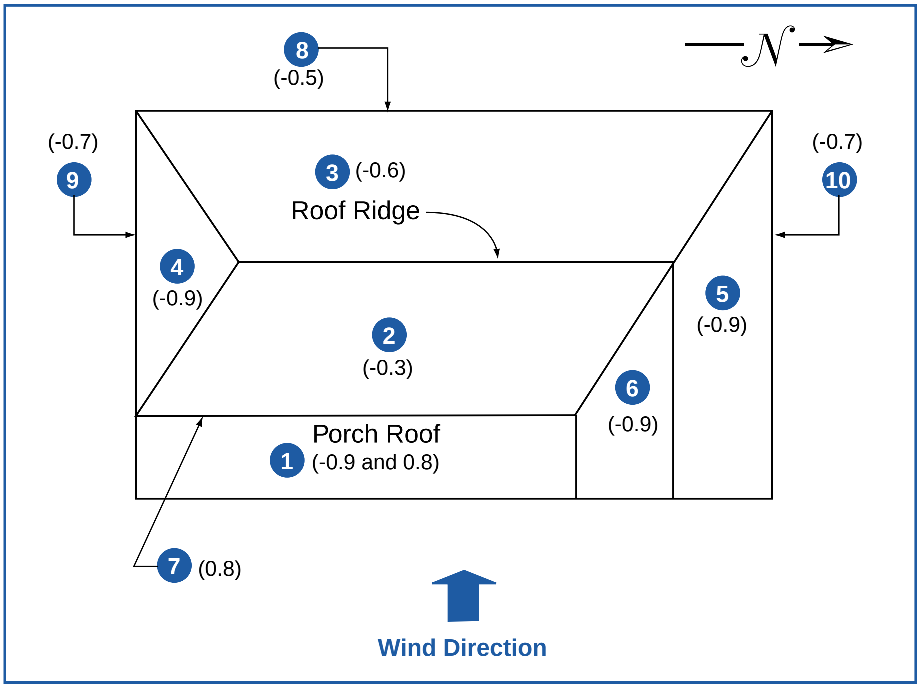



**Figure 11-23**
Wind pressure zones for MWFRS. The external pressure coefficients ($C_p$) for the 10 zones indicated are provided here for the case of wind direction from the east.

**Table 11.7**
Summary of Wind Pressures (p) for MWFRS

| Wind Pressure Zone (see Figure 11-23) | Wind Direction East | West | North | South |
|---|---|---|---|---|
| 1 Porch overhang | **-45.45** | -12.30 | -22.14 | **-45.45** |
| 2 Front roof | -12.63 | -20.01 | **-27.39** | **-27.39** |
| 3 Rear roof | -20.01 | -12.63 | **-27.39** | **-27.39** |
| 4 Left hip roof | **-27.39** | **-27.39** | -20.01 | 4.60 |
| 5 Right hip roof | **-27.39** | **-27.39** | 4.60 | -20.01 |
| 6 Left front gable | **-27.39** | **-27.39** | -20.01 | 4.60 |
| 7 Front wall | 14.44 | -17.55 | **-22.47** | **-22.47** |
| 8 Rear wall | -17.55 | 14.44 | **-22.47** | **-22.47** |
| 9 Left side wall | **-22.47** | **-22.47** | -15.09 | 14.44 |
| 10 Right side wall | **-22.47** | **-22.47** | 14.44 | -15.09 |

**Notes:**

(1) Locations on the building are taken from the east, facing the building.

(2) All pressures are in lbs/ft$^2$ and have been rounded to the nearest two decimal places.

(3) + pressures act toward the building.

(4) – pressures act away from the building.

(5) Highest pressures are indicated in bold.

# Example Wind Load Example Problem (continued)

**2 Wind pressures for components and cladding:**

$p = q_h [(GC_p) - (GC_{pi}c]$

The wind pressure zones for components and cladding are shown in Figure 11-24.

The computed pressures are listed in Table 11.8.

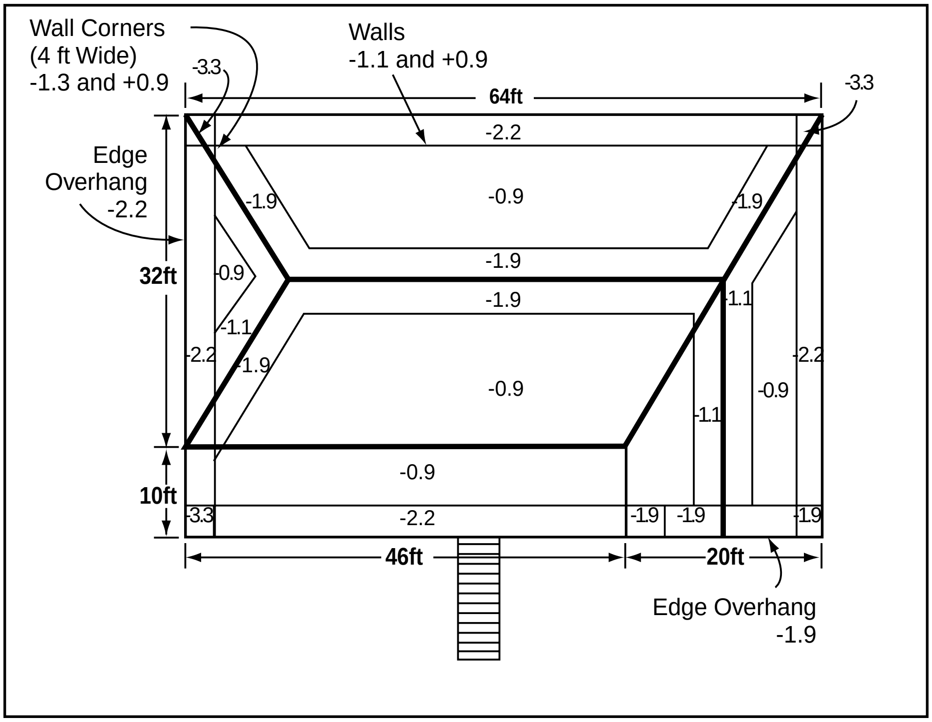



**Figure 11-24**
Wind pressure zones for components and cladding; most severe negative pressure coefficients shown.

**Table 11.8**
Summary of Wind Pressures for Components and Cladding

| Wind Pressure Zone (see Figure 11-24) | $GC_p$ Coefficients | Wind Pressure ($lb/ft^2$) |
|---|---|---|
| Corners of hip roof and front porch | -3.3 | -119.48 |
| Interior of hip and porch roofs | -0.9 | -31.47 |
| Steep-sloped hip and gable roofs | -0.9 | -31.47 |
| Hip roof overhang edge | -2.2 | -87.42 |
| Steep hip and gable and edges | -1.9 | -78.68 |
| Wall corner | -1.3 | -43.13 |
| Main wall area | -1.1 | -37.3 |
| Porch roof edge | -2.2 | -69.36 |

**Notes:**

1. All pressures are in $lbs/ft^2$ and have been rounded to the nearest two decimal places.
2. + pressures act in toward the building.
3. – pressures act away from the building.
4. Effective area of 20 $ft^2$ is assumed.

## 11.9 Tornado Loads

Tornadoes are frequently spawned by hurricanes and other severe weather systems, but tornado wind speeds can be much greater than hurricane wind speeds (see Commentary, Section C6.5.4.3 of ASCE 7-98). ASCE 7-98 specifically excludes tornadoes from consideration in the basic wind speed distributions. Designing an entire building to resist tornado-force winds of F2 or greater on the Fujita Scale is usually beyond the realm of practicality and cost-effectiveness. A more practical approach is to consider constructing an interior room or space that is specifically "hardened" to resist not only tornado-force winds, but also the impact of windborne missiles.

FEMA Publication No. 320, *Taking Shelter From the Storm: Building a Safe Room in Your House* (FEMA 1998), provides general information about tornado and hurricane hazards and includes risk assessment aids. It also includes detailed construction plans for several types of shelters that are designed to provide protection from both extremely high winds and windborne missiles. The research and materials testing work performed for this project were conducted by the Wind Engineering Research Center at Texas Tech University.

## 11.10 Seismic Loads

The 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 1997), published by the Building Seismic Safety Council for FEMA, is considered the state-of-the-art seismic design standard. The 1997 NEHRP (National Earthquake Hazard Reduction Program) Provisions served as the basis for the seismic provisions in the IBC 2000 (ICC 2000a) and the IRC 2000 (ICC 2000b), and are similar in many respects to the seismic provisions of the 1997 *Uniform Building Code*. The seismic provisions in the 1993, 1996, and 1999 editions of *The BOCA National Building Code* and the 1994 and 1997 editions of the *Standard Building Code* are based on the 1991 NEHRP Provisions.

**NOTE**

**The IBC 2000 includes provisions that exempt detached one- and two-family dwellings located in areas where $S_{DS}$ is < 0.4 from the seismic design requirements of the IBC. Designers should consult local officials to clarify specific design requirements.**

This manual uses the seismic provisions of the IBC 2000 to illustrate the seismic design procedure. Because the procedure is complex and is covered thoroughly in the two documents referred to above, as well as in the other model building codes, this manual will present only the basic steps of the procedure. It is important to understand the basic principles of seismic design because some are in conflict with the design requirements for other natural hazards. Finding ways to resolve these design conflicts is an important task for the designer.

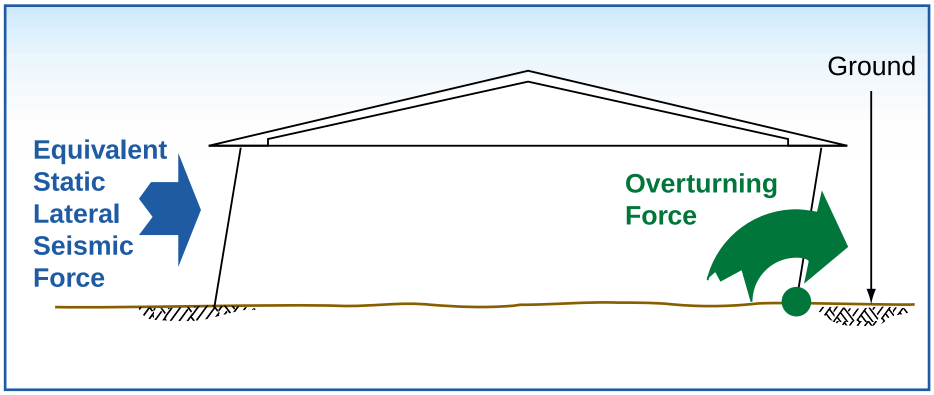
Specific seismic design requirements and calculation methods are presented later in this section, but first, a short discussion of seismic theory will help explain the effects of seismic ground shaking motion on buildings. These effects can be summarized as follows:

- Ground motion during a seismic event is both lateral and vertical, thus the motion of the affected building is also both lateral and vertical.
- 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 site location, the site soil properties, and the building characteristics.
- The ground shaking motion immediately displaces the foundation of the building more than the building mass above the foundation. As a result, the difference between the immediate displacement of the foundation and that of the rest of the building causes deformation and stress in the supporting elements of the building. This is an important consideration for buildings elevated on pile foundations. Because of the height of such buildings and the slenderness of the foundation members, the seismic demands on the foundation of an elevated pile-supported building can be greater than those on the foundation of a ground-level building.
- Irregularities in the shape, mass, and structural stiffness of a building will cause non-uniform displacements when the building mass moves.
- Acceptable building performance for life safety during a major seismic event is defined as non-collapse; some structural and non-structural building damage is acceptable if it does not prevent egress from the building.
- Actual seismic forces can exceed the code design forces; therefore, the ductility of building elements and connections is important. Ductility is the ability to sustain large deformations without losing strength.

Figures 11-25 through 11-27 show how ground motion resulting from a seismic event causes buildings to move and leads to failures.

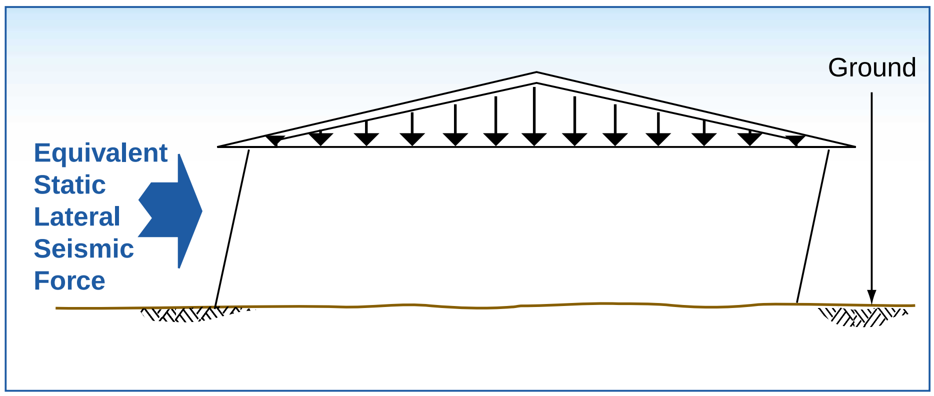
The seismic shaking creates shear and overturning forces and deformation in the walls between the floor and roof line (see Figure 11-25). The entire load path from the roof through the walls and into the foundation must have the capacity to withstand these forces. All of the loaded elements must be able to withstand the force and deformation without failing.

**Figure 11-25**
Shear and overturning forces.



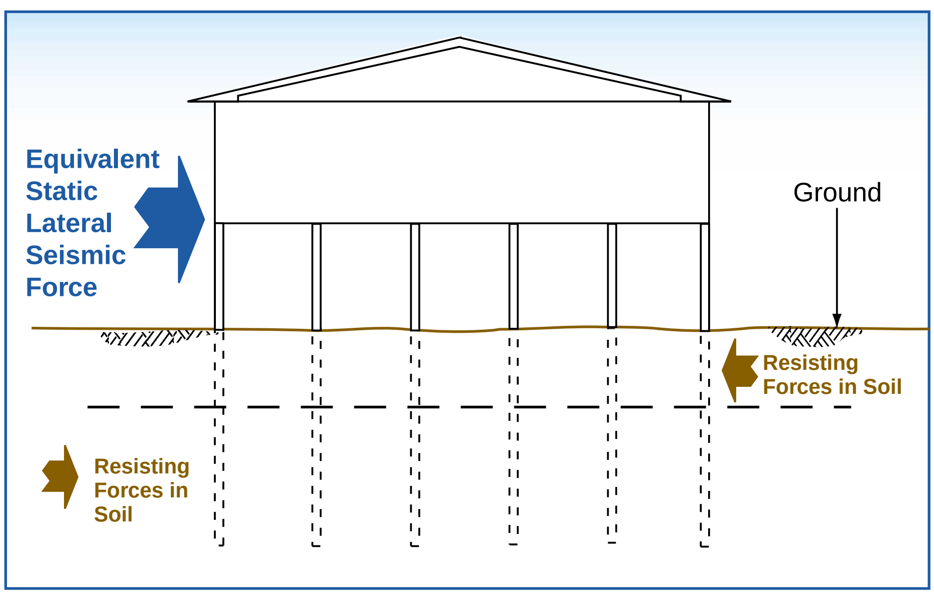

Once the structural frame becomes out of plumb, vertical gravity force can deform it further (see Figure 11-26). This is known as the P-delta effect.

**Figure 11-26**
Gravity further deforms the out-of-plumb frame of the building.




The lateral force on a cantilevered pile-supported building causes the piles to bear against the upper soil in the direction opposite the force and against the lower soil in the direction of the force (see Figure 11-27).

**Figure 11-27**
Effect of seismic forces on supporting piles.



The following is a suggested seismic design procedure in accordance with the Simplified Analysis Procedure for the Seismic Design of Buildings in Chapter 16 of the IBC. The suggested procedure is similar to the wind design procedure in that it involves an evaluation of forces that act in more than one dimension.

## Seismic Load Determination Procedure

| | |
|---|---|
| STEP 1 | Determine the site acceleration values from the Maximum Considered Earthquake Ground Motion maps in Chapter 16 of the IBC. |
| STEP 2 | Determine the Site Class, based on site soil characteristics. |
| STEP 3 | Select an appropriate seismic-force-resisting structural system. |
| STEP 4 | Estimate the dead weight of the building by level. |
| STEP 5 | Determine the additional loads (e.g., snow, storage, equipment) that must be added to the weight of the building in the calculation of the Effective Seismic Weight of the building. |
| STEP 6 | Determine the total seismic force or base shear. |
| STEP 7 | Determine the seismic force at each building level. |
| STEP 8 | Distribute the seismic force into the foundation. |

These steps are described in somewhat more detail below. The complete execution of the steps would require the use of the IBC.

### STEP 1

Using the Maximum Considered Earthquake Ground Motion maps in the Earthquake Loads section of Chapter 16 of the IBC, find the short-period maximum considered earthquake spectral response seismic acceleration for the building location.

### STEP 2

The Site Class, in accordance with the Site Class Definitions table in Chapter 16 of the IBC, can be determined with a geotechnical analysis that measures the site soil shear wave velocity, penetration resistance, and/or unconfined shear strength. The IBC includes guidelines for assuming a Site Class if a geotechnical analysis is not performed.

**STEP 3**

Using the preliminary plans from the building designer, select the structural seismic-force-resisting system or systems in accordance with the Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems table in the IBC. This information is necessary for determining the Response Modification Coefficient R in Step 6.

**STEP 4**

The IBC and many building construction texts provide guidance for determining the dead weight of the building. The dead weight must then be distributed to the appropriate levels of the building (e.g., roof, each story)

**STEP 5**

The IBC states to what degree other loads (e.g., snow, storage, equipment) must be added to the dead weight of the building in the calculation of the Effective Seismic Weight of the building.

**STEP 7**

The determination of the total seismic force, or base shear, is described fully in the IBC and the NEHRP Provisions. The force must be determined for each plan direction. The Simplified Analysis Procedure from the IBC can often be used on smaller coastal buildings such as the residential buildings for which this manual is intended. Larger and more complex buildings will require a more complex analysis procedure.

The analysis procedures require that the Response Modification Coefficient R be determined, based on the structural system assumed in Step 3. Total seismic base shear is then computed with Formula 11.11.

formula

Seismic Base Shear by the Simplified Analysis Procedure

**Formula 11.11 Seismic Base Shear by the Simplified Analysis Procedure**

$$V = 1.2S_{DS}W/R$$

where: $S_{DS}$ = Design Spectral Response Acceleration, calculated using the short-period maximum considered earthquake spectral response seismic acceleration from Step 1 and the Site Class from Step 2

**W** = Effective Seismic Weight of the building in lb from Steps 4 and 5.

**R** = Response Modification Coefficient of structural system

All base shear methods require that the Response Modification Coefficient R be determined. The Response Modification Coefficient is an indicator of structural system seismic behavior. The larger the coefficient, the greater the system ductility and damping. The Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems table in the IBC lists coefficients by structural system, but the descriptions of building frame types in the table do not adequately cover typical coastal pile-supported structural systems. The following additional structural systems with recommended R values have been considered by the Seismology Committee of the Structural Engineers Association of California and the appropriate NEHRP committee.

- Cantilevered piles supporting a light-frame shear-panel building of one to three stories. The piles are sufficiently embedded in the soil to be moment-resisting at the base. **Recommended R = 2.2**
- Diagonal-braced piles supporting a light-frame shear-panel building of one to three stories. The diagonal braces are tension-only braces, and the piles are sufficiently embedded in the soil to be moment-resisting at the base. **Recommended R = 2.8**
- Pole construction in which the pile/pole extends vertically into a light-frame shear-panel building of one to three stories. The drift restraint provided by the shear walls serves to fix the rotation at the top of the piles/poles. The piles are sufficiently embedded in the soil to be moment-resisting at the base. **Recommended R = 4.5**
- Pole construction in which the pile/pole extends vertically into a light-frame shear-panel building of one to three stories. The drift restraint provided by the shear walls serves to fix the rotation at the top of the piles/poles. The depth of embedment is not sufficient to provide moment-resistance at the base. **Recommended R = 2.8**
- Knee-braced piles supporting a light-frame shear-panel building of one to three stories. The piles are sufficiently embedded in the soil to be moment-resisting at the base. The heavy timber knee braces need not carry gravity loads. **Recommended R = 4.5**

A typical coastal building can have one structural seismic-force-resisting system in the elevated foundation and a different structural seismic-force-resisting system for the living spaces. The IBC allows the forces to each system to be calculated separately by the appropriate R value, as long as the superstructure R value is equal to or greater than the foundation R value.

**STEP 7**

Determine the lateral force at each building level. The simplified method presented below (from the IBC) may be used for smaller coastal buildings. Larger buildings, as defined in the IBC, require a more complex analysis.

The distribution of the total horizontal force (shear) into each story of the building is determined by Formula 11.12.

Formula
Vertical Distribution of Seismic Forces

**Formula 11.12**
**Vertical Distribution of Seismic Forces**

$$F_x = 1.2S_{DS}w_x/R$$

where: $S_{DS}$ = (See Formula 11.11)
$F_x$ = force at level x
$w_x$ = portion of the effective seismic weight in lb at level x
$R$ = Response Modification Coefficient for structural system

One of the primary considerations for seismic design in coastal buildings is the structural configuration seismic engineers call an "inverted pendulum." This configuration occurs in elevated pile-supported buildings, where almost all of the weight is at the top of the piles; therefore, almost all of the horizontal shear occurs at the top of the piles. This configuration creates a large overturning moment (force times moment arm or distance above the base) in the overall pile foundation system. It also creates large bending forces and deflection in the individual cantilevered piles, creating a "soft story" effect. Low R values are typically assigned to inverted pendulum structural systems. The problem of designing for this configuration is discussed further in Chapter 12 of this manual.

**STEP 8**

The principles of seismic load determination and wind load determination are similar, but seismic forces rely on building ductility and damping. Although the design wind force may be larger than the design seismic force and thus may govern the lateral force design, seismic design requirements for ductility must still be met.

The calculated seismic or wind force at each story must be distributed into the building frame. The horizontal forces and related overturning moments will be taken into the foundation through a load path of horizontal floor and roof diaphragms, shear walls, bracing, shear connections, and tiedowns.

The total shear force distributed to the bottom floor diaphragm is usually transferred through shear walls. This shear force can be distributed across the floor diaphragm into the overall pile foundation. The shear wall overturning moments must be taken into the floor framing and piles directly below the shear walls. See the Seismic Load Example Problem (on pages 11-55 and 11-56) and Chapter 12 for more information.

# Example Seismic Load Example Problem

**Given:**

1. $S_{MS}$ for the site is $F_aS_S$, which is determined to be (1.2)(0.50) = 0.6.

2. The longitudinal shear walls are the two exterior side walls and one interior wall as shown in Figure 11-28. Dead load for the building is as follows:

   Roof and ceiling – 10 lb/ft$^2$ (roof overhang considered)

   Exterior walls – 10 lb/ft$^2$

   Interior Walls – 8 lb/ft$^2$

   Floor – 10 lb/ft$^2$

   Piles – 409 lb each

3. No live load is added.

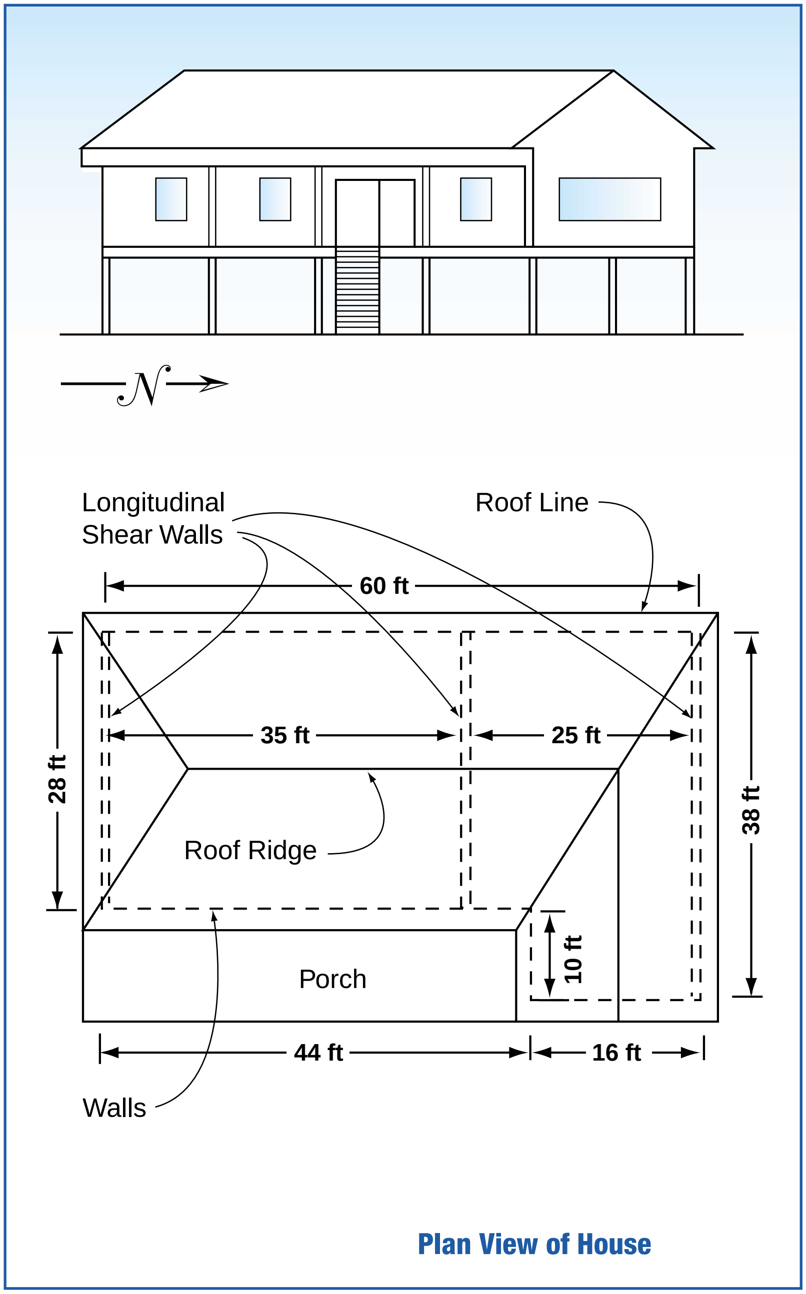



**Figure 11-28**
Building elevation and plan view of roof showing longitudinal shearwalls. Dimensions are wall-to-wall and do not include 2-foot roof overhang.

**Find:**

**(by IBC Simplified Analysis Procedure):**

1. The maximum shear force in the 28-foot side shear wall.

2. The shear force to the pile foundation.

# Example Seismic Load Example Problem (continued)

**1 Maximum shear force in the 28-foot side shear wall:**

Calculate dead weight:

Roof, ceiling, top 1/2 of interior partitions = (10 lb/ft$^2$)(2,248 ft$^2$) + 1/2 (8 lb/ft$^2$)(2,000 ft$^2$) = 30,480 lb

Exterior walls = (10 lb/ft$^2$)(2,088ft$^2$) = 20,880 lb

Top 1/2 of piles, floor framing, floor, bottom 1/2 of interior partitions = 1/2 (409 lb/pile) (31 piles) + (10lb/ft$^2$)(1,840 ft$^2$) + 1/2 (8 lb/ft$^2$)(2,000 ft$^2$) = 32,740 lb

**Total dead weight (Effective Seismic Weight) = 84,100 lb**

$S_{DS}$ = 2/3$S_{MS}$ = (2/3)(0.6) = 0.402, as a percent of the acceleration of gravity

R = 6.5 for light-frame walls with plywood

Force in shear walls is from roof-level weight. This is the weight tributary to the roof level plus 1/2 of the exterior wall weight.

$F_{roof}$ = 1.2$S_{DS}w_{roof}$ /R = [(1.2)(0.402)( 30,480 + (20,880/2))/6.5

**$F_{roof}$ = 3,037 lb**

The 28-foot side shear wall has two 10-foot windows, so $l_{wall}$ = 28 feet – (2x10 feet) = 8 feet

By tributary (simple horizontal beam) distribution among the three walls:

$F_{28\text{-foot wall}}$ = 3,037 lb x 17.5 feet/60 feet = 886 lb

So maximum shear in 28-foot wall, at pier panels beside windows = 886 lb/8 feet

**Maximum shear force = 111 lb/ft**

**2 Shear force to the pile foundation:**

From Step 6 on page 11-52 take R = 2.2 for cantilevered piles. For vertically mixed seismic-force-resisting systems, the draft IBC allows a lower R to be used below a higher R value.

Total dead weight (Effective Seismic Weight) = 84,100 lb

V = 1.2$S_{DS}$W/R = [(1.2)(0.402)(84,100)]/2.2

**V= 18,441 lb**

31 piles tied to floor diaphragm

So V/pile = 18,441 lb/31 piles

**V = 595 lb/pile**

## 11.11 Load Combinations

It is conceivable that more than one type of natural hazard will occur at the same time. It is clearly possible, for example, for a flood to occur at the same time as a high wind event—this happens during most hurricanes. In addition, it is possible to have very heavy rain at the same time as high winds and flooding conditions. ASCE 7-98 addresses the various load combination possibilities. As noted at the beginning of this section, this manual uses ASD as the design method of choice.

The ASD method uses nominal loads, usually without load factors, to compute stresses due to axial loads, bending moments, shears, etc. The induced stresses are considered acceptable if they do not exceed allowable stress values specified in the appropriate material design standard referenced in the building code. This method inherently includes a factor of safety, which is the ratio of the specified strength of the material to the actual induced stress.

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

| | |
|---|---|
| **D** | dead load |
| **L** | live load |
| **F** | load due to fluids with well-defined pressures and maximum heights (e.g., fluid load in tank) |
| $\mathbf{F_a}$ | flood load |
| **H** | loads due to weight and lateral pressures of soil and water in soil |
| **T** | self-straining force |
| $\mathbf{L_r}$ | roof live load |
| **S** | snow load |
| **R** | rain load |
| **W** | wind load |
| **E** | earthquake load |

When loads are combined with the ASD method, they are considered to act in the following combinations for buildings in V zones and coastal A zones (ASCE 7-98, Section 2.4.1), whichever produces the most unfavorable effect on the building or building element:

1. D
2. D + L + F + H + T + ($L_r$ or S or R)
3. D + (W or 0.7E) + L + ($L_r$ or S or R) +1.5$F_a$*
4. 0.6D + W + H + 1.5 $F_a$*
5. 0.6D + 0.7E + H

* For non-coastal A zones, the flood load shall be 0.75$F_a$ instead of 1.5$F_a$.

Note the following:

- In V zones, coastal A zones, and non-coastal A zones, E shall be set equal to 0 in combination 3, above.
- In areas not subject to flooding, combination 3 becomes D + (W or 0.7E) + L + ($L_r$ or S or R)
- In areas not subject to flooding, combination 4 becomes 0.6D + W + H

The *Commentary* in ASCE 7-98 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. In some instances, forces due to wind might exclude those due to earthquake, while ductility requirements might be determined by earthquake loads."

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 11.6.12. It is important to note that the load combinations discussed in this section must be resolved directionally, so that all loads in a given combination are acting in the same direction, either vertically or horizontally. See page 11-59 for an example of how to calculate the appropriate load combination.

# Example Load Combination Example Problem

## Determined Previously:

1. Flood loads (from page 11-33):
   $F_{sta} = 0$
   $F_{dyn} = 21{,}816$ lb
   $F_{brkp} = 4{,}375$ lb
   $F_i = 758$ lb

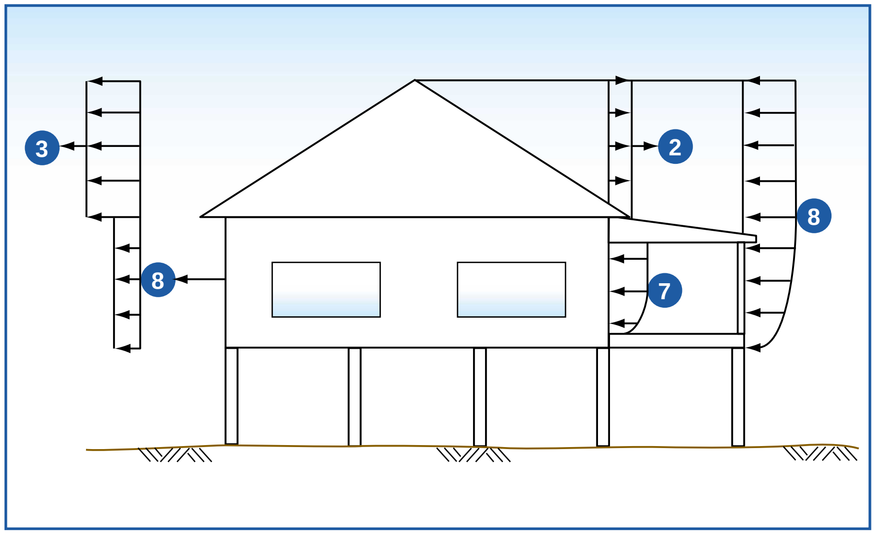



**Figure 11-29** Side view of building shown in Figure 11-23.

2. Horizontal wind loads (using the projected area method):

   $\Sigma$ wind forces to left = (pressure in each area)(area) or

   (p8)(A8) + (p8)(front gable wall area) + (p2)(A2) +(p3)(A3) + (p9)(A9) or

   $\Sigma\,(14.44\ \text{lb/ft}^2)(440\ \text{ft}^2) + (14.44\ \text{lb/ft}^2)(288\ \text{ft}^2)\ \cancel{-(12.63\ \text{lb/ft}^2)(352\ \text{ft}^2)} + (20.01\ \text{lb/ft}^2)(480\ \text{ft}^2) + (17.55\ \text{lb/ft}^2)(600\ \text{ft}^2) = 30{,}648\ \text{lb}$

3. Horizontal seismic shear force at foundation (from page 11-56):
   V = 18,441 lb above ground

## Find:

Total horizontal load required for foundation design.

1. D = 0 in horizontal direction

2. $D + L + F + H + T + (L_r \text{ or } S \text{ or } R)$
   0 + 0 + 0 + 0 + 0 = 0

3. $D + (W \text{ or } 0.7E) + L + (L_r \text{ or } S \text{ or } R) + 1.5F_a$
   $0 + [30{,}648 \text{ or } \cancel{(0.7)(18{,}441)}] + 0 + 0 + 1.5\,(21{,}816 + 4{,}375) = 69{,}934$ lb **(controls)**

4. $0.6D + W + H + 1.5F_a$
   0 + 30,648 + 0 + 39,286 = 69,934 lb **(controls)**

5. $0.6D + 0.7E + H$
   0 + 12,909 + 0 = 12,909 lb

## Load Combination Computation Worksheet

Owner Name: ______________________ Prepared by: ______________________

Address: ______________________ Date: ______________________

Property Location: ______________________

### Variables

D (dead load) =

L (live load) =

F (fluid load) =

$F_a$ (flood load) =

H (lateral soil and water in soil load) =

T (self-straining force) =

$L_r$ (roof live load) =

S (snow load) =

R (rain load) =

W (wind load) =

E (earthquake load) =

### Summary of Load Combinations:

**1.**

**2.**

**3.**

**4.**

**5.**

| Combination No. 1 |
|---|
| **D =** |

| Combination No. 2 |
|---|
| **D + L + F + H + T + ($L_r$ or S or R) =** |

| Combination No. 3 |
|---|
| **D + W + L + ($L_r$ or S or R) + 1.5$F_a$* =** |

| Combination No. 4 |
|---|
| **0.6D + W + H + 1.5$F_a$* =** |

| Combination No. 5 |
|---|
| **0.6D + 0.7E + H =** |

* Use 0.75$F_a$ for non-coastal A zones and 0 for all structures outside SFHAs.

**Note:** The load combinations calculated with this worksheet are arithmetic sums. The combinations **must** be broken down into horizontal and vertical components and summed in each direction.

## 11.12 References

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# Chapter 12: Designing the Building

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# Designing the Building

## 12.1 The Design Process

The design process begins with the loads that were determined from the formulas developed and discussed in Chapter 11. These loads then must be applied to the building; forces and stresses will be determined from these loads, and resistance to the forces and stresses in the form of connectors and materials will be selected. Figure 12-1 illustrates the general design process.

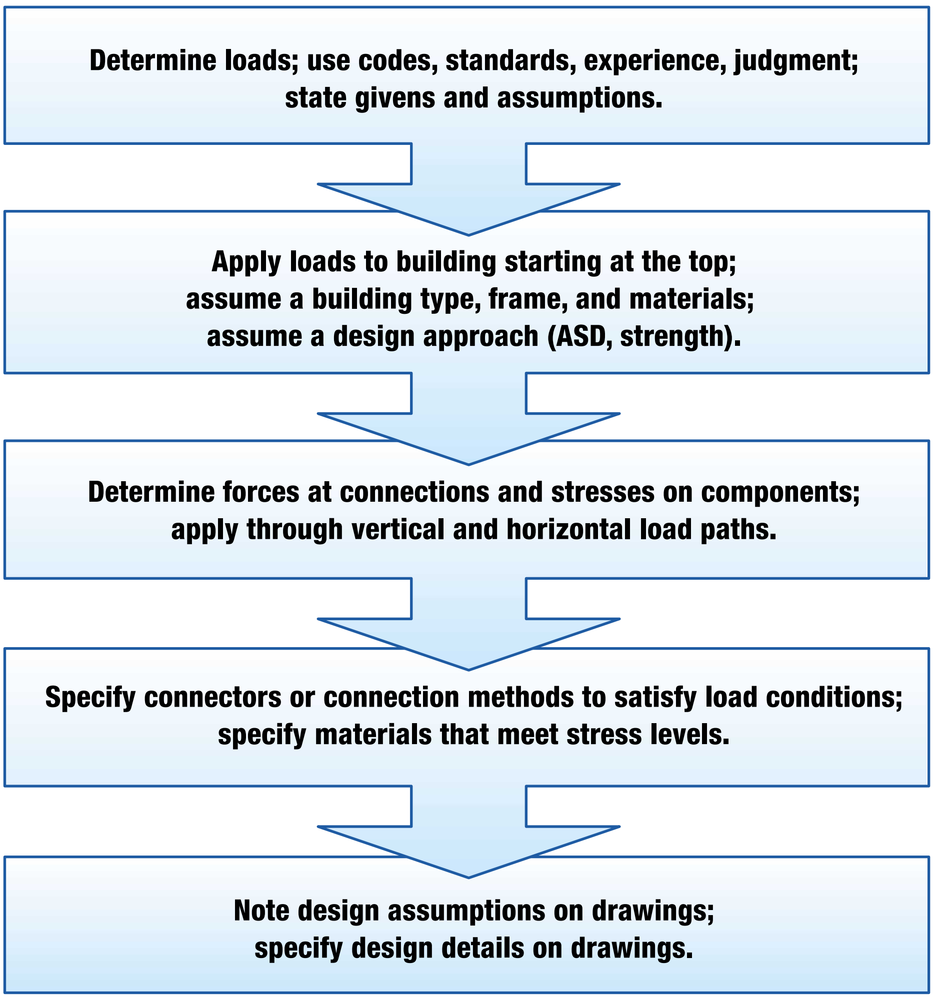



**Figure 12-1**
Coastal construction design process.

**NOTE**
Designers should consider the consequences of damage to, or the failure of, critical design components.

Each step of this process is covered as a separate section of this chapter. The many variations of designs, the location of buildings in different hazard areas, the varieties of building shapes, and other building parameters are discussed where appropriate in each of the design steps. A building located in either a V zone or a coastal A zone that is subject primarily to flood and wind hazards is used throughout this chapter as a demonstration of how to design a building in a coastal flood hazard area.

The design process involves the following:

- determining design loads
- determining the building's foundation, structural frame, and envelope
- determining the connections between individual elements
- determining the elevation, placement, and support for utilities
- selecting the appropriate materials

The entire design process is based on the fundamental premise that anticipated service **and natural hazard loads can and must be transferred through the building in a continuous path to the supporting soils. ANY** weakness in that continuous path is a potential point of failure of the building, and any failure creates the possibility for large property losses and the potential for loss of life.

This manual does not cover all of the almost endless number of combinations of loads, materials, building shapes and functions, hazard zones, and elevations. The designer will find that engineering judgment will need to be applied to a range of problems during the design of a coastal residential building. Therefore, it is the intent of this manual to provide sufficient background and examples so that a designer can effectively design a residential building for construction in a coastal hazard area.

**CROSS-REFERENCE**

Allowable Stress Design (ASD) is described in Section 11.11.

In this manual, the recommended design method is Allowable Stress Design (ASD), so there are factors of safety (FS) built into the development of the material stresses and the forces at the connections. This design method has been chosen for this manual because ASD continues to be the predominant design method in light-frame, residential, wood construction. Most suppliers of wood framing hardware and connectors provide load limits for their products with factors of safety built into the limits. Load and Resistance Factor Design (LRFD) guidance is available for wood if the designer prefers this ultimate strength or limit state design method.

**NOTE**

ASCE 7-98 addresses other loads (e.g., fluid, lateral earth pressure, rain) that may need to be considered depending on the nature of the construction project.

## 12.2 Step 1 – Determining Loads

The types of loads that most commonly act on one- to three-story residential buildings during severe natural hazard events are as follows:

- dead loads
- live loads
- flood loads
- wind loads
- earthquake (seismic) loads
- snow loads

Additional loads caused by long-term and short-term erosion and localized scour can play a significant part in the total loads that are imparted to the structure; therefore these conditions must be accounted for.

Load determination involves calculating each type of load. The most severe load combination required by the applicable building code or standard is then applied to the structure. Therefore, consideration must be given to the following loads and factors that affect loads:

**Dead loads –** The weight of the building and accessory equipment such as tanks, piping, electrical service panels and conduits, and HVAC equipment.

**Live loads –** Combined loads of occupants, furnishings, and non-fixed equipment.

**Flood loads –** Flood depth and velocity, wave effects, expected long-term and short-term erosion as well as localized scour, elevation of the building in relationship to the expected flood conditions, and floating debris impacts.

**Wind loads –** Roof shape and pitch, siting, topography and exposure, and building shape and orientation. The height of the structure also needs to be assessed.

**Seismic loads –** Mass (including elevation, location, and distribution) of the building, soil supporting the building, height of the building above the ground, and additional loads that the building may occasionally support (e.g., snow).

**Snow loads –** Roof shape and pitch, multi-level roofs, and building orientation. Also, drifting snow may cause unbalanced loading on the roof system.

An important part of design is deciding how these loads are imparted to the building. This means that the designer must decide where (and perhaps in what sequence) the loads are to be applied to the building.

## 12.3 Step 2 – Applying Loads to the Building

The following concepts show how one design step leads to the next:

- All design loads create forces in and on the building. The forces are transferred through load paths.
- Load paths always end in the soil that supports the structure.
- Loads should be applied to the building beginning at the top.
- Loads should be determined for both the vertical and horizontal load paths.

- Load transfer creates forces at connections and imparts stresses in the materials. Connections and materials must be strong enough to handle those forces and stresses.
- The load path must be continuous; any break or weakness in the load path "chain" can result in damage or even structural failure.

### 12.3.1 Failure Modes

Building failures most frequently occur by one or more of the following:

**Primary Failure Modes**

**Uplift:** Vertical forces caused by wind or buoyancy exceed the weight of the structure and the strength of the soil anchorage. The building fails by being lifted off its foundation or because the foundation pulls out of the soil.

**Overturning:** The applied moments caused by wind, wave, earthquake, and buoyancy forces exceed the resisting moments of the building's weight and anchorage. The building fails by rotating off its foundation or because the foundation rotates out of the soil.

**Sliding or Shearing:** Horizontal forces exceed the friction force or strength of the foundation. The building fails by sliding off its foundation, by shear failure of components transferring loads to its foundation, or by the foundation sliding.

**Secondary Failure Mode**

**Collapse:** Collapse is a secondary mode of failure. Structural components fail or become out of plumb or level under uplift, overturning, or sliding. The building then becomes unstable and collapses.

Buildings under extremely heavy vertical downward loads, such as snow, can also fail in bending, shear, or compression of primary structural members. For purposes of this manual, it is assumed the designer is familiar with the design concepts used to support these ordinary gravity loads.

#### 12.3.1.1 Uplift

Uplift failure occurs when the vertical forces are greater than the weight of the building, the strength of the structural frame (e.g., fasteners or connections), or the foundation anchorage. This type of failure can occur from high winds or buoyancy. Figure 12-2 illustrates how vertical uplift wind forces ($F_1 – F_6$) from the roof areas shown in Figure 12-16 pull on the structural components. The forces are the products of the pressures shown in Table 11.7, in Chapter 11, and the applicable area of the building.

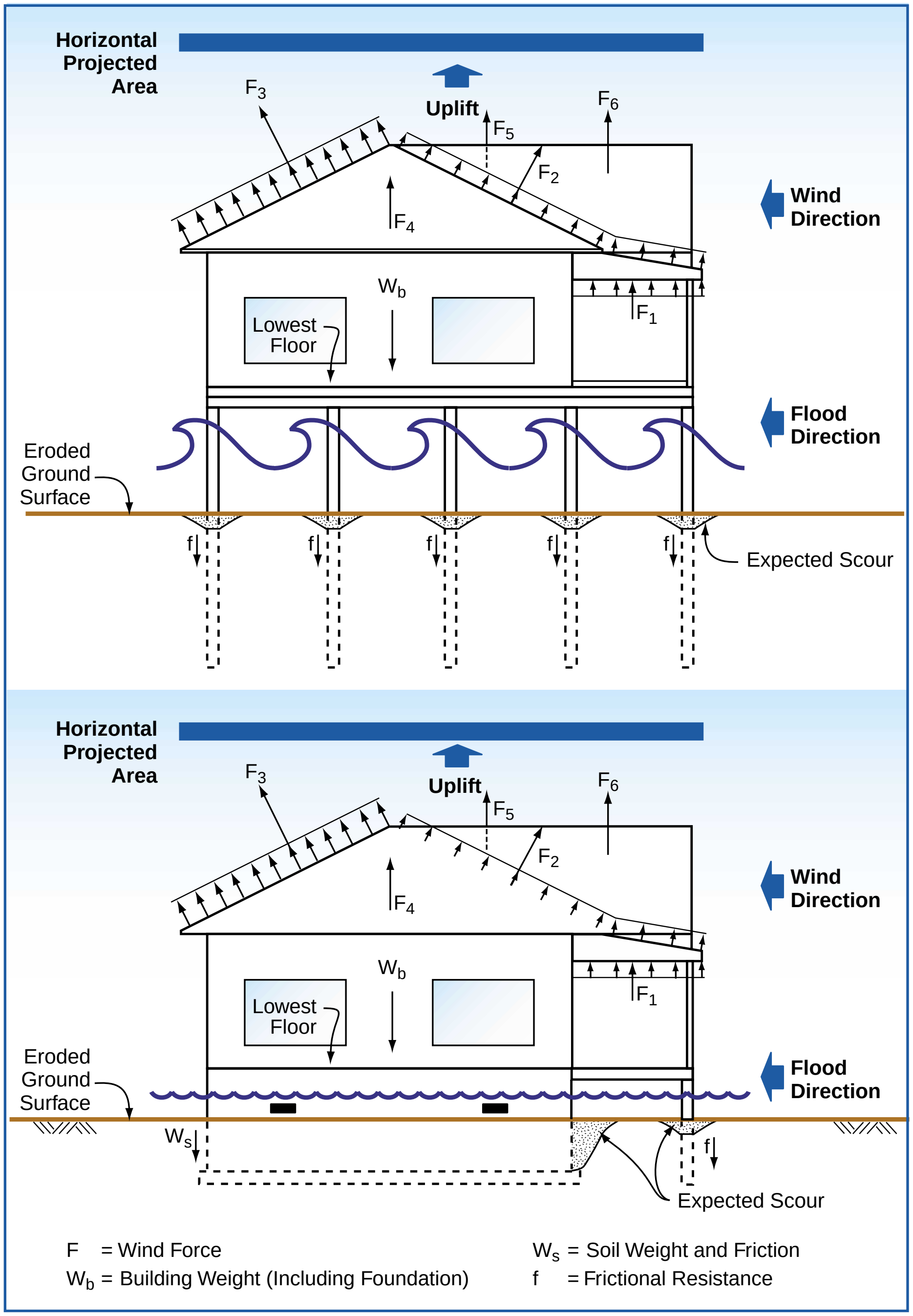



**Figure 12-2**
Uplift.

This manual will show how to calculate forces with the projected area method, which will simplify calculations. The horizontal projected area, shown above the buildings in Figure 12-2, is used for calculating uplift. The horizontal projected area for each roof segment must be multiplied by the pressure per unit area for that segment and all of the forces summed to arrive at a total uplift force. This method is similarly applicable to other failure modes.

Field investigations indicate that the failure of houses with wood-framed roofs often occurs first at the roof, often at improper fastening between the roof sheathing and building frame. Proper fastening of the roof sheathing is also important because there is little dead load at the roof to resist uplift. After wind and/or rain has entered the building (in a hurricane or other storm event), forces on other building components increase and cause additional failure.

The progressive nature of failures is illustrated in Figure 12-3, in which the collapsed trusses are an indication that, once the sheathing was removed, the trusses lost the lateral support required for stability and for resistance to lateral wind forces.

**Figure 12-3**
Hurricane Andrew (1992), Dade County, Florida. Roof structure failure due to inadequate bracing.

The progressive nature of uplift failure is further demonstrated in Figure 12-4. The sheathing is missing at the right end of the structure, and several trusses collapsed because of the loss of lateral support otherwise provided by the roof sheathing. The roof damage allowed the wind to enter the structure and pull the wall panel down. Obviously, when the structural failure progresses to the stage shown in this figure, significant interior damage comes from rain entering the building.

Note the eave overhang in Figure 12-4. Uplift failures frequently occur at wide overhangs. In addition to imposing uplift forces, or suction pressures, on the roof surface itself, wind pushes up on the roof sheathing from underneath the eaves or other overhangs. The combination of these forces can cause either a failure in the roof sheathing or a failure in the connection between the roof framing and the exterior wall.

**Figure 12-4**
Hurricane Andrew (1992), Dade County, Florida. Second-story wood framing (on first-story masonry). End gable and wall failure.

Porch roofs are very susceptible to uplift failure. They generally have a large surface area, are unprotected from the wind (and thus as open structures experience higher wind pressures), are relatively light, and are normally supported on widely spaced columns. The loss of the connection between the roof and its support causes failures like that shown in Figure 12-5. This figure demonstrates the importance of providing framing connections that can resist uplift.

**Figure 12-5**
Hurricane Diana (1984). Uplift failure of a porch roof.

Preventing wind from entering the building significantly reduces the potential for uplift failure and involves protecting the building envelope. Wind pressures on "partially enclosed" buildings are approximately 30 percent higher than wind pressures on enclosed buildings—see Table 6-2 in ASCE 7-98 (ASCE 1998b).

The openings of the house in Figure 12-6 were protected, and only minimal damage to the structure is evident in the photograph.

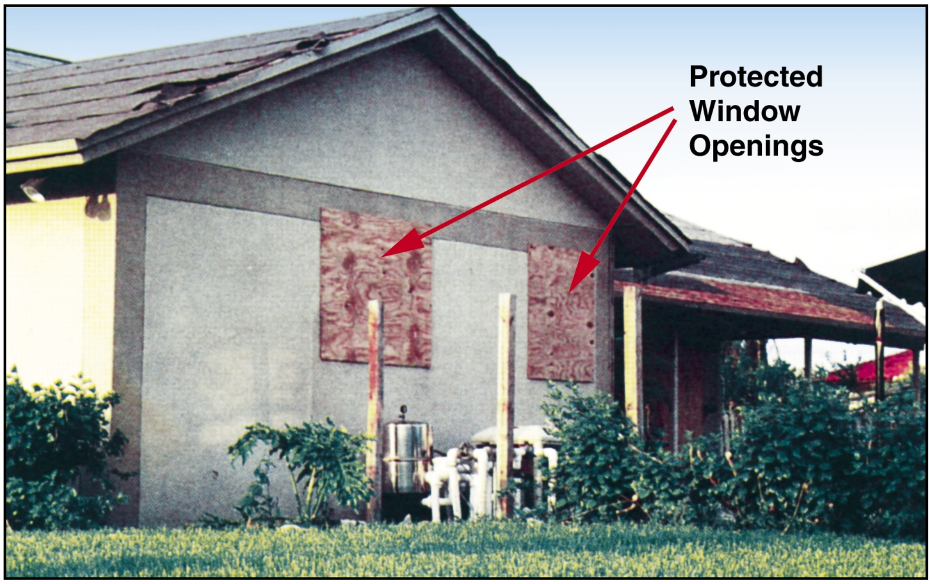



**Figure 12-6**
Hurricane Andrew (1992), Dade County, Florida. Openings in house protected with plywood panels.

Failure from uplift can occur some distance inland from the coastline. Areas of Exposure D (ASCE 7-98 exposure classification) can extend 1,500 feet inland (approximately 1/4 mile). These areas include inland waterways, the Great Lakes, and coastal areas of California, Oregon, Washington, and Alaska but exclude shorelines in hurricane-prone regions, which are now in Exposure C. Design wind pressures in Exposure D are 18.5 percent higher than those in Exposure C.

#### 12.3.1.2 Overturning

The next possible failure mechanism is overturning. Overturning can occur when insufficient weight or anchorage exist to prevent the building from rotating about a “pivot” point along one side of the building. Figure 12-7 illustrates this failure mode. To prevent this failure, the resisting moment capacity must be greater than the overturning moment. Moment, measured in foot-pounds (ft-lb), is the force times the distance (**d**) from the centroid of applied force to the pivot point. For the structure to be in equilibrium, the sum of the overturning moments must be less than the righting moment capacity.

In natural hazard events, overturning can occur from high-wind, seismic, or flood events. Floods can cause overturning if the building is below the flood level and inundated by moving water. Figure 12-8 shows a building that was overturned by flood and wind forces. The projected area method is again used to determine the moments. In the design of an elevated building, the vertical projected area is multiplied by the pressure per unit area for each roof and wall segment and then by the distance from the pivot point to the point at

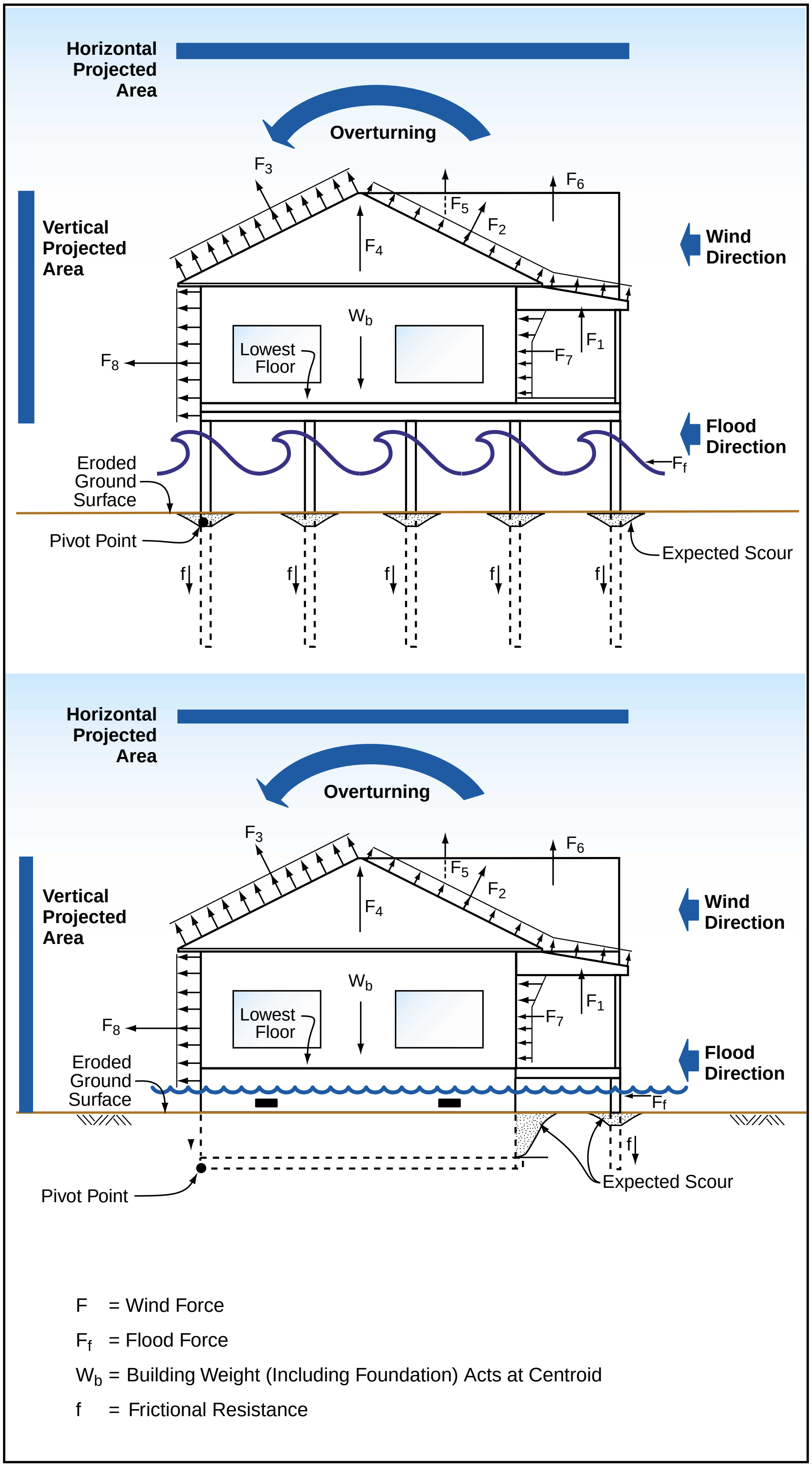



**Figure 12-7**
Overturning.

which the force acts. The uplift force on the roof also causes the building to turn over about the pivot point and is included in the overturning moment equilibrium analysis.

For lightweight structures (e.g., manufactured homes, appurtenant structures such as storage sheds, garages, outdoor pool enclosures, and gazebos), there is a high risk of overturning failure. Failure occurs in the anchorage of the building to the foundation or by the foundation rotating out of the ground. Figure 12-9 shows a lightweight building (manufactured home) overturned by high winds.

**Figure 12-8**
Hurricane Fran (1996), North Carolina. House overturned by flood and wind forces.

**Figure 12-9**
Hurricane Fran (1996), North Carolina. Lightweight building (manufactured home) overturned by wind forces.

### 12.3.1.3 Sliding or Shearing

If the building adequately resists uplift and overturning, it can still fail in sliding or shearing. Sliding failure occurs where the building connects to the foundation or where the foundation "connects" to the supporting soil. Figure 12-10 illustrates this failure mode.

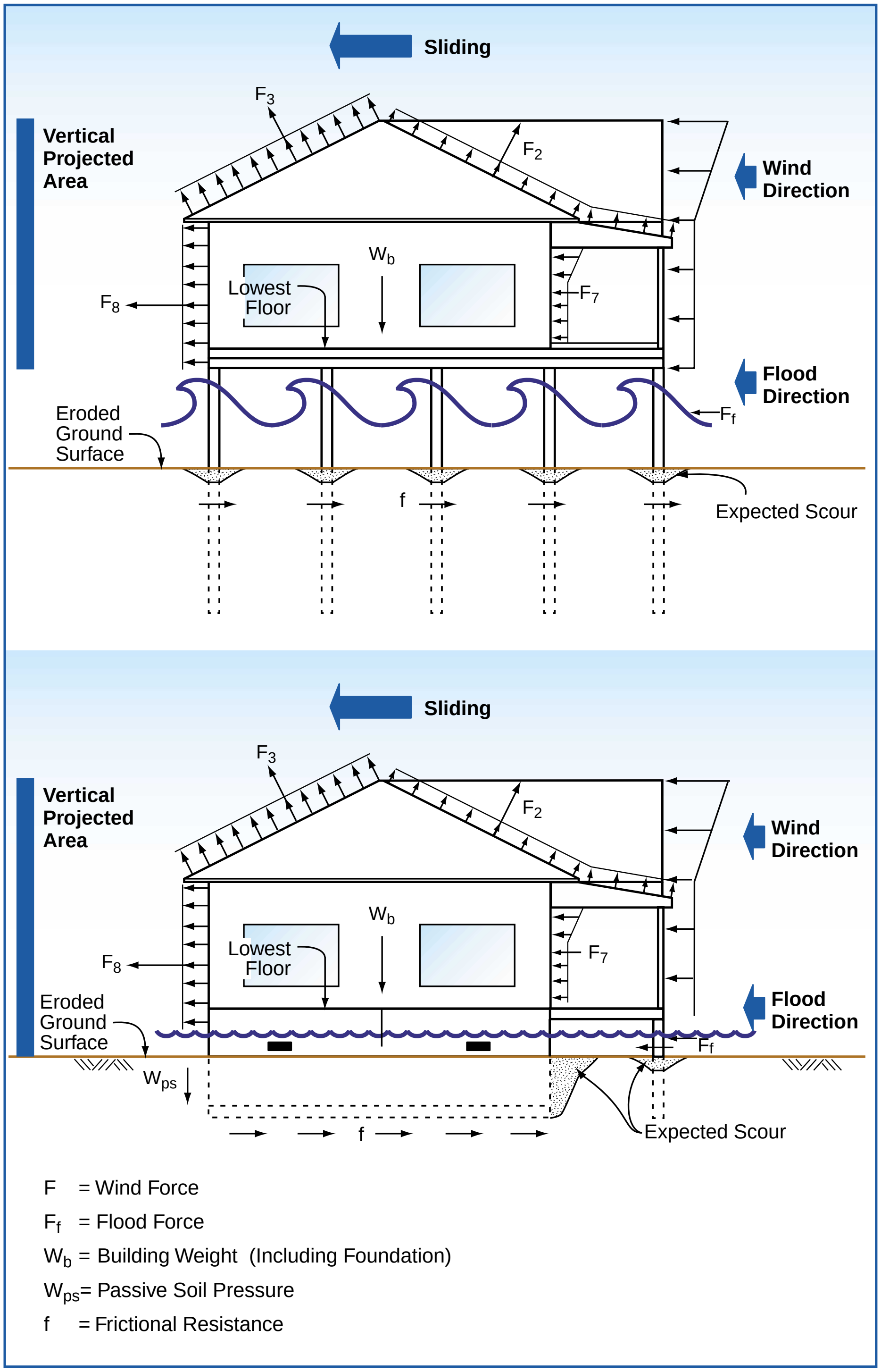



**Figure 12-10**
Sliding failure.

In pile-supported buildings, a more likely failure is excessive lateral movement, potentially resulting in building collapse. Resistance to shear is provided by the foundation and eventually the soil.

During natural hazard events, sliding failures can occur when erosion and scour have removed soil needed to prevent sliding. Figure 12-11 shows how erosion and scour can affect a foundation's ability to resist sliding.

**Figure 12-11**
Hurricane Fran (1996), North Carolina. Failure of a coastal A-zone building constructed on a masonry wall and slab-on-grade foundation. The failure resulted from undermining of the foundation by severe scour.

Sliding failures can also occur when stiff foundation elements such as masonry "shear off" or when flexible foundation elements such as wood piles snap. Figure 12-12 shows piles snapped off.

**Figure 12-12**
Hurricane Fran (1996), North Carolina. Building pilings (circled) snapped at top.

Figure 12-13 shows an entire building that withstood uplift and overturning forces, but slid inland after the connection between the building and the foundation failed.

**Figure 12-13**
Hurricane Fran (1996), North Carolina. Building that moved off its foundation. Original location indicated by black line. Note that the porch roof failed on the house on the left.

The lateral resistance of the soil is a function of the internal friction of the soil. The frictional resistance of the soil is determined by Formula 12.1.

**Formula 12.1 Frictional Resistance of Soil**

$$F = (\tan \phi)\ (\text{building weight})$$

where: **F** = sliding resistance

$\phi$ = angle of internal friction of soil

(Angles of internal friction are available in numerous engineering texts.)

Formula

Frictional Resistance of Soil

Sliding can also be resisted by passive soil pressure against a vertical surface of a belowgrade foundation.

### 12.3.1.4 Collapse

When structural elements fail or lose alignment, other undamaged elements can fail. In extreme cases, such failures can cause the building to collapse. Figure 12-14 shows several collapsed buildings near a temporary inlet cut by flood flows during Hurricane Fran. The water flowing through the new inlet caused the buildings to collapse.

**NOTE**

The case study building shown in Chapter 11 to which flood, wind, and seismic loads were applied will be followed throughout this chapter.

**Figure 12-14**
Hurricane Fran (1996), North Carolina. Building collapse caused by the force of water flowing through an inlet created across Topsail Island during the storm.

Now that the conditions under which light-frame buildings can and have failed have been discussed, the concept of a continuous load path will be examined.

### 12.3.2 Load Path

A load path can be thought of as a "chain" running through the building. Because all applied loads must be transferred to the foundation, the load path chain must connect to the foundation. To be effective, each "link" in the chain must be strong enough to transfer loads without breaking.

Figure 12-15 shows the load path and the links studied throughout this chapter. This load path was selected because it includes a hip roof, a window opening, a shearwall, and a pile support. Figures 12-16 and 12-17 show roof plans and elevations of the case study building used to illustrate the load and design computations presented in Chapter 11 and in this chapter.

The design loads must be applied to each link to determine what loads exist. Each link can then be designed to prevent failure. This detailed study begins in Section 12.4.

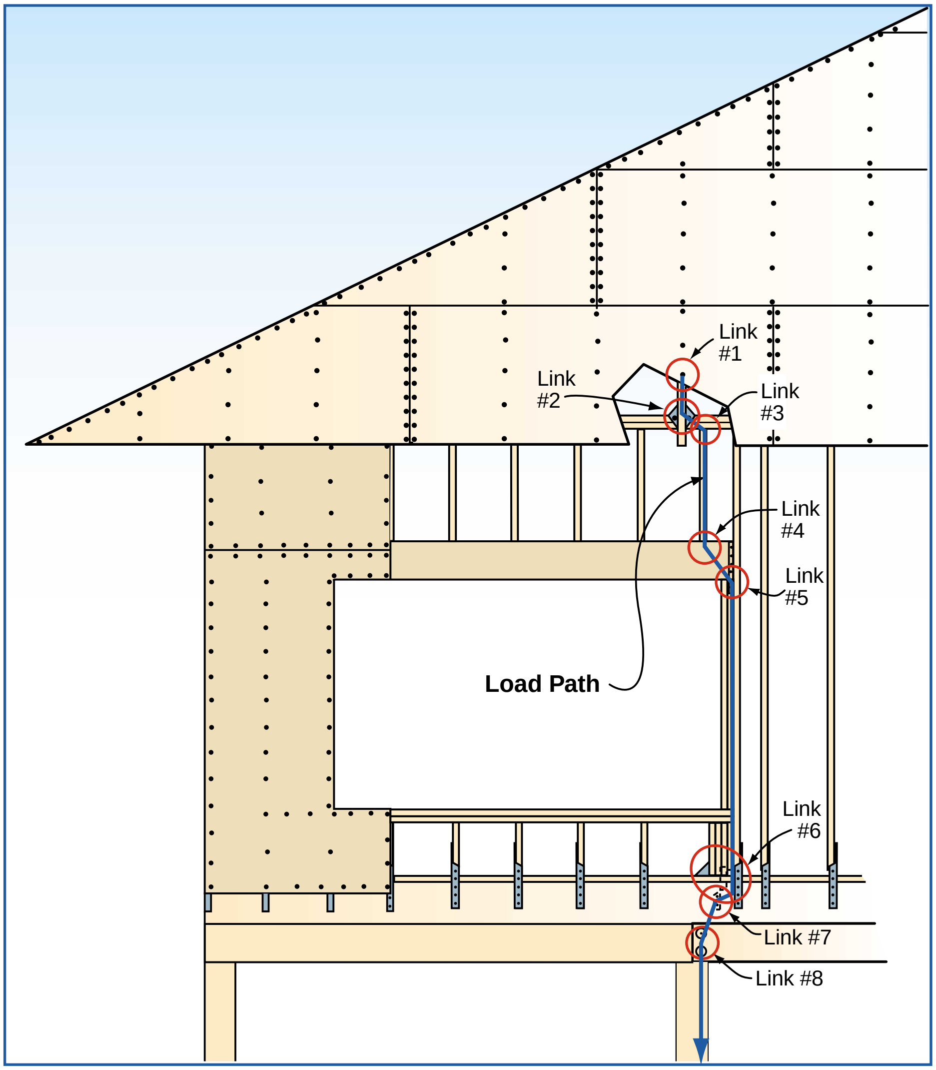



**Figure 12-15**
Load path at southwest corner window of case study building.

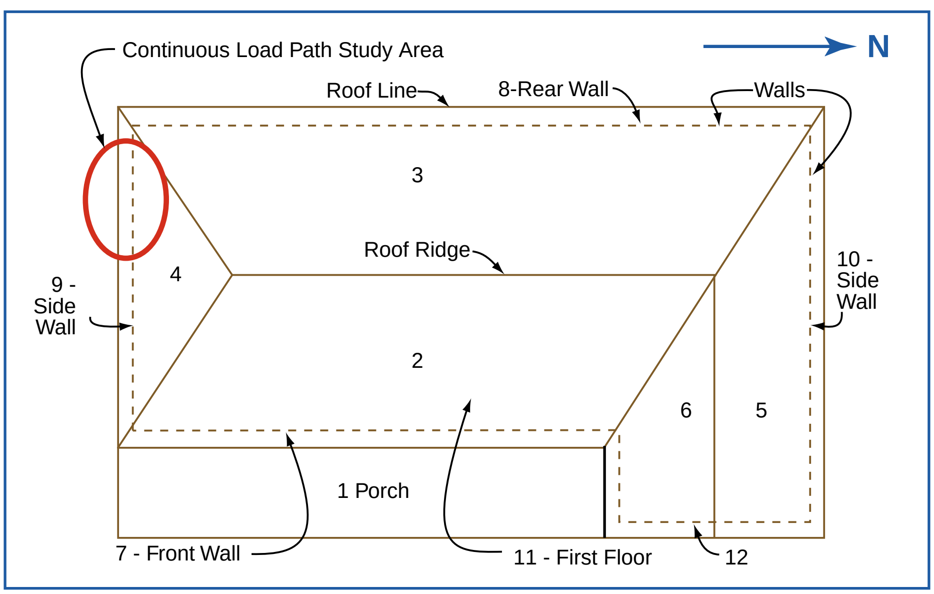



**Figure 12-16**
Roof plan of case study building.

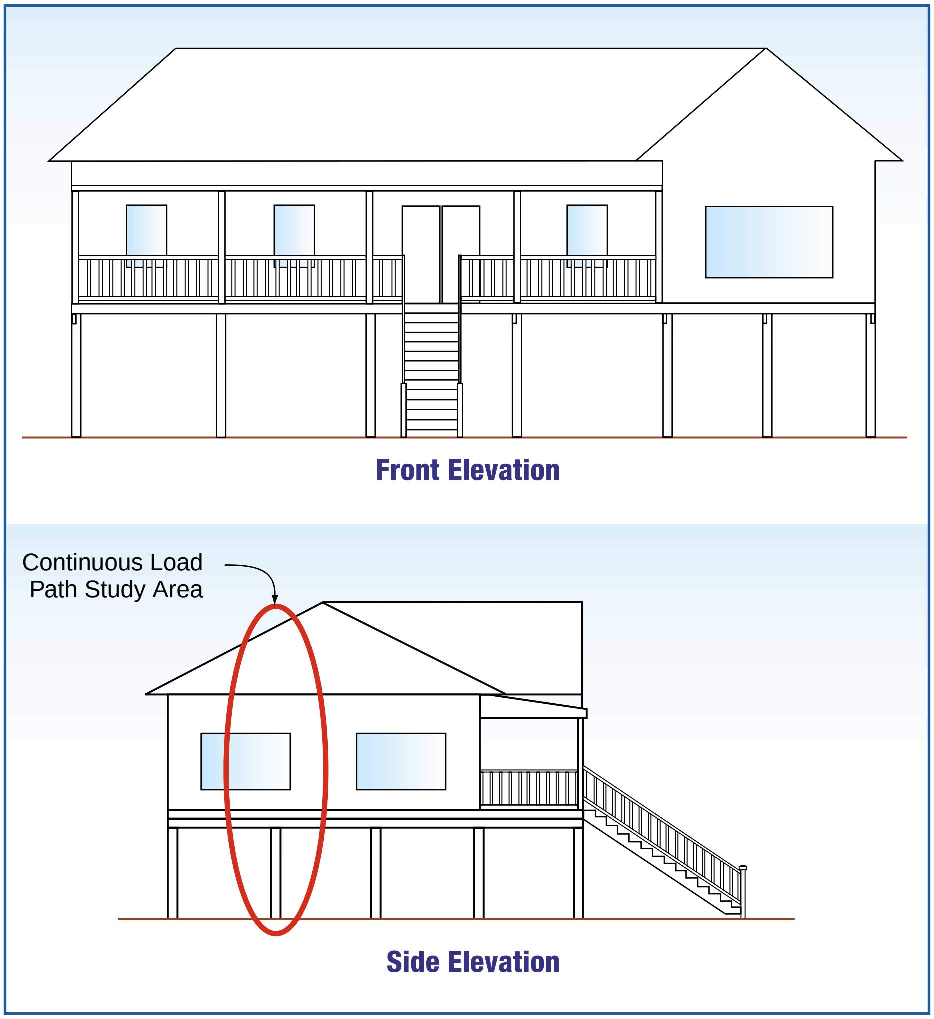
### 12.3.3 Structural Building Systems

Four primary structural building systems are used in most residential construction:

- platform framing
- balloon framing
- post-and-beam framing
- concrete/masonry

The structural system used for a building must either be known or assumed before the building can be analyzed.

**Figure 12-17**
Elevations of case study building.




#### 12.3.3.1 Platform Framing

Across the United States, this is by far the most common method of framing a wood-stud or steel-stud residential building. In the platform framing method, a floor assembly consisting of beams, joists, and a subfloor creates a "platform" that supports the exterior and interior walls. The walls are normally laid out and framed flat on top of the floor, tilted up into place, and attached at the bottom to the floor through the wall bottom plate. The walls are attached at the top to the next-level floor framing or (in a one-story building) to the roof framing. Figure 12-18 is an example of platform framing in a two-story building.

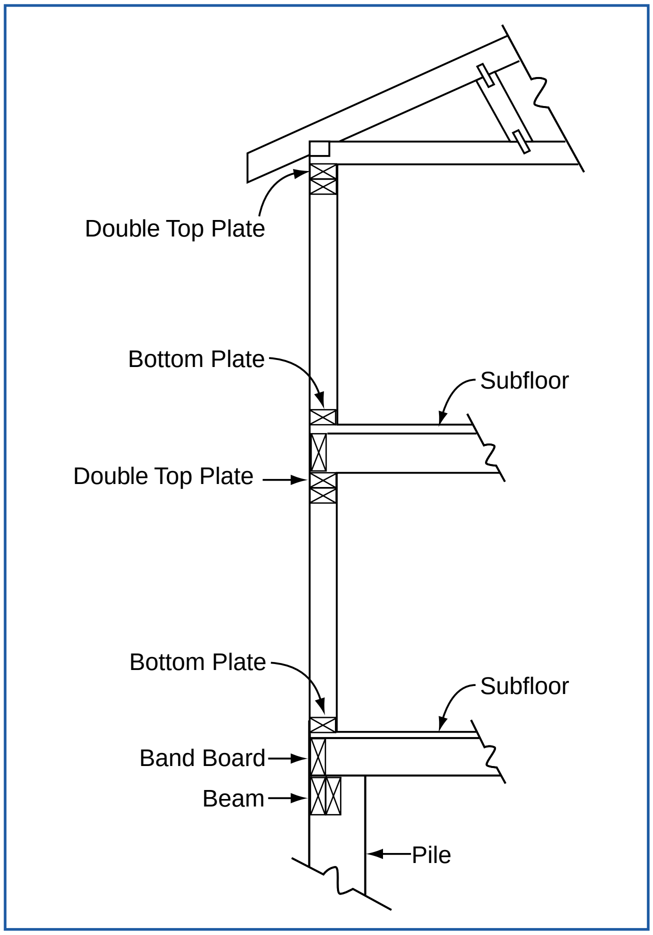



**Figure 12-18**
**Example of two-story platform framing on a pile-and-beam foundation. (For clarity, drawing is not to scale.)**

This method is commonly used on all types of foundation systems, including walls, piles, piers, and columns consisting of wood, masonry, and concrete materials. Some of the advantages and disadvantages of this framing method are listed below.

Advantages

- Walls can be built on the floor and tilted into place.
- Lumber for standard-height walls can be purchased pre-cut.
- Fireblocking at floors is created at each floor line.
- Plumb walls are easy to build.
- The construction process is faster than in other framing methods.

Disadvantages

- Creates potential failure planes in the building at every wall/floor/roof joint.
- Does not leave much chase space in exterior walls between floors.
- Makes proper detailing and construction of connections critical, because the building is constructed with a skeleton frame and is relatively light,

**CROSS-REFERENCE**

See Figure 12-43 on page 12-48 for the illustration of Link # 6, the wall to floor connection.

Platform framing contains an inherent weakness in that a failure plane in shear is built into floor/wall and wall/gable roof connections as shown in Figure 12-18. The designer must design these connections so load is adequately transferred across these failure planes.

### 12.3.3.2 Balloon Framing

The balloon method uses continuous exterior wall studs that extend up from the foundation. The floor framing is supported on either blocking or a ribbon board attached to the studs. This method offers some benefits over platform framing, but is more expensive. Figure 12-19 is an example of balloon framing in a two-story building.

Some of the advantages and disadvantages of this framing method are listed below.

Advantages

- Results in less vertical shrinkage than the platform framing method.
- Can be used in the top story of a gable end wall without undue cost premium.
- Reduces the number of load transfer connections and the possibility of a failure plane from shear.

Disadvantages

- Requires splicing or scabbing of wall studs, because one-piece lumber long enough to extend to the roof is not available.
- Costs more than platform framing.
- Firestopping is required at each floor level.
- Method is not as familiar to construction trades.

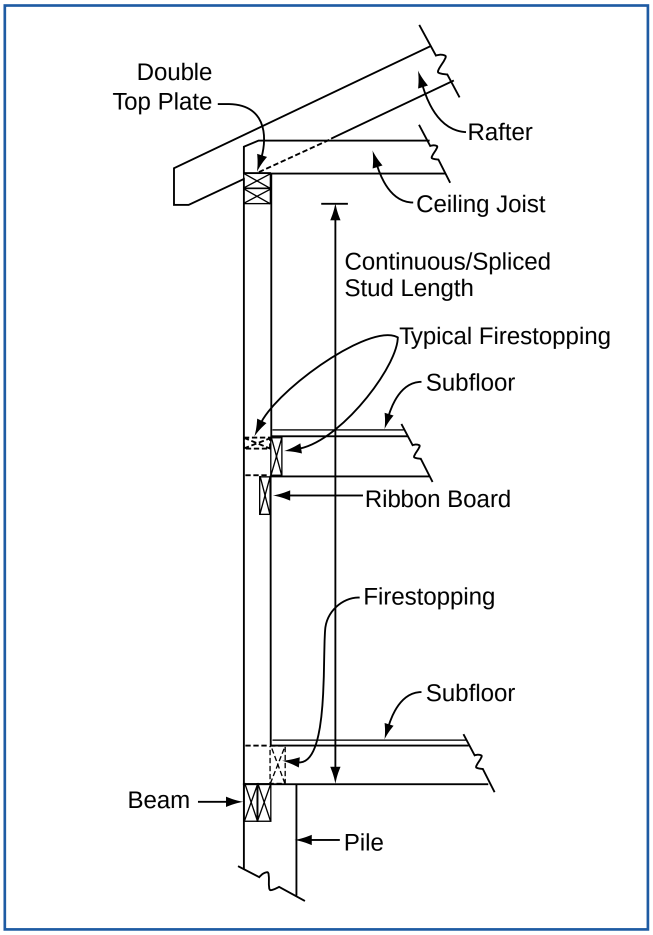


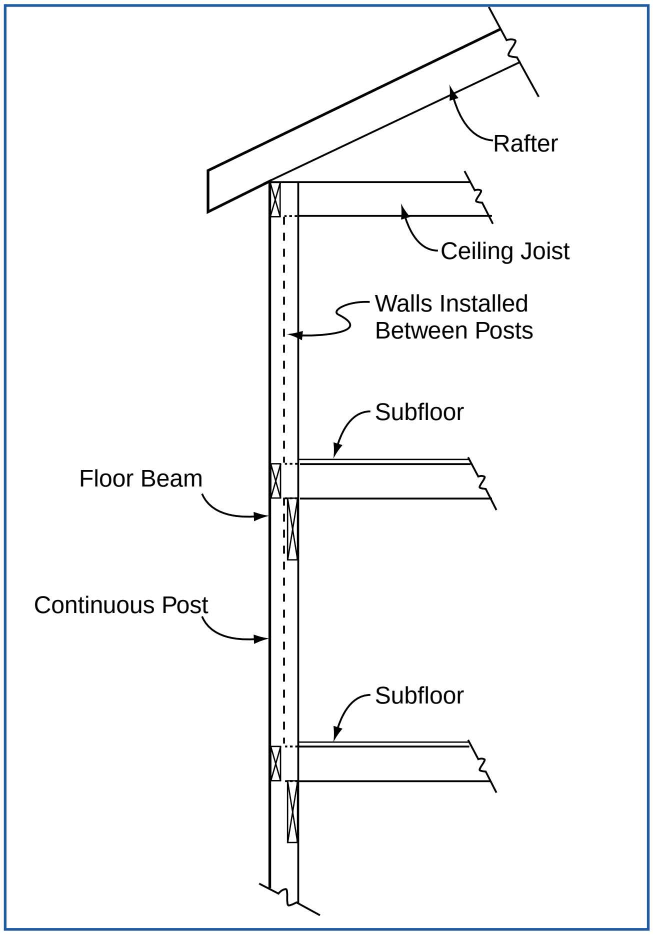

**Figure 12-19**
Two-story balloon framing on a pile-and-beam foundation. (For clarity, drawing is not to scale.)

Because it produces continuous exterior walls, balloon framing lacks the failure planes inherent in the platform method and therefore produces a more rigid frame.

### 12.3.3.3 Post-and-Beam Framing

The post-and-beam method uses continuous posts installed as part of the foundation system with support provided by beams that also support the floors and roof. The posts and beams form the primary frame of the building. Exterior walls installed between the posts form the building enclosure. Figure 12-20 is an example of post-and-beam framing in a two-story building.

**Figure 12-20**
Typical post-and-beam framing. (For clarity, drawing is not to scale.)




Some of the advantages and disadvantages of this framing method are listed below.

Advantages

- Offers a clearly defined and continuous load path for vertical loads.
- Allows the frame to be an extended portion of the foundation.

Disadvantages

- Costs more than other wood-frame methods.
- May make attachment of walls and portions of the building envelope more difficult.
- Requires lumber of greater lengths and therefore limited availability.

The performance of the entire building, as in other framing methods, relies on good connections between the structural frame and the exterior walls, the primary failure plane in this type of frame.

#### 12.3.3.4 Concrete/Masonry

In certain parts of the United States, concrete/masonry building systems are the prevalent construction method. When masonry is used as the exterior wall material, the walls are normally constructed to full height (similar to wood balloon framing) and then wood floors and the roof are framed into the masonry. Fully or partially reinforced and grouted masonry is preferable in high wind areas and required in seismic hazard areas. Floor framing is normally supported by a ledger board fastened to the masonry and the roof is anchored into the top course of masonry. Figure 12-21 is an example of masonry wall construction in a two-story building. Some of the advantages and disadvantages of this method are listed below.

Advantages:

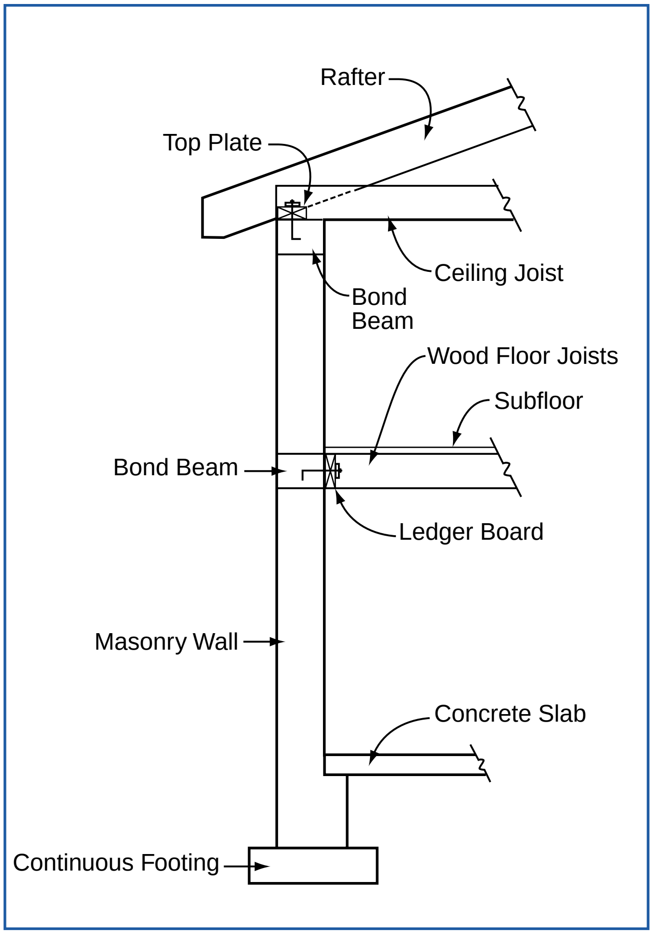
- Results in continuous exterior walls and thus a stronger structural frame.
- Results in exterior walls more resistant to windborne debris impacts.
- Results in exterior walls that require less maintenance.
- Provides greater fire resistance than wood.

In addition, reinforced masonry buildings have good high-wind damage history.

Disadvantages:

- Requires additional construction trades.
- Requires more construction time than wood framing.
- Requires special reinforcement in high-wind and seismic hazard areas.
- May require additional framing inside the exterior walls for interior finishes.
- Increases difficulty of insulation without additional interior wood walls.
- Requires more extensive foundation system, because of greater exterior wall weight.

**Figure 12-21**
Two-story masonry wall with wood floor and roof framing.




## 12.3.4 Construction Materials

In most coastal areas of the United States, wood is the primary material for coastal construction. It is strong, lightweight, and easy to work with. Wood is available in standard dimension lumber as well as new engineered materials such as laminated beams and I-sections. Other materials are used for structural systems, including reinforced concrete, light-gauge steel studs, reinforced masonry, and heavy steel framing.

**NOTE**
National Evaluation Report NER-272 (National Evaluation Service, Inc. 1997) includes prescriptive nailing schedules for nails used in pneumatic nail guns. These nails are listed by length and diameter.

Each material offers benefits and drawbacks. At this stage, the designer must select materials so that dead loads can be determined. The material selection must be based on cost, availability, and the ability to properly connect building elements. Table 12.1 lists some of the wood products available in the marketplace for each of the major building components.

In most areas of the country, builders employ timesaving techniques such as the use of pneumatic nailers. These "nail guns" typically use nails fastened together and produced in sleeves. Normally these nails differ slightly from common and

**Table 12.1**
**Wood Products Used for Structural Frame Components**

| Component | Primary Product | Primary Sizes* |
|---|---|---|
| Floor Joists | Dimension lumber | 2 x 8 – 2 x 12 |
| | Engineered floor beams or trusses | Varies |
| Subfloor | Wood structural panel | $^5/_8$ – $^3/_4$ |
| Wall Studs | Dimension lumber | 2 x 4 – 2 x 6 |
| Wall Sheathing | Wood structural panel | $^1/_2$ – $^5/_8$ |
| Roof Framing | Dimension lumber | 2 x 8 – 2 x 12 |
| | Trusses | Varies |
| Roof Sheathing | Wood structural panel | $^1/_2$ – $^5/_8$ |

* Dimensions are in inches. Does not preclude the use of other sizes or thicknesses. Nominal sizes are listed.

box nails in length and/or diameter**. Designers and builders are encouraged to specify nailed connections by length and diameter of nail required** instead of the more customary method of "pennyweight" or xd nail size.

A number of alternatives to wood are available for the components of the structural frame (see Table 12.2).

## 12.3.5 Building Layouts and Architectural Shapes

The layout of the building plays the most significant role in the application of loads to the building. Layout is a function of the following:

- number of stories
- orientation of the building in relation to the water or street (building access and view from the building)
- building shape
- openness of the floor plan
- cathedral ceilings or unusually high ceilings
- placement of building equipment, including mechanical systems, elevators, baths, and kitchens
- use of areas below the first floor
- use of outdoor areas such as decks, gazebos, and pools
- proximity to neighbors

**Table 12.2**
Alternatives to Wood Products for Structural Frame Components (NAHB Research Center 1994)

| Alternative | Definition | Building Component |
|---|---|---|
| Laminated Fiberboard Structural Sheathing | Fibrous plys laminated under pressure and covered with foil or polyethylene | Wall sheathing |
| Light-Gauge Structural Steel | Galvanized steel framing components as a direct substitute for conventional wood framing systems | Floor, wall, and roof systems |
| Structural Foam Sandwich Panels | Structural panel consisting of two stiff skins separated by a foam core | Wall and roof systems |
| Insulated Concrete Wall Forms | Concrete cast between two foam panels or into the hollow cores of stackable, interlocking insulation blocks | Wall systems |
| Insulated Concrete Wall System | Concrete cast over a polystyrene board in the center of conventional forms | Wall systems |
| Welded-Wire Sandwich Panels | Shotcrete applied over a steel-reinforced foam panel | Wall and roof systems |
| Conventional Concrete Block | Mixture of cement, aggregate, and water compacted and cured into blocks | Wall systems |
| Insulated Concrete Block | Conventional block cores filled with either plastic inserts or foam insulation | Wall systems |
| Structural Lightweight Concrete | Use of admixture and/or lightweight aggregates with conventional concrete | Wall systems |

(Source: NAHB Research Center 1994)

**NOTE**
The application of some of the materials listed in this table is governed by either standards or industry-specific design and installation guidance.

The layout considerations that impact building design for natural hazards include the following:

- Roof spans and shapes are influenced by building size, number and placement of interior walls, and building height restrictions.
- Floor plan openness and space utilization affect the number and placement of interior walls that may be needed for shearwalls.
- Equipment placement may affect building weight and placement of some framing members.
- Floor plans and building orientation dictate pile (foundation) layouts.

- Floor plans and space utilization dictate the orientation of floor support beams.
- Orientation and plan openness of the building affect the torsional response to a seismic event.
- Building orientation, plan openness, and space utilization all affect the number and placement of openings such as windows and doors.

Table 12.3 lists some of the impacts that these layout considerations have on design issues that will affect the building's performance during a natural hazard event.

**Table 12.3**
Building Layout and Impacts on Natural Hazard Design

| Layout Consideration | Impact on Flood Design | Impact on Wind Design | Impact on Seismic Design |
|---|---|---|---|
| Floor space | ▲ | ▲ | ▲ |
| Number of stories | | ▲ | ▲ |
| Building orientation | ▲ | | |
| Plan openness | | ▲ | ▲ |
| Design of area below first floor | ▲ | | ▲ |
| Building equipment | ▲ | ▲ | ▲ |
| Use of outdoor space | ▲ | ▲ | |

## 12.4 Step 3 – Determine Forces at Connections and Stresses in Materials

Each link in the load path will now be examined to see how loads are applied to each link and, thus, how designs for buildings are developed. This examination will be done using the case study example of a building subject to flood and high-wind hazards. Emphasis will be placed on those links where failures have typically occurred; however, under large forces caused by natural hazards, failure will occur at the weakest link so that improving performance at historically weakest links may create a failure at another link.

### 12.4.1 Getting Started

The first step is to determine the appropriate building areas and select the design constraints for the building so that when it is time to perform an analysis of a particular connection, the basic information has already been developed. Loads and pressures developed in the example problems in Chapter 11 will be used in the analysis of forces and stresses at the links. Figures 12-15, 12-16, and 12-17 show the case study building and the location of the continuous load path to be studied. This analysis is for only

**NOTE**

**The example presented in this chapter uses the same foundation beneath both the main structure and the porch. This approach is recommended. Using a smaller foundation beneath the porch increases the likelihood of porch and roof failure.**

one of many load paths in the building. The selected load path is representative of all of the loads that need to be analyzed for the south wall. For the analysis for this building complete, other load paths would need to be determined and other wind directions analyzed.

The areas of the building that will be affected by the natural hazard event must be determined at the start of the design process. In the case study building, the study area is near the left rear (southwest) corner of the building, so the areas shown in Figures 12-16 and 12-17 and listed below in Table 12.4 are of importance. The areas listed in Table 12.4, and used in the study of global uplift, overturning, and sliding forces in this chapter, are the projected areas and are the same areas shown in Figure 12-16. This approach simplifies the calculations, as will be shown. Table 12.4 is specific not only to the loads being studied, but also to the direction from which the loads are applied. In this example, the wind is assumed to be coming from the east.

Other building design information for this case study example includes the following:

- The roof covering is assumed to be asphalt shingles. (dead load consideration)

**Table 12.4**
Building Areas Affected by Wind Hazard for the Case Study Building When Primary Wind Direction Is From the East

| Area* | Horizontal Projected Area ($ft^2$) | Vertical Projected Area ($ft^2$) |
|---|---|---|
| Porch roof (1) | 440 | 0 |
| Front roof (2) | 704 | 352 |
| Rear roof (3) | 864 | 480 |
| Left hip roof (4) | 160 | 0 |
| Right hip roof (5) | 340 | 0 |
| Left front gable (6) | 180 | 0 |
| Front wall (7) | 0 | 440 |
| Rear wall (8) | 0 | 600 |
| Left side wall (9) | 0 | 0 |
| Right side wall (10) | 0 | 0 |
| First floor (11) | 1,840 | 0 |
| Front gable wall (12) | | 288 |

* Numbers refer to Figure 12-16.

- The siding is assumed to be lightweight (e.g., vinyl or wood) with structural sheathing beneath. (dead load consideration)
- The structural frame of the building is wood. The roof and floor framing could be trusses or composite framing members such as plywood web roof rafters or floor beams. (dead load consideration)
- All openings are assumed to be protected from breakage or wind penetration; therefore, the building is considered enclosed. (wind pressure consideration)
- The design wind speed is 120 mph, 3-sec peak gust (the equivalent fastest-mile wind speed is approximately 100 mph). (wind pressure consideration)
- The BFE is 14.0 feet National Geodetic Vertical Datum (NGVD). A freeboard of 1.0 foot is required. (flood load consideration)
- The eroded ground surface elevation is 5.5 feet NGVD, not including 1.8 feet of local scour, which results in a ground elevation of 3.7 feet NGVD adjacent to the piles. (scour effect consideration)
- The soil is medium sand with a submerged unit weight of $\gamma = 65$ lb/ft$^3$. (foundation reaction consideration)

**WARNING**

**The assumption that the openings are protected is very important. If openings are not protected, the forces must be determined assuming a partially enclosed building.**

The first step is to perform a global check of the uplift, overturning, and sliding or shearing forces described in Section 12.3.1.

#### 12.4.1.1 Uplift

The vertical components of all forces are shown in Figure 12-22.

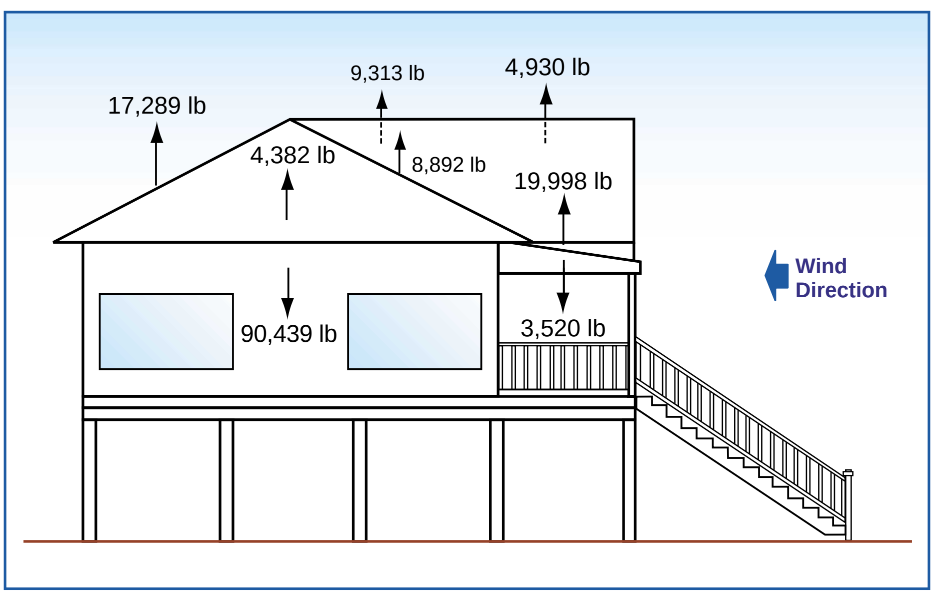



**Figure 12-22**
Uplift and gravity forces on the case study building.

**NOTE**

The uplift forces shown in Figure 12-22 are equal to the horizontal projected areas from Table 12.4 multiplied by the corresponding wind pressures shown in Table 11.7 for the east wind direction.

Calculate the uplift using the building areas from Table 12.4 and the pressures (p) from Table 11.7 (page 11-46). The sum of the uplift forces minus the building weight and pile uplift capacity must be less than or equal to zero (with up being the positive sign convention) in order for the building to remain in place without vertical displacement. The dead weight of the building is taken from standard unit weights and is shown as Calculation 12.1. It is assumed that uplift from wind pushing up on the underside of the pile-supported house and suction of the underside down towards the ground will not occur and thus will not affect the building behavior in uplift.

**NOTE**

**Designers should determine actual dead loads from building components.**

Weight of this case study house on pile foundation (main house only, porch determined separately):

Roof = (10 lb/$ft^2$)(2,248 $ft^2$) = 22,480 lb

Exterior walls = (10 lb/$ft^2$)(2,088 $ft^2$) = 20,880 lb

Floor = (10 lb/$ft^2$)(1,840 $ft^2$) = 18,400 lb

Interior walls = (8 lb/$ft^2$)(2,000 $ft^2$ – assumed) = 16,000 lb

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

**TOTAL WEIGHT** = 90,439 lb

The front porch roof weighs (8 lb/$ft^2$)(440 $ft^2$) = 3,520 lb **[12.1]**

Formula 12.2 shows how to determine the net uplift force in terms of the building components and uplift pressures. This formula is used to determine overall building stability.

**Formula**

**Net Uplift Force**

**Formula 12.2 Net Uplift Force**

**Net Uplift Force = (building component projected horizontal area) (uplift wind pressure) – dead weight of building**

**NOTE**

**Load combination no. 4 in ASCE 7-98 is the most stringent in this case of uplift; it requires that the dead load be reduced by a factor of 0.6.**

The application of Formula 12.2 is illustrated below with the wind pressures determined in the Wind Load Example Problem on page 11-45, the building areas shown in Table 12.4, and the uplift force diagram in Figure 12-2. See Figure 12-22 for the vertical components of the forces shown below.

Net uplift force = F1 + F2 + F3 + F4 + F5 + F6 – weight of building(0.6) – weight of porch roof(0.6)

Net uplift force (lb) = 19,998 + 8,892 + 9,313 + 4,930 + 4,382 + 14,243 – (90,439)(0.6)–(3,520)(0.6)

Uplift = 64,804 – 56,375 lb = 8,429 lb **[12.2]**

**Since the net uplift force is positive (upward), frictional resistance of the piles must be relied on to resist the uplift force.**

Note that the porch roof is subject to uplift failure in that:

Porch roof uplift = (440 ft$^2$)(45.45 lb/ft$^2$) – (3,520) = 16,478 lb net uplift **[12.3]**

The connections at the top and bottom of the porch roof and at the building must be strong enough to resist this 16,478-lb net uplift force.

### 12.4.1.2 Over turning

The net overturning moments are determined by establishing the pivot (hinge) point for the analysis, multiplying individual forces (generally wind, water, and weight) by their respective moment arms (distances from their line of force to the pivot point), and summing all moments. Righting moments are added; overturning moments are subtracted. The forces and lines of action on the case study building are shown in Figure 12-23. Forces that act to push the building over and forces that lift the building up have been included. Engineering judgment is required in this situation—forces that may act to counterbalance the overturning have been excluded.

To prevent overturning, the resulting moment must be greater than or equal to zero. If the resulting moment is less than zero, pile uplift frictional resistance must be considered, and anchorage or additional dead load must be added to prevent failure.

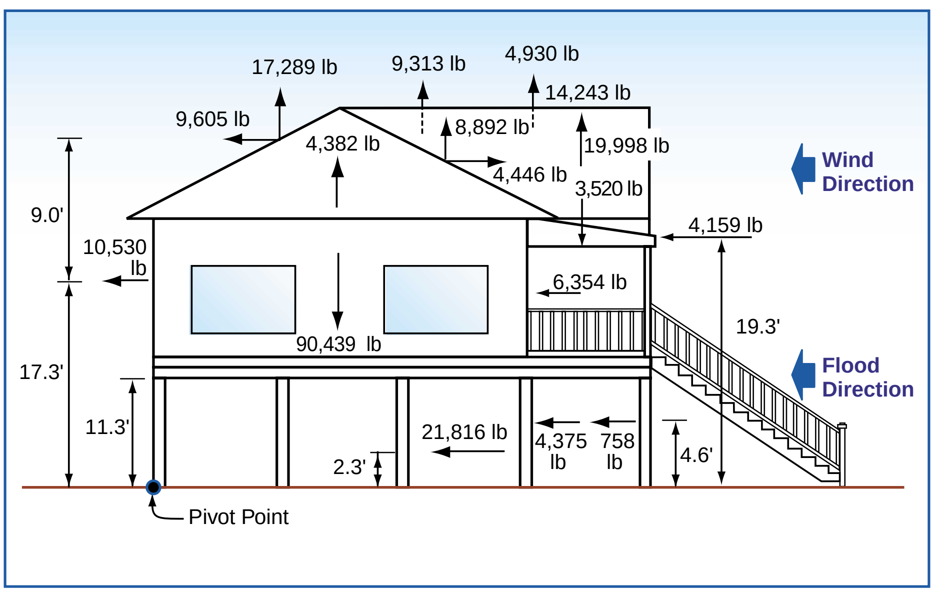



**Figure 12-23**
Overturning moment on the case study building.

Formula
Overturning Moment

**Formula 12.3 Overturning Moment**

Righting Moment = (building weight) (distance from center of gravity to pivot point)(factor of safety)

Overturning Moment = Σ[(wind forces) (distances from line of force to pivot point)] + 1.5 (flood forces) (distance from pivot point)

**For equilibrium: Righting Moment – Overturning Moment > 0**

**NOTE**

The 1.5 factor applied to flood forces in Formula 12.3 is discussed in Chapter 11. This factor is required by ASCE 7-98 for load combinations that include flood loads in coastal flood hazard areas.

**NOTE**

The strikethrough in Calculation 12.4 indicates that this force will be ignored. The flood load was determined by the flood load combination procedure shown in the Flood Load Example Problem in Chapter 11, which begins on page 11-30. From the loads on page 11-33, the flood load is: (909 lb)(24 piles) + 4,375 lb + 758 lb = 26,949 lb

The application of Formula 12.3 is illustrated below with the wind pressures determined from the Wind Load Example Problem on page 11-45, the flood forces determined from the Flood Load Example Problem on page 11-30, the building areas shown in Table 12.4, and the overturning moment diagram in Figure 12-7. Formula 12.3 is used to determine overall building stability.

Righting moment = (90,439 lb)(14 ft)(0.6) + (3,520 lb)(33 ft)(0.6) = 829,384 ft-lb

Overturning moment from lateral forces and vertical projected areas = ~~F2(26.3 ft)~~ + F3(26.3 ft) + F7(17.3 ft) + F8(17.3 ft) + F12(19.3 ft) + ($F_f$)(1.5)(4.6 ft or 2.3 ft) **[12.4]**

Overturning moment from vertical forces and horizontal projected areas = F1(33 ft) + F2(22 ft) + F3(6 ft) + F4(14 ft) + F5(20.7 ft) + F6(28.4 ft)

Righting moment – Overturning moment = 829,384 – 2,089,088 = -1,259,704 ft-lb **[12.5]**
**Equilibrium is not satisfied and overturning must be resisted by frictional resistance of the piles.**

### 12.4.1.3 Sliding

The sliding (or shear) forces are shown in Figure 12-24. The resistance to sliding on a pile foundation is provided primarily by the building-to-pile connections and the resistance of the piles to bending and breaking. For a building on a foundation wall and footings, the sliding or shear resistance is provided by the friction between the building and the soil and passive soil pressure against the side at the foundation below the scour line. Friction forces are proportional to normal forces, so the normal (vertical) forces caused by net uplift must be accounted for.

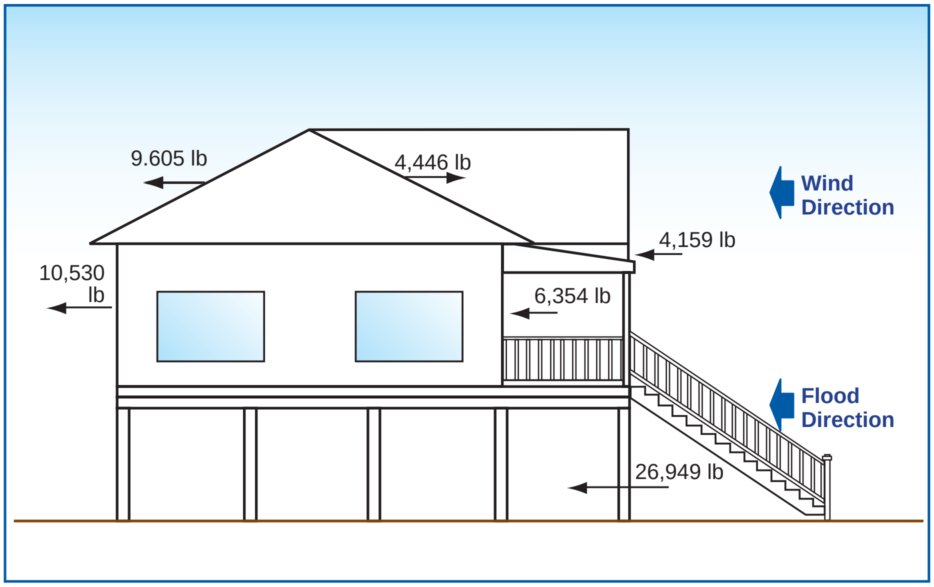



**Figure 12-24**
Sliding forces on the case study building. Engineering judgment is required in this situation—wind forces opposite the sliding direction are assumed to not significantly resist this sliding action.

The sliding forces are determined by adding the horizontal wind and flood forces multiplied by the appropriate load combination factors. Wind forces should be applied to the walls and projected vertical roof area that is perpendicular to the wind flow direction to determine the horizontal sliding forces. The determination of sliding forces in this case study will use the wind pressures determined in the Wind Load Example Problem in Chapter 11 (page 11-45), the building areas shown in Table 12.4, and the sliding force diagram in Figure 12-10. Because the flood forces act at a point below the first floor line, sliding forces must be determined at the floor-to-pile connection and at the pile-ground intersection.

Sliding forces at floor-to-pile connection =
F3 + F7 + F8 + F12 – ~~F2~~

Sliding forces at floor-to-pile connection for each of 31 piles = 9,605 + 6,354 + 10,530 + 4,159 – ~~4,446~~ = 30,648 lb or
**989 lb/pile** **[12.6]**

Sliding forces at the pile/ground intersection = (wind load calculated in Calculation 12.6) + (flood load of 26,949 lb)(1.5)

Sliding forces at the pile/ground intersection for each of 31 piles = 30,648 + 40,423 = 71,071 lb or
**2,293 lb/pile** **[12.7]**

**NOTE**
The information on the force at each floor-to-pile connection will be used in Section 12.4 in analyzing the building-to-pile connection.

Formula

Sliding Resistance of Foundation Walls and Footing

NOTE

Load paths for roof framing include the connection between the roof rafter and the ridge board. The ridge board must be treated as a beam when it is subject to upward bending from uplift forces unless ridge straps are installed across the ridge board to resist withdrawal of the board from the ridge.

CROSS-REFERENCE

See Section 12.5 for a discussion of the actual connection at each link.

If the case study building was installed on a foundation wall and footing, the sliding resistance of the soil could be determined with Formula 12.4:

**Formula 12.4**
**Sliding Resistance of Foundation Walls and Footing**

**Sliding Resistance = (tan $\phi$)(N) + Passive Force at Vertical Foundation Walls**

where: $\phi$ = internal angle of soil friction

N = net normal force (building weight – uplift forces)

### 12.4.2 Analyze Load Path Links

The concept of load path links is discussed throughout this chapter. In this section, individual links in a sample load path will be analyzed (see Figure 12-15). At each link, the maximum forces must be determined so that the building can be adequately designed and detailed. As mentioned previously, links have been selected for the analysis of areas that historically have a high incidence of failure. Other critical links exist that will need to be analyzed. In addition to analyzing individual links, this section investigates related structural elements.

#### Link #1 – Roof Sheathing to Roof Framing

This link, shown in Figure 12-25, connects the roof sheathing to the roof framing. The connection can fail in withdrawal, shear, or by the sheathing "pulling over" the fastener. The roof sheathing is considered part of *Components and Cladding* in ASCE 7-98; therefore, pressure coefficients are higher than those for similar areas of the Main Wind Force Resisting System (MWFRS).

Withdrawal:

From the Wind Load Example Problem in Chapter 11 (page 11-47), the pressure p is -87.42 lb/ft$^2$ on this connection (hip roof overhang edge), which is the uplift pressure for components and cladding, as roof sheathing is considered cladding. Link #1 is a field nail in the sheathing; from ASCE 7-98, the effective wind area for this cladding fastener is reduced to 10 ft$^2$, but for this overhang condition, the pressure p remains the same –87.42 lb/ft$^2$. This pressure acts normal to the roof surface, and it acts on the fastener through the effective area.

Figure 12-26 shows the effective area for the roof sheathing field fasteners. The width of the effective area is the spacing of the roof framing members. The actual area in which the wind uplift acts on the fasteners is the tributary area, which is the nail spacing times the spacing of the roof framing members.

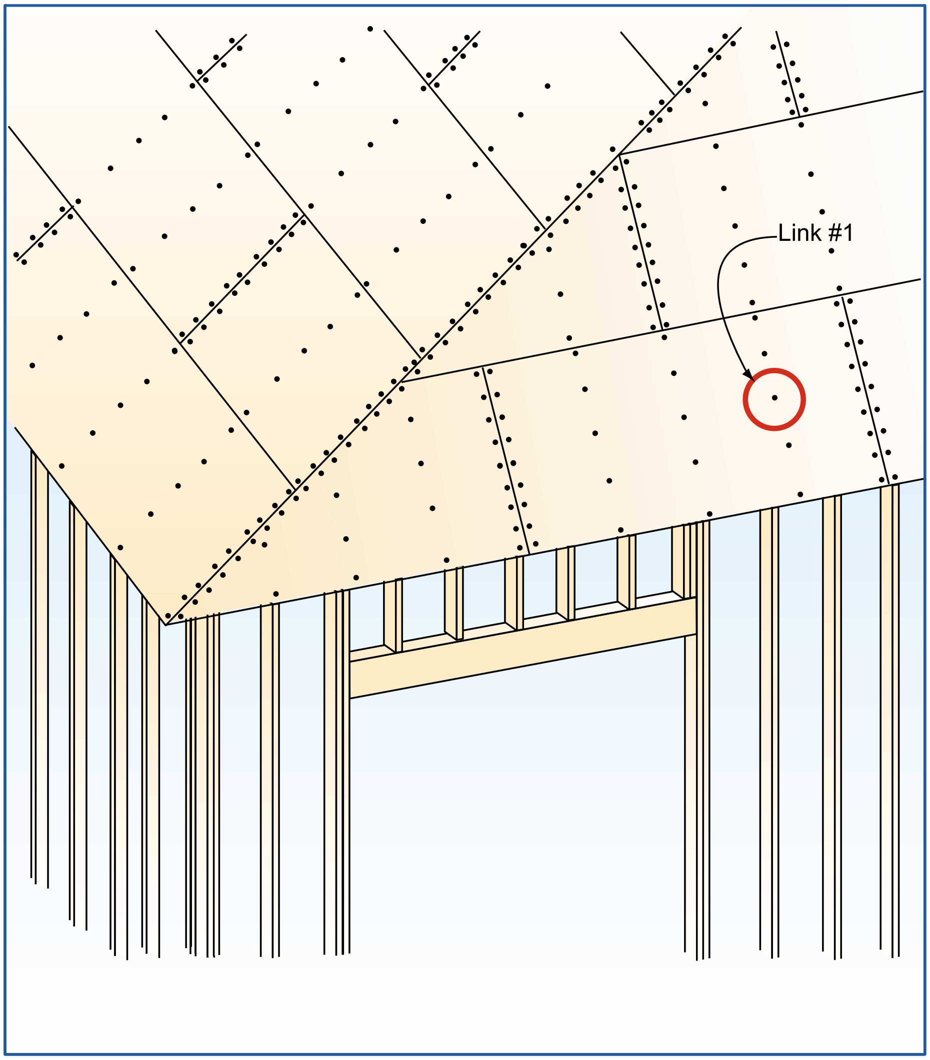
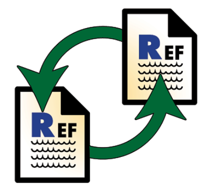



**Figure 12-25**
Link #1 – attachment of roof sheathing to roof framing. (See Table 12.6, on page 12-36, for additional information on nail spacing for various nailing zones.)



## CROSS-REFERENCE

Nail spacing is not uniform throughout the roof; rather, it is based on the nailing zones shown in Figure 12-28.

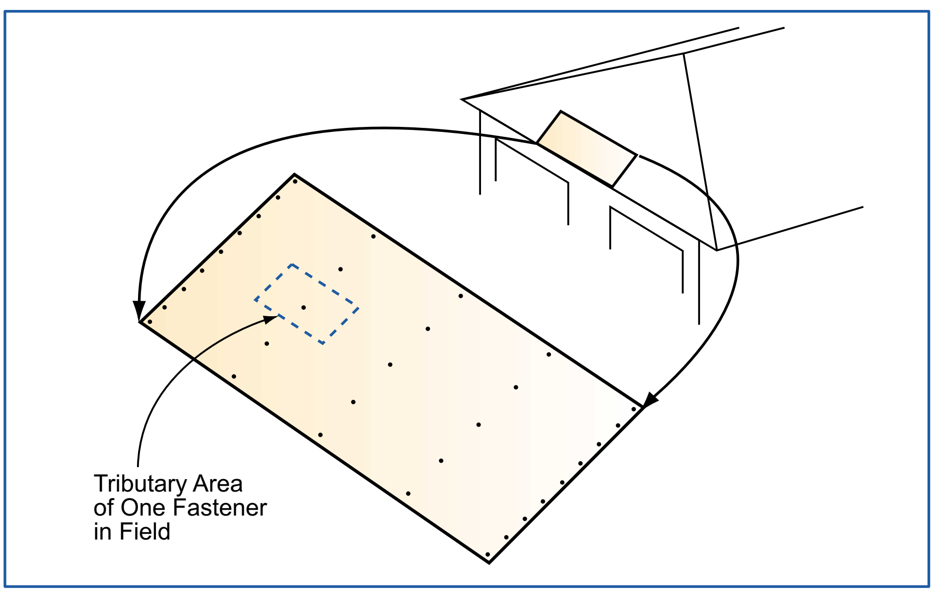



**Figure 12-26**
Effective area of a roof sheathing fastener in the field.

Nails must be spaced close enough to prevent withdrawal. Nail spacing depends on the following:

- nail withdrawal value
- effective area
- uplift pressure

Nail withdrawal values depend on the following:

- specific gravity of the framing lumber
- depth of nail penetration into the roof framing member
- nail diameter
- nail shank characteristics (e.g., smooth, screw, ring-shank)

Edge nail spacing (inches) = {[allowable withdrawal per nail (lb)]/[uplift load (lb/ft$^2$)]/[rafter spacing (feet)} x 12 in/ft)] (Note: value is rounded down to be evenly spaced in a 48-inch-wide sheathing panel.) **[12.8]**

Table 12.5 lists the nail spacings required in the field of the roof to prevent failure of the sheathing at Link #1 (hip roof edge at overhang) in withdrawal.

**Table 12.5**
Nail Spacing Required in Field for 1/2-Inch Roof Sheathing Using Various Size Nails at 120 mph (3-sec peak gust) at Link #1 for the Case Study Example

| Nail SIze | Allowable Withdrawal per Nail (lb) | Nail Spacing For 2-ft Roof Framing Member spacing | |
|---|---|---|---|
| | | Calculated Spacing | Specified Spacing |
| 8d (common) 2-1/2" x 0.131" dia. | 44 x 1.6 = 70.4 | 4.8 " o.c. | 4 " o.c. |
| 10d (common) 3" x 0.148" dia. | 62.5 x 1.6 = 100 | 6.9" o.c. | 6" o.c. |
| 8d (box) 2-1/2" x 0.113" dia. | 38 x 1.6 = 60.8 | 4.2" o.c. | 4" o.c. |
| 10d (box) 3" x 0.128" dia. | 42 x 1.6 = 67.2 | 4.6" o.c. | 4" o.c. |
| 8d (nail gun) 2" x 0.133" | 33.6 x 1.6 = 54 | 3.7" o.c. | 3" o.c. |

**Notes:**

1. Withdrawal values are derived from Table 12.2A, 1997 NDS, and are for withdrawal from hem-fir roof framing, Specific Gravity = 0.43.
2. Load duration factor is 1.6 per 1997 NDS.

Most prescriptive standards for nail withdrawal values recommend that spacing in the field of the roof not exceed 12 inches on center (o.c.). The American Plywood Association's Report T325, *Roof Sheathing Fastening Schedules for Wind Uplift* (APA 1997) recommends a minimum fastening schedule of 6 inches o.c. at the panel edges and 12 inches o.c. in the field. To illustrate forces on roof sheathing, a finite element analysis of a 4-foot x 8-foot roof panel included in the APA report showed that there were two critical fasteners in the field when one fastener is either missing or ineffective. This sheathing deflection model and the critical fasteners are shown in Figure 12-27. Uplift resistance of roof sheathing is also a function of support at panel edges (blocked or unblocked). Designers should consider specifying blocked panel edges to increase uplift resistance.

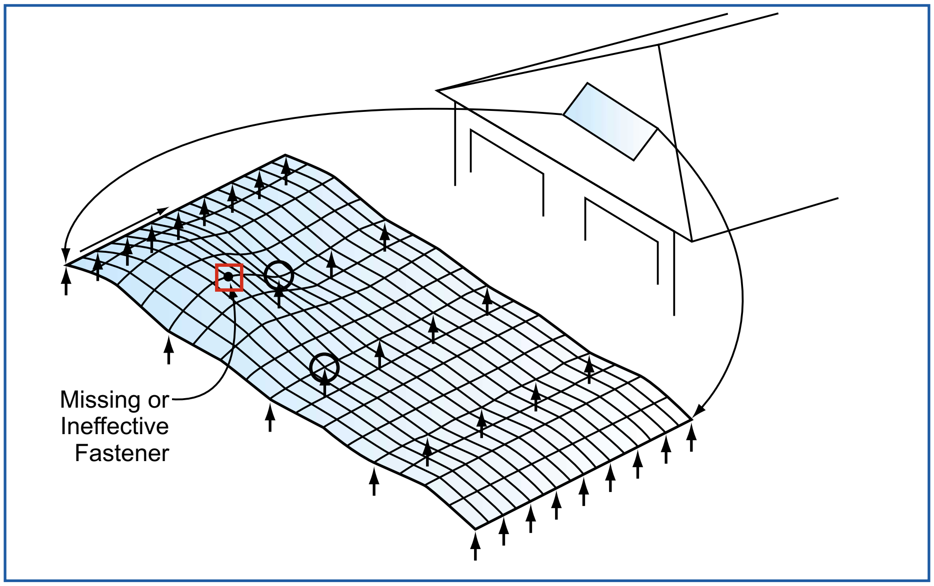



**Figure 12-27**
Deflection of an unblocked edge sheathing panel with one missing field nail due to wind uplift loading (deflected shape scaled for perspective). The missing (or otherwise ineffective) fastener in the field has a significant impact on uplift resistance. Fastener locations made critical by the missing nail are circled.

Using the uplift pressures from the Wind Load Example Problem (page 11-45), Figure 12-28 shows the nailing zones for roof sheathing on a building in a 120-mph, 3-sec peak gust wind area. Table 12.6 lists the nailing requirements for each zone. Nail spacings are rounded to the nearest 2 inches for convenience.

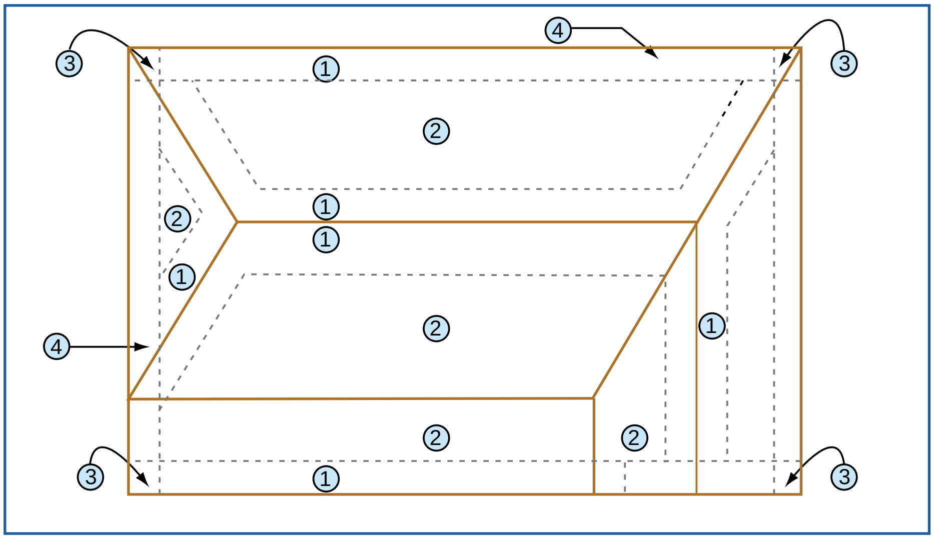



**Figure 12-28**
Nailing zones for roof sheathing in 120-mph peak gust wind zone for the case study building (see Table 12.6).

**Table 12.6**
Nailing Requirements for the Case Study Roof – 120-mph, 3-sec Peak Gust, 1/2-Inch-Thick Roof Sheathing with 8d Common Nails

| | Zone 1 | Zone 2 | Zone 3 | Zone 4 |
|---|---|---|---|---|
| **Field** | 8" o.c. | 12" o.c. | 3" o.c. | 4" o.c. |
| **Panel Edges** | 4" o.c. | 6" o.c. | 3" o.c. | 3" o.c. |

**NOTE**

On this simple hip roof case study house, there are four nailing zones. Designers should consider reducing the number of zones to a central and edge zone with nail spacings that will be conservative enough to provide adequate fastening of the sheathing anywhere in that zone.

**NOTE**

For prescriptive nailing requirements and gable-end bracing guidance, see SSTD 10-97, *Standard for Hurricane Resistant Construction* (Southern Building Code Congress International 1997) or the *SBC High Wind Edition* of the *Wood Frame Construction Manual for One and Two-Family Dwellings* (American Forest & Paper Association 1995).

**NOTE**

ASCE 7-98 (ASCE 1998) provides wind pressure coefficients for various roof shapes.

Screws can be used as an alternative to nails for attaching the roof sheathing to the roof framing. The withdrawal capacities of screws are approximately three times those of the same diameter nail per inch of penetration into the wood framing; however, shear capacities of screws are lower than those of nails of the same diameter. The implication for designers and builders is that it may be possible to use screws with diameters smaller than those of the required nails **if** the most likely failure is withdrawal of the screws, not shear failure. Spacing of screws must be about the same as that of nails because a greater spacing of the fasteners will allow more of the roof sheathing to lift up from the framing (regardless of the type of fastener used). This condition increases the likelihood that the sheathing will pull over the fastener or break the roof framing member. For guidance concerning the use of screws, designers should refer to the American Forest & Paper Association (AFPA) *National Design Specification for Wood Construction* (AFPA 1997), hereafter referred to as the NDS.

Nail spacing required to resist withdrawal will generally satisfy the requirement for shear. Values for single shear for nails are given in Tables 12.3A and 12.3B of the NDS. **Given the critical importance of maintaining the integrity of the roof deck for structural and envelope protection, proper roof sheathing attachment provides a useful and inexpensive safety factor. Designers should clearly specify nailing requirements.**

Roof shape plays a significant role in roof performance — both the structural aspect and the covering. Compared to other types of roofs, hip roofs generally perform better in high winds because they have fewer sharp corners and fewer distinctive building geometry changes. Gable-end roofs require detailing for transferring the lateral load against the gable-end wall into the structure. (See Figure 12-63, in Section 12.4.4.1, for a gable-end bracing suggestion.) Steeply pitched roofs usually perform better than flat roofs. Figures 12-29 and 12-30 show two roofs that experienced the winds of Hurricane Marilyn. The gable roof in Figure 12-29 failed, while the hip roof in Figure 12-30 survived the same storm with little to no damage. These houses are in areas of approximately the same terrain.

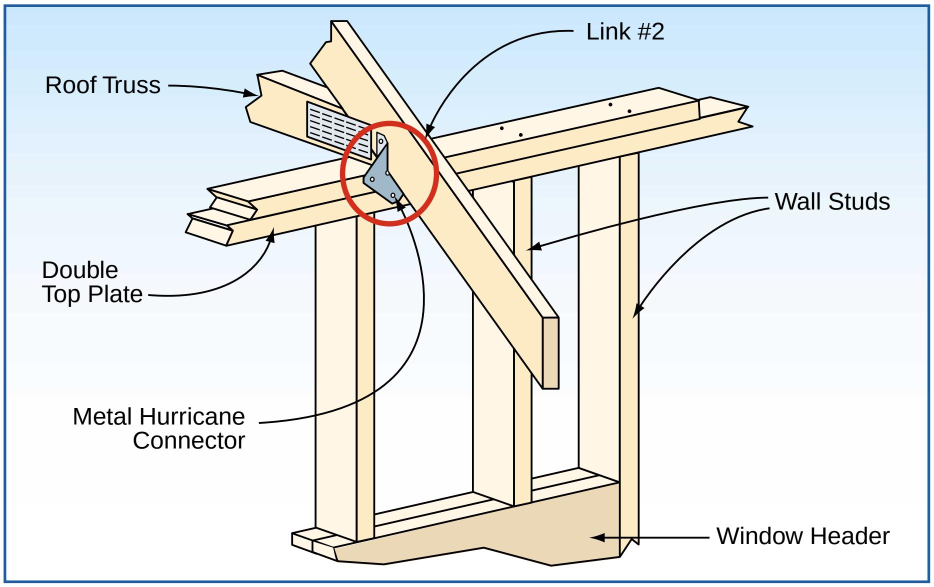
**Figure 12-29**
Hurricane Marilyn (1995), U.S. Virgin Islands. Gable-end failure caused by high winds.

**Figure 12-30**
Hurricane Marilyn (1995), U.S. Virgin Islands. Hip roof that survived high winds with little to no damage.

### Link #2 – Roof Framing to Exterior Walls

Link #2 is the connection between the roof framing member (truss or rafter) and the top of the wall below (see Figure 12-31 ). The link must be analyzed in three directions: uplift and two lateral directions. Calculating the load at this connection requires determining the tributary area and the load over that area.

**Figure 12-31**
**Link #2 – connection of roof framing to exterior wall.**




The hip roof shape changes the tributary area along the 28-foot width of the case study house. For purposes of this example, the uplift force on Link #2 will be determined and the same force will be assumed for all other roof/wall connections along this side of the building. It is important to note that this method may not be sufficiently conservative in all cases, and that loads at the other roof/wall connections should be checked.

The width of the tributary area for the roof/wall connection is the spacing of the roof framing members. For purposes of this case study, the roof framing spacing is assumed to be 24 inches o.c., and the area extends to the end of the roof overhang (eave area).

For the MWFRS, the roof pressure $p = -27.39$ lb/ft$^2$ (taken from the Wind Load Example Problem in Chapter 11, page 11-46). The components of the force are shown in Figure 12-32.

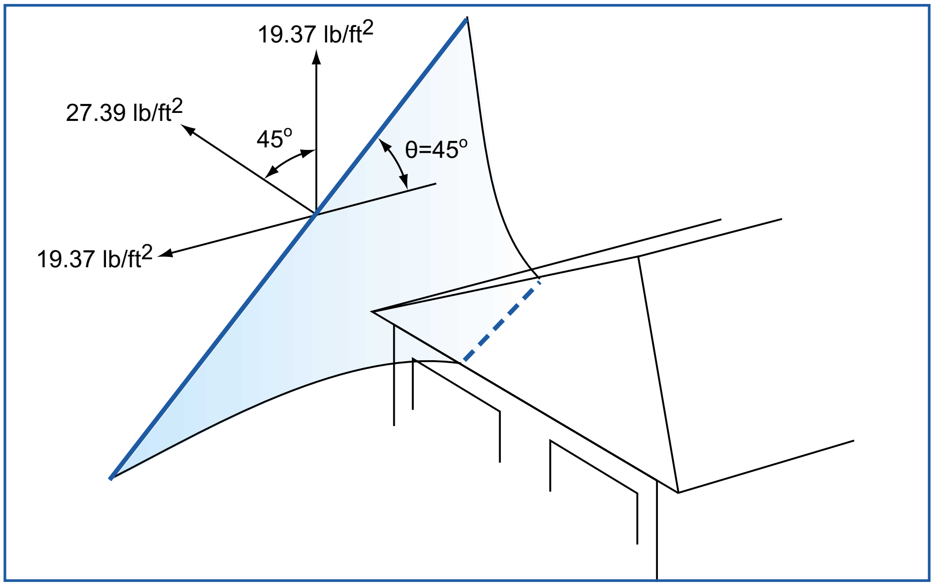



**Figure 12-32**
Uplift force components on roof.

Forces on the connection of the roof framing to the exterior wall are determined by multiplying the tributary area (including eave overhang) by the roof pressure. For rafter framing, the rafter and connection are modeled as a simply supported beam with overhang. Formulas for determining support reactions of a simple beam are available in engineering texts. The force on the link itself is the uplift reaction force on the beam. Forces on the rafter are illustrated in Figure 12-33; Formula 12.5 shows the method for calculating the uplift force at this connector.

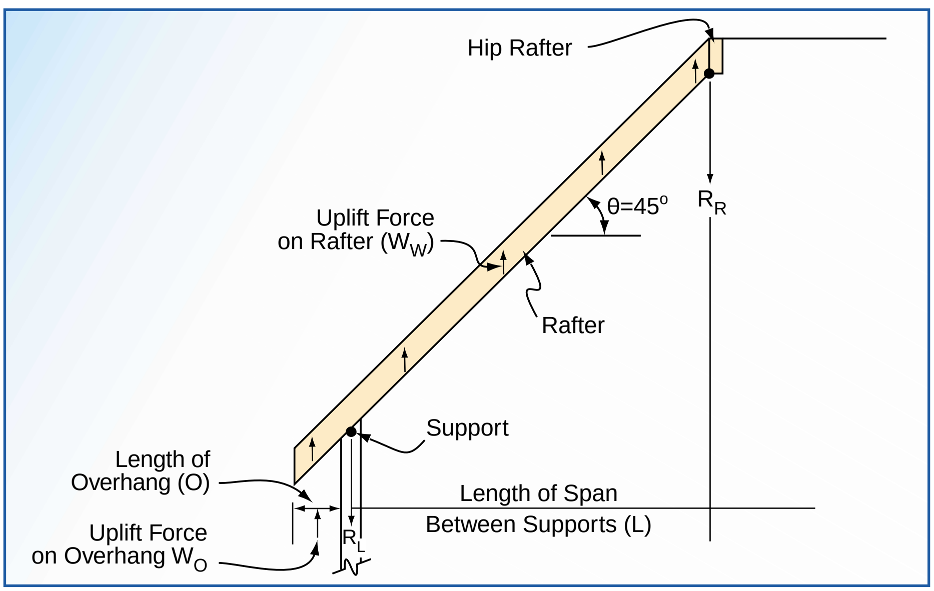



**Figure 12-33**
Uplift forces on the connection of the roof framing to the exterior wall for this case study example.

formula

Determining Uplift Forces at Each Connection in Hip Roof

**Formula 12.5**
**Uplift Forces at Each Connection in Hip Roof**

$\Sigma F = 0 = R_L + R_R - [(w_w)(L+O) + (w_o)(O)](\cos\theta)$

$\Sigma M_R = 0 = (R_L)(L) - [(w_w)(L+O)(L+O)/2 - (w_o)(O)(L+O/2)](\cos\theta)$

Solve for $R_L$ to determine uplift force at the roof-to-wall connection.

**Uplift force = $R_L$ =**

$$\frac{[(27.39\text{lb}/\text{ft}^2)(L+O)(L+O)/2 + 27.39\text{lb}/\text{ft}^2\,(O)(L+O/2)]\cos\theta\ (\text{2ft rafter spacing})}{L}$$

where:

**$R_L$** = reaction left in lb
**$R_R$** = reaction right in lb
**$W_w$** = uplift force on rafter in lb/ft$^2$
**L** = length of span between supports in ft
**O** = length of overhang in ft
**$W_0$** = uplift force on overhang in lb/ft$^2$
$\theta$= angle of roof, as measured from horizontal plane

The uplift forces at each connection along one half of the hip roof are listed in Table 12.7 and are illustrated in Figure 12-34.

**Table 12.7**
Uplift Forces at Each Connection in Hip Roof for the Case Study Example

| Connection Point | Uplift Force (lb) |
|---|---|
| **1** | 401 |
| **2** | 335 |
| **3** | 332 |
| **4** | 343 |
| **5** | 361 |
| **6** (point at Link #2 –load path) | 383 |
| **7** | 405 |
| **8** | 430 |

**NOTE**

Guidance on design requirements is available in Table 2.2A and Table 3.3, "Uplift Connection Loads from Wind," Commentary, in the *SBC High Wind Edition of the Wood Frame Construction Manual for One and Two Family Dwellings* (AFPA 1995). Care must be exercised in using prescriptive tables because often many parameters must be accounted for.

The forces do not vary significantly along the hip under the conditions of the case study, i.e., 2-foot overhang, hip roof, and short spans. In this case, one uplift connector could be selected and used at each connection point.

In-plane and normal horizontal forces to this connection must also be determined. The normal horizontal roof (perpendicular to the wall) forces at Link #2 can be determined by:

(6-ft rafter span)(2-ft rafter spacing)(19.37 lb/ft$^2$) = 232 lb **[12.9]**

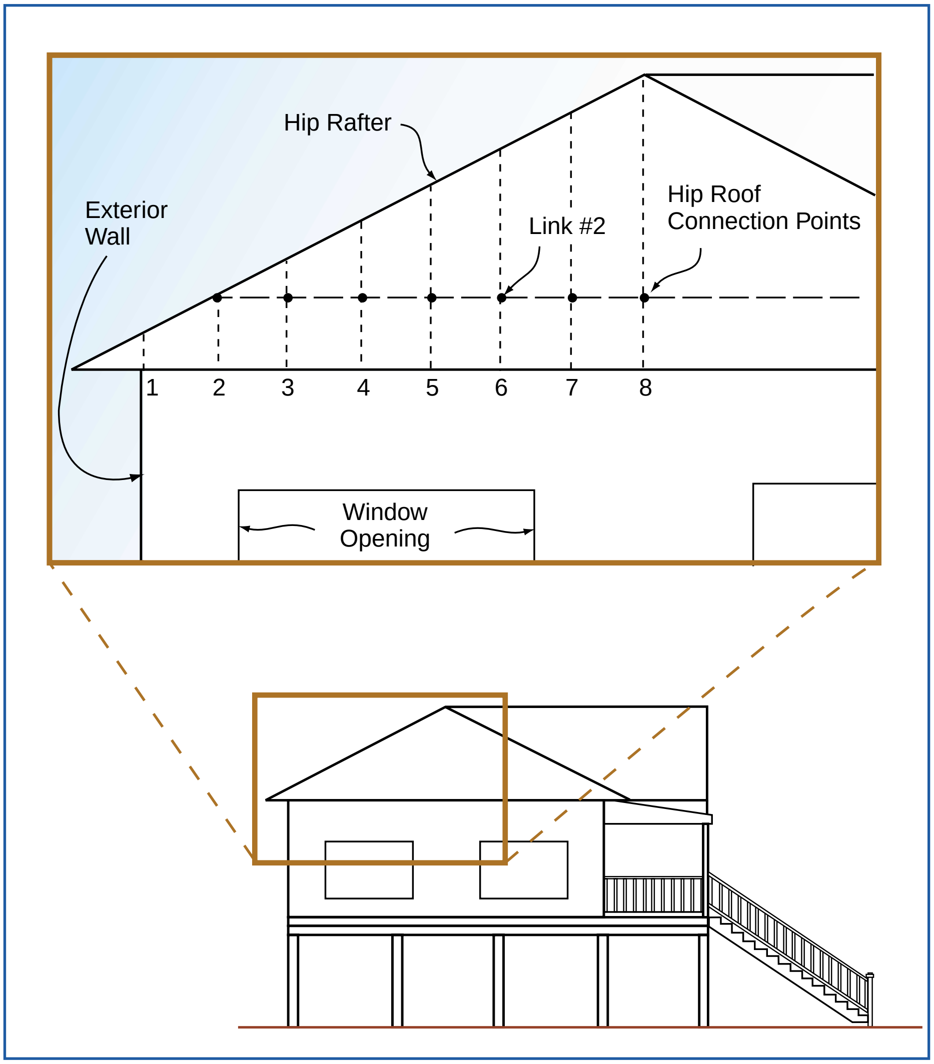



**Figure 12-34**
Uplift connections on hip roof.

There is also a normal force from suction pressure on the side wall below this connection. This force is 225 lb at the rafter:

> So the total normal force = 232 lb + 225 lb = 457 lb
>
> The in-plane lateral force at this individual Link is (the in-plane lateral force determined by Calculation 12.12 and shown in Figure 12-45) (2-ft rafter spacing)/(wall length) = (4,000)(2)/(28) = 286 lb

It is assumed the designer is familiar with bending and shear analyses of beams. Rafters respond identically in both negative (gravity) and positive (uplift) bending and shear. Therefore, rafters are generally adequate to resist uplift unless the uplift pressures are greater than the minimum live load requirements. Trusses, however, can respond differently under uplift than under gravity loading. Designers should consult truss manufacturers to ensure adequate resistance to uplift exists.

Three other important parameters must also be checked with the loads at Link # 2:

- roof framing size under upward bending, both midspan and at the rafter notch for the top plate for cantilever bending of the overhang
- roof framing size under shear force
- stability of the unbraced bottom of the rafter under compression from upward bending

Figures 12-35 and 12-36 show typical truss-to-wood-wall connections made with metal connectors. Figure 12-37 shows a typical rafter-to-masonry-wall connector that is embedded into the concrete-filled or grouted masonry cell.

**Figure 12-35**
Typical connection of truss to wood wall.

**Figure 12-36**
Typical connection of truss to wood wall.

**Figure 12-37**
Hurricane Andrew (1992), Dade County, Florida. These properly placed hurricane straps were cast into the concrete bond-beams atop a reinforced masonry wall and wrapped over roof trusses.

### Link #3 – Top Wall Plate to Wall Studs

Link #3 connects the top wall plates and the vertical wall stud over the window header. Figure 12-38 shows Link #3 for a wood wall. The forces at this link are the same as those at Link #2. There is an additional dead load from the weight of the top wall plates, but this downward load is insignificant and is ignored for purposes of this illustration.

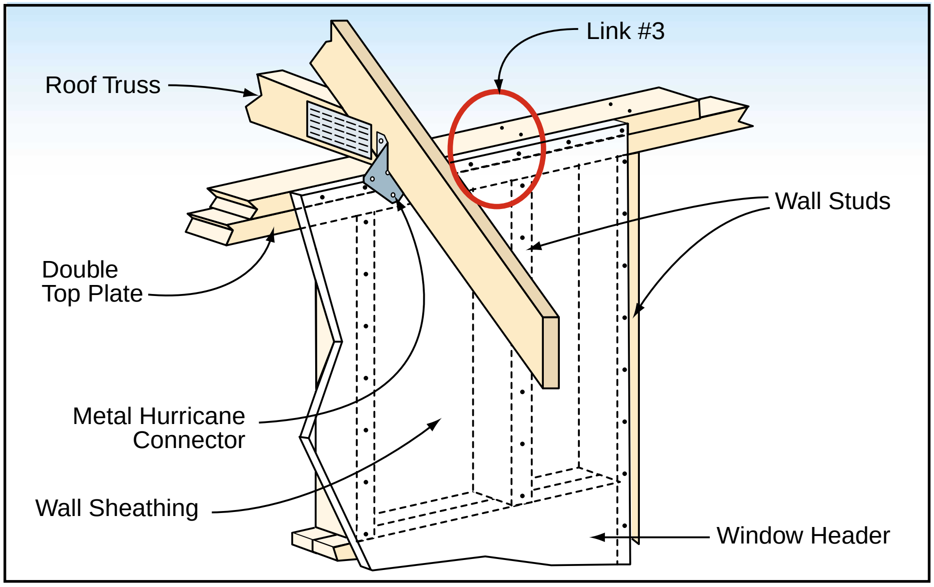
Instead of calculating forces at each of the links, designers can consult references such as Table 3.3B of the *High Wind Edition* of the *Wood Frame Construction Manual for One and Two Family Dwellings* (AFPA 1995), which provides prescriptive nailing requirements for uplift straps for roof-to-wall and wall-to-wall connections. This link is frequently made with wall sheathing acting as the connector, particularly when each roof framing member is not supported over a stud (as in the case study example). The in-plane lateral force at Link #2 must be transferred to Link #3 through the connector at Link #2.

**NOTE**

As in other examples, the shear capacity from the 1997 NDS is multiplied by a 1.6 load duration factor. The wood is assumed to be hem-fir. The sheathing is 1/2 inch thick.

The shear capacity of 8d box nails (used commonly to install sheathing) is 78.4 lb each.

**Figure 12-38**
Link #3 – connection of top wall plate to wall stud.




The uplift load at this link is 383 lb, which requires 383/78.4 = 4.9 or 5 nails over a 24-inch truss spacing. **[12.10]**

For masonry or concrete walls, if a top wood sill plate is installed (so wood roof framing can be installed), the plate must be connected to the masonry with anchor bolts or cast-in straps. Uplift forces along the wall must be resisted by the anchor bolts so the bolts must be spaced to resist pullout and the plate must resist bending and splitting at the bolts. The important part about this connection is that the wall plate must resist bending in the weak axis; placing anchor bolts close together will assist in reducing the bending stress, but placing the bolts too close together can promote splitting along the grain. There are design requirements in SSTD 10-97 (SBCCI 1997a) for the installation of anchor bolts in masonry.

### Link #4 – Wall Sheathing to Window Header

Link #4 connects the wall sheathing and the window header. The link is illustrated in Figure 12-39 for a wood wall. A failure at this link will normally occur from uplift or shear in the plane of the wall (racking). The uplift force is the same as that at Link #2 minus some additional dead load, which, for purposes of this illustration, is ignored. This approach again produces a conservative design. The uplift forces can be resisted by wall sheathing nailed to the header as described for Link #3. In-plane forces go to the full-height wall segment through the double top plate as shown in Figure 12-45.

The window header must be checked for resistance to bending from gravity

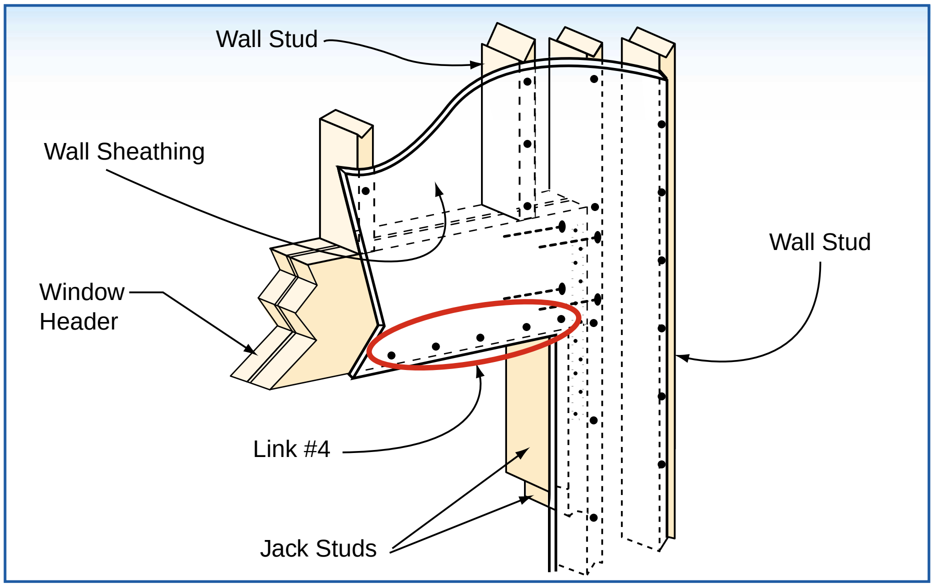



**Figure 12-39**
Link #4 – connection of wall sheathing to window header.

loads and uplift and for wall-out-of-plane bending. These can be analyzed with statics and beam theory. A header consisting of two 2x12's is required in the case study example.

In masonry construction, a masonry or concrete bond beam is required over the window opening. This beam must also be able to resist bending in both the plane of the wall and normal to the wall. SSTD 10-97 (SBCCI 1997a) includes prescriptive designs for masonry headers. Concrete and masonry are inherently weak in tension (bending). Reinforcing steel must be placed in the bond beam in order for the beam to adequately resist the bending stresses. The design of these members is beyond the scope of this manual; the prescriptive methods used in SSTD 10-97 or other concrete and masonry references should be used.

### Link #5 – Window Header to E xterior Wall

Link #5, illustrated in Figure 12-40, connects the window header to the adjacent wall framing. The link must be checked in uplift and shear out of the plane of the wall.

Uplift forces at this link are the same as the resultant force at the end of the window header, which is 731 lb (determined by summing all of the uplift loads on the header and finding the resultant force on this end of the header). This uplift force can be transferred by a strap (shown in Figure 12-40) or by end-nailing, as described on page 12-47.

The outward force at this connection results from the negative (outward) pressure of wind as it travels around the south side of the building. The wind produces a suction pressure of 22.47 lb/ft$^2$ on the MWFRS, but nearly two times that (43.13 lb/ft$^2$) on the components and cladding (see Wind Load Example Problem on page 11-45). This force acts on a tributary area that includes part of the window and is calculated with pressure coefficients for components and cladding. Figure 12-41 shows the tributary area for this link.

The outward force at Link #5 is determined by multiplying the outward pressure (p) by the tributary

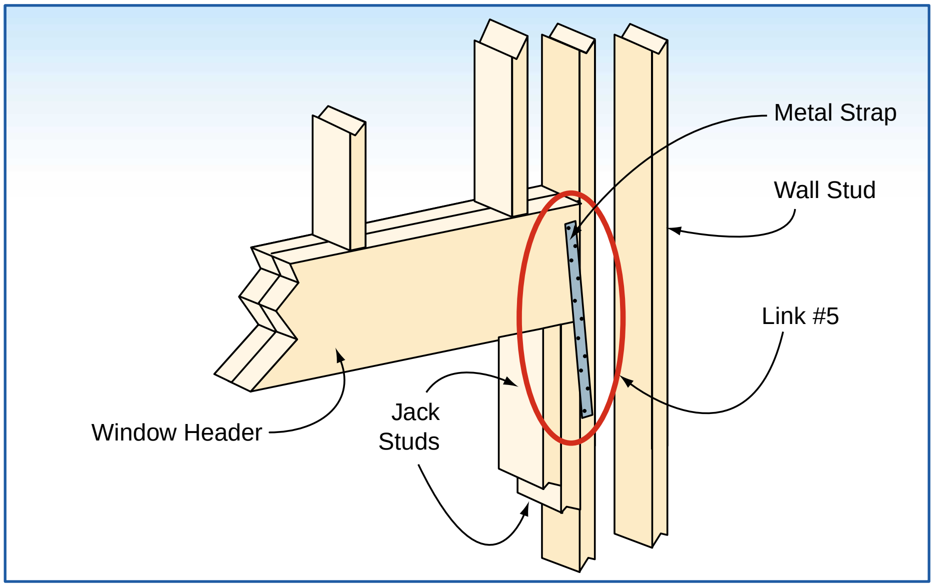



**Figure 12-40**
Link #5 – connection of window header to exterior wall.

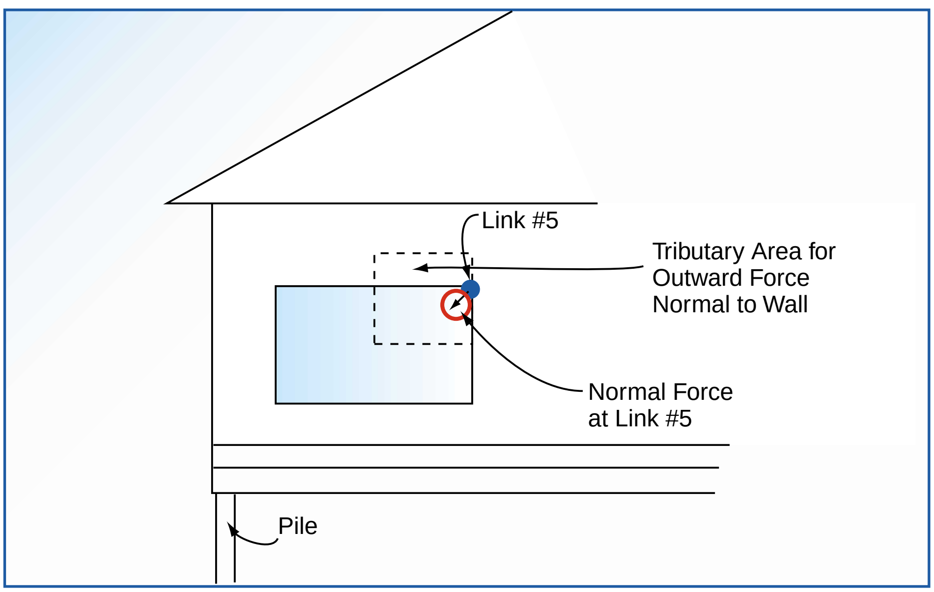



**Figure 12-41**
Link #5 – tributary area for wind force normal to wall.

area (A). This force is: (22.47 lb/ft$^2$)(20.0 ft$^2$) = 449.4 lb on the MWFRS. The total outward force on the 7-ft by 5-ft window is (43.13 lb/ft$^2$)(35 ft$^2$) = 1,510 lb.

This is somewhat conservative in that the effective wind area used for the pressure calculation in Chapter 11 was 20 ft$^2$, so the actual pressure coefficient $GC_p$ is -1.2, not -1.3. **[12.11]**

For the MWFRS, four 16d nails can be used in shear (142-lb shear capacity per 16d nail—see Table 12.8) if the header is end-nailed through the adjacent stud to resist the outward pressure, as shown in Figure 12-39. Figure 12-42 shows a metal connector used to resist uplift at a connection similar to Link #5.

Fasteners for the cladding (window) also need to be determined in withdrawal and can be calculated by dividing the total outward force on the window by the linear dimension of the window attachment device (e.g., nailing flange, window trim). The designer should verify that the window unit selected for the project has been tested to the calculated wind suction pressures with the same attachment method as that used in the field.

**NOTE**

It is particularly important for designers to verify that window units have been tested in the same way they are to be installed (e.g., with nailing flanges attached) and for windows to be installed in accordance with the manufacturer's recommendations.

**Table 12.8**
Shear Resistance Provided by 16d Box Nails

| Number of 16d Nails | Shear (lb) |
|---|---|
| 1 | 142 |
| 2 | 284 |
| 3 | 426 |
| 4 | 568 |
| 5 | 710 |
| 6 | 852 |
| 7 | 994 |
| 8 | 1,136 |

**Notes:**

1.Hem-fir with Specific Gravity = 0.43 is assumed.

2. 1/2" thick side member for withdrawal and 1-1/2" side member for shear is assumed.

3. Wind load factor = 1.6 is included.

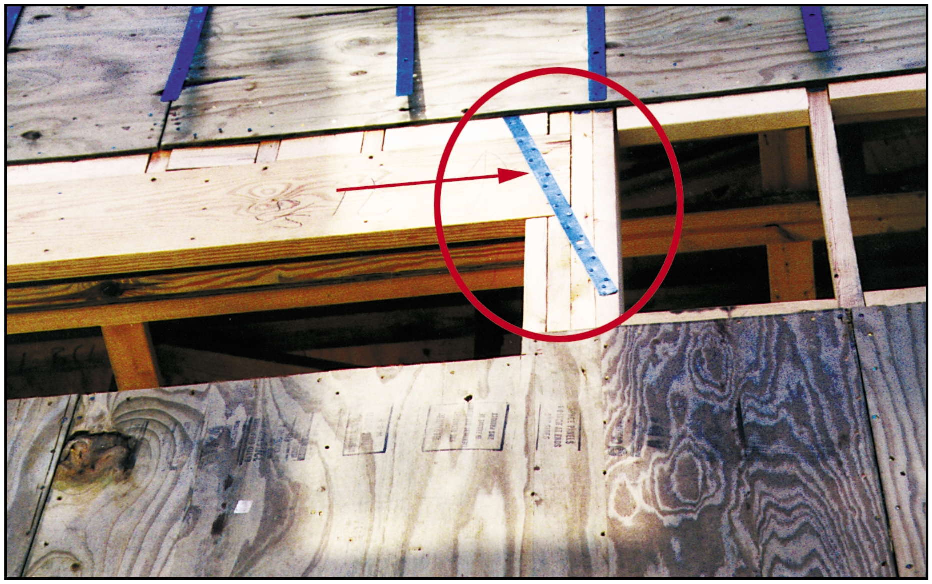


**Figure 12-42**
Metal connector at window opening.

### Link #6 – Wall to Floor Framing

Link #6 connects the wall and floor framing (see Figure 12-43). This link is very important. Sufficient uplift resistance must be provided to keep the adjacent shear panel from overturning under horizontal wind (or seismic) loads.

The uplift forces and overturning moments are significant in this case; the largest forces exist on the portion of the wall that functions as a shearwall. Link #6 is part of that shearwall.

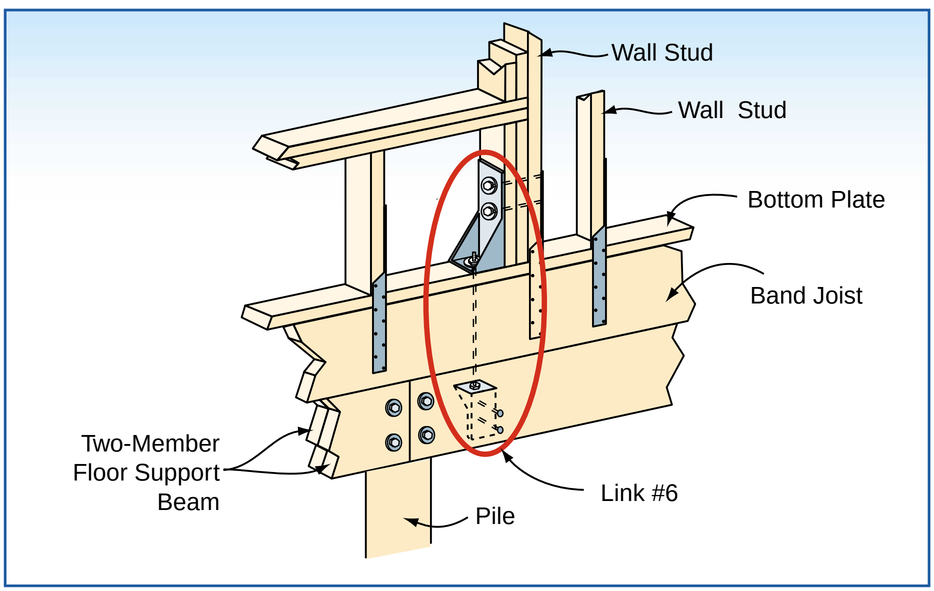



**Figure 12-43**
Link #6 – connection of wall to floor framing.

Before loads at Link #6 are determined, the concept of a shearwall and its associated loads will be discussed.

**Shearwalls**

Shearwalls collect applied lateral forces and transfer those forces into the foundation. In this case study, the shearwall that contains Link #6 collects the forces applied to the windward wall, the leeward wall, and the leeward roof areas.

Roof loads are collected by the diaphragm action of the roof and distributed to the top plates of the wall supporting the roof. The diaphragm-to-wall connection transfers shear to in-plane walls. It will be assumed that all exterior walls and one interior wall will be used for shear distribution. The maximum spacing between braced walls in seismic zones is 25 feet; therefore, 25 feet will be used in this example as the maximum distance between shear walls.

The front and rear walls “collect” the windward and leeward wind loads and distribute them to the ends of the front and rear walls as a distributed load. This shear distribution and the exaggerated deflected shape of the roof and walls are illustrated in Figure 12-44.

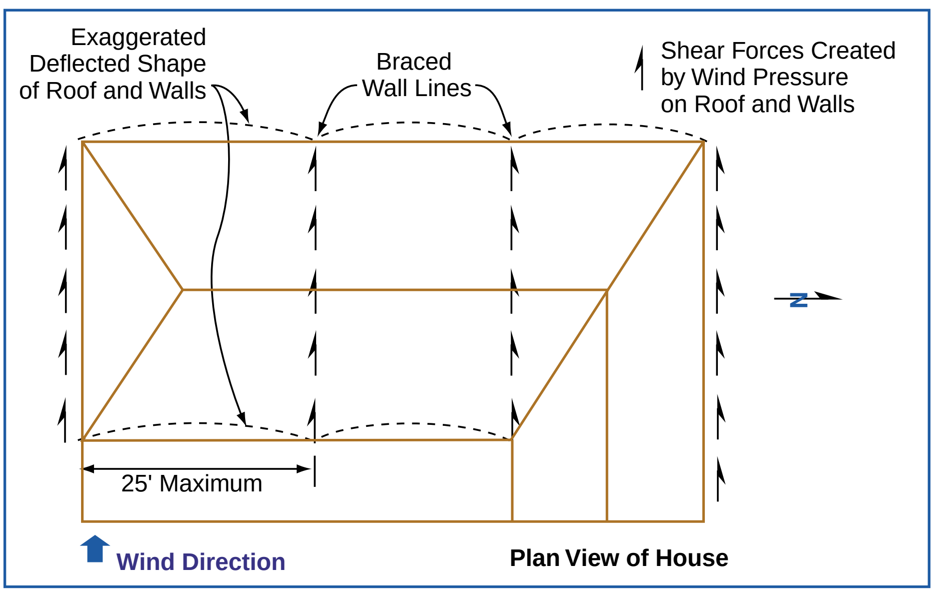



**Figure 12-44**
Shear force distribution from east wind at roof diaphragm.

Using the wind forces from Figure 12-10, the total applied force at the top of the shearwall is:

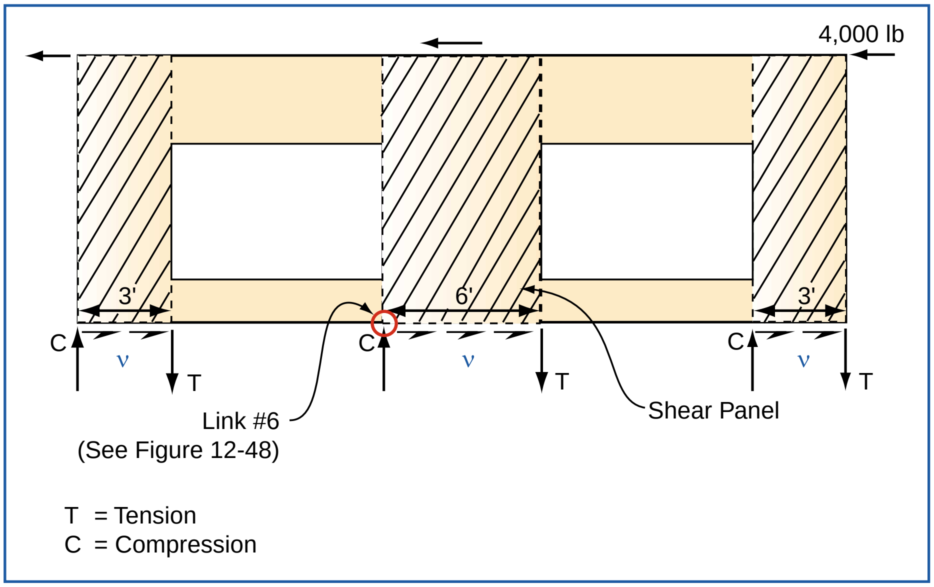
$$(p_7)(10\text{ ft})(25\text{ ft})/4 + (p_3)(8\text{ ft})(25)/2 + (p_8)(10\text{ ft})(25\text{ ft})/4 = 4{,}000\text{ lb} \qquad [12.12]$$

The lateral load (shear) at the bottom of the front and rear walls is taken into the floor diaphragm.

The reaction at the top of the shearwall of 4,000 lb must be transmitted into the shearwall; this is done by means of a strut. The strut will transmit forces across the top of openings; the strut in this case is the double top plate of the shearwall. The reaction at the strut will be in compression, and the design of this strut for lateral forces is not a consideration. However, the design of the top plate is an important consideration in the distribution of the horizontal forces from the roof diaphragms. See *Design of Wood Structures* (Breyer 1993) for explanations of how to handle this design. In shearwall design, it is commonly assumed that areas over and under the openings do not help resist shear.

Figure 12-45 shows the south wall elevation and the loads that must be collected by this 28-foot-long wall. This figure also shows where resistance to the overturning forces must be located. Loads are much more difficult to handle properly when large openings exist in the wall. See references such as *Diaphragms and Shearwalls* (Diekmann, undated ) for additional information on perforated shearwalls.

**Figure 12-45**
Loads on south shearwall.



The effective shear segments are limited to areas that are full height and continue uninterrupted from the floor to the roof and to those segments where the aspect ratio (ratio of height of segment to width) is no more than 3-1/2:1. In areas of high seismic hazard, the maximum aspect ratio is 2:1. The segments of the south wall that meet the criteria are the two corners (3 feet wide each) and the 6-foot space between the window openings shown in Figure 12-45. The unit shear, ν, is:

4,000 lb/(6 ft + 3 ft + 3 ft) = 333.3 lb/ft **[12.13]**

The overturning moment on the shearwall (and thus the force that must be resisted by a connector) is:

(unit shear ν)(wall height) **[12.14]**

and the shearwall holddown force T is the overturning moment/shearwall segment length, so the shearwall holddown force T is:

(333.3 lb/ft)(10 ft) = 3,333 lb
+ 731 lb (direct uplift from Link #5) = 4,064 lb **[12.15]**

In this case study example, this uplift resistance load is transferred to the beam that spans between the piles, as shown in Figure 12-43. Figures 12-46 and 12-47 show shearwall holddown connectors attached to a wood beam and a concrete beam, respectively.

**Figure 12-46**
Shearwall holddown connector with bracket attached to a wood beam.

**Figure 12-47**
Shearwall holddown connector (without bracket to post) attached to a concrete beam.

Shearwall deflection and aspect ratio must be checked against the criteria established in the *International Building Code* (IBC) 2000 (ICC 2000a) and the *International Residential Code for One- and Two-Family Dwellings* (IRC) 2000 (ICC 2000b). Chapter 23 of the IBC 2000 includes maximum aspect ratios for shearwalls. On the case study shearwalls, the aspect ratio is 1.67 (10/6) and 3.33 (10/3). If the aspect ratio is less than 3-1/2:1, the IBC criterion is met.

Horizontal deflection of the panel must be checked (see the equation given in Chapter 23 of the IBC 2000). The predicted deflection in the braced frame may be greater as a result of factors such as crushing of the wood fibers at the holddown support members and movement of the holddown brackets. The calculated deflection from the IBC formula is 0.946 inch.

The wall has two large windows, so nearly a 1-inch deflection is likely to be excessive. Glass breakage and water entry would probably result. To prevent this, a larger effective shear area or additional shear walls should be provided; however, these changes may radically affect the building layout.

From the prescriptive shearwall design in Chapter 23 of the IBC 2000, one layer of 3/8-inch Structural 1 Grade sheathing secured with 8d common nails, 4 inches o.c., has a shear capacity of 360 lb/ft when 2x_ wall framing lumber is used and 400 lb/ft when 3x_ wall framing lumber is used.

To prevent glass breakage (which can contribute to failure of the entire building through internal pressurization), a steel moment frame would need to be installed around openings. The moment frame does not rely on the shear capacity of the framed walls to resist horizontal forces and is more effective at preventing excessive deflection. Moment frame design is discussed below.

Alternative methods of increasing shear capacity include the following:

- Installing additional interior shearwalls to further reduce lateral loads on the shearwall being designed.
- Decreasing the size of window openings to increase the effective width of the shearwall. In this case study, for every 1-foot increase in the effective shearwall length, the shear force per foot is reduced about 40 lb.
- Widening the building to increase the shearwall length while leaving the windows the same size. Again, for every 1-foot increase in the effective shearwall length, the shear force per foot is reduced by about 40 lb.

The forces at Link #6 are as follows:

- normal force = 168 lb, which is resisted by sill plate nailing into the band joist
- horizontal force = unit shear of 333 lb x 1.33 = 443 lb, which also is resisted by sill plate nailing
- uplift force = 731 lb, except that at this exact location, the shearwall connector will be installed with a capacity of 4,064 lb; therefore, an additional uplift connector is not required

These forces and directions are illustrated in Figure 12-48.

### Moment Frames

More moment-resistant frames are being built and installed in coastal homes (Hamilton 1997). The need for this special design results from more buildings in coastal high hazard areas being constructed with large glazed areas on exterior walls, with large open interior areas, and with heights of two to three stories. Figure 12-49 shows a typical steel moment frame.

Large glazed areas pose challenges to the designer because they create:

- large openings in shearwalls,
- large deflection in shearwalls, and
- difficulties in distributing the shear to the foundation.

**Figure 12-48**
Load summary at Link #6.

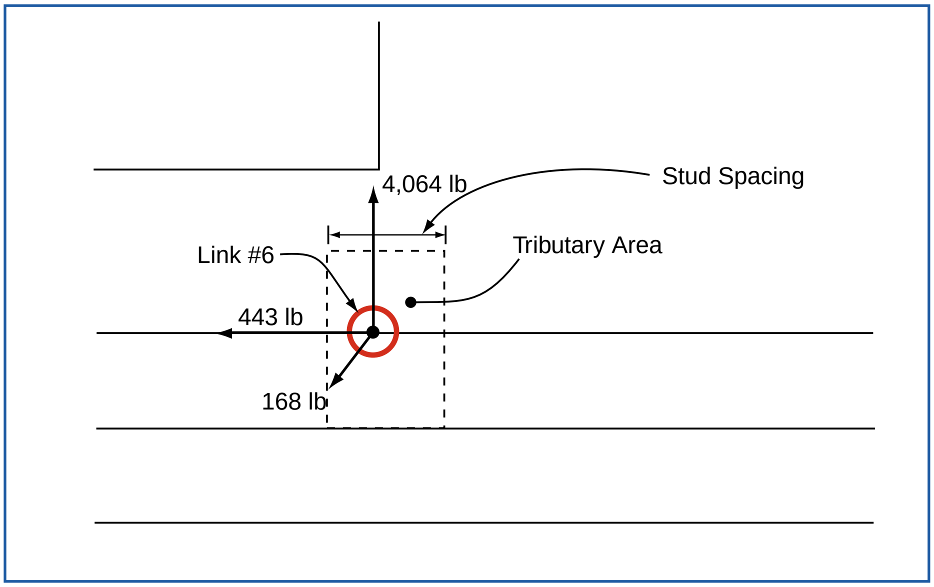



**Figure 12-49**
Steel moment frame at large opening.

In residential construction, moment frames are frequently tubular steel. Tubular steel shapes can be selected that are close to the size of nominal framing lumber. This approach alleviates the need for special, time-consuming methods required to make the steel frame compatible with wood; however, frames made with tubular steel are more difficult to build than frames made with "H" or "WF" flange shapes.

The advantages of the moment frames at window and door openings include the following:

- The shear force/foot of wall is reduced or eliminated because the shear is transferred into the steel frame.
- The deflection is reduced because the load is transferred into the stiffer steel frame.

### Link #7 – Floor Framing to Support Beam

The connection between the floor framing and the floor support beam is Link #7 (see Figure 12-50). The forces on this link include uplift and lateral forces transferred from the roof and walls.

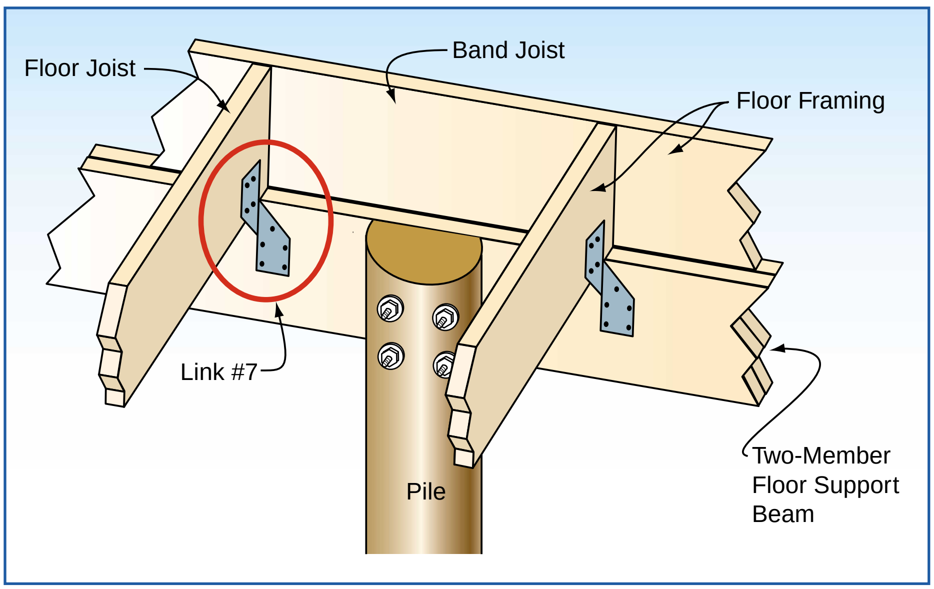
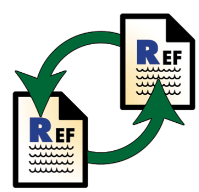



**Figure 12-50**
Link #7 – connection of floor framing to support beam.



**CROSS-REFERENCE**
See Figure 12-52 for an example of the use of wood blocking as an alternative to metal straps.

The uplift (731 lb) and horizontal (443 lb) loads have been determined previously except to account for the dead load from exterior walls and floors and any shearwall uplift taken directly to the beam.

The connection at Link #7 is normally made with metal connectors (see Figure 12-51); however, most metal connectors cannot be used for both uplift and lateral load transfer. When the connectors shown in Figure 12-50 are used for uplift resistance, additional clips must be used for lateral force transfer. This connection can also be made with a wood block nailed to the beam and floor joists (see Figure 12-52). Advantages of a wood connection, provided it is sufficient to transfer the uplift forces, are that it eliminates the need for a metal connector, which is easily corroded, and can be used for both uplift and lateral load transfer.

**Figure 12-51**
Metal joist/beam connector.

**Figure 12-52**
Wood joist/beam connector.

Table 12.8 (on page 12-47) lists the uplift resistance provided by 16d nails acting in single shear. It is common practice to frame wood buildings with 16d nails, so they have been used in this table of values. The wood floor members will normally be 2x10s so there is sufficient width of the floor joist to install four nails on each side of the beam-to-wood uplift connector, for a total of eight. This connection can then be used for a maximum uplift of 1,136 lb shown for eight nails in Table 12.8 (page 12-47).

### Link #8 – Floor Support Beam to Foundation (Pile)

Link #8 connects the floor support beam to the top of the pile. This link, illustrated in Figure 12-53, must resist uplift and lateral loads placed on the building by wind forces (and, when necessary, seismic forces). The pile at Link #8 must resist 1/31 of the total lateral load because there are 31 piles in the foundation system. The total lateral force per pile was determined in Calculation 12.6. The wind load per pile is 989 lb.

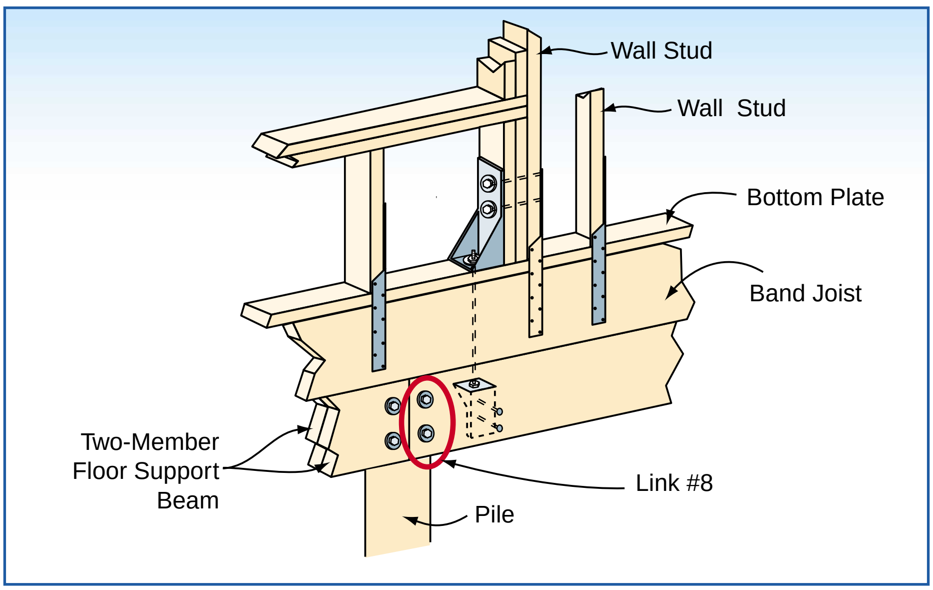



**Figure 12-53**
Link #8 – connection of floor support beam to foundation.

The connection between the beam and the top of the pile is critical. Bolts, strong enough to resist shear forces at this link, are typically used. For this calculation, it is assumed the pile is sufficiently stiff and embedded to resist twisting so the connection does not have to resist rotation. However, we know that, in practice, this is not true and that wind and flood forces can come from a direction such that the entire building can rotate about the tops of the piles. The rotation **could** be reduced by the installation of braces at each corner pile between the floor support beam in the plane of the floor (see Figure 12-54). An alternative method is sheathing the underside of the floor framing.

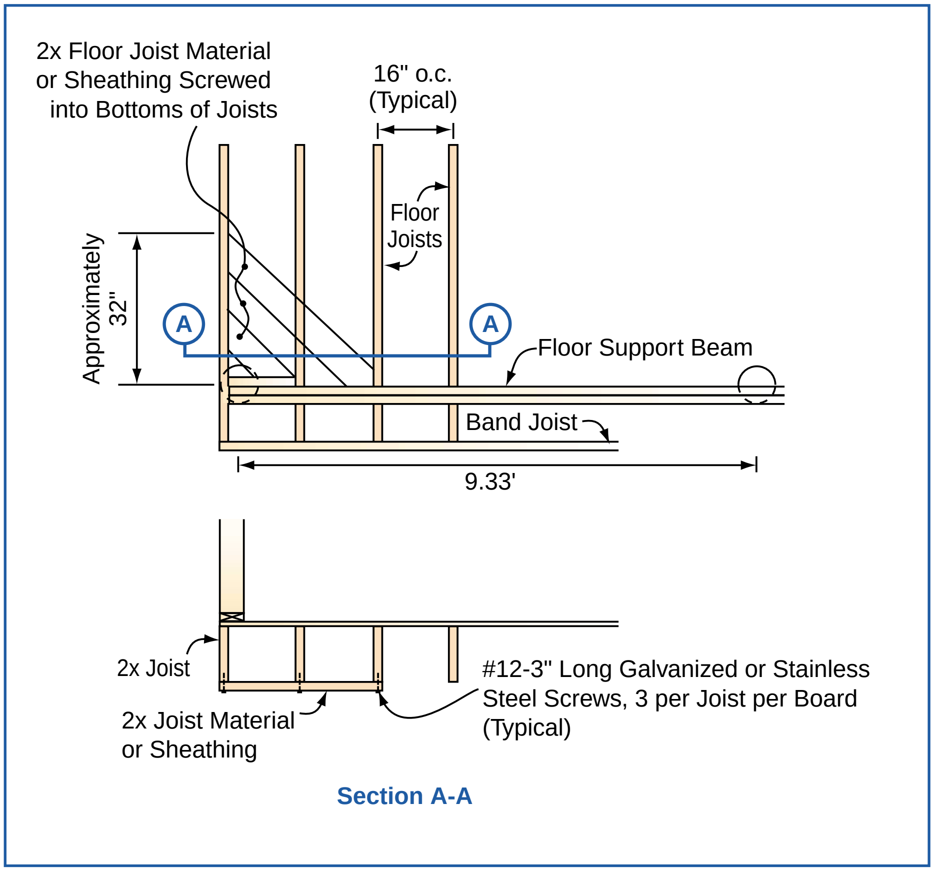



**Figure 12-54**
Corner pile bracing to reduce pile cap rotation.

The connection at Link #8 transfers loads from the beam to the pile in shear through the bolts. The bolts rarely fail in shear, but can yield (become plastic and bend) or can crush wood fibers in the beam or the pile. The strength of the connection depends on the following:

- size of the main member (notched pile) and side member(s) beam
- species of the main and side members
- size and yield stress of the bolts
- configuration of the connection (single shear or double shear)
- sufficient edge and end distances for the bolt holes
- adequate support or seat for gravity loads

This connection also transfers the shearwall uplift load and must be able to resist the tension–compression couple associated with shearwall connections. Where possible, double-shear (three-member) connections should be chosen over single-shear (two-member) connections. Connections using double shear are more efficient in transferring loads. Figure 12-55 shows both single- and double-shear connections.

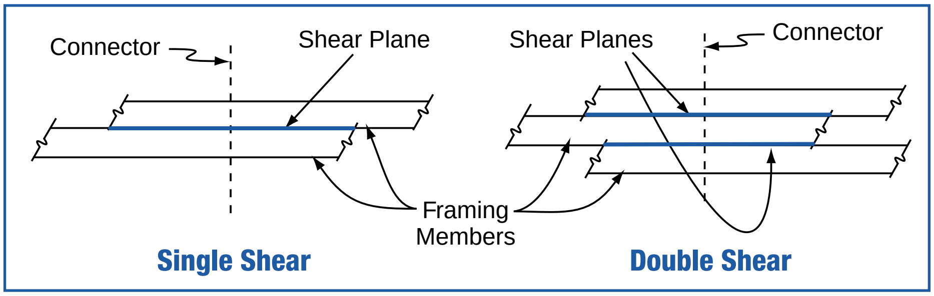



**Figure 12-55**
Single and double shear connections.

Determining yield modes of bolted connections is outside the scope of this manual. Chapter 8 of the NDS lists allowable loads on a variety of bolted connections.

The allowable shear for one 5/8-inch-diameter steel bolt installed as follows is 2,324 lb:

- in a 7-1/2-inch-thick notched section of a pile
- supporting two 1-1/2-inch-thick side members (3 inches total)
- both members are southern pine (Specific Gravity = 0.55)
- in double shear

Resisting the 4,064-lb load (from Calculation 12.15) would require two bolts.

**NOTE**
See Table 12.13, on page 12-69, for a discussion of pile installation methods.

**NOTE**
The minimum recommended pile spacing is 8 feet. This spacing will allow debris to flow between piles.

## 12.4.3 Foundation Design

### 12.4.3.1 Pile Design

To determine pile capacities and embedment depths of driven piles, soil properties must be either known or assumed. In the case study building, the soil is assumed to have the following characteristics:

- medium dense sand, graded SP
- $\gamma = 65$ lb/ft$^3$ (unit weight of saturated sand)
- $\phi = 30°$ (angle of internal friction)

The pile design now must be completed for the following load cases:

- gravity (compressive) loads
- uplift (tension) loads
- horizontal loads and the moments developed by horizontal loads

Pile capacity formulas used in this example are obtained from the U.S. Navy's Design Manual 7.2, *Foundation and Earth Structures* (U.S. Department of the Navy 1982).

**NOTE**
Pile design involves complex soils mechanics theory; pursuing soil mechanics beyond what is included in the formulas for compression and tension pile capacity is beyond the scope of this manual. This manual does not provide a single pile embedment depth recommendation for all coastal areas.

### Gravity Loads

For gravity loads, a 15-foot embedment depth will be assumed. With the pile extending approximately 9 feet above grade, the total pile length will be approximately 24 feet. Formula 12.6 determines the allowable compression load on the pile. The resistance of the pile results from end bearing and friction.

Formula

Ultimate Compression Capacity of a Single Pile (Developed by End Bearing and Frictional Resistance)

**Formula 12.6 Ultimate Compression Capacity of a Single Pile**

$$Q_{ult} = P_T N_q A_T + \Sigma(k_{HC})(P_0)D(\tan \delta)(s)$$

where:

$Q_{ult}$ = ultimate load capacity in compression (lb)

$P_T$ =effective vertical stress at pile tip (lb/ft$^2$)

$N_q$ = bearing capacity factor

$A_T$ = area of pile tip (ft$^2$)

$k_{HC}$ = earth pressure coefficient in compression

$P_0$ = effective vertical stress over depth of embedment, D (lb/ft$^2$)

$\delta$ = friction angle between pile and soil

$s$ = surface area of pile per unit length (ft)

$D$ = depth of embedment (ft)

When the variables associated with the soil type given in this case study are used in the formula, $Q_{ult}$ = 34,015 lb. The foundation guide recommends an FS of 3.0; thus, $Q_{allow}$ = 34,015/3 = 11,338 lb is the allowable capacity in compression for this pile with an embedment depth of 15 feet.

From the determination of dead load (Calculation 12.1), plus a live load of 20 lb/ft$^2$ on the roof and 40 lb/ft$^2$ on the floor of the tributary area, the total dead and live load on the case study pile is 5,974 lb. The compression load from overturning adds another 4,736 lb, so the total compression load is 10,710 lb. The allowable compression load is greater than the required load, so the pile is adequate for dead load, live load, and compression and overturning loading.

Recalculating the allowable compression load to account for 1.8 feet of scour (see Section 11.6.11 in Chapter 11 of this manual) reduces the allowable compression load to 10,594 lb, which is still adequate to support the required dead and live loads. If this pile were augered into place instead of driven, the allowable compression load would be reduced to 4,466 lb, or less than 40 percent of the allowable load for a driven pile. For comparison purposes, see Table 12.9, on page 12-62, for allowable loads on driven, augered, and jetted wood piles for this case study.

### Uplift Loads

Pile capacity in tension loads is given in Navy Design Manual 7.2 (USDN 1982). Tension capacity is required to resist uplift and overturning loads. Formula 12.7 determines the tension capacity in a single pile.

**Formula 12.7 Ultimate Tension Capacity of a Single Pile**

$$T_{ult} = \Sigma(k_{HT})(P_0)(\tan\delta)(s)(D)$$

where:

$T_{ult}$ = ultimate load capacity in tension (lb)

$k_{HT}$ = earth pressure coefficient in tension

$P_0$ = effective vertical stress over depth of embedment, D (lb/ft$^2$)

$\delta$ = friction angle between pile and soil

$s$ = surface area of pile per unit length (ft)

$D$ = depth of embedment (ft)

Formula

Ultimate Tension Capacity of a Single Pile (Developed by Frictional Resistance)

The formula predicts a tension capacity ($T_{ult}$) (with 1.8 feet of scour) of 14,965 lb. With the recommended FS of 3.0, the allowable tension capacity of the 15-foot pile is $T_{allow}$ = 14,965/3 = 4,988 lb.

The frictional resistance required of the row of piles under the front porch if only those piles resisted the net overturning moment shown in Calculation 12.5 is 1,259,704 ft-lb/38 feet/7 piles or 4,736 lb/pile. Therefore, a 15-foot embedment satisfies overturning tension as well as compression.

Direct tension must also be checked. Figure 12-56 shows the tributary area that creates this tension. Depending on shearwall connection location, it is also possible that the tension or compression load from the shearwall connector will be concentrated on one pile (as it is in this case study).

The total tension load on the pile highlighted in Figure 12-56 is 8,586 lb, which exceeds the allowable tension load for the 15-foot embedment. This pile needs to be embedded 25 feet to resist the total tension load in this case study.

Table 12.9 summarizes the allowable compression and tension (uplift) capacities of wood piles placed 15 feet below the ground surface.

**Figure 12-56**
Uplift tributary area from hip roof to a single pile.

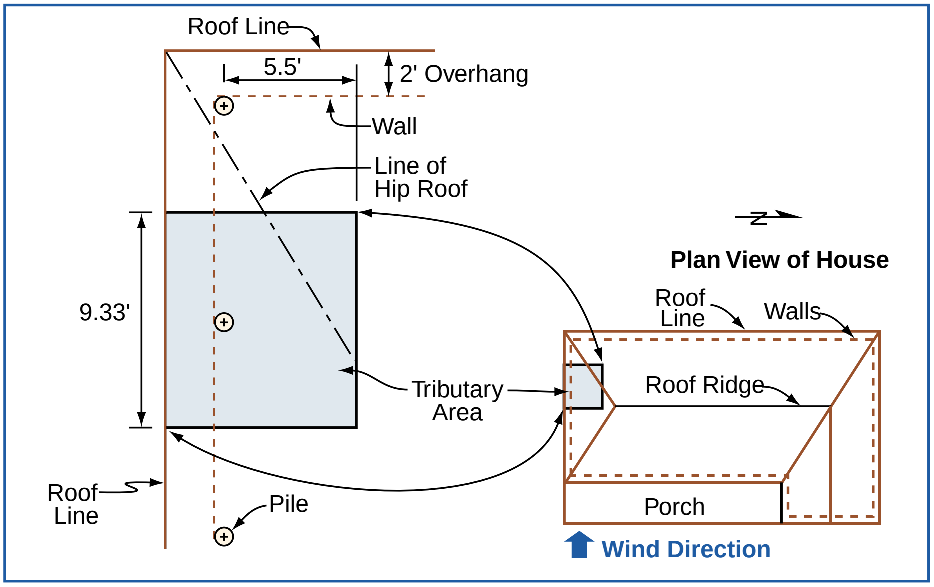



**Table 12.9**
Allowable Capacities of Wood Piles (lb) for the Case Study Example Only, Based on End Bearing and Frictional Resistance

| | Compression | | Tension | |
|---|---|---|---|---|
| **Pile Type** | **No Scour** | **1.8 ft Scour** | **No Scour** | **1.8 ft Scour** |
| **Driven** | 11,338 | 10,594 | 5,477 | 4,988 |
| **Augered** | 4,764 | 4,466 | 1,905 | 1735 |
| **Jetted** | 6,099 | 5,822 | 2,143 | 1,952 |

### Combined Bending and Compressive Loads

Structural members that carry both compressive and bending loads are more susceptible to buckling failure than those carrying only one or the other type of load. The effect of the combination of these loads is checked by an interaction formula shown as Formula 12.8.

Formula
Combined Axial and Bending Stresses

**Formula 12.8 Combined Axial and Bending Stresses**

$$(f_c/F_c')^2 + f_b/[F_b(1-f_c/F_{cE})] \leq 1.0$$

where:

$f_c$ = actual compressive stress (lb/in$^2$)

$F_c'$ = allowable compressive stress (lb/in$^2$)

$f_b$ = actual bending stress (from Formula 12.9)

$F_b$ = allowable bending stress (lb/in$^2$)

$F_{cE}$ = Euler-based buckling stress (lb/in$^2$)

Actual compressive stress $f_c$ is 10,710 lb (from page 12-60)/pile area of 63.62 in$^2$ = 168 lb/in$^2$. The allowable compressive stress $F_c'$ from the NTPC (1995) is:

$$F_c' = F_c C_d C_{ld} C_t C_{pt} C_{sp} C_p C_{cs}$$

$$F_c' = (1{,}278)(1.0)(1.6)(1.0)(0.9)(0.8)(0.15)(1.054) = 233 \text{ lb/in}^2 \qquad [12.16]$$

From the NTPC (1995), the allowable bending stress $F_b'$ equals:

$$F_b' = F_b C_d C_{ld} C_t C_{pt} C_f C_{sp} C_{cs}$$

$$F_b' = (2{,}612)(1.0)(1.6)(1.0)(0.9)(1.0)(0.77)\ (1.054) = 3{,}052 \text{ lb/in}^2 \qquad [12.17]$$

The adjustment factors are listed in the NTPC. $C_d C_t C_{pt}$ are functions of the pile material and loading; $C_f C_{sp} C_{cs}$ are functions of the pile dimensions and length.

From Formula 12.9, $f_b$ = (989 lb)(13.5 ft) (12 in/ft)/71.57 in$^3$ = 2,239 lb/in$^2$

**Formula 12.9**
**Determination of Actual Bending Stress**

$$f_b = (PxL)(12 \text{ in/ft})/S$$

where:

**$f_b$** = actual bending stress (lb/in$^2$)

**P** = applied lateral load (lb)

**L** = allowable unbraced pile length (feet) (from Formula 12.10)

**S** = section modulus (in$^3$)

### Horizontal Loads

The horizontal load on an unbraced pile causes the pile to deflect as illustrated in Figure 12-57. ASCE 24-98, Section C5.2.4.7 (ASCE 1998a), includes a formula (also included in this manual as Formula 12.10) for determining the allowable length of a pile above the point of fixity. The point of fixity is the point at which the pile is assumed to be fixed for translation and rotation, and it occurs at some distance (d) below the ground surface.

In Figure 12-57, the distance H includes the height above original ground plus the additional height resulting from any expected erosion and scour. In this case study, H = 9.5 feet + 1.8 feet = 11.3 feet. The total effective length L is determined by Formula 12.10.

**NOTE**

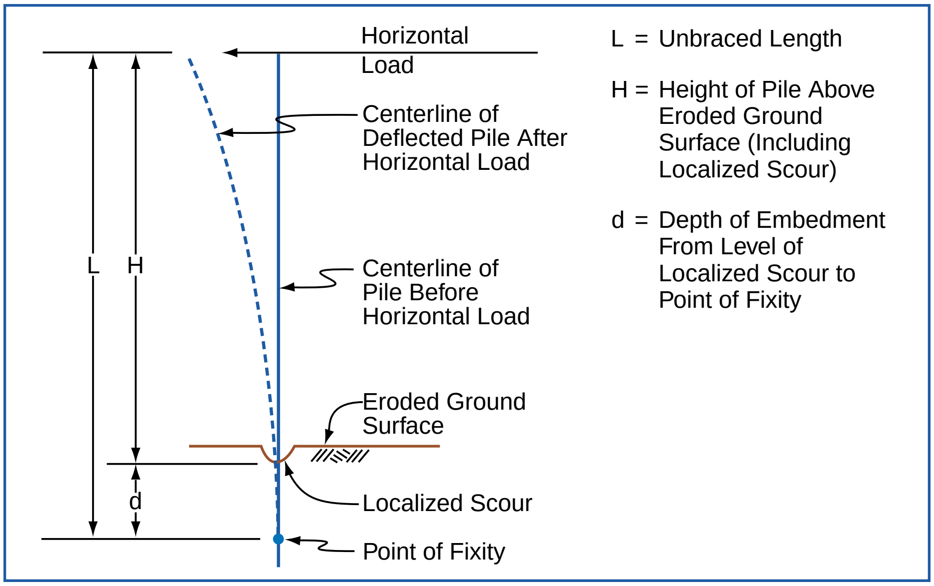
$C_p$ in Calculation 12.16 is a column stability factor. The value of this factor depends on the effective column length from Formula 12.10. Section 3.7 of the NDS provides a formula for $C_p$.

The adjustment factors used in Calculations 12.16 and 12.17 are described in detail in the *Technical Guidelines for Construction With Treated Round Timber Pilings*, by the National Timber Piling Council (NTPC 1995).

Formula

Determination of Actual Bending Stress

**Figure 12-57**
Deflected pile shape for an unbraced pile. The effective length of this pile is equal to L x (K=2.1).




formula
Effective Length of Unbraced Pile

**Formula 12.10 Effective Length of Unbraced Pile**

$$L = K(H + d/12)$$

where:

**L** = length of pile (feet)

**d** = **depth of embedment from level of localized scour to point of fixity = $(1.8)(EI/n_h)^{1/5}$ (inches)**

**E** = Modulus of Elasticity of pile material (lb/in$^2$)

**I** = moment of inertia of pile material (in$^4$)

**$n_h$** = modulus of subgrade reaction (lb/in$^2$)

**K** = effective length factor based on end conditions

**H** = length of pile between horizontal load and eroded ground surface (see Figure 12-57) (feet)

Table 12.10 lists recommended values for $n_h$, the modulus of subgrade reaction, for a variety of soils (Bowles 1996), which is used in Formula 12.10.

**Table 12.10**
Values of $n_{h,}$ Modulus of Subgrade Reaction

| Soil | $n_h$ Modulus of Subgrade Reaction (lb/In$^3$) |
|---|---|
| Dense sandy gravel | 800-1,400 |
| Medium dense coarse sand | 600-1,200 |
| Medium sand | 400-1,000 |
| Fine or silty fine sand | 290-700 |
| Medium clay (wet) | 150-500 |
| Soft clay | 6-150 |

The pile is assumed to have the following characteristics:

- material: southern pine with a modulus of elasticity (E) of 1,500,000 lb/in$^2$
- tabulated fiber bending stress: $F_b$ = 2,612 lb/in$^2$ 1995 National Timber Piling Council (NTPC)
- section: round timber with a nominal 9-inch diameter (8 inches at base; 10 inches at top); moment of inertia (I) = 322 in$^4$; section modulus (S) = 71.57 in$^3$ (at 9-inch section)
- end restraints: free at the top and fixed at the base; the fixed/free end conditions of a cantilever pile produce an effective length factor (K) of 2.1 (from Appendix G of the NDS).

Calculating the pile length L for a round timber pile using Formula 12.10, assuming $n_h$= 700 lb/in$^3$:

$$d = (1.8)[(1.5x10^6)(322)/700]^{1/5} = 26.5 \text{ in}$$

$$L = (11.3 \text{ ft} + 26.5/12) = (11.3 + 2.2) = 13.5 \text{ ft} \qquad [12.18]$$

This length represents the unbraced length of the pile to its point of fixity. This length is used to determine the actual bending stress $f_b$ (Formula 12.9) and the $C_p$ factor used in Calculation 12.16

Now using Formula 12.8, the effect of combined bending and axial loads is determined. Using $f_b$ = 2,239 lb/in$^2$, $F_b$ = 3,052 lb/in$^2$, $f_c$ = 168 lb/in$^2$, and $F_c$ = 233 lb/in$^2$, the interaction formula = 3.15, which is much greater than 1.0. Therefore, this pile design does not work. The alternatives are as follows:

- larger piles
- reduced spacing
- shorter effective length
- piles with greater strength than wood

In order for a wood pile to be acceptable, the effective length shown in the last column of Table 12.11 must be greater than approximately13.5. This length is determined by working backwards through Formula 12.8 and the factor $C_p$ to determine what the maximum effective length could be when Formula 12.8 yields a value of 1.0. As can be seen from Table 12.11, the pile must be at least 12 inches in diameter for the load and height criteria to be met.

Another way to make this pile design acceptable is to shorten the effective length with bracing. This method is discussed in Section 12.4.5

**Table 12.11** Maximum Unbraced Lengths of Various Pile Materials for the Case Study Loads[a]

| Pile | Material[b] | E (lb/in$^2$) | I (in$^4$) | S (in$^3$) | $F_B$ (lb/in$^2$) | Length (ft) |
|---|---|---|---|---|---|---|
| Wood – 9" diameter | P.T. S.Y.P. | 1,500,000 | 322 | 71.57 | 3,052 | 7.1 |
| Wood – 10" x 10" square | P.T. S.Y.P. | 1,500,000 | 678.8 | 142.9 | 3,052 | 12.8 |
| Wood – 12" diameter | P.T. S.Y.P. | 1,500,000 | 1,018 | 169.65 | 3,052 | 15.1 |

[a] Buckling and the interaction formula not considered.

[b] P.T. = Pressure Treated; S.Y.P. = Southern Yellow Pine.

### Allowable Shear Stress

From the NTPC, the allowable shear stress $F_v'$ equals:

$$F_v' = F_v C_{ld} C_t C_{pt}$$

$$F_v' = (110)(1.6)(1.0)(0.9) = 158 \text{ lb/in}^2$$

The actual average shear stress is the horizontal load at the pile-ground intersection from Calculation 12.7 divided by the area of the pile, or 2,293 lb/ 63.62 in$^2$ = 36 lb/in$^2$. The maximum shear stress is 4/3 the average shear stress, or 48 lb/in$^2$. Thus shear is not a failure mode for the pile under the case study conditions.

### Deflection

From beam theory, the deflection at the top of the case study 13.5-foot-long (H+d only) cantilevered pile horizontally deflected by a 989-lb load is 2.9 inches. This is an excessive amount of deflection that most building occupants would find unacceptable. This deflection is being caused by very high winds, however, in what is expected to be an unoccupied house. This deflection can be reduced by bracing to reduce the unbraced length. See Section 12.4.5 for additional information.

### Other Pile Types

Other types of piles and methods of installation are available, including the following:

- precast concrete
- cast-in-place concrete
- steel H-sections
- concrete filled steel pipe piles

Some advantages of, and special considerations for, of each of these are listed in Table 12.12.

**Table 12.12**
Pile Material Selection

| Pile Material Type | Advantages | Special Considerations |
|---|---|---|
| Wood (ASTM-D25) | • Comparatively low initial cost<br>• Permanently submerged piles are resistant to decay<br>• Relatively easy to drive in soft soil<br>• Easy to connect to wood framing<br>• Suitable for friction and end bearing pile | • Difficult to splice<br>• Subject to eventual decay when in soil or intermittently submerged in water<br>• Vulnerable to damage from driving — easy to split<br>• Comparatively low compressive load<br>• Relatively low allowable bending stress |
| Concrete (ACI 318 for concrete, ASTM-A15 for reinforcing steel) | • High compressive load capacity for precast/ prestressed piles<br>• Corrosion resistant<br>• Can be driven through some hard material<br>• Suitable for friction and end bearing pile<br>• Reinforced piles have high bending resistance | • High initial cost<br>• Difficult to splice<br>• Vulnerable to breakage<br>• Must be reinforced, no bending allowed in plain concrete<br>• More difficult to attach to wood framing<br>• Not usable in high seismic areas |
| Steel (ASTM-A36) | • High resistance to bending<br>• Easy to splice<br>• Available in many lengths and sizes<br>• Able to drive through hard subsurface material<br>• Suitable for end bearing<br>• High compressive capacity | • Vulnerable to corrosion<br>• May be permanently deformed if impacted by heavy object<br>• High initial cost<br>• Some difficulty attaching wood framing |

There are several methods for installing piles, including driving, augering, and jetting.

**Driving** a pile is hitting the top of the pile with a pile driver hammer until the pile reaches the desired depth. In some areas, piles are driven with vibratory hammers, which generate vertical oscillating movements that reduce the soil stress against the pile and make the piles easier to drive. Ultimate load resistance is achieved by a combination of end bearing of the pile and frictional resistance between the pile and the soil. A record of the blow counts from the pile driver can be used with a number of empirical formulas to determine capacity.

**Augering** is drilling a full or partial pile-diameter hole in the soil to some predetermined depth, installing the pile, and then driving the pile into the hole until it reaches refusal.

**Jetting** is removing soil with a jet of water (or air) as the pile is driven, and then driving the pile down until it reaches refusal. Both augering and jetting remove natural, undisturbed soil along the side of the pile. Load resistance for both of these methods is achieved by a combination of end bearing and frictional resistance, although the frictional resistance is much less than that provided by driven piles.

Figure 12-58 illustrates the three pile installation methods. Table 12.13 lists advantages and special considerations for each method.

It is important to note from Table 12.13 that the 40-percent reduction in the k factors for compression and tension loads on augered and jetted piles reduces the allowable loads by the same percentage. Therefore, for the case study building, if the piles were augered or jetted into place, the allowable load in compression would be reduced to 2,710 lb, which is then less than the required load of 10,710 lb. The allowable load in tension is reduced to 1,220 lb, which is less than the net uplift load of 8,429 lb. Consequently, if the piles were installed by augering or jetting, they would have to be embedded further to resist both the compression and tension loads .

### 12.4.3.2 Masonry or Concrete Columns

While local floodplain regulations require the use of open foundations in V zones, foundation materials are not restricted to wood as long as the foundation is anchored to resist flotation, collapse, and lateral movement. Solid foundation walls may be constructed in A zones, but must comply with the NFIP and have openings to equalize hydrostatic pressure. Columns constructed of concrete or reinforced masonry can be used, but must be constructed on a solid foundation such as a spread footing or a pile cap to distribute loads onto undisturbed soils below the anticipated erosion and scour depth. This type of foundation is normally assumed to be fixed at its base.

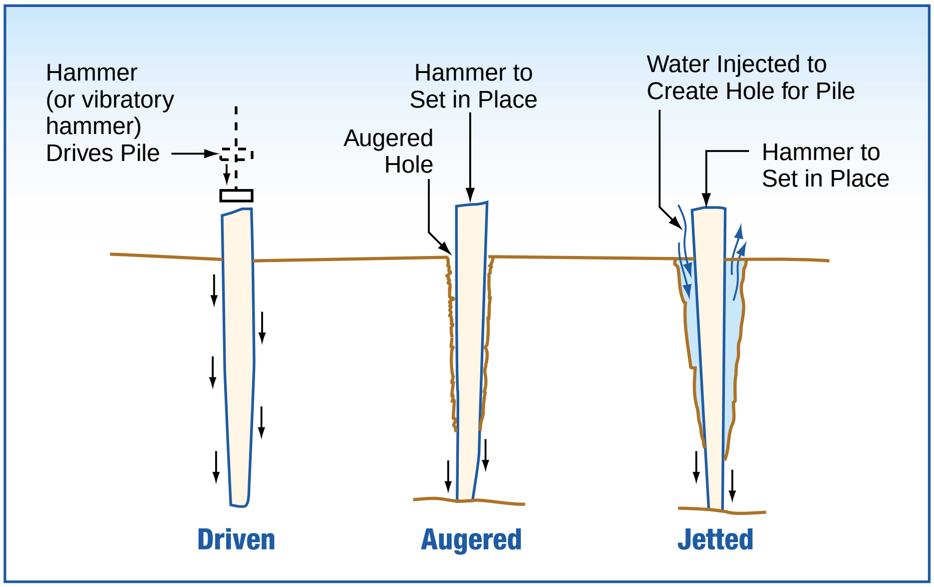


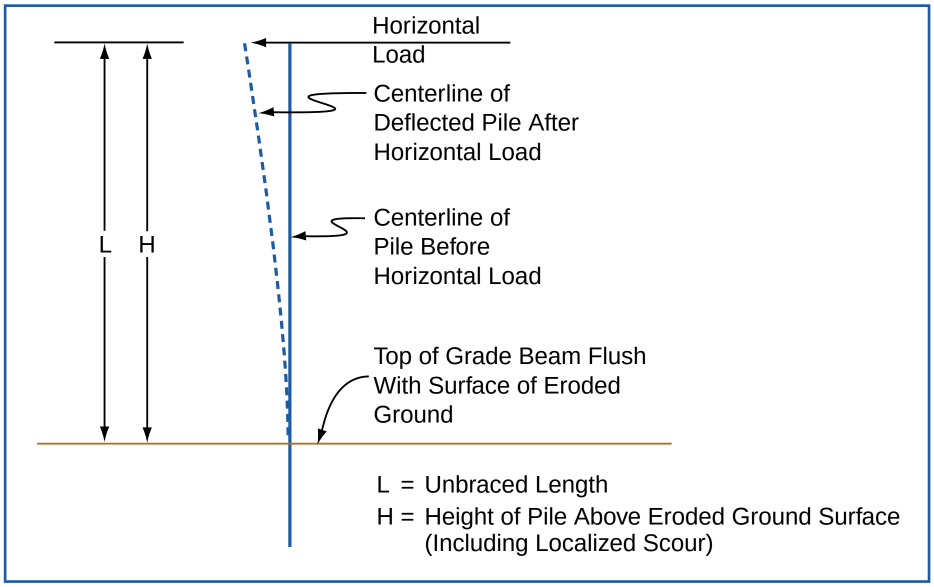

**Figure 12-58**
Pile installation methods.

**Table 12.13** Selection of Pile Installation Method

| Pile Installation Method | Advantages | Special Considerations |
|---|---|---|
| Driven | • Well suited for friction pile<br>• Common construction practice<br>• Pile capacity can be determined empirically | • Difficult at times to reach terminating soil strata, which is not necessary for friction piles<br>• Difficult to maintain plumb during driving and thus maintain column lines |
| Augered | • Economical<br>• Minimal driving vibration to adjacent structures<br>• Well suited for end bearing<br>• Visual inspection of some soil stratum possible<br>• Convenient for low headroom situations<br>• Easier to maintain column lines | • Requires subsurface investigation<br>• Not suitable for highly compressible material<br>• Disturbs soil adjacent to pile, thus reducing earth pressure coefficients $k_{HC}$ and $k_{HT}$ to 40 percent of that for driven piles<br>• Capacity must be determined by engineering judgment or load test |
| Jetted | • Minimal driving vibration to adjacent structures<br>• Well suited for end bearing<br>• Easier to maintain column lines | • Requires subsurface investigation<br>• Disturbs soil adjacent to pile, thus reducing earth pressure coefficients $k_{HC}$ and $k_{HT}$ to 40 percent of that for driven piles<br>• Capacity must be determined by engineering judgment or load test |

A way to reduce this unbraced length is to "fix" the column in both translation and rotation at some point along its length. A timber pile can be fixed in only translation by the addition of horizontal grade beams. These grade beams may be constructed of concrete or wood; they must be installed so that they are self-supporting and the effects of scour have been considered. The effectiveness of grade beams in "fixing" the piles is subject to engineering judgment. See Figure 12-59 for an illustration of this design concept.

**Figure 12-59**
Design method for reducing effective column length of a cantilever pile.




Guidance on the design of concrete or masonry piers on spread footings follows. It will be obvious why the use of these foundation methods in V zones and coastal A zones is often not appropriate. It should be noted, however, that foundations of this type can be appropriate in V zones and coastal A zones where bedrock, coral rock, or some other stratum that terminates erosion and scour is present.

First for V zones, the design example that follows will use the loads developed in previous sections. From previous examples and the Flood Load Example Problem presented on page 11-30 in Chapter 11, L = 13.3 feet (see calculation at left). It is assumed that the allowable soil bearing pressure is 2,000 lb/ft$^2$. Figure 12-60 illustrates a column on a spread footing with the axial ($P_a$) and lateral ($P_l$) forces determined previously.

## CALCULATION OF UNBRACED PILE LENGTH (L)

Distance from top of pile to DFE = 2 feet

Distance from DFE to eroded ground surface = 9.5 feet

Scour = 1.8 feet

**L = 2 feet + 9.5 feet + 1.8 feet = 13.3 feet**

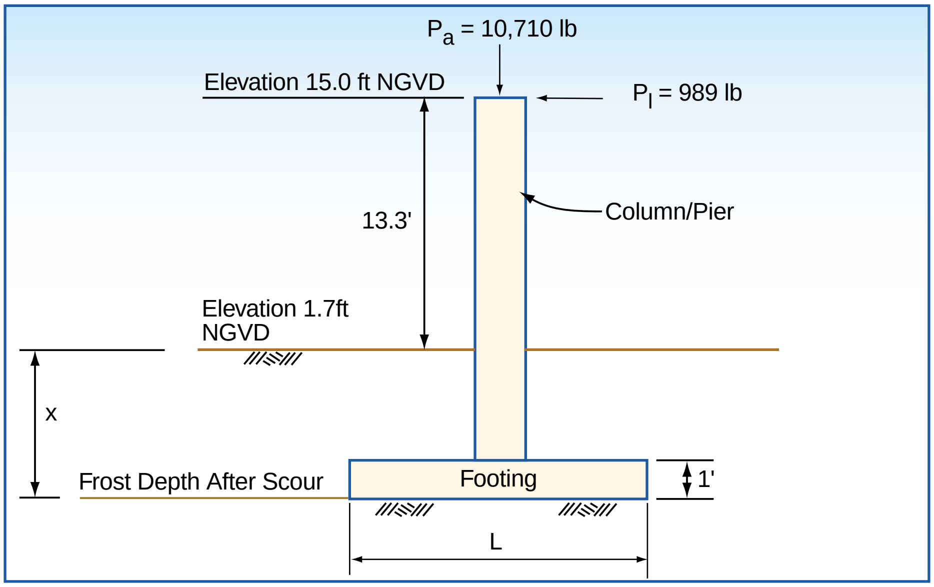



**Figure 12-60**
Column/pier on a spread footing.

The following foundation elements and design parameters must be determined:

- soil bearing pressure assuming a footing size or a footing size assuming 2,000-lb/ft$^2$ soil bearing pressure
- bending stresses in the column, to determine the amount of reinforcing steel to place in the column
- compressive stresses in the column, to determine whether these stresses exceed the allowable for the material selected for the column
- bending stresses in the footing, to determine the amount of reinforcing steel to place in the footing

This foundation type must be designed for the same conditions as the pile – gravity loads, uplift loads, horizontal loads, and the resulting bending that occurs in the column.

### Gravity Loads

The footing must be large enough to carry the following loads:

- weight of the foundation
- axial load $P_a$

Formula

Determination of Square Footing Size for Gravity Loads for the Case Study Building

The footing size can be determined by Formula 12.11.

**Formula 12.11**
**Determination of Square Footing Size for Gravity Loads**

$$L = (\{P_a + [col.\ h + (x\text{-}1)](col.\ w)(col.\ t)(w_c)\}/[q - (foot.\ t)w_c])^{0.5}$$

where:

**L** = square footing dimension

$P_a$ = axial load

**h** = column height above scour elevation

**x** = depth of bottom of footing from scour elevation

**w** = column width

**t** = column thickness

$w_c$ = unit weight of column and footing material

**q** = soil bearing pressure

**foot. t** = footing thickness

For a concrete column and footing with h = 13.3 feet, x = 2 feet, column dimensions are 16 inches square, foot.t = 1 foot, q = 2,000 lb/ft$^2$, $P_a$ = 10,710 lb, and the unit weight of concrete is 150 lb/ft$^3$, the required footing size to support only the gravity load is 2.8 feet square.

### Uplift Loads

Uplift resistance is provided by the weight of the foundation and column and the weight of soil on top of the footing. In this case, where the height of the column is 13.3 feet and concrete weighs 150 lbs/ft$^3$, the footing and column weight of 4,920 lb does not exceed the uplift load of 8,429 lb. The uplift load can be resisted by a footing that is 5.6 feet square.

### Horizontal Loads

Overturning of the foundation must be resisted by the allowable soil bearing pressure and the weight of soil above the "heel" of the footing. This footing design concept is very similar to that of a retaining wall. The centroid of resistance of the soil to the overturning forces acts at a point R located as shown in Figure 12-61. It is assumed for this analysis that q is always in the middle one-third of the footing, so overturning stability is satisfied.

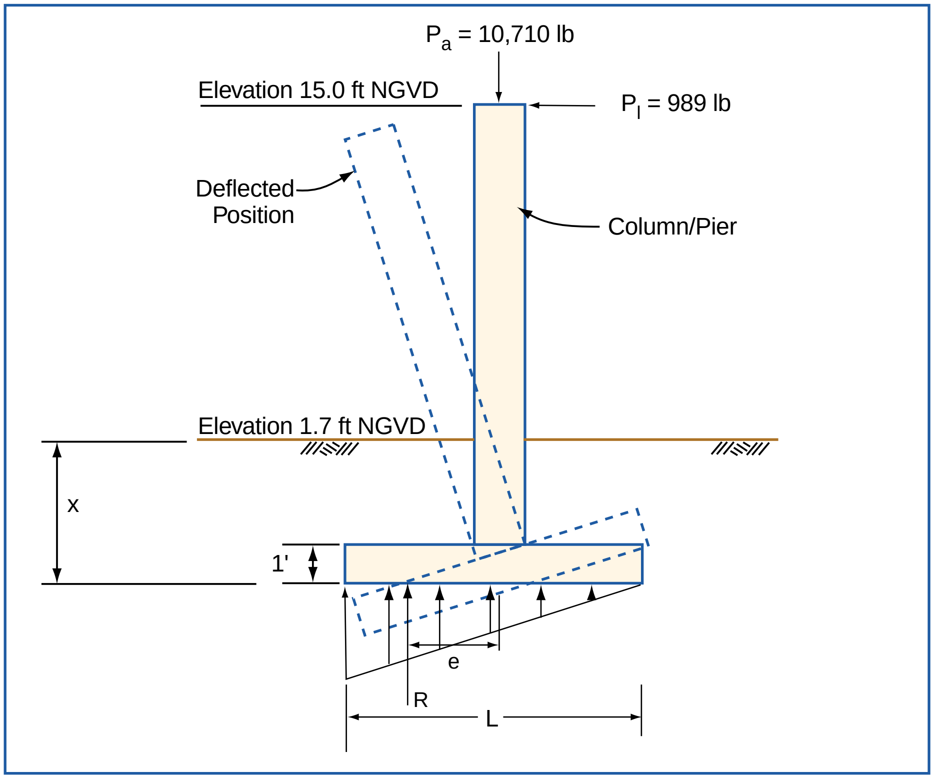



**Figure 12-61**
Horizontal load diagram for columns/piers on spread footings.

As illustrated, if the soil bearing capacity is not adequate to support the overturning forces, the column and footing rotate. Sufficient rotation will result in either failure of the foundation due to excessive settlement or failure in the connection at the top of the column. See *Foundation Analysis and Design* (Bowles 1996) for additional information on foundation design.

Determining the length of footing L required to resist the loads given in this case study without exceeding the 2,000-lb/ft$^2$ allowable soil bearing pressure is an iterative process. Using Formula 12.12, the square footing length L must be such that $q_{max}$ is 2,000 lb/ft$^2$ and $q_{min}$ is 0 lb/ft$^2$. The footing size for this case study and this unique set of parameters is 5 feet.

**Formula 12.12 Determination of Soil Pressure for the Case Study Building**

$$q = P_t/L^2 \pm 6M/L^3$$

where:

**q** = soil bearing pressure

**$P_t$** = total vertical load, including $P_a$, weight of column and weight of footing

**L** = square footing dimension

**M** = moment = Pl(13.3 + x), where x = the depth of the bottom of the footing from the scour elevation

**e** = eccentricity (see Figure 12-61) = M/P, which cannot exceed L/6

Formula

Determination of Soil Pressure for the Case Study Building—From *Foundation Analysis and Design* (Bowles 1996)

In order for the pier and footing to act as one unit, the connection between the footing and the base of the pier must be fixed. This fixed condition can only be developed with sufficient steel reinforcement that is adequately anchored into both the footing and the pier.

When the uplift load of 8,429 lb is considered with the lateral loads (which would be the case for the windward- or seaward-most row of piles), and $q_{max}$ is 2,000 lb/ft$^2$ and $q_{min}$ is 0 lb/ft$^2$, the required footing size is 9.7 feet square. Thus the footing size is governed by the combination of uplift and lateral forces.

A 9.7-foot-square footing is not practical when the pile spacing is 9.33 feet o.c. in one direction and 11 feet o.c. in the other direction. In addition, excavation in sand for a 9.7-foot-square footing 4 feet below grade results in, for all practical purposes, a continuous 1-foot-thick concrete mat under the house, approximately 4 feet below grade. This type of foundation, known as a “mat” or “raft” foundation, is used in some locations along the coast of the United States.

**Allowable Bending Stress and Moment**

Design for bending in concrete or masonry involves designing steel reinforcement to resist tensile stresses (because neither concrete nor masonry has much tensile strength) and designing the section to resist failure in compression or crushing. Design guidance can be found in many of the references listed at the end of this chapter. The determination of these stresses involves effort beyond the scope of this manual and is left to the designer.

If the case study building were placed in an A zone where the breaking wave height is just less than 3 feet (see Chapter 6 for a description of A zones) and all other loads on the building were the same as the example above, the following design parameters would apply:

- The lateral force is minimally reduced.
- The height of the building above the ground (after scour) is only 1 foot lower than the V zone building (see Figure 11-13 for BFEs for the case study) so “moment arms” are practically the same as before.
- With the moments on the concrete or masonry piers practically the same as before and forces only slightly reduced, the footing length is practically the same as the footing length required in the V zone.

**As a result, and for other reasons noted in previous chapters, this manual recommends that the foundation standards applied in V zones also be used for coastal A zone construction.**

Prescriptive designs, including column sizes, reinforcing requirements, and allowable heights, are available for both masonry and concrete materials. Designers should refer to the *Masonry Designers' Guide* (ACI 1993) and *Building Code Requirements for Masonry Structures*, ACI 530, (ACI 1999a) for information on how to design masonry columns, and to *Building Code Requirements for Structural Concrete and Commentary*, ACI 318, (ACI 1999b) for information on how to design concrete columns.

### 12.4.4 Other Important Load Paths

Several additional design considerations must be investigated for the case study building in order for the design to be complete. The details of these investigations are left to the designer, but they are mentioned here to more thoroughly cover the subject of continuous load paths and to point out that many possible paths require investigation.

From Figure 12-16, the following vertical load paths should also be investigated:

- uplift of the front porch roof
- uplift of the main roof section that spans the 28-foot width of the building

The following lateral load paths also need to be investigated:

- lateral load on the front gable wall (see "Shearwalls" on page 12-49)
- interior shearwalls noted on Figure 12-44
- possible twisting of the L-shaped front during either a wind or seismic event (see Section 12.4.4.2)

#### 12.4.4.1 Gable Wall Support

There are many cases of failures of gable-end walls during high-wind events. The primary failure modes in gable-end walls are as follows:

- A gable wall that is not braced into the structure collapses, and the roof framing falls over (see Figure 12-62).
- An unsupported gable-end eave ladder used for overhangs is lifted off by the wind and takes the roof sheathing with it.
- The bottom chord of the truss is pulled outward, twisting the truss and causing an inward collapse.

**Figure 12-62**
Hurricane Andrew (1992), Dade County, Florida. Gable-end failure.

It is common in non-coastal areas to expect the roof sheathing to provide the required lateral bracing of the roof system. However, the wind pressures are too great in coastal areas, and certainly during high-wind events, to expect that the sheathing is sufficient to provide the required lateral support.

Recommendations for additional gable wall bracing are provided in the *Guide to Wood Construction in High Wind Areas*, by the Wood Products Promotion Council (WPPC 1996), and in retrofitting and rebuilding guidance for the U.S. Virgin Islands. The bracing recommendation shown in Figure 12-63 is based on an illustration in the WPPC publication. In addition, truss manufacturers and the Truss Plate Institute recommend that permanent lateral bracing be installed on all roof truss systems.

#### 12.4.4.2 Building Eccentricities

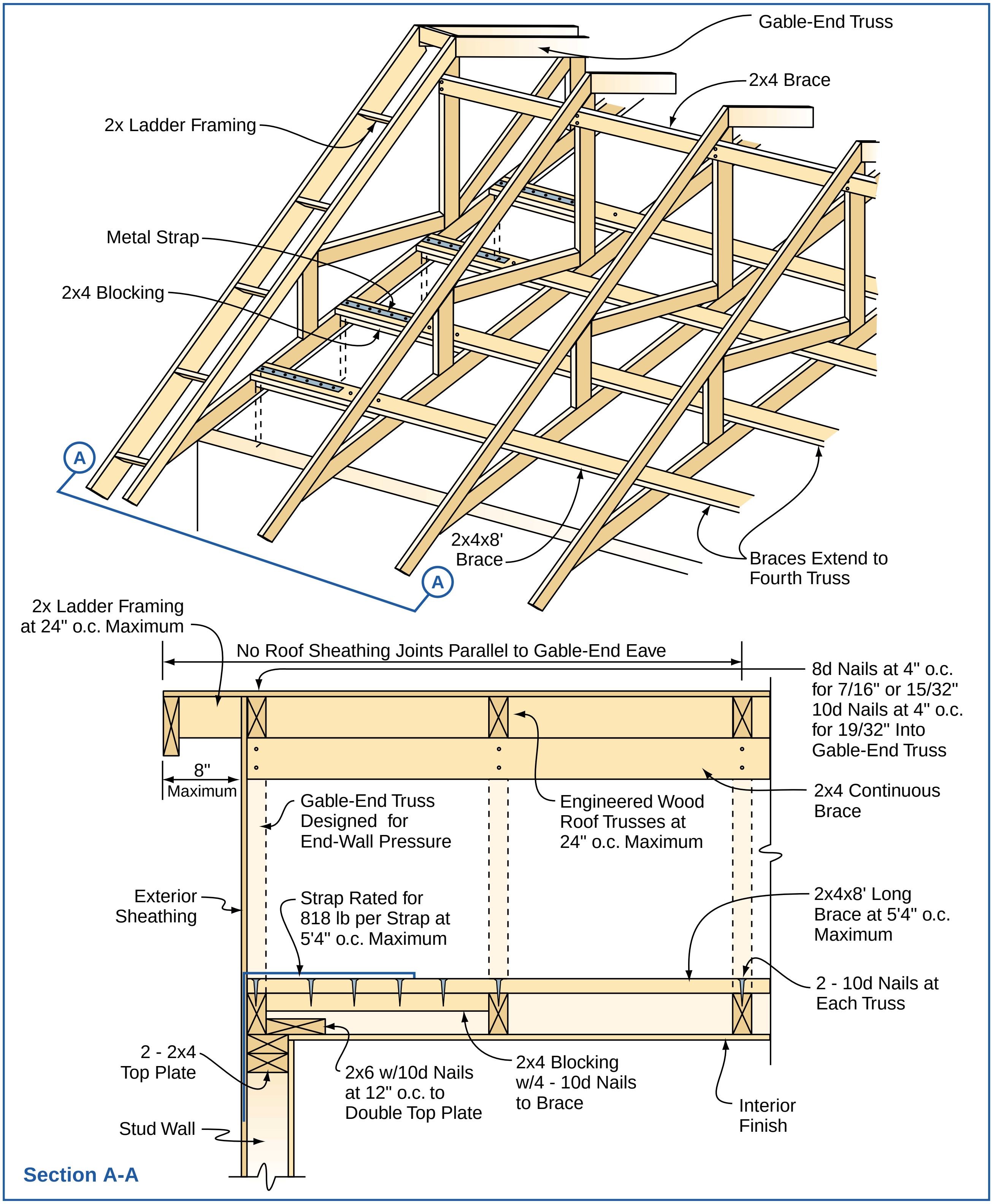
In the simple one-story, L-shaped case study building, the L-shape contributes an eccentricity to the distribution of forces. High winds can cause the L to rotate about the inside corner where the extension meets the main portion of the building. A seismic event can also create rotation about the inside corner. The differences in the two “masses” located at the two centers of gravity will create stress at the inside corner and on the pile-supported foundation.

### 12.4.5 Bracing

In a pile foundation, piles can be braced in three ways:

- grade beams
- diagonal bracing across the full pile heights
- knee bracing

**Figure 12-63** Gable-end bracing recommendations.




Source: Wood Products Promotion Council (1996)

**Grade beams** provide support in the horizontal plane parallel to the floors. **Diagonal braces** are normally attached to the pile near the top and secured to the adjacent pile either near the ground surface or at the height that reduces the unbraced length to the required height. **Knee braces** are normally installed at 45° angles between the floor framing and the pile and are usually placed within 4 feet of the top of the pile. Figures 12-64, 12-65, and 12-66 show examples of these bracing methods.

**Figure 12-64**
Grade beams.

**NOTE**
This manual recommends that if full-height diagonal bracing is used, it be placed parallel to the direction of current and waves.

**Figure 12-65**
Diagonal bracing.

**NOTE**
ASCE 24-98 (ASCE 1998a) recommends that designers strive to establish a stable design free of bracing and use bracing only to add rigidity to the design for the comfort of the occupants. As noted in ASCE 24-98, past experience has shown that cross-bracing often fails during a storm event and does not provide the expected degree of support.

**Figure 12-66**
Knee bracing.

Bracing is used to reduce the effective unbraced pile length, which will reduce the lateral movement of a cantilever foundation system when a lateral load is applied and reduce the stresses in the pile.

### 12.4.5.1 Grade Beams

Because grade beams are usually placed at or below grade, they are normally constructed with either wood or concrete. The maximum allowable unbraced pile length in this case study is 7.1 feet. In the case study, erosion and scour expose 11.3 feet of pile, thus grade beams cannot be used in this case to reduce the effective pile length.

Grade beams have advantages and disadvantages.

Advantages:

- When grade beams are placed near the ground surface, they do not obstruct the area between piles, making the space more usable.
- Grade beams reduce the potential for catching debris during a storm because they do not block the area between the piles, the floor, and the ground surface.

Disadvantages:

- Grade beams become an obstruction around which high-velocity flow must be redirected. Scour potential around and under grade beams can be significant.

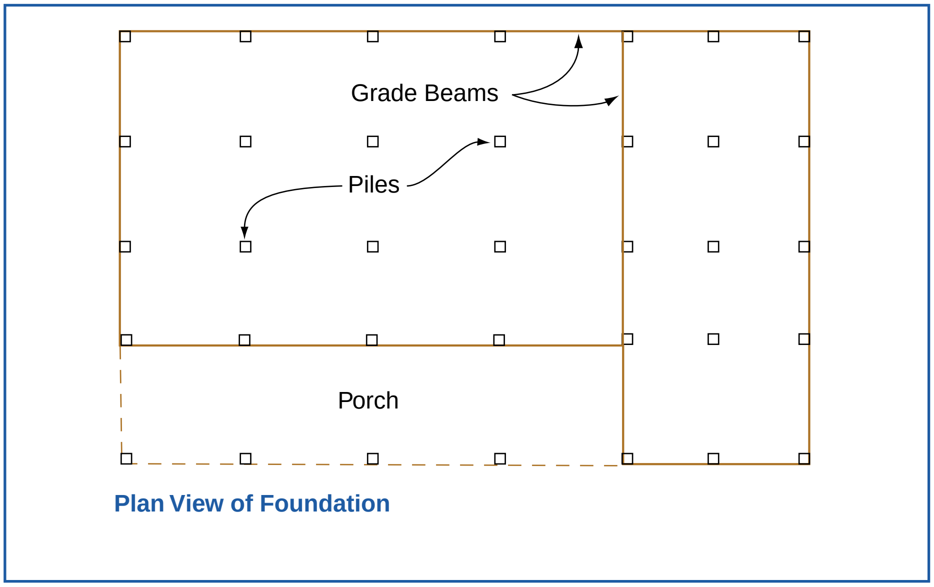
**COST CONSIDERATION**

See Tables 9.5 and 9.6, in Chapter 9, regarding the financial implications of having the lowest floor below the BFE.

- Grade beams shall not be used as structural support for a concrete slab below an elevated building in a V zone. Although such a slab may serve as the floor of a ground level enclosure-usable only for parking, storage, and building access-the slab must be independent of the building foundation. If a grade beam is used to support the slab, the slab becomes lowest floor of the building, the beam becomes the lowest horizontal structural member supporting the lowest floor, and the bottom of the beam becomes the reference elevation for flood insurance purposes. The NFIP, IBC and IRC, require buildings in V zones to have the bottom of the lowest horizontal structural member supporting the lowest floor elevated to or above the BFE. Therefore, to avoid the slab being considered the lowest floor, slabs must be separate from grade beams. This requires that the slab not be monolithic with the grade beam, and that the slab and grade beam not be tied together through such means as reinforcing steel. (Chapter 9 describes how the elevation of the bottom of the lowest horizontal structural member can affect flood insurance premiums.)

Lateral resistance to bending and excessive deflection is required for all piles that support lateral loads, so installing lateral bracing along the perimeter of the building and at any intermediate shearwall placed in either direction is one way to provide the lateral resistance. Grade beams that provide lateral bracing must be continuous in order to transfer the horizontal loads to adjacent piles. Figure 12-67 illustrates a grade beam layout.

**Figure 12-67**
Grade beam layout.



### 12.4.5.2 Diagonal Bracing

Diagonal bracing is normally constructed with wood framing members or steel rods or angles. This bracing is attached at the top of one pile and near grade level at adjacent piles. Diagonal bracing usually acts in tension only, so the bracing attachment to the pile must be capable of resisting that tension force.

Diagonal bracing has advantages and disadvantages.

Advantages:

- Bending stresses in piles are greatly reduced, so piles can be taller, and fewer piles may be required
- Piles can be braced to practically any required unbraced length as long as the lumber or steel used for the bracing is manufactured in lengths that will reach from one pile to the next.
- There are no obstructions placed at grade; therefore, the risk of scour at and around the piles is reduced.

Disadvantages:

- Diagonal bracing creates an obstruction between the piles, thereby increasing the risk that debris will become trapped. When trapped debris obstructs the flow of flood water, loads on the foundation are increased.
- Bracing only in the direction parallel to flood flow (which would be acceptable if properly done) is difficult with diagonal bracing.
- Braces provide unwanted support to breakaway walls (see Section 12.4.6 and FEMA NFIP Technical Bulletin 9 in Appendix H) and therefore can prevent breakaway walls from failing as intended. When this occurs, loads on the foundation are increased.

Figure 12-68 shows the forces that occur at the top of the pile when diagonal bracing is used to reduce unbraced length.

The tension force in the diagonal is determined as follows:

$T_{diagonal}$ = (989/cos 45°) = 1,399 lb. The interaction of the soil with the pile is expected to fully resist the tension load on the pile from the diagonal. **[12.20]**

The restraint provided by the brace reduces the effective length factor K for the 9-inch-diameter wood pile to about 1.5, and the bending stress is reduced because the restraint at the top of the pile causes a double curvature in the

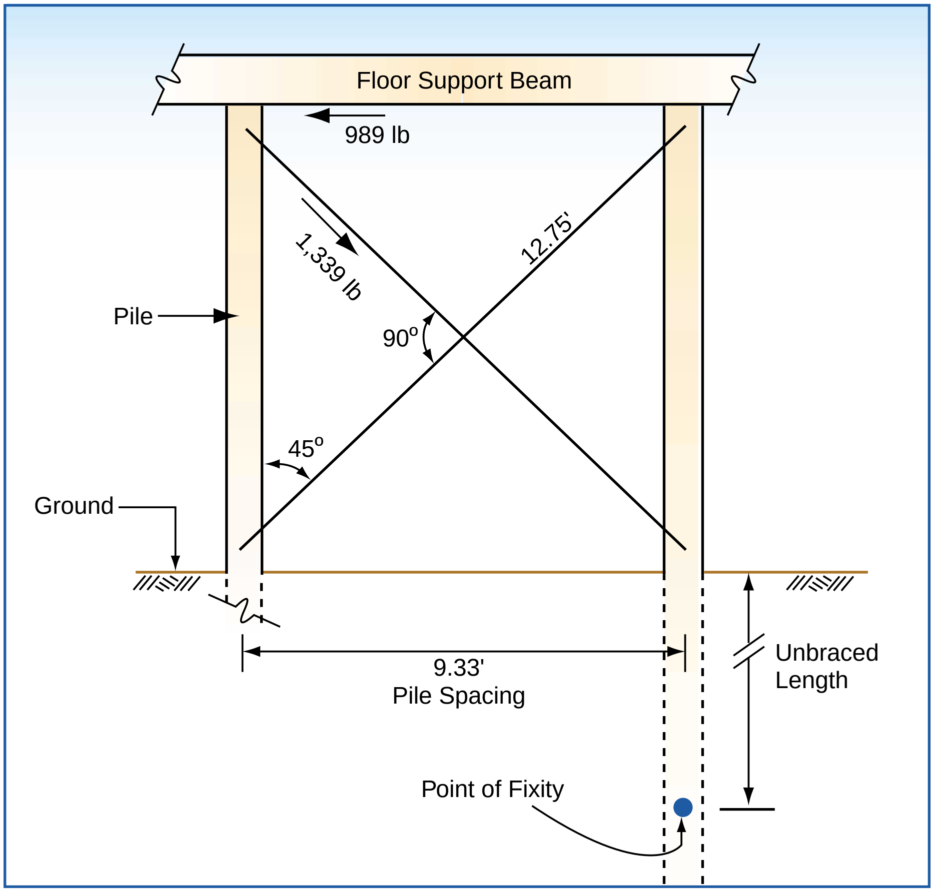



**Figure 12-68**
Force diagram for diagonal bracing.

**NOTE**

Steel rods are subject to corrosion in salt air environments. For buildings in such environments, designers should specify rods with a greater cross-sectional area, or stainless steel or hot-dip galvanized rods.

**WARNING**

Because of considerations regarding serviceability, grade of lumber, and out-of-plane bending from flood impacts, the minimum size wood brace recommended is 2 inches x 6 inches.

pile. These effects reduce the result of the interaction formula (Formula 12.8) to less than 1.0, thus this method of reducing stress and effective length has provided a means by which the 9-inch-diameter wood pile can be used throughout for the case study building.

Assuming southern pine, the allowable tension stress in the brace is:

$$F_t = (825\ \text{lb/in}^2)(C_d) = (825)(1.6) = 1{,}320\ \text{lb/in}^2. \quad [12.21]$$

The required size of the southern pine bracing is:

$$A_t = 1{,}399\ \text{lb}/1{,}320\ \text{lb/in}^2 = 1.06\ \text{in}^2. \text{ The minimum size brace is 1 in x 3 in with an area of } 1.875\ \text{in}^2. \quad [12.22]$$

If steel rods are used, the minimum size of the bracing is 1,399 lb/(60,000 lb/in$^2$ yield strength)(0.67 allowable stress factor) = 0.015 in$^2$ or a bar with a diameter of at least 3/16 inch that has an area of 0.03 in$^2$. Steel rods and most 1x and 2x wood braces are used only in tension.

The connection between the diagonal brace and the pile must be sufficient to transfer loads from the pile to the brace. Through-bolts are the preferred method. Lag bolts, while they may have sufficient capacity, can work their way out of the wood as the wood dries and as the brace is loaded and unloaded.

Assuming 1x bracing and an 8-inch-diameter pile, the 1,399-lb load requires two 5/8-inch-diameter bolts (1997 NDS, Table 8.2A). Section 8.5 of the 1997 NDS is used to determine the bolt placement. Figure 12-69 illustrates one method of placing the two bolts.

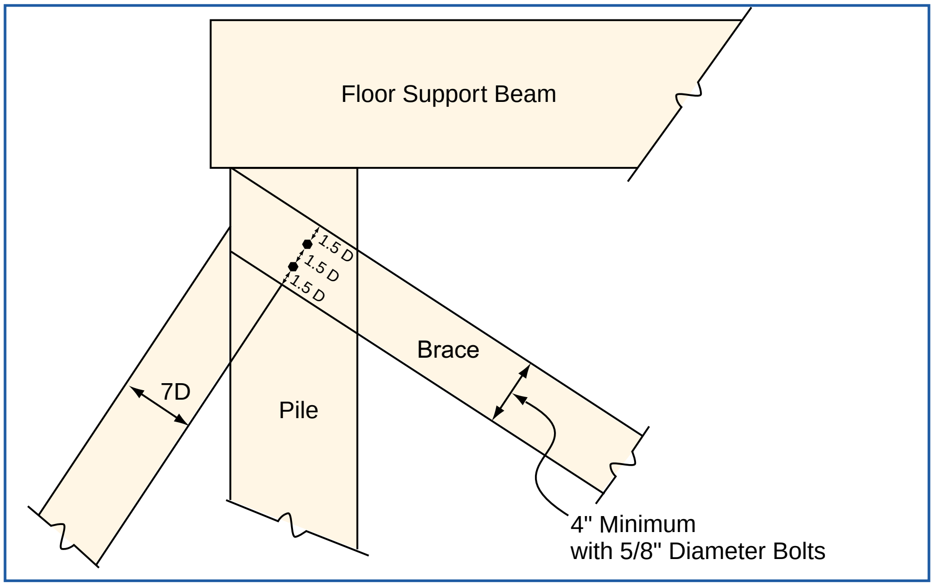



**Figure 12-69**
Bolt layout for diagonal brace. A space equal to 1.5 bolt diameters (1.5D) should be provided between bolts and between the bolts and the edge of the brace. This spacing dictates a minimum brace width of 2.8 inch, which means the minimum brace size is 1 inch x 4 inch for this case study example.

### 12.4.5.3 Knee Bracing

Like grade beams and diagonal bracing, knee bracing is installed to reduce the unbraced pile length. The unbraced length starts at the point where the knee bracing is connected to the pile (see Figure 12-70).

Knee bracing has advantages and disadvantages.

Advantages:

- Piles can be braced with shorter members. Using shorter bracing reduces the obstructions between piles, and therefore the potential for catching debris, and increases the usability of the area between the piles.
- No obstructions are placed at grade, which reduces the risk of scour around the piles.

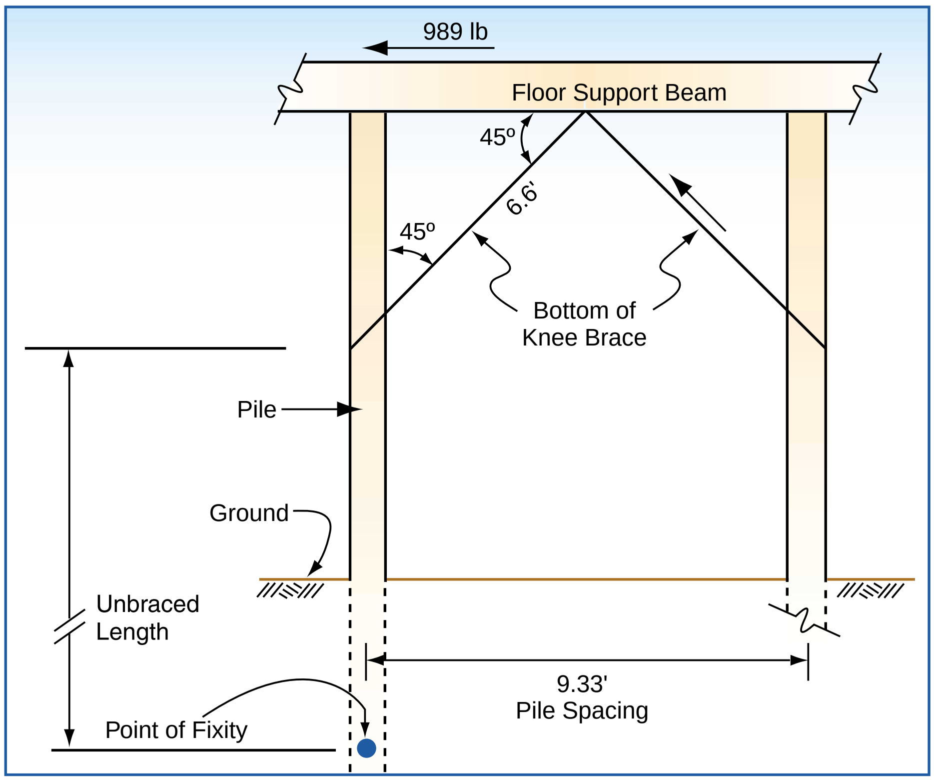
Disadvantages:

- Knee bracing is not as effective as full-height diagonal bracing in reducing the bending stress in piles subjected to high lateral loads.
- It is difficult to brace only in the direction parallel to flood flow (which would be acceptable if properly done).

- Braces provide unwanted support to breakaway walls (see Section 12.4.6 and FEMA NFIP Technical Bulletin 9 in Appendix H) and therefore can prevent breakaway walls from failing as intended. When this occurs, loads on the foundation are increased.
- Shear at top of pile is increased.

The bottoms of the knee braces are placed to reduce the unbraced length. Figure 12-70 illustrates the placement of knee braces on the pile.

**Figure 12-70**
Knee brace placement and forces.




**WARNING**

**Because of considerations regarding serviceability, grade of lumber, and out-of-plane bending from flood impacts, the minimum size wood brace recommended is 2 inches x 6 inches.**

The tension force in the knee brace is larger than that in the diagonal brace. In compression, the 1,098-lb load requires a minimum member size of 1 inch x 12 inches or 2 inches x 3 inches to prevent buckling failure. The knee brace has reduced the effective length factor K to 1.2 for the 9-inch-diameter pile and has reduced the bending stress because the restraint at the top causes double curvature in the pile. These effects have reduced the result of the interaction formula to less than 1.0, which indicates that this bracing method also has provided a means by which the 9-inch-diameter pile can be used.

### 12.4.6 Breakaway Wall Enclosures

In V and coastal A zones, breaking waves are almost certain to occur simultaneously with peak flood conditions. As breaking waves pass an open piling or column foundation, the foundation experiences cyclic fluid impact and drag forces. The flow peaks at the wave crest, just as the wave breaks. Although the flow creates drag on the foundation, most of the flow under the building is undisturbed. This makes open foundations somewhat resistant to wave actions and makes pile and column foundations a manageable design.

When a breaking wave hits a solid wall, the effect is quite different. When the crest of a breaking wave impacts a vertical surface, a pocket of air is trapped and compressed by the wave. As the air pocket compresses, it exerts a high-pressure burst on the vertical surface, centered at the stillwater level. The pressures can be extreme. For example, a 5-foot wave height can produce a peak force of 4,500 lb/$ft^2$, roughly 100 times the force caused by a 130-mph wind. These extremely high loads make designing solid foundation walls for small buildings impractical in areas subject to the effects of breaking waves. Prudent design dictates elevating buildings on an open foundation above potential breaking waves. In fact, the IBC 2000 (ICC 2000a) and the IRC 2000 (ICC 2000b) require that new, substantially damaged, and substantially improved buildings in V zones be elevated above the BFE on an open (e.g., pile, post, column, or pier) foundation.

**CROSS-REFERENCE**

**NFIP compliance provisions, as described in the IBC 2000 and the IRC 2000, are discussed in Chapter 6 of this manual.**

The IBC and IRC prohibit obstructions below elevated buildings but allow enclosures below the BFE as long as they are constructed with insect screening, lattice, or walls designed and constructed to fail under the loads imposed by floodwaters. Because such enclosures will fail under flood forces, they will not transfer additional significant loads to the foundation. Regulatory requirements and design criteria concerning enclosures below elevated buildings in V zones are discussed in FEMA NFIP Technical Bulletin 9 (see Appendix H). As explained in the bulletin, breakaway walls may be of wood- or metal-frame or masonry construction.

Figure 12-71 shows how a failure begins in a wood-frame breakaway wall—with the failure of the connection between the bottom plate of the wall and the floor of the enclosed area. Figure 12-72 shows a situation in which utility components placed on and through a breakaway wall prevented it from breaking away completely.

If screening is used it may be either metal or synthetic. Lattice is available in pre-manufactured 4-foot x 8-foot sheets. Either wood or plastic lattice is acceptable, provided the material used to fabricate it is no thicker than 1/2 inch and the finished sheet has an opening ratio of at least 40 percent. Figure 12-73 shows lattice used to enclose an area below an elevated building.

**Figure 12-71**
Hurricane Hugo (1989), South Carolina. Typical failure mode of breakaway wall beneath an elevated building—failure of the connection between the bottom plate of the wall and the floor of the enclosed area.

**Figure 12-72**
Hurricane Opal (1995), Florida. Utility penetrations prevented this breakaway wall panel from breaking away cleanly.

## 12.5 Step 4 – Develop Connections at Each Link

### 12.5.1 Connection Choices

Alternatives for joining building components include the following:

- mechanical connectors such as those available from a variety of manufacturers
- fasteners such as nails, screws, bolts, pegs, and reinforcing steel
- connectors such as wood blocks
- alternative materials such as adhesives and strapping

**Figure 12-73**
Lattice installed beneath an elevated house in a V zone.

Most commercially available mechanical connectors that have been approved by the model building code product evaluation services are fabricated metal devices formed into shapes designed to fit snugly around components such as studs, rafters, and wall plates. To provide their rated load, these devices must be properly nailed or bolted as specified by the manufacturer. Mechanical connectors are typically galvanized with 1 to 2 oz of zinc per $ft^2$. **Thicker coatings are recommended because they provide greater protection against corrosion.** Welded steel products are generally hot-dip galvanized or painted for corrosion protection. Stainless steel (A304 and A316 SS) connectors are also available. Because exposed metal fasteners (even when galvanized) can corrode within a few years of installation in coastal areas, **stainless steel is recommended where rapid corrosion is expected.**

**NOTE**

Table 1 in FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas* (see Appendix H), lists recommendations for corrosion-resistant connectors.

Connector manufacturers provide specifications and ratings for their products. Often this information is not reviewed sufficiently and improper connectors are selected. Therefore, the designer should review the specifications carefully. Particular attention should be given to the following:

- corrosion protection provided
- wood species or lumber type used in framing (e.g., ordinary framing lumber, wood I-beams, LVL products)
- ultimate capacity of connector for all modes of failure (e.g., shear, uplift, gravity loading)
- corrosion protection provided for the nails
- nail size and type required to achieve rated loads
- use in new or retrofit applications

In the following sections, each link in the studied load path is investigated and connection alternatives are suggested.

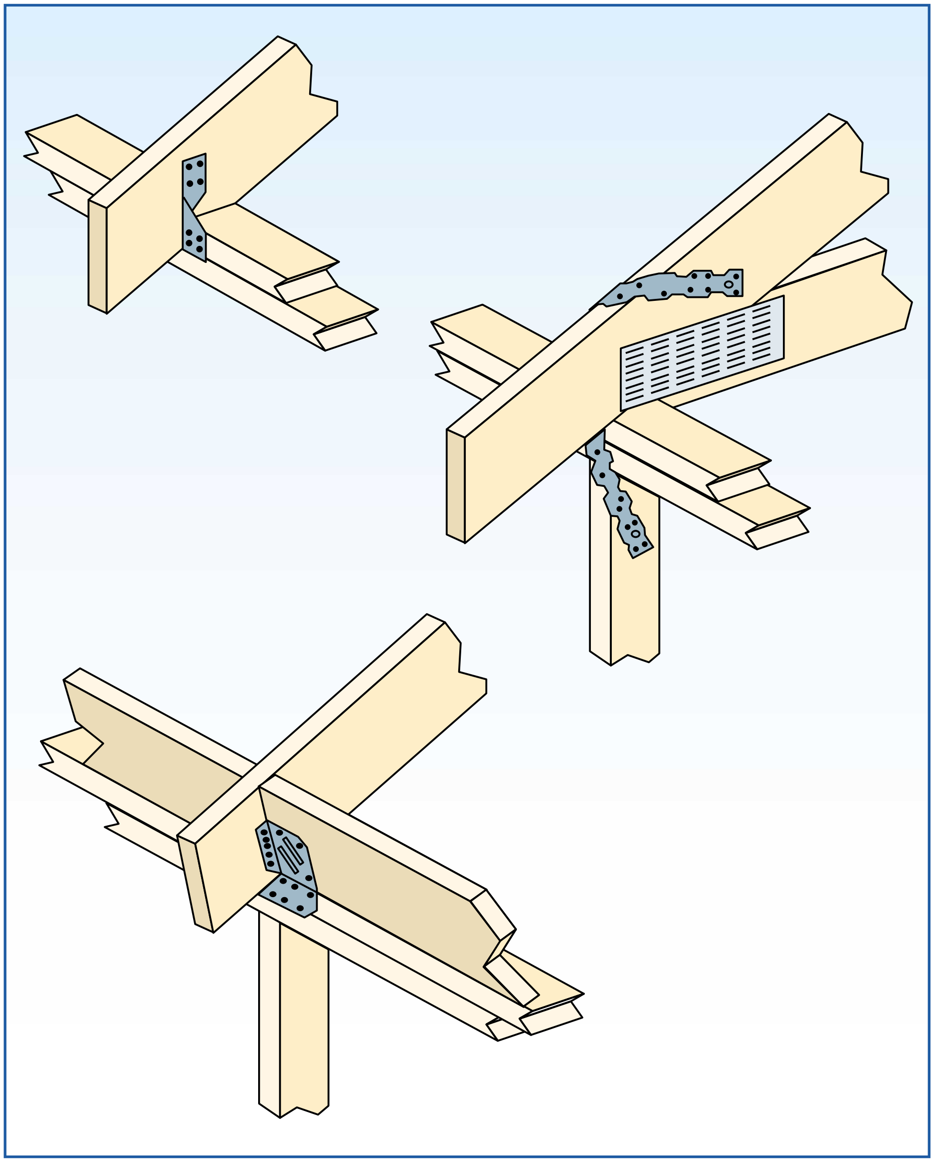
### 12.5.2 Link #1 – Roof Sheathing to Roof Framing

Link #1 connects roof sheathing to roof framing. This connection can be made with nails or screws.

### 12.5.3 Link #2 – Roof Framing to Exte rior Walls

Link # 2 is the connection between the roof rafter or truss and the top plate of the exterior wall. This connection is usually made with a metal strap designed to resist uplift and lateral loads (see Figure 12-74). The connector must be sized so that loads in all three planes are adequately resisted.

**Figure 12-74**
Metal connectors.



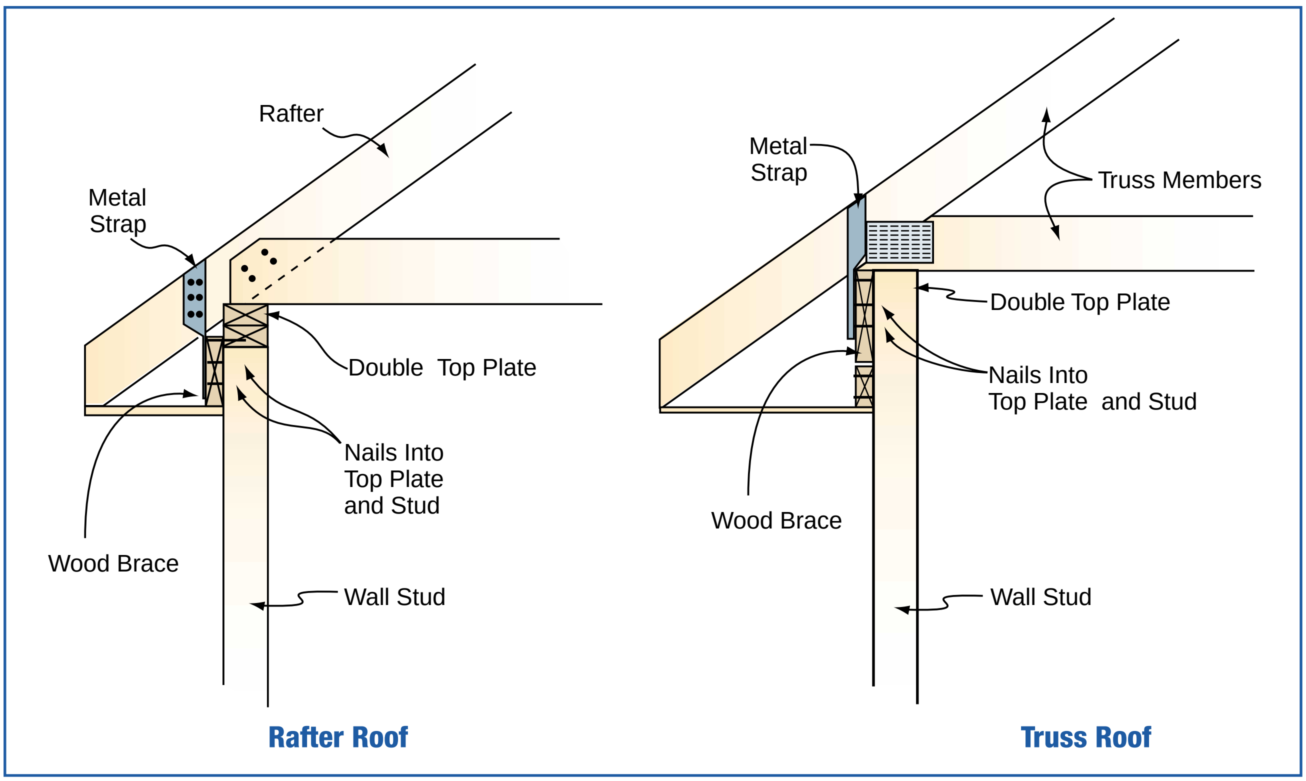
The loads at Link #2 (from Section 12.4.2) are as follows:

- uplift = 383 lb (from Table 12.7)
- lateral force in plane of the wall =286 lb (from page 12-41)
- normal force = 457 lb (from page 12-41)

The designer must specify connectors and their locations and capacities in the plans. A number of manufacturers supply mechanical connectors for use in residential construction.

In a retrofit situation, it is possible to nail a connecting board to both the roof member and the top plate of the wall. Figure 12-75 illustrates this connection with wood. The installation must be done carefully so that no wood splits during the installation of the nails. Wood splitting will significantly reduce the capacity of the member in any of the primary directions. Each of the primary loads can be resisted with 16d nails that each resist 224 lb in single shear [from 1997 NDS where shear value is (122)(1.6)(1.15)] and 85 lb in withdrawal. For this connection, the uplift can be resisted by two 16d nails and by two to four 16d nails in lateral and normal directions.

**Figure 12-75** Board connected to both the roof member and the top plate of the wall.



### 12.5.4 Link #3 – Top Wall Plate to Wall Studs

This link is the connection between the top wall plate and the wall studs. Because roof framing members are normally spaced 24 inches o.c. and wall studs are spaced 16 inches o.c., the same connector usually cannot be used to link the roof framing to the wall stud. Where the spacings are the same, a connector that accomplishes a link at both locations may be preferable. This link can normally be accomplished with wall sheathing.

Figure 12-76 shows a typical metal connector that can be used when such a connector is required. Calculation 12.10 indicated the connection with wall sheathing could be completed with five 8d box nails over a 24-inch truss spacing.

The loads at Link #3 (same as Link #2) are:

- uplift = 383 lb
- lateral force in plane of the wall = 176 lb
- normal force = 232 lb

**Figure 12-76**
Top-plate-to-wall-stud metal connector.

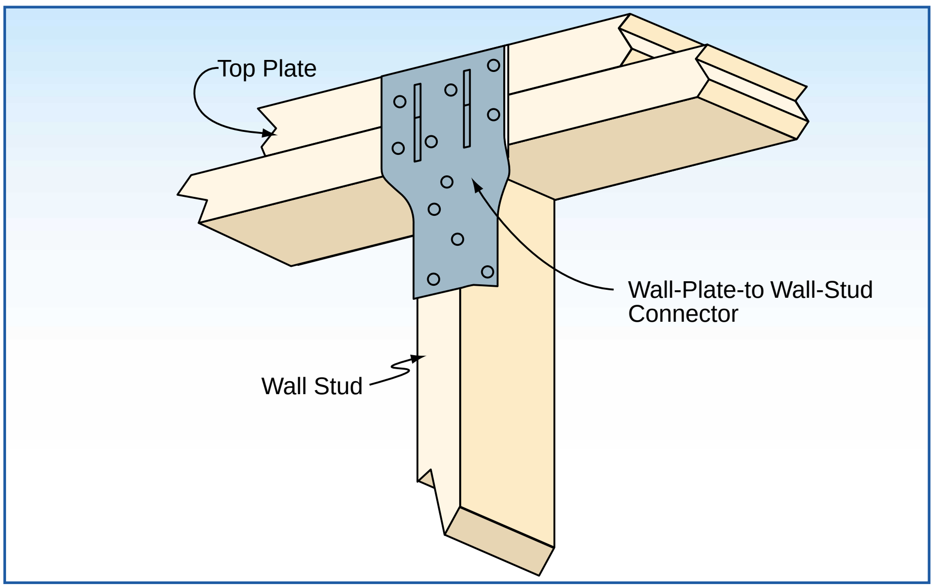



Table 12.14 lists the capacities for 8d box nails for Link #3.

**Table 12.14**
Nail Capacities for Link # 3

| Product | Uplift (lb) | Lateral Force (lb) | Normal Force (lb) |
|---|---|---|---|
| Required load | 383 | 176 | 232 |
| 8d box nails | 392 | 342 | 305 |

### 12.5.5 Link #4 – Wall Stud to Window Header

The connection method and the loads required at this connection are the same as those used at Link #3.

### 12.5.6 Link #5 – Window Header to Exterior Wall

This link is the connection between the end of the window header and the wall studs. This connector will normally have to be placed flat against the framing, so it will be in the shape of a strap. The connector must be sized so that loads in all three planes are adequately resisted.

The loads at Link # 5 (from Section 12.4.2) are:

- uplift = 731 lb (from page 12-45)
- lateral force in plane of the wall is distributed into shearwall
- normal force = 449 lb (from Calculation 12.11) (Use end nails through stud into header.)

### 12.5.7 Link #6 – Wall to Floor Framing

This link is the connection between the bottom of the wall framing and first floor. The connector used here is also the shearwall holddown connector. It is bolted to a stud (or more than one stud) and through the floor framing into the beam (as in the case study house) or, in a multi-story house, to the wall below. This connector must be sized to resist tension. Figure 12-77 shows a shearwall holddown connector. The uplift load at this holddown is 4,064 lb (from Calculation 12.15).

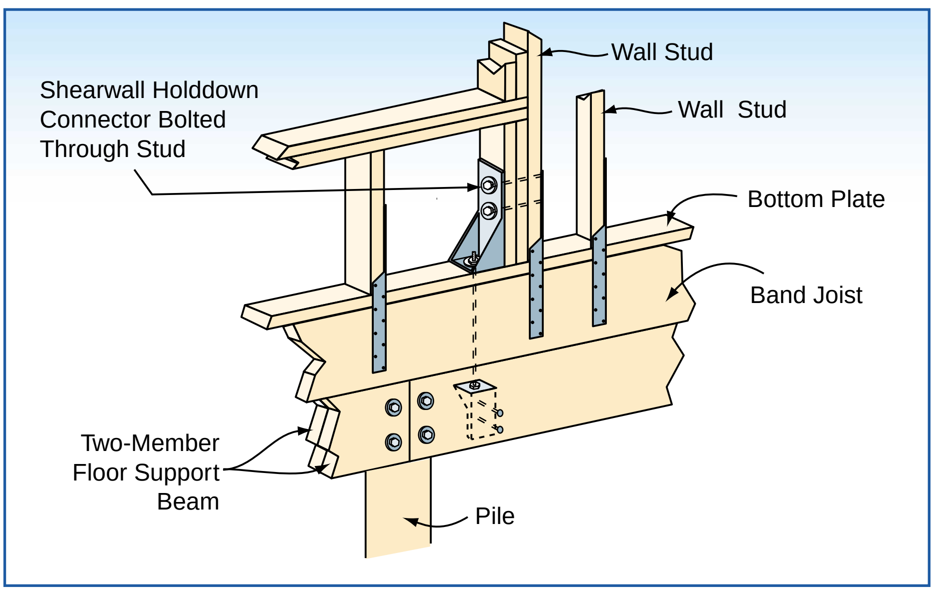



**Figure 12-77**
Shearwall holddown connector.

### 12.5.8 Link #7 – Floor Framing to Support Beam

This link is the connection between the floor framing and the floor support beam and does not include the shearwall overturning uplift. This connection can be made with either a metal connector (see Figure 12-51) or a block of wood installed between the floor support beam and nailed to the floor joists (see Figure 12-52). This connector must be sized so that loads in all three planes are adequately resisted.

The loads at Link #7 (from Section 12.4.2) are:

- uplift – 4,064 lb
- lateral force in plane of the wall – 443 lb (from page 12-53)
- normal force – 168 lb (from page 12-53)

**WARNING**

**Overnotching of pilings during construction is a common problem. Designers should specify maximum pile notching depths.**

### 12.5.9 Link #8 – Floor Support Beam to Pile Foundation

The connection between the beam and the top of the pile has already been discussed. This connection is normally completed with hex head bolts (bolts installed through the beam and top of the pile). When this connection is made by notching the pile to provide a "seat" for the beam, the pile is often overnotched. The top of the pile at the cut is a failure plane in shear.

For the case study example problem, the lateral load at the pile top is 989 lb. When the top of the pile is held rigidly by bracing, the load could shear off the top of the pile. Using guidelines from the NTPC (1995), the allowable shear stress is $(F_v)(C_d)(C_{pt}) = (110 \text{ lb/in}^2)(1.6)(0.90)$

$= 158.4 \text{ lb/in}^2$ (and as noted on page 12-66, the maximum shear stress is higher than this average shear stress) **[12.23]**

The minimum area of the pile required to resist shear failure is 989 lb(1.5 factor for rectangular shape)/(158.4) lb/in$^2$. The minimum area = 9.4 in$^2$.

The area of a 9-inch-diameter pile is 63.62 in$^2$, so the minimum area required is **15 percent** of the pile area. Overnotching to provide a beam "seat" does increase the risk of shear failure in the pile. Figure 12-78 illustrates failure of a pile that has been overnotched. Reinforcing the pile at the overnotched section with steel plates on both sides of the pile is an acceptable method of increasing the shear capacity at this critical point. This detail must, however, be carefully designed.

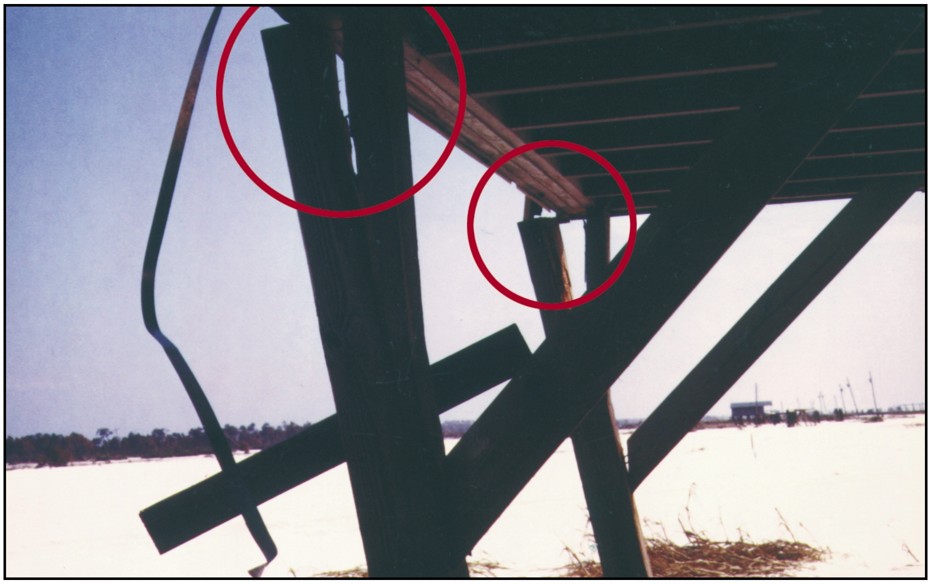


**Figure 12-78**
Failure of overnotched pile.

# 12.6 Step 5 – Select Building Materials

## 12.6.1 Introduction

The designer and builder of coastal structures must choose the materials to use for all parts of the building, including the foundation, structural frame, exterior envelope, and interior finishes. The foundation and structural frame are usually wood, concrete, steel, or masonry. The exterior envelope and interior finishes can be of these materials or of a wide selection of other materials.

The choice of materials will be influenced by many considerations, including whether they will be used above or below the DFE. Below the DFE, the risk of inundation by seawater must be anticipated. Significant forces due to wave action, water velocity, and waterborne debris impact must also be considered. Materials intermittently wetted by flood water below the BFE are subject to corrosion and decay.

Above the DFE, building materials still face significant environmental effects. The average wind velocity increases with height above ground. Wind-driven salt water spray can cause corrosion and moisture intrusion. The evaporation of salt water leaves crystalline salt that retains water and is corrosive.

Each of the commonly used materials (wood, concrete, steel, and masonry) has characteristics that can be advantageous or that can require special consideration when the materials are used in the coastal environment (see Table 12.15). Usually, a coastal residential structure will have a combination of these materials, with each used to advantage in a specific application.

**Table 12.15**
General Guidance for Selection of Materials – Advantages and Special Considerations

| Material | Advantages | Special Considerations |
|---|---|---|
| Wood | • Generally available and commonly used<br>• With proper design, can generally be used in most structural applications<br>• Variety of products available<br>• Can be treated to resist decay | • Easily over-cut, over-notched, and over-nailed<br>• Many designs done by "rule-of-thumb"<br>• Requires special treatment and continued maintenance to resist decay and damage from termites and marine borers<br>• Requires protection to resist weathering<br>• Subject to warping and deterioration |
| Steel | • Used for larger forces than wood can resist<br>• Can span long distances<br>• Can be coated to resist corrosion | • Not corrosion-resistant<br>• Heavy and not easily handled and fabricated by carpenters<br>• May require special connections such as welding |
| Reinforced Concrete | • Resistant to corrosion if reinforcing is properly protected<br>• Good material for compressive loads<br>• Can be formed into a variety of shapes<br>• Pre-stressed members have high load capacity | • Salt water infiltration into concrete cracks will cause reinforcing steel corrosion<br>• Pre-stressed members require special handling<br>• Water intrusion and freeze-thaw cause deterioration and spalling |
| Masonry | • Resistant to corrosion if reinforcing is properly protected<br>• Good material for compressive loads<br>• Material commonly used in residential construction | • Not good for beams and girders<br>• Water infiltration into cracks will cause reinforcing steel corrosion<br>• Requires reinforcement to resist loads in coastal areas |

## 12.6.2 Selection of Materials for Foundations and Enclosures Below the DFE

The use of flood-resistant materials below the BFE is covered in FEMA NFIP Technical Bulletin 2 (see Appendix H). The introduction in this bulletin indicates that "All construction below the lowest floor is susceptible to flooding and must consist of flood-resistant materials. Uses of enclosed areas below the lowest floor in a residential building are limited to parking, access, and limited storage—areas that can withstand inundation by floodwater without sustaining significant structural damage." The IBC 2000 and IRC 2000 require that all new construction and substantial improvements in the Special Flood Hazard Area be constructed with materials resistant to flood damage.

Compliance with these requirements in coastal areas means that the only building elements that will be below the BFE include the following:

- foundations – either treated wood, concrete or steel piles, concrete or masonry piers, or concrete, masonry, or treated wood walls
- breakaway walls
- enclosures used for parking, building access, or storage below elevated buildings
- garages in enclosures under elevated buildings or attached to buildings

Material choices for these elements are limited to the following:

- pressure-treated lumber, 0.40 CCA or equivalent (see Appendix J and references listed at the end of this chapter for additional information on wood preservatives)
- naturally decay-resistant lumber
- concrete – 5,000 lb/in$^2$ minimum compressive strength recommended in coastal environments, with a 0.40 water-cement ratio
- masonry – reinforced and fully grouted in coastal environments
- steel – must resist corrosion
- closed-cell foam insulation
- other flood-resistant materials approved by local building officials

Each of these materials has characteristics that can be advantageous or that can require special consideration when the materials are used for various building elements. Some of these are presented in Table 12.16. Additional information about material selection for various locations in the building and for various uses is included in Appendix J.

**NOTE**

Although the NFIP regulations, IBC 2000 (ICC 2000a), and IRC 2000 (ICC 2000b) specify that flood-resistant materials be used below the BFE, this manual recommends that flood-resistant materials be used below the DFE.

**CROSS-REFERENCE**

NFIP compliance provisions, as described in the IBC 2000 and the IRC 2000, are discussed in Chapter 6 of this manual.

**CROSS-REFERENCE**

Table 9.6 in Chapter 9 lists sample flood insurance premiums for buildings in which the lowest floor is below the BFE and in which there is an enclosure below the BFE.

**Table 12.16**
Selection of Materials for Use in Foundation Elements Below the DFE – Advantages and Special Considerations

| | Material/Use | Advantages | Special Considerations |
|---|---|---|---|
| Deep Foundations | Tapered Round Wood Piles | • Normally longer than square piles<br>• Shape is advantageous for pressure treatment to resist decay<br>• Cross-sectional area and stiffness greater than square piles | • May not be straight, thus driving accuracy is affected<br>• May be more costly than square wood piles |
| Deep Foundations | Square Wood Piles | • May be more economical than round piles in some areas<br>• May be more available than round piles in some areas<br>• Can be pressure-treated to resist decay<br>• May be easier to frame to flat surfaces | • Knots can be located at pile edge and weaken pile (see Figure 12-79)<br>• Pressure-treated square wood piles can twist<br>• Smaller cross-section make these piles weaker and less stiff than round piles |
| Deep Foundations | Pre-Stressed Concrete Piles | • Constructed straight and true to any specified length and strength<br>• Can support large vertical loads<br>• Material is decay-resistant | • Reinforcing steel must be protected to prevent corrosion<br>• More expensive than wood piles<br>• Special handling equipment may be required because of weight |
| Deep Foundations | Steel Piles | • Delivered straight and true to any specified length<br>• Can support large vertical and lateral loads<br>• Material is easily cleaned after flooding | • Special handling equipment may be required because of weight<br>• Must be coated and maintained to be corrosion-resistant<br>• More expensive than wood piles |
| Shallow Foundations | Reinforced Concrete Piers or Walls on Footings | | • Reinforcing steel must be protected to prevent corrosion<br>• Requires extensive excavation in order to be used as deep foundation |
| Shallow Foundations | Masonry Piers or Walls on Footings | • Easy to construct | • Reinforcing steel must be protected to prevent corrosion<br>• Requires extensive excavation in order to be used as deep foundation<br>• Performance very sensitive to quality of construction |

**Figure 12-79**
Hurricane Fran (1996), North Carolina. Square wood pile failure at edge knot.

### 12.6.3 Selection of Materials for Use Above the DFE

The selection of materials for use above the DFE is usually a function of one or more of the following:

- exposure – outside or inside the building envelope
- use – structural support or finish material
- structural requirement – span, deflection, load
- availability – cost and delivery

Long-term durability as well as architectural and structural considerations will normally be the most important determiners of material selections. Material in the coastal environment is very susceptible to weathering, corrosion, termite damage, and decay from water infiltration, in addition to the stresses induced by loads from natural hazard events. These influences must be considered in the selection of the appropriate materials. Appendix J contains additional information about a variety of wood products and considerations important in their selection and use.

### 12.6.4 Materi al Combinations

Materials are frequently combined in the construction of a single residence. The most common combinations are as follows:

- masonry or concrete lower structure with wood on upper level
- wood piles supporting concrete pile caps and columns that support a wood superstructure
- steel framing with wood sheathing

For the designer of coastal buildings, the important design considerations when combining materials include:

1. There must be **material compatibility** so that corrosion is not caused by the contact of dissimilar metals in the presence of salt and moisture. Appendix J addresses a possible problem with galvanized fasteners and hardware in contact with certain types of treated wood.
2. **Connecting the materials together** is crucial. The connector must be properly embedded (if into concrete or masonry) and placed so that alignment and vertical or horizontal load path continuity are maintained. Altering a connector location after it has been cast into concrete or grout is a difficult and expensive task.
3. Material combinations used for the same building add the **complexity of additional skills** being required to construct the project. This is normally of more concern to the builder than to the designer, but may impact decisions regarding which materials will be acceptable. Figure 12-80 shows a coastal house being constructed with ordinary wood piles that support a welded steel frame that will be used as the floor frame support beams.
4. Material properties such as stiffness of one material relative to another will affect movement or deflection of one material relative to the other. This **difference in material behavior** can affect the resultant damage to the building. For example, movement that occurs in a wood-frame building during high wind can fail masonry piers because masonry is less flexible than wood and will fail with small deflection.

**Figure 12-80**
House being constructed with a steel frame on wooden piles.

### 12.6.5 Fire Safety Considerations

Construction of multi-family coastal dwellings that must withstand natural hazards and meet the building code requirements for adequate fire separation presents some challenges. In multi-family buildings, the IBC 2000 and IRC 2000 require that the common walls between living units be constructed of materials that provide a minimum of 2-hour fire resistance. The code requires that the units be constructed such that if a fire were to occur in one unit, the structural frame of that unit would collapse within itself and not affect either the structure or the fire resistance of adjacent units.

For townhouse-like units, the common framing method is to use the front and rear walls for the exterior load-bearing walls so that firewalls can be placed between the units. If these walls are also used as the support for the floor framing, the floor support beams must be supported by the side walls (or side wall support structure such as piles). This beam orientation may make the structure more vulnerable to flood damage. Figure 12-81 illustrates the framing system for a series of townhouses and the potential difficulties in framing these units to minimize flood and wind damage.

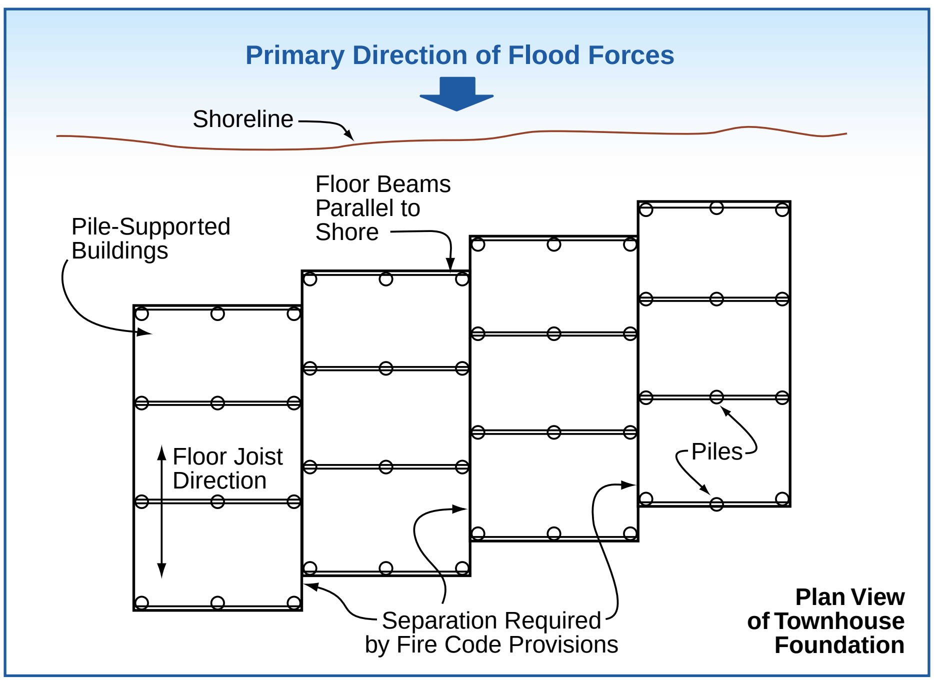



**Figure 12-81**
Townhouse framing system.

**NOTE**

Consult the local building code or building official to determine how to resolve the conflict between fire-resistant construction and flood-resistant construction.

The figure suggests several design difficulties that must be overcome. These difficulties include, but are not limited to the following:

1. The floor support beams are parallel to the shore and perpendicular to the expected flow, and may therefore create an obstruction during a greater than design flood event.

2. The fire separation between the units makes structural rigidity difficult to achieve because the interior walls between townhouse units cannot be attached to the framing system and used for support. Therefore, the transfer of lateral loads to the foundation, particularly for multi-directional loads from wind or earthquake forces, is difficult to achieve.

3. The exposed undersides of buildings elevated on an open foundation (e.g., pile, pier, post, or column) must be protected from fire with a 2-hour rated material. Where sheathing is desired on the underside of the floor framing—for example, in a below-building enclosure used for parking, storage, or building access—the use of fire-resistant gypsum board will comply with the requirement for fire protection; however, gypsum board is not a flood-resistant material. A better approach would be to use cement-fiber board, which has a greater resistance to damage from flood waters. If the building design requires fire separation and the fire-protection material is not flood-resistant (e.g., gypsum board), larger wood framing members or other techniques may be required in order to meet the competing demands of flood- and fire-resistance.

4. The requirement for separation of the foundation elements between townhouse units makes structural rigidity in the direction parallel to the shore difficult to achieve. If the houses in Figure 12-81 were located in a seismic hazard area, the designer might want to place a shearwall or diagonal bracing in the direction parallel to the shore (i.e., perpendicular to the primary flood flow direction). Shearwall segments or diagonal bracing will provide rigidity, but when they are constructed below the first-floor framing (and thus below the DFE) and perpendicular to the expected flood flow, they will create an obstruction below the DFE. The designer should consult FEMA NFIP Technical Bulletin 5, *Free of Obstruction Requirements for Buildings Located in Coastal High Hazard Areas* (see Appendix H) for information about the types of construction that constitute an obstruction.

Designers should consult local building officials for guidance about how to create a design compatible with both fire and natural hazard design requirements. Understanding local requirements will influence how the designer resolves potential conflicts between various code requirements.

### 12.6.6 Corrosion

Modern construction techniques often rely heavily on metal fasteners and connectors to resist the forces of various coastal hazards. To be successful, these products must have lifetimes comparable to those of the other materials used for construction. The metal materials in common use have proved to have an adequate lifetime in most inland applications. However, for some uses near saltwater coastlines, corrosion has been found to drastically shorten the lifetime of standard fasteners and connectors. Corrosion is one of the most underestimated hazards affecting the overall strength and lifetime of coastal buildings. To be successful, hazard-resistant buildings must match the corrosion exposure of each component with the proper corrosion-resistant material. Although standard materials may not have sufficient corrosion resistance for some uses in coastal buildings, a variety of corrosion-resistant materials and techniques are readily available, often at a small increase in cost.

**NOTE**

See FEMA NFIP Technical Bulletin 8 (Appendix H) for additional information about corrosion of metal connectors in coastal construction.

FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas,* represents the current state of knowledge and research concerning this subject. This bulletin is included in Appendix H.

### 12.6.7 Additional Environmental Considerations

In addition to water intrusion and possible resulting decay, several other environmental factors must be considered in the selection of materials to be used in coastal buildings. The coastal environment is extremely harsh, and materials should be selected that will not only provide protection from the harsh elements, but also require minimal maintenance. The following environmental factors will be discussed:

- sun (heat and ultraviolet [UV] radiation)
- wind-driven rain

#### 12.6.7.1 Sun

Buildings at or near the coast are typically exposed to extremes of sun, which produces high heat and UV radiation. This exposure creates the following effects:

- The sun will bleach out many colors.
- The heat will build up in enclosed spaces like attics so the design must consider ways to reduce the heat buildup.
- The heat and UV will shorten the life of many organic materials such as asphalt roof shingles.
- The heat and UV affect the life of sealants, protective materials placed on siding, and exterior wood used for decks, walkways, and other external components.

- The heat dries out oils and lubricants such as those contained in door and window operating mechanisms.

To overcome these problems:

- use materials that are heat/UV-tolerant,
- shield heat/UV susceptible materials with other materials, and
- perform periodic maintenance and repair.

### 12.6.7.2 Wind-Driven Rain

Wind-driven rain is primarily a problem for elements of the building envelope, particularly elements that operate such as windows and doors. High winds can carry water droplets into the smallest of openings, even up, into, and behind flashings, vents, and drip edges. When houses are built to provide what is considered to be complete protection from the effects of natural hazards, any small “hole” in the building envelope becomes an area of weakness into which sufficiently high wind can drive a large amount of rain.

The designer must address any area of potential weakness in the building envelope with the proper technique—flashings, sealants, product design of windows and doors, attachment of elements that penetrate the building envelope (e.g., decks, porches, window boxes)—as further discussed in Section 12.7. Figure 12-82 illustrates a design detail for the attachment of a deck that minimizes penetrations of bolts through the building envelope.

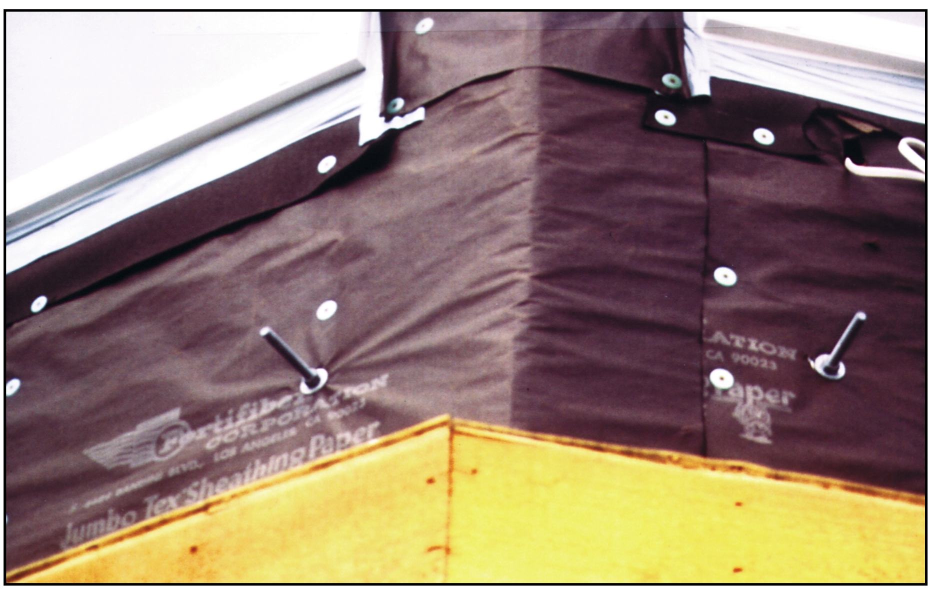



**Figure 12-82**
Deck attachment scheme that minimizes penetration of the building envelope.

## 12.7 Design of the Building Envelope

The building envelope comprises the roof covering, exterior wall covering, and exterior doors and windows. For buildings elevated on open foundations, the floor is also considered a part of the envelope .

**NOTE**

Special attention to the design of the building envelope must be given to buildings located in areas subject to wind speeds greater than 90 mph (3-sec peak gust).

To avoid injury and minimize damage to a building and its contents, good structural system performance is critical; however, it does not ensure occupant or building protection. Good performance of the building envelope and exterior-mounted mechanical and electrical equipment is also necessary. Satisfactory building envelope performance is critical for buildings exposed to high-wind and wildfire hazards.

Satisfactory performance of the envelope depends on suitable design, materials, application, maintenance, and repair. Of these, design is the key element. Although design inadequacies frequently cannot be compensated for by the other four elements, good design (to some extent) can compensate for other inadequacies.

Breaching of the building envelope and subsequent water infiltration has historically been the predominant cause of damage to buildings and their contents during high-wind events. Breaching includes catastrophic failure (e.g., loss of the roof covering or windows) as well as water infiltration through small openings at walls, windows, doors, or the roof. Water infiltration also occurs during high winds because wind-driven rain will penetrate into even the smallest opening. Preventing this type of damage requires the application of a high-quality sealant around components such as windows and doors. Other openings that must be properly sealed include mechanically attached roof or wall penetrations such as exhaust fans and vents.

Building envelope components have also been the predominant source of windborne missiles generated from damaged buildings. Close design attention should be given to buildings in special wind regions (see Figure 11-18, in Chapter 11) or in areas where the basic wind speed is greater than 90 mph (3-sec peak gust).

Building integrity in earthquakes also is dependent on the performance of the building envelope, particularly the integrity of roof-to-wall connections and diaphragm-to-wall connections and detailing. Residential buildings have historically performed very well when the structural integrity of these components of the building envelope has been maintained.

In addition, poor resistance of building envelopes to wildfires has historically resulted in building losses as illustrated in Figure 12-83.

Basic design issues and general good practices that are applicable to all residential design are not addressed in this section. Rather, this section builds on the basics by addressing the special design considerations related to buildings susceptible to natural hazards. It provides recommendations regarding doors, floors, roofs, walls, and windows. Flooding influence on the building envelope is not addressed, because it is assumed that the envelope will not be inundated by water; however, envelope resistance to wind-driven rain is addressed. The recommended measures taken to protect against wind-driven rain should also be adequate to protect against wave spray.

**Figure 12-83**
The fire-resistant walls and roof of this house helped it survive a wildfire while surrounding houses were destroyed. Photograph courtesy of Decra Roofing Systems.

**NOTE**
For A-zone buildings elevated on solid walls with openings, a number of manufacturers produce vents that reduce heat loss in cold-weather environments but still allow the entry and exit of flood waters.

## 12.7.1 Floors for Elevated Buildings

### 12.7.1.1 Corrosion and Wave Spray

For buildings near the ocean, sheathing the underside of the bottom-floor joists or trusses helps minimize corrosion of framing connectors and fasteners. The sheathing also protects insulation installed between the joists/trusses from wave spray. (If fiberglass insulation is installed, the paper or foil face should be installed adjacent to the underside of the floor decking, or the insulation should be unfaced so that downward water vapor migration is not impeded.) For long-term durability, exterior grade sheathing is recommended for the exposed sheathing and it should be fastened with stainless steel or hot-dip galvanized nails or screws.

### 12.7.1.2 High Winds

For buildings in high-wind areas, if sheathing is applied to the underside of joists or trusses, its attachment should be specified in order to avoid blowoff.

### 12.7.1.3 Wildfires

For buildings in areas prone to wildfires, sheathing the underside of joists or trusses with a fire-resistant material such as cement-fiber panels is recommended. Cement-fiber panels should be attached with stainless steel or hot-dip galvanized screws.

### 12.7.2 Exterior Walls and Soffits

High winds and wildfires are the natural hazards that can cause the greatest damage to exterior wall systems. Seismic events can also damage heavy wall systems or coverings. Although hail can damage walls, significant damage is not common.

A variety of systems can be used for wall construction. The following wall coverings are commonly used over wood-frame construction: aluminum siding, cement-fiber panels or siding, exterior insulating finishing system (EIFS), stucco, vinyl siding, sawn wood siding boards, and wood panel siding. Concrete or masonry wall construction may also be used. These systems are discussed in the following sections.

#### 12.7.2.1 High Winds

The discussion of air-permeable roof coverings in Section 12.7.5 is also applicable to air-permeable wall coverings such as siding. Research on special pressure coefficients has not been conducted for air-permeable wall claddings. Therefore, according to ASCE 7, these claddings have to be designed for the full wind load. Wind-load resistance of non-load-bearing walls, wall coverings, and soffits should be based on testing in accordance with ASTM E 1233. Walls and soffits must be designed to resist positive and negative wind pressure.

**NOTE**

**Throughout this manual, references to ASTM standards are based on the *Annual Book of ASTM Standards* (ASTM 1998).**

Siding, panels (e.g., textured plywood), and stucco over masonry and concrete typically perform well during high winds. More blowoff problems have been experienced with vinyl siding than with other siding or panel materials. Aluminum and cement-fiber siding problems have also occurred. The key to the successful performance of siding and panel systems is attachment with a sufficient number of proper corrosion-resistant fasteners (based on design loads and tested resistance) that are correctly located. Blowoff of stucco applied directly to concrete walls (i.e., wire mesh was not applied over the concrete) has occurred. This problem can be avoided by leaving the concrete exposed or painted.

A secondary line of protection against wind-driven water infiltration (e.g., an air-barrier film) is recommended underneath wall coverings. Designers should specify that horizontal laps be installed so that water is allowed to drain from the wall (i.e., the top sheet should lap over the bottom sheet so that water running down the sheets remains on their outer face). The bottom of the secondary protection needs to be detailed to allow drainage.

EIFS can be applied over wood-frame, concrete, or masonry construction. The EIFS assembly is composed of several types of materials. Some of the layers are adhered to one another, and one or more of the layers is typically mechanically attached to the wall. If mechanical fasteners are used, they need to be correctly located, of the proper type, and of sufficient number (based on design loads and tested resistance). Failures have been observed where the rated wind load capacity of the fasteners was not sufficient for a design event. Proper application of the components that are adhered together is also necessary to avoid blowoff. It is strongly recommended that if EIFS is used, it be designed with a drainage system that allows for dissipation of water leaks.

**WARNING**

**There have been documented problems with deterioration of wall sheathing in EIFS systems that are non-drainable. The use of these systems has been restricted in some jurisdictions. The designer should consult with local authorities to determine whether such restrictions exist, and what their effect may be on design and construction.**

Punctures of EIFS by windborne debris are also common in high-wind events. EIFS must be installed on a solid substrate such as plywood in order to provide enhanced resistance to missile penetration and thus occupant protection. A minimum plywood thickness of 15/32-inch is recommended.

Concrete and masonry walls (or veneers) typically provide excellent windborne missile resistance provided they are adequately designed and constructed to resist the wind load.

Durability: To avoid corrosion problems, stainless steel or hot-dip galvanized fasteners (preferable heavy-duty hot dip galvanized) are recommended for buildings located within 3,000 feet of an ocean shoreline. If air can freely circulate in a cavity (e.g., above a soffit), access panels should be provided so components within the cavity can be periodically observed for corrosion.

In areas with severe termite problems, if wood is specified, it should be pressure-treated. See Appendix J for additional information.

### 12.7.2.2 Wildfires

For buildings in areas prone to wildfires, concrete, masonry, stucco, or cement-fiber panels or siding offer the greatest protection. If one of these wall surfaces is specified, a fire-resistive system should also be specified for soffits (e.g., stucco or cement-fiber).

Gable and soffit vents should have openings covered with wire mesh that has openings no greater than 1/4 inch, in order to inhibit the entry of burning brands. For added protection, noncombustible hinged shutters that can quickly be placed in the closed position could be designed and installed.

### 12.7.2.3 Seismic

Where required by code, concrete and masonry walls (or veneers) need to be designed for the seismic load. When a heavy covering such as stucco, cement-fiber panels or siding, or brick veneer, is specified, the seismic design should account for the added weight of the material, and its connection to the base material in the case of veneer. Inadequate connection of veneer material to the base substrate has been a problem in past earthquakes and can result in a life safety hazard.

Some non-ductile coverings such as stucco and cement-fiber products can be cracked or spalled during seismic events. If these coverings are specified in areas prone to large ground motion accelerations, the structural sheathing behind the covering should be designed with additional stiffness to minimize damage to the wall covering.

### 12.7.2.4 Flashings

Poor performance of flashing and water intrusion protection is a common problem in many coastal homes. In areas that frequently experience strong winds, enhanced flashing details are recommended. Enhancements include use of flashings that have extra-long flanges, and use of sealant and tapes. General guidance is offered below, but it is recommended that designers also attempt to determine what type of flashing details have successfully been used in the area where the residence will be constructed.

Flashing design should recognize that wind-driven water can be pushed vertically. The height to which water can be pushed increases with wind speed. Water can also migrate vertically and horizontally by capillary action between layers of materials (e.g., between a flashing flange and housewrap).

**NOTE**

**Housewrap is a material used on the exterior skin of the house prior to siding installation that is primarily intended to reduce air infiltration into the building.**

Conceptually, the exterior siding should not be thought of as the only barrier to water intrusion. The housewrap (if used), flashings, and underlayment must be used to shed and direct water away from openings in the building envelope. The overriding principle of successful water diversion is to install the layers of building materials correctly so that water can not get behind any one layer and into an opening.

- Roof/wall flashing: Where enhanced protection is desired, use step flashing that has a 2- to 4-inch-longer vertical leg than normal. Alternatively (or for a more conservative design, in addition to the long leg), tape the top of the vertical flashing to the wall sheathing with 4-inch-wide self-adhering modified bitumen roof tape (apply about 1 inch of tape on the metal flashing, 3 inches on the sheathing). Extend the housewrap over the flashing in the normal fashion. Do not seal the housewrap to the flashing—if water reaches the housewrap further up the wall, it needs to be able to drain out at the bottom of the wall. Figure 12-84 illustrates a good roof/wall flashing detail. This detail has been used successfully by a builder on the Delaware coast.
- Window flashings: For windows with nailing flanges, apply a generous bead of butyl sealant to the wall sheathing before setting the window. Place the sealant inward of the fasteners. At sheathing joints, place sealant over the joint, from the window opening out past the flange. Place the housewrap over the head trim flashing, and tape the flange to the housewrap with duct tape or modified bitumen roof tape as illustrated in Figure 12-85.

**CROSS-REFERENCE**

**The designer should consult *Nail-On-Windows, Installation & Flashing Procedures for Windows & Sliding Glass Doors* (Bateman 1995) for additional information.**

**Figure 12-84** Roof/wall flashing detail. Courtesy of Journal of Light Construction.

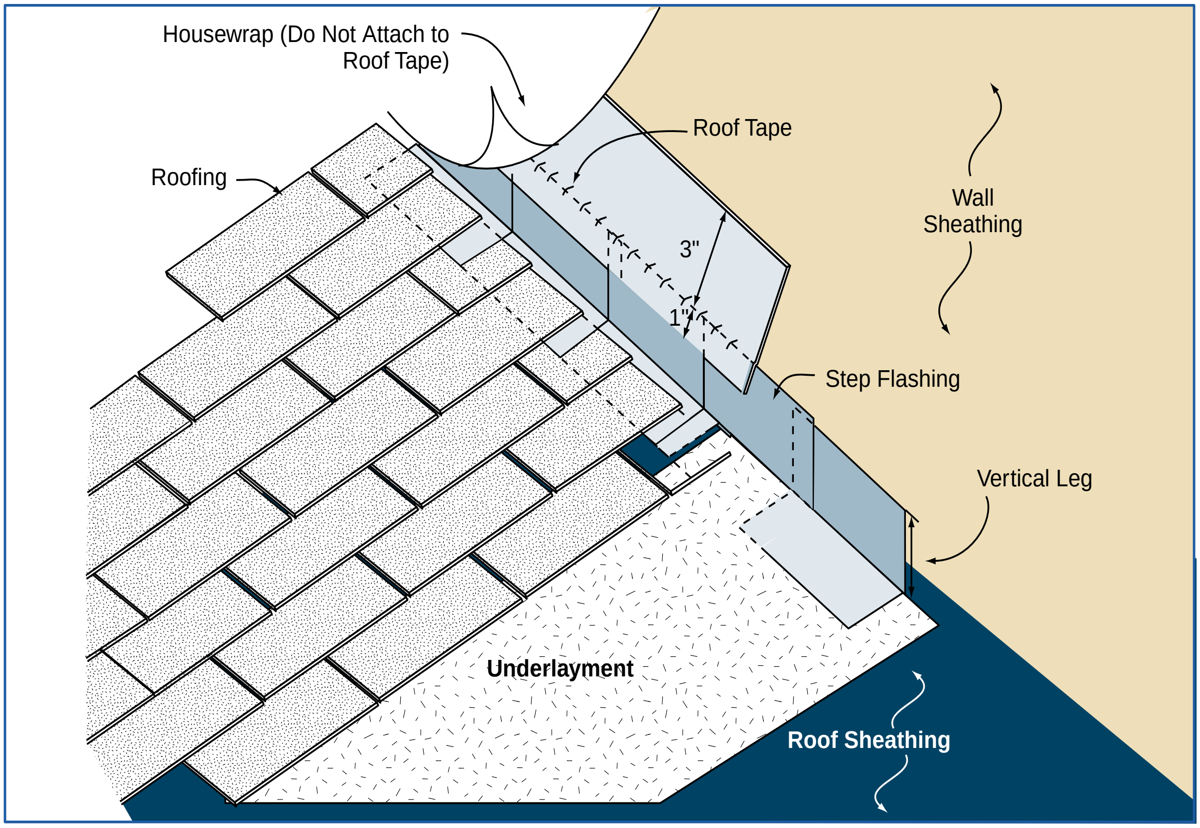
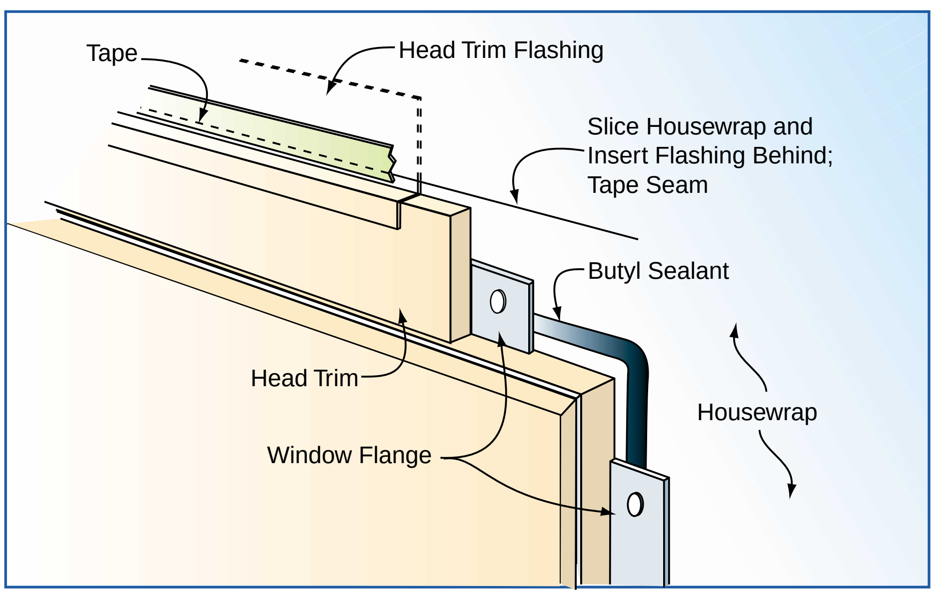



**Figure 12-85**
Window flashing detail. Courtesy of Journal of Light Construction.




### 12.7.3 Doors and Door Assemblies

High winds and wildfires are the natural hazards that present the greatest threat to exterior doors.

### 12.7.3.1 High Winds

Loads and resistance: The door assembly (i.e., door, hardware, frame, and frame attachment to the wall) should be of sufficient strength to resist positive and negative design wind pressures (see Section 11.8 and the Wind Load Example Problem on page 11-45). The assembly should be specified to comply with wind load testing in accordance with ASTM E 1233. (Note: ASTM E 330 is the most commonly used method to evaluate door assemblies; however, because E 1233 is a cyclic test method, whereas E 330 is a static test, E 1233 is the preferred test method, particularly in hurricane-prone areas.) It is important to specify frame attachment to the wall, either by performance or prescriptive criteria, as inadequate frame attachment due to lack of design guidance is a common problem.

Except for glazing, doors typically do not need to be resistant to loads from hurricane-generated windborne missiles. If a door is hit with a missile, the missile may penetrate the door, but, in most cases, the missile opening will not be large enough to result in significant water infiltration problems or in a substantial increase in internal pressure. For information on glazing in doors, see Section 12.7.4.2. If enhanced missile resistance of solid doors is desired for occupant protection, specify door assemblies that have demonstrated compliance with testing discussed in Section 12.7.4.2.

**CROSS-REFERENCE**

**Section 12.7.4.2 provides information about windborne missiles and glazing in doors.**

Durability: To avoid corrosion problems with metal doors or frames, aluminum or galvanized steel units are recommended for buildings located within 3,000 feet of an ocean shoreline. Stainless steel frame anchors and hardware are also recommended. Fiberglass doors may also be used with wood frames. Galvanized steel doors and frames should be painted for additional protection.

In areas with severe termite problems, metal door assemblies are recommended. If concrete, masonry, or metal wall construction is used to eliminate termite problems, it is recommended that wood not be specified for blocking or nailers. If wood is specified, see Appendix J, for information on wood treatment methods.

Water infiltration: Hurricanes and coastal storms present enormous wind-driven water infiltration problems. Leakage can occur between the door and frame or between the frame and wall. Because of the extremely high design wind pressures and numerous opportunities for development of leakage paths, some leakage should be anticipated when design wind-speed conditions are approached. Examples of design responses to possible water infiltration include the following:

- Vestibule – designing a vestibule is one method of accounting for the infiltration problem. With this approach, both the inner and outer doors can be equipped with weatherstripping, and the vestibule itself can be designed to tolerate water. For example, water-resistant finishes (e.g., concrete or tile) can be specified, and the floor can be equipped with a drain.

- Door swing – if weatherstripping is specified, out-swinging doors offer an advantage over in-swinging doors. With out-swinging doors, the weatherstripping is located on the interior side of the door, where it is less susceptible to degradation. Also, some interlocking weatherstripping assemblies are available for out-swinging conditions. However, there is a security disadvantage to out-swinging doors; without door stops, an out-swinging door is much easier to break into.

- Pan flashing – adding flashing under the door threshold will help prevent penetration of water into the subflooring. This is a common place for water entry and subsequent wood dry rot in coastal homes.

A variety of pre-manufactured weatherstripping components are available, including drips, door shoes and bottoms, thresholds, and jamb/head weatherstripping. A few examples of weatherstripping options are presented below:

- Drips are intended to divert water away from the opening between the frame and door head, and the opening between the door bottom and the threshold as shown in Figures 12-86 and 12-87. Alternatively, a door sweep can be specified (Figure 12-88); however, for high-traffic doors, periodic replacement of the neoprene will be necessary.

- Door shoes and bottoms are intended to minimize the gap between the door and the threshold. Figure 12-87 illustrates a door shoe that incorporates a drip. Figure 12-89 illustrates an automatic door bottom. Door bottoms can be surface-mounted or mortised. For high-traffic doors, periodic replacement of the neoprene/vinyl will be necessary.

- Thresholds are available in a variety of configurations. Thresholds with high vertical offsets offer enhanced resistance to water; however, where handicap access thresholds are required, the offset is limited. Thresholds that can be interlocked with the door, or thresholds with a stop and seal, are recommended. Examples are shown in Figure 12-90. Designers should specify setting the threshold in sealant to avoid water infiltration between the threshold and floor. Butyl sealant is recommended. If the threshold has a drain pan (Figure 12-90), designers should specify that the weep holes not be obstructed during installation.

- Adjustable jamb/head weatherstripping is recommended. These units offer good door contact because they have wide sponge neoprene and they can be adjusted to fit the door (Figure 12-91).

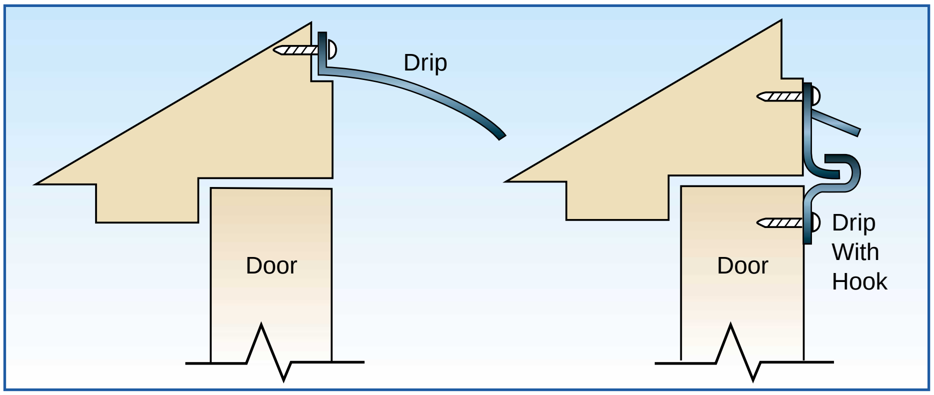



**Figure 12-86**
Drip at door head and drip with hook at head.

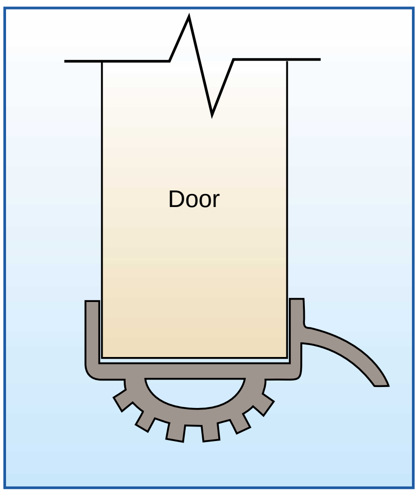



**Figure 12-87**
Door shoe with drip and vinyl seal.

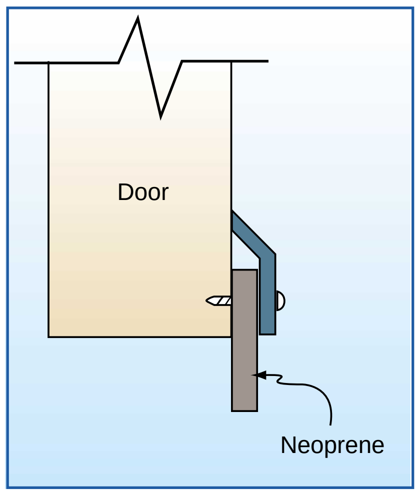



**Figure 12-88**
Neoprene door bottom sweep.

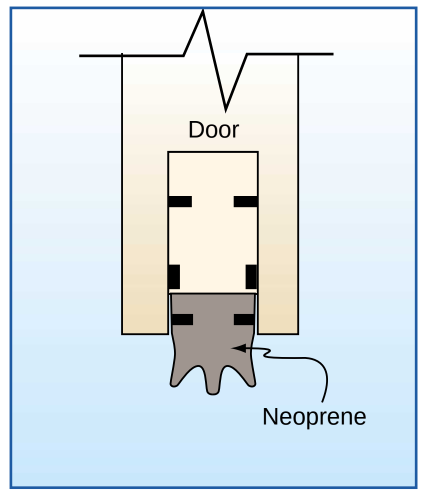



**Figure 12-89**
Automatic door bottom.

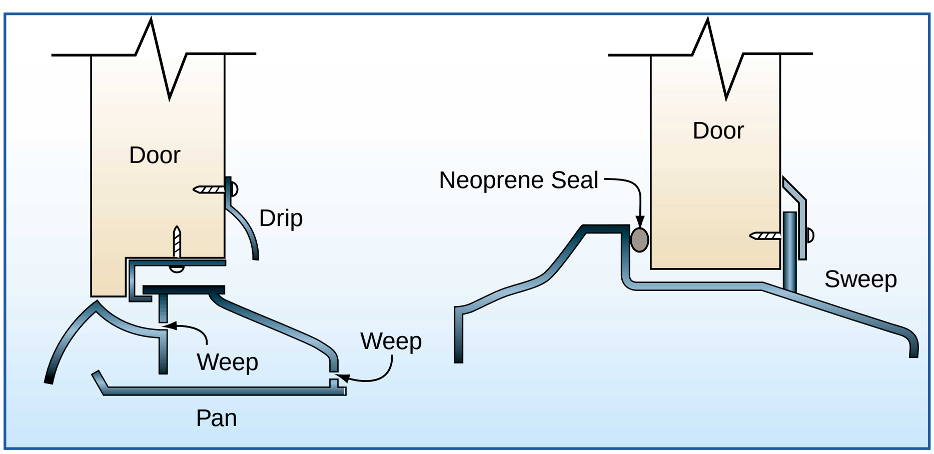
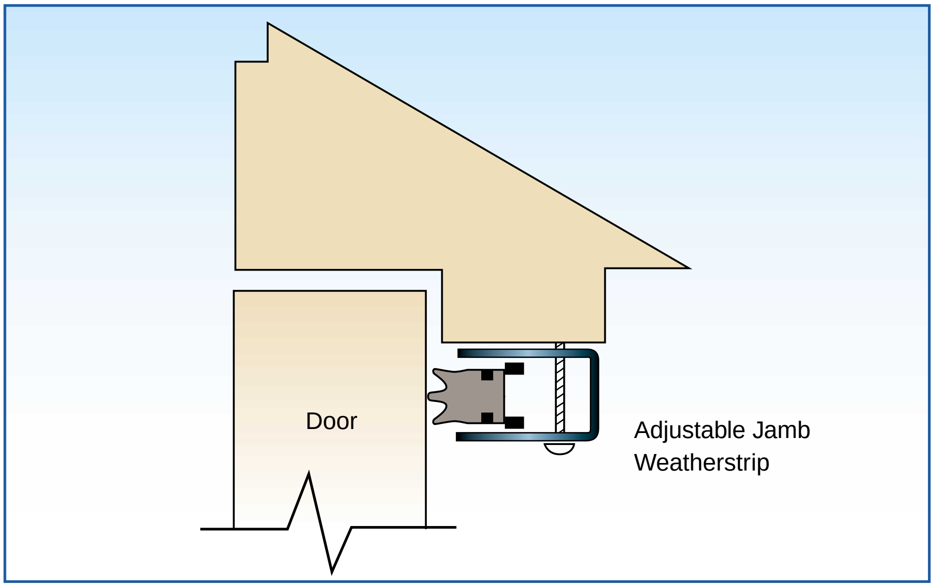



**Figure 12-90**
Interlocking threshold for doors with exit hardware.

**Figure 12-91**
Adjustable jamb/head weatherstripping.




### 12.7.3.2 Wildfires

For buildings in areas prone to wildfires, fire-rated metal doors and frames are recommended. Door/frame assemblies are available in 3/4-, 1-1/2-, and 3-hour ratings.

Although construction of the door and frame are usually the same for these three ratings, the amount of allowable glass opening is decreased as the hourly rating is increased. Maximum protection is provided by a 3-hour rated assembly with no glazing. When glazed openings are desired, maximum protection is provided by fire-rated glass ceramic (this type of glass is discussed in Section 12.7.4.7).

For glass sliding doors, protection of the opening with fire-rated glass ceramic or a metal shutter is recommended; however, because a sliding glass door/ shutter assembly has not been subjected to fire testing, its reliability is unknown. See Section 12.7.4 for shutter information.

## 12.7.4 Windows, Shutters, and Skylights

High winds and wildfires are the natural hazards that present the greatest threat to windows and skylights. Strong seismic events can also damage windows, but, in residential construction, this is not common. Hail can be very damaging to skylights and occasionally causes window breakage. Although this section focuses on windows, shutters, and skylights, it also addresses door glazing.

### 12.7.4.1 High Winds

Window and skylight assemblies (i.e., glazing, hardware for operable units, frame, and frame attachment to the wall/curb) must be of sufficient strength to resist positive and negative design wind pressures. The assembly should be specified to comply with wind load testing in accordance with ASTM E 1233. (Note: ASTM E 330 is the most commonly used method for evaluating window assemblies; however, because E 1233 is a cyclic test method and E 330 is a static test, E 1233 is the preferred test method, particularly in hurricane-prone areas). Designers must specify the frame attachment to the wall, either by performance or prescriptive criteria, as inadequate frame attachment due to lack of design direction is a common problem as illustrated by Figure 12-92.

**Figure 12-92**
Hurricane Georges (1998), Puerto Rico. Inadequate window frame attachment caused the window frame to be pulled out of the wall.

### 12.7.4.2 Windborne Missiles

A special consideration of glazing is its susceptibility to breakage by windborne missiles (debris). When a missile penetrates most materials, only a small opening is made; when a missile penetrates most glazing materials, however, it can result in a very large opening. The opening is often sufficiently large to increase the building's internal pressure, which may overstress other building envelope components or the structure itself. Increased internal pressure can also damage interior partitions and ceilings. A substantial amount of wind-driven water may also enter through the breached openings. In windstorms other than hurricanes, the probability of a window or skylight being struck by a missile is extremely low; however, in hurricane-prone regions, the missile load issue is of concern. Figure 12-93 shows a large window broken by windborne debris.

**NOTE**

Because of the extensive interior damage that can be caused by wind and rainwater if glazing is breached, and because of the large internal pressure created by a breach of the building envelope, this manual recommends the use of shutters or impact-resistant glass rather than the design of partially enclosed buildings.

Missile impact criteria are included in the IBC 2000 and IRC2000. ASCE 7-98, the IBC, and the IRC require that one of the following conditions be met for glazing (including skylights) in windborne debris regions:

1. The glazing must either be designed to resist missiles or be protected by shutters, or
2. The building in which the glazing is used must be designed as a partially enclosed building.

According to ASCE 7-98, the IBC, and the IRC, windborne debris regions are areas within hurricane-prone regions located:

- within 1 mile of the coast where the basic wind speed is equal to or greater than 110 mph (3-sec peak gust) and in Hawaii, or
- in all areas where the basic wind speed is equal to or greater than 120 mph (3-sec peak gust), including Guam, Puerto Rico, the U.S. Virgin Islands, or American Samoa.

**Figure 12-93**
Hurricane Georges (1998), Puerto Rico. Window broken by windborne debris.

It is recommended that in addition to being tested for air pressure (i.e., ASTM E 1233 and E 330), glazed assemblies in windborne debris regions have sufficient strength to resist missile loads specified in the IBC and IRC and be tested in accordance with ASTM E 1886-97 and E 1996-99. (Note: A special impact-resistant glazing assembly or shutters will be necessary.) Glazing protected with shutters should also be designed to resist the positive and negative design wind pressure.

The typical missile test criteria include the following provisions:

- Small missiles weighing 2 grams shall impact the surface at a speed of 80 ft/sec (54 mph).
- Each test specimen shall receive 30 small missile impacts; these 30 impacts are to be distributed over the surface of the window/door unit.
- Large missiles shall be wood 2x4s weighing 9 lb and impacting the surface at a speed of 50 ft/sec (34 mph).
- Each test specimen shall receive two large-missile impacts, one in the center of the assembly, and one near the corner.

In addition to the missile impact standards, each of the window/door assemblies must pass a cyclic pressure test. The assembly must first pass the missile impact test, then that same assembly is used in the cyclic pressure test. A pressure loading sequence for both inward- and outward-acting pressure, a range of pressure, the number of cycles, and the duration of each cycle is given. Tests should also be conducted for shutters, skylights, and glazing protective film.

### 12.7.4.3 Durability

To avoid corrosion problems, aluminum, wood or vinyl frames are recommended for buildings located within 3,000 feet of an ocean shoreline. Stainless steel frame anchors and hardware are also recommended in these areas. In areas with severe termite problems, wood frames should either be treated or not used.

### 12.7.4.4 Water Infiltration

Hurricanes and other coastal storms present enormous wind-driven rainwater infiltration problems. Leakage can occur at the glazing/frame interface, at the frame itself, or between the frame and wall. Because of the high design wind pressures and numerous opportunities for leakage path development, some leakage should be anticipated when design wind speed conditions are approached. A design approach to deal with this problem is to not run carpet all of the way to walls that have a large amount of glazing. Instead, a strip of water-resistant material such as tile could be specified along the wall. During a storm, towels could be placed along the strip to absorb water infiltration. These actions can help protect carpets from water damage.

It is recommended that window units be tested in accordance with ASTM D 1233 for water infiltration. The challenge with prefabricated window units is meeting the need for successful integration between the window units and the walls. To the extent possible, detailing of the interface between the wall and window units should rely on sealants as the secondary line of defense against water intrusion, rather than making them the primary protection.

The design of joints between walls and window units should consider the shape of the sealant joint (i.e., a square joint is typically preferred) and type of sealant to be specified. The sealant joint should be detailed so that the sealant is able to bond on only two opposing surfaces (i.e., a backer rod or bond-breaker tape should be specified). Butyl is recommended for concealed sealants, and polyurethane is recommended for exposed sealants. During installation, cleanliness of the sealant substrate is important (particularly if polyurethane or silicone sealants are specified), as well as tooling of the sealant.

A removable stop, as illustrated in Figure 12-94, protects the sealant from direct exposure to the weather, and the protection can reduce the wind-driven rain demand on the sealant.

On-site water infiltration testing in accordance with ASTM E 1105 can be specified in instances where water infiltration is particularly important; however, this is an expensive test.

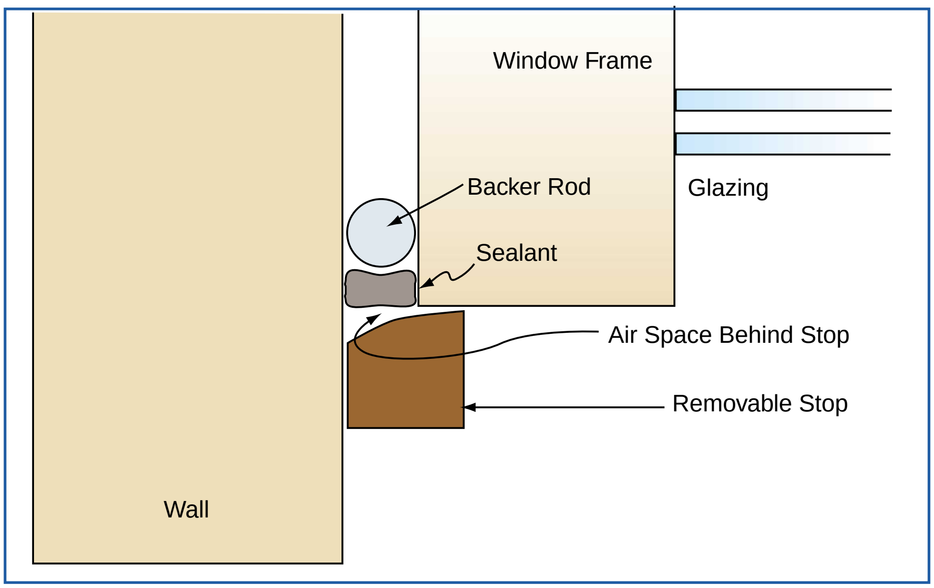



**Figure 12-94**
Protection of sealant with a stop.

**NOTE**

Missile impact standards (ASTM E 1886 and E 1996) cover criteria for small and large missiles. Shutters can be costly, so the designer or owner may want to evaluate debris and impact potential vs. the cost of shutters for the design wind event. The most important envelope protection goal is to prevent an opening from allowing internal pressure increases in the building and to prevent windblown water from causing interior damage.

### 12.7.4.5 Shutters

If shutters are used to provide missile protection, they should be tested as discussed above for glazing (see Section 12.7.4.2). Miami-Dade County, Florida, has established a product approval mechanism for shutters to ensure that these important protective devices and the method used for securing them to the building are rated for particular wind and missile loads. A variety of shutter designs and materials are available. For windows that are difficult to reach, motor-driven rollup shutters are available as illustrated in Figure 12-95. Shutter designs that use permanently installed tracks facilitate rapid attachment of shutter panels as illustrated in Figure 12-96.

Plywood panels can also provide protection, provided they are thick enough (a minimum thickness 15/32 inch is recommended). Attaching the panels with closely spaced screws will help prevent them from blowing off. Figure 12-97 illustrates an attachment scheme for plywood panels on either wood-frame or masonry walls.

Shutters should be attached to the wall rather than the window frame, because the attachment between the window frame to the wall may be weak, as illustrated in Figure 12-92. Shutters also need to be stiff enough, or set far enough away from the glazing, that under design wind loads they do not deflect and break the glass. Shutters constructed of 2x4 boards can also provide protection from missile impacts as shown in Figure 12-98.

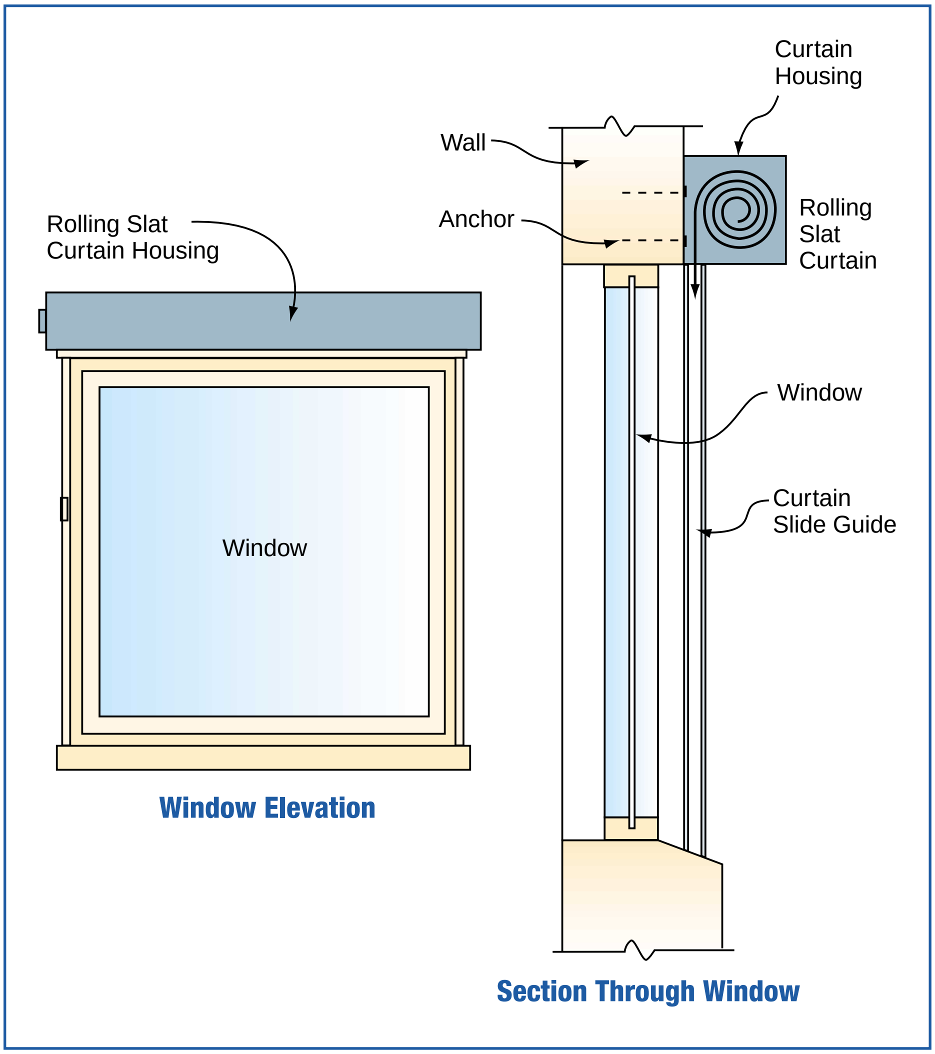



**Figure 12-95**
Motor-driven rollup shutters.

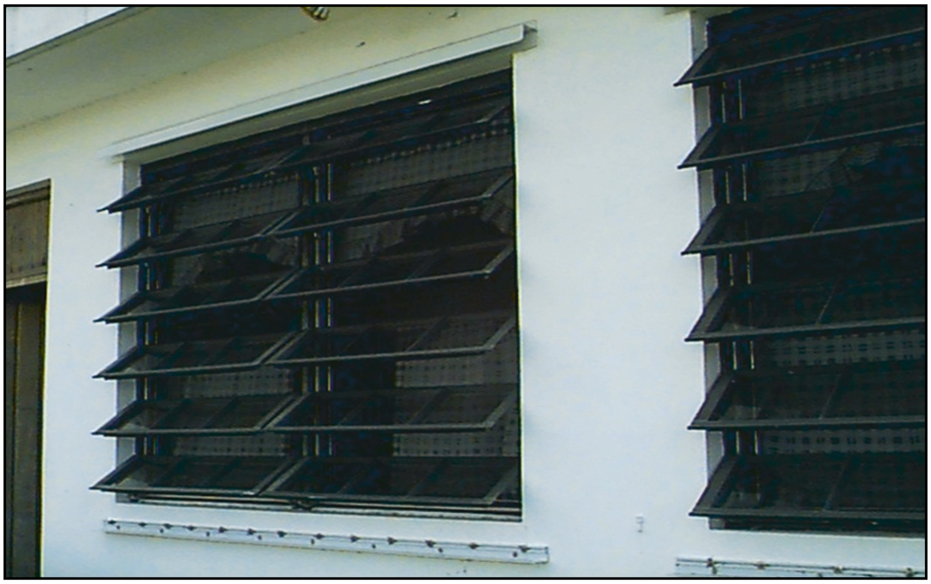


**Figure 12-96**
Hurricane Georges (1998), Puerto Rico. Permanently installed shutter tracks.

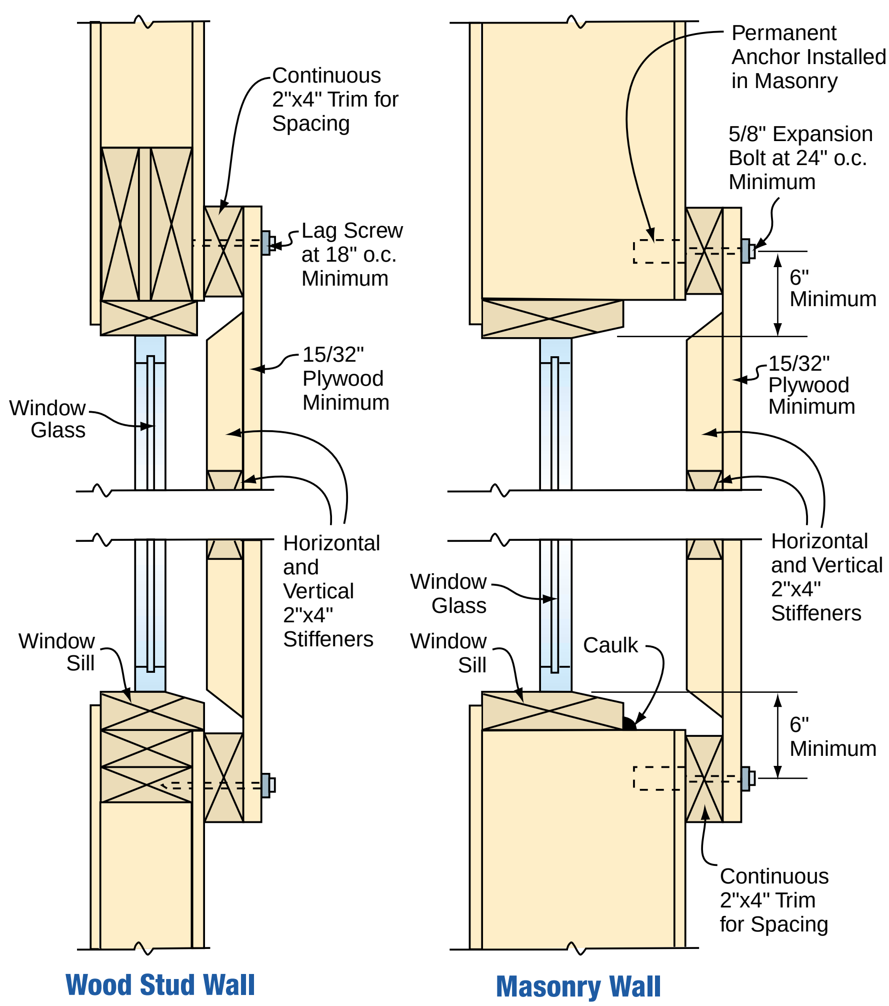



**Figure 12-97**
Plywood “shutter” attachment on either wood-frame or masonry walls.

**NOTE**

The Engineered Wood Association (formerly the American Plywood Association) has prepared five brochures that present hurricane shutter designs for wood-frame and masonry buildings. The brochures are available on-line at the Engineered Wood Association website at http://www.apawood.org/

**Figure 12-98**
Hurricane Marilyn (1995), U.S. Virgin Islands. Shutters constructed of 2x4 lumber.

### 12.7.4.6 Seismic

Glass breakage due to in-plane wall deflection is unlikely; however, special consideration should be given to walls that have a high percentage of windows and limited shear capacity. In these cases, it is important to analyze the in-plane wall deflection to verify that it will not exceed the limits prescribed in the building code.

### 12.7.4.7 Wildfires

Limited data and guidelines are available related to performance of glazing subjected to wildfires**.** Reports from California (McMullen 1994) indicate that insulated (i.e., double-pane) windows were found to be an effective insulator against radiant heat exposure. Fire-rated glass ceramic is also a very heat-resistant material, but it is relatively expensive. Insulated windows could be composed of a fire-rated glass ceramic for the outer lite and annealed glass for the inner lite. With these approaches, metal frames should provide greater reliability than wood or vinyl frames.

Another approach is to protect the window openings with metal shutters similar to those shown in Figures 12-95 and 12-96; however, because these types of shutter assemblies have not been subjected to fire testing, their reliability is unknown.

### 12.7.4.8 Hail

A test method has not been developed specifically for testing skylights for hail-resistance. However, ASTM E 822, for testing hail-resistance of solar collectors, could be used for assessing the hail-resistance of skylights.

**NOTE**

The IBC and IRC include requirements for roof coverings in high-wind areas.

### 12.7.5 Roof Coverings

High-winds, seismic events, wildfires, and hailstorms are the natural hazards that can cause the greatest damage to roof covering systems. In addition to damage that can be inflicted on the roof covering itself, there are secondary issues that are of greater importance. When high winds damage the roof covering, water infiltration commonly causes significant damage to the interior of the building and its contents. Water infiltration may also occur after very large hail impact. During seismic events, heavy roof coverings such as tiles or slate may be dislodged and fall from the roof and present a hazard if the building is occupied during the event. A roof covering system that is not highly resistant to fire exposure can result in the destruction of the building during a wildfire.

**NOTE**

Roofing failures in high-wind areas are one of the most common causes of substantial building damage and dollar loss.

Residential buildings typically have steep-slope roofs (i.e., a slope greater than 3:12). Some residential buildings have low-slope roofs, which can be roofed with a variety of systems. Low-slope roof systems are discussed in Section 12.7.5.8.

A variety of products can be used for steep-slope coverings. The following commonly used products are discussed below: asphalt composition shingles, cement-fiber shingles, liquid-applied membranes, tiles, metal panels, metal shingles, slate, and wood shingles and shakes. All of these steep-slope roofing coverings are air-permeable, except for the liquid-applied membrane and metal panel systems, which are air-impermeable. The wind loads derived in Chapter 11 represent the pressure differential between the exterior and interior surfaces of the roof system. For air-impermeable coverings, the roof covering should typically be designed to resist the full wind load; however, because of partial air-pressure equalization provided by air-permeable coverings, the wind loads derived in Chapter 11 can overestimate the load on air-permeable coverings. ASCE 7 allows the designer to use the full design load or calculate the load by using a pressure coefficient that was specifically developed for the air-permeable element being considered. Research on special pressure coefficients have been developed only for asphalt shingles and tiles. Therefore, according to ASCE 7, the other air-permeable roofing products have to be designed for the full wind load, which is problematic for several of the coverings as discussed below.

**NOTE**

When retrofitting in high-wind areas, it is recommended that the existing roof covering be removed so that the roof deck can be checked for deterioration and to verify that it is adequately attached.

#### 12.7.5.1 Asphalt Shingles

The discussion of asphalt shingles relates only to shingles with self-seal tabs. Mechanically interlocked shingles are not addressed because of their limited use.

**High Winds**

A method for calculating uplift loads on asphalt shingles has been developed (Peterka et al. 1997). A pressure coefficient for use in the calculation has also been developed; however, that coefficient may not be applicable to all asphalt shingles. Additional research needs to be undertaken to determine coefficients on all asphalt shingles. Until work on the uplift coefficients is complete, it is not possible to analyze loads on asphalt shingles. Therefore, a prescriptive solution based on results of field investigations and judgement is necessary. However, with this approach, the uplift resistance provided by the system is unknown.

**WARNING**

**ASTM D 3161 and Underwriters Laboratories (UL) 997 are virtually identical test methods for determining uplift resistance of asphalt shingles. However, complying with these methods does not ensure successful performance of shingles during high-wind events.**

The key elements to successful wind performance include the bond-strength of the self-seal strip, the mechanical properties of the shingle, and correct application of the shingle fasteners. If the tab lifts, the number of fasteners used to attach the shingle may influence whether or not shingles are blown off. Bond strength can be assessed by ASTM D 6381.

Underlayment

In the event of shingle blowoff, subsequent water infiltration damage can be avoided if the underlayment remains attached and is adequately sealed at penetrations. However, to achieve reliable secondary protection, an enhanced underlayment design is needed. The design enhancements include increased blowoff resistance of the underlayment itself, increased resistance to water infiltration (primarily at penetrations), and increased resistance to extended weather exposure.

If shingles are blown off, in some cases the underlayment is exposed for only a week or two before a new roof covering is installed. But many roofs damaged by hurricanes go unrepaired for several weeks. If a hurricane strikes a heavily populated area, roof covering damage is typically extensive. Therefore, because of the great work load, large numbers of roofs go unrepaired for several months. It is not uncommon for some roofs to go unrepaired for nearly a year.

**WARNING**

**If the roof has a ridge vent that is blown off, the underlayment recommendations do not provide water infiltration protection at ridge vent slots. To avoid leakage at ridge vent slots, gable end vents and/or well-anchored stack vents in lieu of ridge vents should be considered.**

The longer an underlayment will be exposed to weather, the more durable it will need to be to provide adequate water infiltration protection for the residence. The three options presented on page 12-122 are listed in order of decreasing resistance to long-term weather exposure. Option 1 provides the greatest reliability for long-term exposure—it is advocated in areas that are heavily populated and where the design wind speed is equal to or greater than 120 mph. Option 3 provides limited protection and is advocated only in areas that have a modest population density and the design wind speed is less than or equal to 110 mph (3-sec peak gust).

**Option 1.** Specify taping the joints of the plywood sheathing with roof tape (i.e., self-adhering modified bitumen). Specify that the tape be a minimum of 4 inches wide, applied to a broom-clean deck, and rolled with a roller. Seal around deck penetrations with roof tape, sealant, or asphalt roof cement. Apply a single layer of ASTM D 226 Type II (#30) underlayment felt, attached with low-profile capped-head nails or thin metal disks ("tin caps") attached with roofing nails. Fasten at approximately 6 inches on center (o.c.) along the laps and at approximately 12 inches o.c. along two rows in the field of the sheet between the side laps. Apply a single layer of self-adhering modified bitumen complying with ASTM D 1970. Seal the modified bitumen sheet to the deck penetrations with roof tape or asphalt roof cement.

Note: (1) Because of its enhanced resistance to weather exposure, plywood is recommended in lieu of OSB, in the event the underlayment is blown off. (2) The purpose of the tape over the plywood joints is to provide secondary protection if the underlayment is blown off. (3) As an alternate to the modified bitumen sheet, install two plies of felt complying with ASTM D 2178 Type IV. Set each sheet in a continuous mopping of hot asphalt complying with ASTM D 312 Type IV.

**Option 2.** Specify taping the sheathing joints and sealing around penetrations as described in Option 1. Specify two plies of underlayment felt with offset side laps, complying with ASTM D 226 Type I (#15). Attach the underlayment with low-profile capped-head nails or thin metal disks ("tin caps") attached with roofing nails. Fasten at approximately 6 inches o.c. along the laps and at approximately 12 inches o.c. along a row in the field of the sheet between the side laps.

Note: If the building is within 3,000 feet of salt water, stainless steel or hot-dip galvanized fasteners are recommended for the underlayment attachment.

**Option 3.** Specify taping the sheathing joints and sealing around penetrations as described in Option 1. Apply a single layer of ASTM D 226 Type I (#15) underlayment felt in the normal fashion.

Note: (1) If the roof slope is less than 4:12, tape and seal the sheathing and follow the recommendations given in *The NRCA Roofing and Waterproofing Manual* (NRCA 1996). (2) With this option, the underlayment has limited blowoff resistance. Water infiltration resistance is provided by the taped and sealed sheathing panels. This option is intended where temporary or permanent repairs are likely to be made within several days after the roof covering is blown off.

### Shingle Products

If fiberglass-reinforced shingles are desired, specify shingles that comply with ASTM D 3462. If organic-reinforced shingles are desired, specify shingles that comply with ASTM D 225. SBS modified bitumen shingles are another option to consider. Because of the flexibility imparted by the SBS polymers, if a tab on a modified bitumen shingle lifts, it is less likely to break and blowoff. Compared to fiberglass-reinforced shingles, organic-reinforced shingles typically have substantially lower resistance to pulling over fastener heads. ASTM D 3462 specifies a minimum fastener pull-through resistance of 20 lb/ft at 70° F; however, fiberglass-reinforced shingles are available with a fastener pull-through resistance in excess of 30 lb/ft. In high-wind (less than or equal to 90 mph 3-sec peak gust) areas, it is recommended that a minimum pull-through resistance of 25 lb/ft be specified. In areas with extremely high design wind speeds (greater than 120 mph 3-sec peak gust), it is recommended that a minimum value of 30 lb/ft be specified.

At the time this manual was produced, data on bond-strength of the self-seal adhesive were typically unavailable. Bond strengths range from approximately 3 to 20 lb/ft at 70° F. It is recommended that bond-strength data be sought from manufacturers and that products be specified with a minimum of 12 lb/ft in high-wind areas. In areas with extremely high design wind speeds, it is recommended that a minimum value of 17 lb/ft be specified.

### Attaching and Sealing

Specify attachment with six nails (rather than staples) per shingle. Locate the nails as indicated in *The NRCA Steep Roofing Manual*. For roofs within 3,000 feet of the ocean, hot-dip galvanized (rather than electroplated [EG]) or stainless steel nails are recommended.

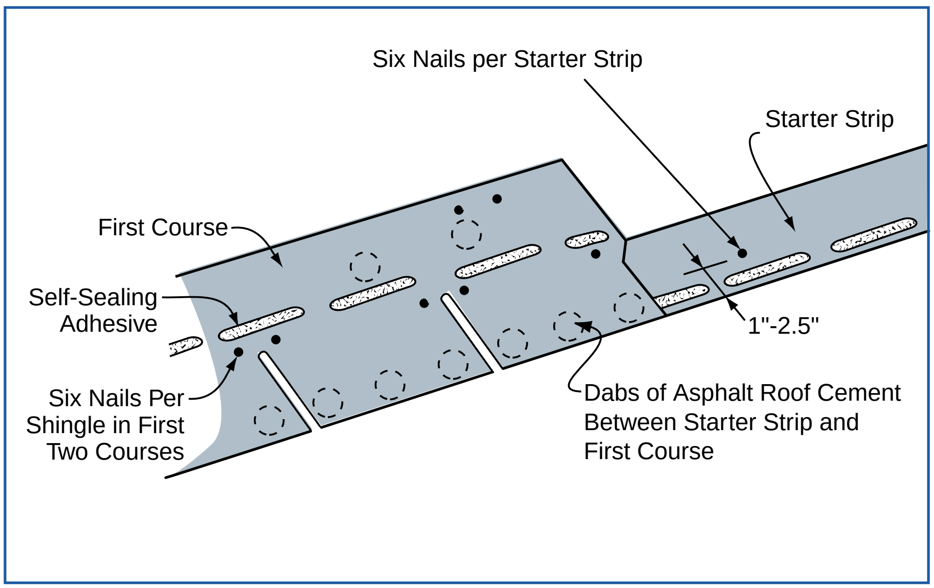
It is recommended that the designer specify that the starter strip be nailed approximately 1–2-1/2 inches from the eave edge of the starter strip. One inch is preferred, but framing conditions may require that the nails be placed further away. The fasteners should be placed in sheathing or framing lumber, rather than in trim boards. Six nails per starter strip are recommended. Specify that the starter strip and first course overhang the eave metal by approximately 1/4 inch. The 1/4-inch overhang will provide a drip edge without exposing much shingle beyond the eave for wind to lift. It is recommended that the designer specify sealing the first course of shingles by placing three (approximately 1-inch-diameter) dabs of asphalt roof cement over the starter strip so that the overlying tab of the first course will be adhered. The eave detail is illustrated in Figure 12-99.

It is recommended that the designer specify hand tabbing rakes, ridges, and hips as illustrated in Figures 12-100 and 12-101. At rakes, two dabs of asphalt roof cement are placed on the shingle about 1 inch from the rake edge, and two dabs of cement are placed on the metal drip edge. The next shingle is then set in place and fasteners, except for the one at the rake, are installed. The rake end of the shingle is then pressed to set the shingle in the dabs of cement. Finally, the fastener at the rake is installed. (Note: If a bleeder strip is used at the rake, omit the dabs of cement on the drip edge.) Figure 12-100 shows a rake detail. Specify that the rake shingles overhang the metal drip edge by approximately 1/4 inch.

At hips and ridges, two dabs of cement are placed on each side of the hip/ridge line on a hip/ridge shingle that has been installed already, as shown in Figure 12-101**.** Two additional dabs are placed on the field shingles to adhere the next hip/ridge shingle. For the starter hip/ridge shingle, four dabs of tab cement are placed on the field shingles.

After the next hip/ridge shingle is set in place, it is pressed to set the shingle in the dabs of cement and one nail is applied on each side of the hip/ridge line.

**Figure 12-99**
Eave detail for asphalt shingles.



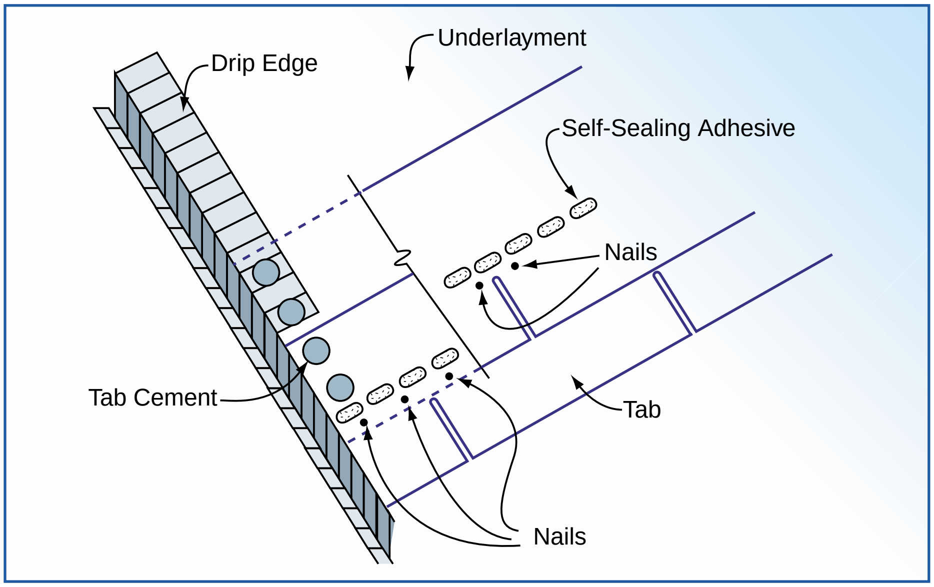



**Figure 12-100**
Rake detail for asphalt shingles.

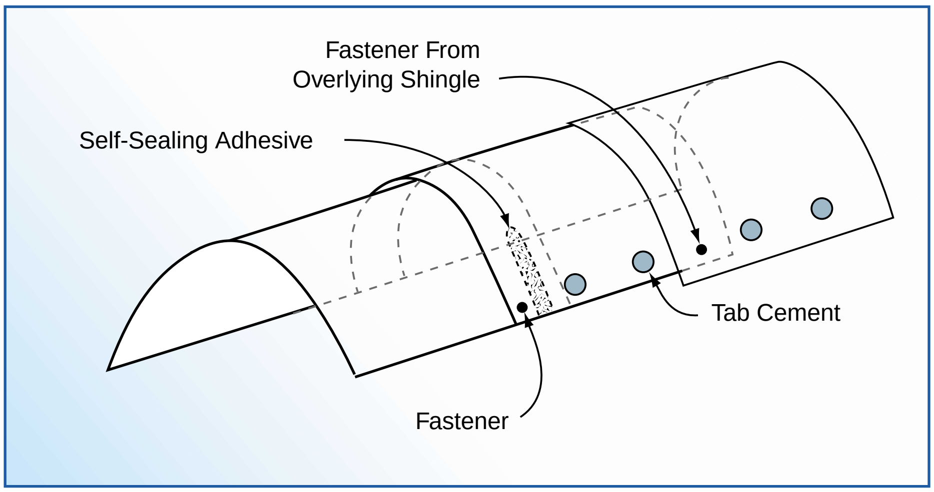



**Figure 12-101**
Hip and ridge detail for asphalt and composition shingles.

Hip and ridge shingles are normally cut from a strip shingle. Premanufactured hip/ridge shingles are also available and may or may not have self-sealing adhesive. If they do not, specify application of cement to replicate the adhesive on the strip shingles.

Because of extra thickness of materials at hips and ridges, longer nails than those used in the roof field are needed. All nails should penetrate the underside of the sheathing, or penetrate at least 3/4 inch into wood plank decks.

Some building codes require that hips, ridges, rakes, eaves, and valleys be set in wide continuous bands of asphalt roof cement; however, this practice may result in shingle blistering, which could shorten shingle life and be aesthetically objectionable to the homeowner. If the code requires wide continuous bands of cement, it is recommended that the designer request the building official to allow the use of dabs of cement as recommended above.

Ridge Vents

Ridge vents should be evaluated for susceptibility to penetration by wind-driven rain. Test methods or specific design guidance have not been developed. See Section 12.7.6 for additional information.

**Wildfires**

Organic-reinforced shingle systems achieve only a Class C fire-resistance rating, which indicates very little fire resistance. Fiberglass-reinforced shingle systems achieve a Class A fire-resistance rating, which is the highest rating achieved through ASTM E 108 fire testing. Under typical fire exposure conditions, fiberglass-reinforced shingle systems normally provide sufficient fire resistance; however, they typically do not provide sufficient resistance to the extreme conditions induced by wildfires. Where enhanced resistance to wildfires is desired, another type of roof covering is recommended.

**Hail**

UL 2218 is a test method for assessing simulated hail resistance of roofing systems. This test yields four ratings (Classes 1–4). Systems rated Class 4 have the greatest impact resistance. Asphalt shingles are available in all four classes. It is recommended that asphalt shingle systems on buildings located in areas vulnerable to hail be specified to pass UL 2218, at a Class level commensurate with the hail load. Hail resistance of asphalt shingles depends partly on the condition of the shingles at the time they are impacted and is likely to decline with roof age.

### 12.7.5.2 Cement-Fiber Shingles

Cement-fiber shingles are made to simulate the appearance of slate, tile, wood shingles, or wood shakes. The material properties of various cement-fiber products vary due to differences in material composition and manufacturing processes.

**High Winds**

Because of limited market share of cement-fiber shingles in areas where research has been conducted after high-wind events, few data are available on their wind performance. At the time this manual was produced, manufacturers had not conducted research to determine a suitable pressure coefficient or coefficients; therefore, demonstrating compliance with ASCE 7 is problematic with these products until such a coefficient(s) is developed. A consensus test method for uplift resistance has not been developed for these products.

In high-wind areas, mechanical attachment should be specified in corner and perimeter roof zones. With some of the lighter-weight products, mechanical attachment in the field of the roof may also be prudent. In extremely high-wind areas, mechanical attachment in the field of the roof should be specified. Additional mechanical attachment near the tail of the shingle should also be specified in perimeter and corner zones, and perhaps in the field of the roof. Because these prescriptive attachment suggestions are based on limited information, the uplift resistance that they provide is unknown.

To minimize water damage in the event of blowoff of the cement-fiber covering, the underlayment recommendations for asphalt shingles are suggested. For cement-fiber systems, however, a Type II (i.e., #30) felt is recommended in order to provide more puncture resistance during application. Stainless steel or hot-dip galvanized fasteners are recommended for roofs within 3,000 feet of the ocean. Note that cement-fiber systems are vulnerable to breakage from missile impact. See the discussion in Section 12.7.5.4**.**

#### Seismic

Cement-fiber products are relatively heavy and, unless they are adequately attached, they can be dislodged during strong seismic events and fall from the roof. At time this manual was produced, manufacturers had not conducted research or developed design guidance for use of these products in areas prone to large ground motion accelerations. The guidance provided in the Section 12.7.5.4 is suggested until specific guidance is developed for cement-fiber products.

#### Wildfires

Most cement-fiber systems can achieve a Class A fire-resistance rating; however, some only achieve a Class B. The Class A systems should provide relatively good fire resistance during wildfires; however, because cement-fiber products typically have less thermal mass than tiles, they probably do not offer as much protection as tiles.  Cement-fiber products may not be as fire-resistant as metal-panel or metal-shingle systems.

#### Hail

At the time this manual was being produced, one cement-fiber assembly had passed UL 2218 with a Class 2 rating. It is recommended that cement-fiber shingle systems on buildings in areas vulnerable to hail be specified to pass UL 2218, at a class level commensurate with the hail load. If products with the desired class are not available, another type of product should be considered.

### 12.7.5.3 Liquid-Applied Membranes

Liquid-applied membranes are not common on the U.S. mainland, but they are common in Guam, the U.S. Virgin Islands, Puerto Rico, and American Samoa.

**High Winds**

Investigations following strong hurricanes and typhoons have revealed that liquid-applied membranes installed over concrete and plywood decks have offered excellent performance (provided the deck remains attached to the building). This type of roof covering has extremely high wind-resistance reliability.

It has been found that unprotected concrete roof decks could eventually experience problems with corrosion of the slab reinforcement. It is recommended that all concrete roof decks be covered with some type of roof covering.

**Wildfires**

Liquid-applied membranes over concrete decks should provide excellent wildfire-resistance. A Class A rating can be achieved for liquid-applied membranes over a plywood deck;  however, where enhanced resistance to the extreme conditions induced by wildfires is desired, a roof covering other than a liquid-applied membrane over plywood is recommended.

**Hail**

At the time this manual was being produced, no liquid-applied membranes over concrete or plywood decks had been evaluated by UL 2218.

### 12.7.5.4 Tiles

Clay and extruded concrete tiles are available in a variety of profiles and attachment methods.

**NOTE**

For additional information, see SBCCI standard SSTD 11-97, *Test Standard for Determining Wind Resistance of Concrete or Clay Roof Tiles* (SBCCI 1997b) for additional information.

**High Winds**

Loads and Resistance

Methods to calculate uplift loads and load resistance on tiles have been developed and incorporated into the *IBC* for loose-laid and mechanically attached tiles. Coefficients have not been determined for mortar-set or foam-set systems.

Storm damage investigations have revealed poor performance of wire-tied systems as illustrated in Figure 12-102.

**Figure 12-102**
These wire-tied tiles (installed over a concrete deck and attached with stainless steel clips at the perimeter rows) failed during Typhoon Paka (1997) in Guam.

Use of mortar to adhere tiles is problematic. For a variety of reasons, performance of mortar-set tile systems during Hurricane Andrew in South Florida was poor. Foam-set securement of tiles was developed after Hurricane Andrew. At the time this manual was being produced, foam-set systems had not been exposed to high-wind conditions; therefore it is unknown how they will perform.

**WARNING**

In high-wind areas, there is a significant potential for tile blowoff. The resulting wind-borne debris can injure people and damage adjacent buildings and vehicles.

#### Missile Loads

Tile roof coverings are very vulnerable to breakage from windborne missiles. Even when their attachment is well-designed and the tiles are properly installed, because of their brittle nature, they can easily be broken by relatively low-energy missiles. If a tile is broken, debris from a single damaged tile can impact other tiles on the roof, which can lead to a progressive cascading failure. In addition, a substantial number of high-energy missiles can be injected into the wind field.

In windstorms other than hurricanes, the probability of a roof being struck by a missile is extremely low; however, in hurricane-prone regions where the basic wind speed is equal to or greater than 110 mph (peak gust), the missile impact issue is of concern.

To minimize water damage in the event of damage to the tile covering, the underlayment recommendations for asphalt shingles are suggested; however, for tiles, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application. If a mortar-set or foam-set system is specified, a mineral surface cap sheet adhered to a nailed base sheet is suggested.

For roofs within 3,000 feet of the ocean, stainless steel fasteners are recommended.

### Seismic

Tiles are relatively heavy. Unless they are adequately attached, they can be dislodged during strong seismic events and fall away from the roof. Manufacturers have conducted laboratory research on seismic resistance of tiles; however, specific design guidance for use of these products in areas prone to large ground motion accelerations has not been developed. As shown in Figures 12-103, 12-104, and 12-105, investigations after seismic events have revealed that tiles can be dislodged if they are not adequately secured.

**Figure 12-103**
Most of the tiles on this roof were nailed to batten strips. However, in one area, several tiles were not nailed. Because of the lack of nails, the tiles were shaken off the battens. Northridge Earthquake, California.

**Figure 12-104**
These tiles were nailed to thin wood sheathing. During the earthquake, the tail of the tiles bounced and pulled out the nails. Northridge Earthquake, California.

**Figure 12-105**
Northridge Earthquake, California. The tile in the center of this photograph slipped out from underneath the hip tiles. The tile that slipped was trimmed to fit at the hip. The trimming eliminated the nail holes and no other attachment was provided. The friction fit was inadequate to resist the seismic forces.

The following attachment guidance is recommended in seismic areas where short period acceleration exceeds 0.5g.

- When tiles are only loose-laid on battens, they can be shaken off as shown in Figure 12-103. If tiles are laid on battens, supplemental mechanical attachment is recommended.
- Tiles nailed only at the head may or may not perform well. If they are attached with a smooth-shank nail into a thin plywood or oriented strand board (OSB) deck, pullout can occur (see Figure 12-104). Specifying ring-shank or screw-shank nails or screws is recommended; however, even with these types of fasteners, the tail of the tile can bounce, causing enlargement of the nail hole by repeated pounding. To overcome this problem, specify wind clips near the tail of the tile.
- Tiles that are attached by only one fastener experience eccentric loading. This problem can be overcome by specifying wind clips near the tail of the tile.
- Two-piece barrel (i.e., mission) tiles attached with straw nails can slide downslope a few inches because of deformation of the long straw nail. This problem can be overcome by specifying a wire-tied system or proprietary fasteners that are not susceptible to downslope deformation.
- When tiles are cut to fit near hips and valleys, the portion of the tile with the nail hole(s) is often cut away (see Figure 12-105). Supplemental securement is necessary to avoid displacement of these loose tiles.
- Securement of rake, hip, and ridge tiles with mortar is ineffective. If mortar is specified, it should be augmented with mechanical attachment.

- Rake trim tiles fastened just near the head of the tile often slip over the fastener head because the nail hole is enlarged by repeated pounding. Additional restraint is needed for the trim pieces. Also, the design of some rake trim pieces makes them more inherently resistant to displacement than other rake trim designs.

For roofs within 3,000 feet of the ocean, stainless steel fasteners are recommended.

**Wildfires**

Tiles are noncombustible and have a relatively large thermal mass. Therefore, tile roof systems should provide excellent wildfire-resistance. Lightweight tile products are available, but it is recommended that normal-weight tiles be specified in lieu of lightweight tiles when enhanced wildfire resistance is desired.

**Hail**

At the time this manual was being produced, no tile assembly had passed UL 2218. Tile manufacturers assert that UL 2218 is not a good test method to assess non-ductile products such as tiles. A proprietary alternative test method is available to assess non-ductile products; however, at the time this manual was being produced, it had not been recognized as a consensus test method.

### 12.7.5.5 Metal Panels and Metal Shingles

A variety of metal panel and shingle systems are available. Some of these products simulate the appearance of tiles or wood shakes.

**High Winds**

Because of the great differences in system designs, the wind performance of metal systems varies widely. For metal-panel systems, it is recommended that uplift resistance be based on the ASTM E 1592 test method; for metal-shingle systems, it is recommended that uplift resistance be based on ASTM E 330. Both of these systems should be considered as air-impermeable for the purpose of calculating uplift loads.

For panel systems, it is recommended that two rows of fasteners be placed along the eaves, hip, and ridges (see Figure 12-106). The first row should be near the edge/end of the panel (i.e., within 2–3 inches), and the next row should be approximately 3–4 inches from the first row. Hip and ridge flashings should be attached with a double row of fasteners. For exposed fastener systems, the hip and ridge flashing fasteners can also serve as the panel fasteners. For concealed clip systems, the hip and ridge fasteners are in addition to the two clips along either side of the hip and ridge. Screws rather than nails should be used to attach clips, panels, and flashings.

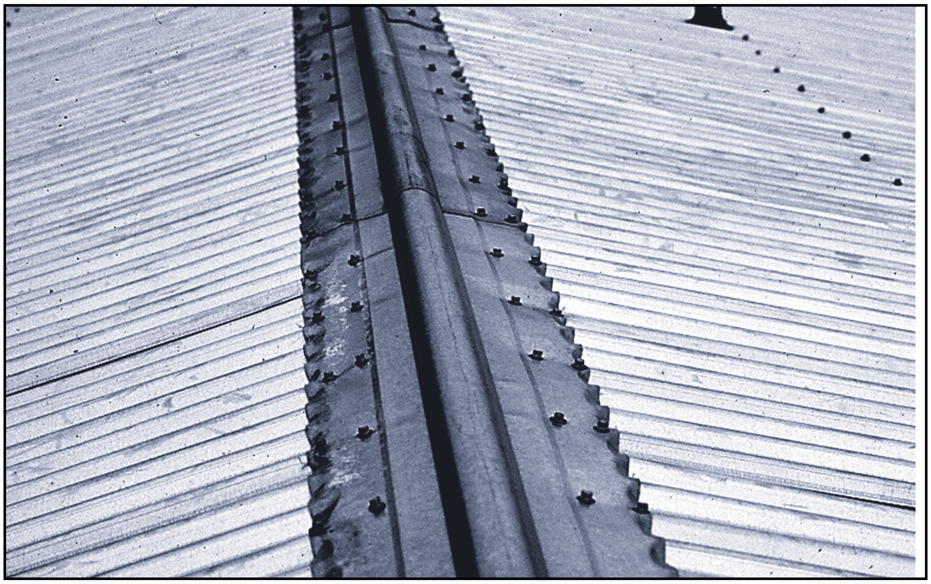


**Figure 12-106**
Hurricane Marilyn (1995), U.S. Virgin Islands. The ridge flashing on this corrugated metal panel roof was well attached, with a double row of closely spaced fasteners on each side of the ridge.

Stainless steel clips and fasteners are recommended. For enhanced corrosion protection of steel panels/shingles, an aluminum zinc alloy (Galvalume®) coating is recommended in lieu of galvanizing. (Additional design information regarding corrugated metal roofing is provided in Appendix K.)

To minimize water damage in the event of damage to the metal covering, the underlayment recommendations for asphalt shingles are suggested; however, for metal coverings, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application.

### Wildfires

Metal panel/shingle systems are noncombustible; however, they can readily transmit heat to the substrate below. If metal roofing is installed over a combustible substrate such as wood, a thermal barrier is needed between the substrate and metal to achieve enhanced resistance to wildfires. The design should be based on system testing.

### Hail

Several metal panel and shingle systems have passed UL 2218. Although metal systems have passed Class 4 (the Class with the greatest impact resistance), they often are severely dented by the testing. Although they may still be effective in inhibiting water entry, they can be aesthetically objectionable. (Note: The appearance of the system is not included in the UL 2218 evaluation criteria.)

### 12.7.5.6 Slate

Some cement-fiber and tile products are marketed as “slate”; however, slate is a natural material. Cement-fiber and tile products that simulate slate are addressed in Section 12.7.5.2.

#### High Winds

Because of limited usage of slate in areas where research has been conducted after high-wind events, few data are available on its wind performance. Manufacturers have not conducted research to determine a suitable pressure coefficient. Demonstrating slate’s compliance with ASCE 7 is problematic until such time as the coefficient is developed. A consensus test method for uplift resistance has not been developed for slate.

In extremely high-wind areas, mechanical attachment near the tail of the slate should be specified in perimeter and corner zones, and perhaps in the field. Because this prescriptive attachment suggestion is based on limited information, the uplift resistance that it provides is unknown.

To minimize water damage in the event of slate blowoff, the underlayment recommendations for asphalt shingles are recommended for slate as well; however, for slate systems, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application.

#### Seismic

Slate is relatively heavy. Unless adequately attached, it can be dislodged during strong seismic events and fall away from the roof. Manufacturers have not conducted research or developed design guidance for use of slate in areas prone to large ground motion accelerations. The guidance provided for tiles, in Section 12.7.5.4, is suggested until specific guidance is developed for slate.

#### Wildfires

Slate is noncombustible and has a relatively large thermal mass. Therefore, slate roof systems should provide excellent wildfire-resistance.

#### Hail

At the time this manual was being produced, no slate assembly had passed UL 2218.

### 12.7.5.7 Wood Shingles and Shakes

#### High Winds

Research conducted after high-wind events has shown that wood shingles/shakes can perform very well during high winds if they are not deteriorated and if they have been attached in accordance with standard attachment recommendations.

At the time this manual was produced, manufacturers had not conducted research to determine a suitable pressure coefficient. Demonstrating compliance with ASCE 7 is problematic with wood shingles/shakes until such a coefficient is developed. At this time, a consensus test method for uplift resistance has not been developed for wood shingles/shakes.

For enhanced durability, preservative-treated wood is recommended for shingle/shake roofs on coastal buildings. For those roofs located within 3,000 feet of the ocean, stainless steel nails (normally Grade 316) are recommended. See Figure 12-107 for an example of shingle loss due to corrosion of the nail fasteners.

**Figure 12-107**
Loss of wood shingles on this North Carolina house during Hurricane Bertha (1996) was due to corrosion of fasteners.

#### Wildfires

Wood shingle and shake roofs (including those with fire-resistive treatment) are not recommended in areas prone to wildfires.

#### Hail

At the time this manual was being produced, no wood-shingle assembly had passed UL 2218; however, heavy shakes have passed Class 4 (the class with the greatest impact resistance) and medium shakes have passed Class 3.

The hail resistance of wood shingles/shakes depends partly on the condition of the shingles/shakes at the time they are impacted and is likely to decline with roof age.

#### 12.7.5.8 Low-Slope Roof Systems

Roof coverings used on low-slope roofs need to be waterproof membranes, rather than water-shedding coverings as used on steep-slopes. Although most of the low-slope membranes can be used on dead-level substrates, it is always preferable (and typically required by building codes) to install them on substrates that have some slope (e.g., 1/4 inch in 12 inches [2 percent]). The most commonly used coverings on low-slope roofs are built-up, modified bitumen, and single-ply. Liquid-applied membranes (Section 12.7.5.3), structural metal panels (Section 12.7.5.5), and sprayed polyurethane foam may also be used on low-sloped roofs. Information on low-slope roof system can be found in *The NRCA Roofing and Waterproofing Manual* (NRCA 1996).

Low-slope roofing makes up a very small percentage of the residential roofing market. However, when low-slope systems are used on residences, the same principles that apply to commercial roofing also apply to residential work. The natural hazards presenting the greatest challenges to low-sloped roofs are high-winds, earthquakes, wildfires, and hail.

**High Winds**

Edge Flashings/Copings

Roof membrane blowoff is almost always a result of lifting and peeling of metal edge flashings (gravel stops) or copings, which serve to clamp down the membrane at the roof edge. For many years, continuous cleats have been recommended to restrain the vertical leg of edge flashings and the outer leg of copings. However, because light-gauge metals are typically used, the cleat and flashing/coping legs deflect outward during high winds. As a result, the flashing/coping leg often disengages from the cleat. The disengaged leg is then free to roll up and allow the wind to lift and peel the membrane.

An effective and reliable deterrent to leg deformation is to face-fasten vertical legs of flashings/copings. Figure 12-108 shows this attachment with concrete spikes. For attachment to other building materials, #14 stainless steel screws with stainless steel washers are recommended. To determine screw spacing and thickness of the flashing/coping refer to ANSI/SPRI ES-1 *Wind Design Standard for Edge Systems Used with Low Slope Roofing Systems*. This standard can also be used in the design of edge flashings/copings that are fastened with continuous cleats; however, cleat-attached systems are not as reliable as face-attached systems. The standard can also be used in selecting pre-manufactured edge flashings and copings.

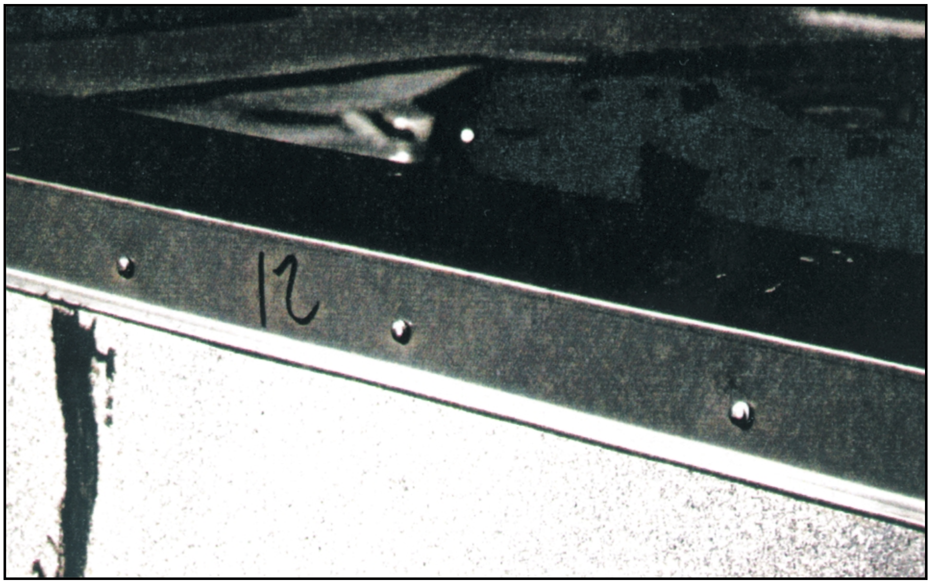



**Figure 12-108**
Typhoon Paka (1997), Guam. 24-gauge coping attached with stainless steel concrete spikes at 12 inches o.c.

### Built-Up Roofs (BUR)

BURs can offer exceptionally good high-wind performance, provided the edge flashing/coping does not lift. These membranes are also resistant to penetration by low- and moderate-energy missiles. After the uplift load is determined (see Chapter 11), a system should be selected that has demonstrated adequate capacity to meet the uplift load through testing in accordance with Factory Mutual (FM) 4470, UL 580, or UL 1897. A factor of safety (FS) should be applied to the test load results (an FS of 2 is commonly used).

To avoid the potential hazard of aggregate blowoff, it is recommended that a cap sheet or field-applied coating be specified in lieu of aggregate surfacing.

### Modified Bitumen

These systems are related and are similar to BURs. The BUR information presented above is applicable to modified bitumen systems.

### Single-Ply

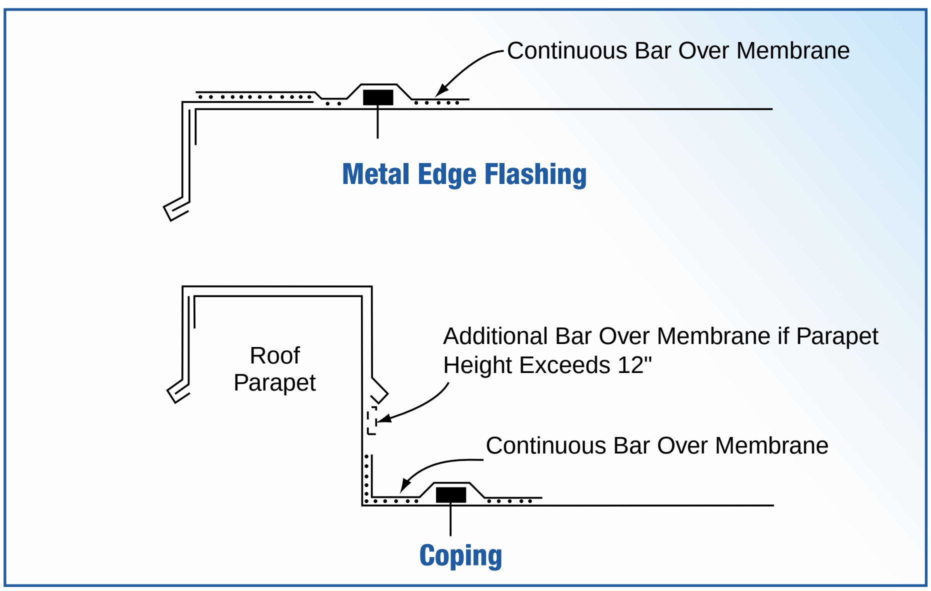
There are four primary types of single-ply membranes: CSPE (Hypalon), EPDM, PVC, and TPO. They are normally attached by ballasting, fully adhering to the substrate, or by rows of mechanical fasteners. Because single-ply membranes do not possess the stiffness or thickness of BURs or modified bitumen membranes, single-plies are more vulnerable to low-energy windborne missiles and they are more susceptible to lifting and peeling from the substrate.

To avoid the potential hazard of aggregate blowoff, it is recommended that heavyweight concrete pavers (i.e., 17 lb/ft$^2$ minimum) be specified in lieu of aggregate if a ballasted system is desired.

Because of the complex load distribution that occurs in most mechanically attached single-ply systems, it is recommended that a fully adhered membrane be specified in lieu of one that is mechanically attached. After the uplift load is determined (see Chapter 11), a fully adhered system should be selected that has demonstrated adequate capacity to meet the uplift load through testing in accordance with FM 4470, UL 580, or UL 1897. An FS should be applied to the test load results (again, an FS of 2 is commonly used).

It is recommended that a "bar" be placed over the membrane near the edge flashing/coping as illustrated in Figure 12-109. The purpose of the bar is to provide secondary protection against the membrane lifting and peeling in the event that the edge flashing/coping fails. A strong bar specifically made for "bar-over" mechanically attached systems is recommended. The bar needs to be very well attached to the parapet or deck. Depending upon wind conditions, a spacing between 6 inches o.c. and 12 inches o.c. is recommended. A gap of a few inches should be left between each bar to allow for water flow across the membrane. After the bar is attached, it is stripped over with a piece of membrane.

**Figure 12-109**
Continuous bar near the edge of flashing/coping.



Sprayed Polyurethane Foam (SPF)

SPF systems typically provide excellent wind pressure performance, provided the substrate to which they are applied does not lift. SPF roofs are typically surfaced with a coating; however, they can be surfaced with aggregate. To avoid the potential hazard of aggregate blowoff, it is recommended that a coating be specified. Note that these systems are vulnerable to puncture by large windborne missiles.

### Seismic

If a ballasted system is specified, its weight should be considered during seismic load analysis of the structure (see Chapter 11). Also, a parapet should extend above the top of the ballast to restrain the ballast from falling over the roof edge during a seismic event.

### Wildfires

Many low-slope systems are available with a Class A fire-resistance rating. However, enhanced protection can be provided by a heavyweight concrete paver ballasted roof system. If pavers are to be placed over a BUR or modified bitumen membrane, it is recommended that a layer of extruded polystyrene intended for protected membrane systems be specified over the membrane. Additionally, for smooth-surfaced BUR and modified bitumen membranes, it is recommended that a sheet of polyethylene (4-mil minimum) be specified between the membrane and polystyrene to keep the polystyrene from bonding to the membrane. To protect the base flashings, it is recommended that a mortar-faced extruded polystyrene board be installed over the base flashings as shown in Figure 12-110.

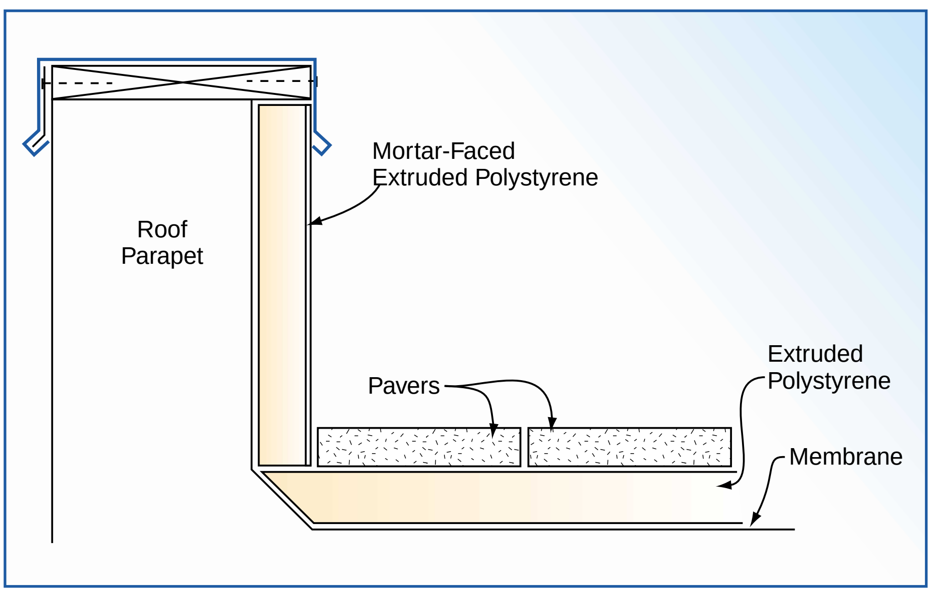



**Figure 12-110**
**Mortar-faced extruded polystyrene over base flashing for wildfire protection.**

### Hail

It is recommended that a system that has passed the Factory Mutual Research Corporation's severe hail test be specified. Enhanced hail protection can be provided by a heavyweight concrete-paver-ballasted roof system, as discussed in the section above.

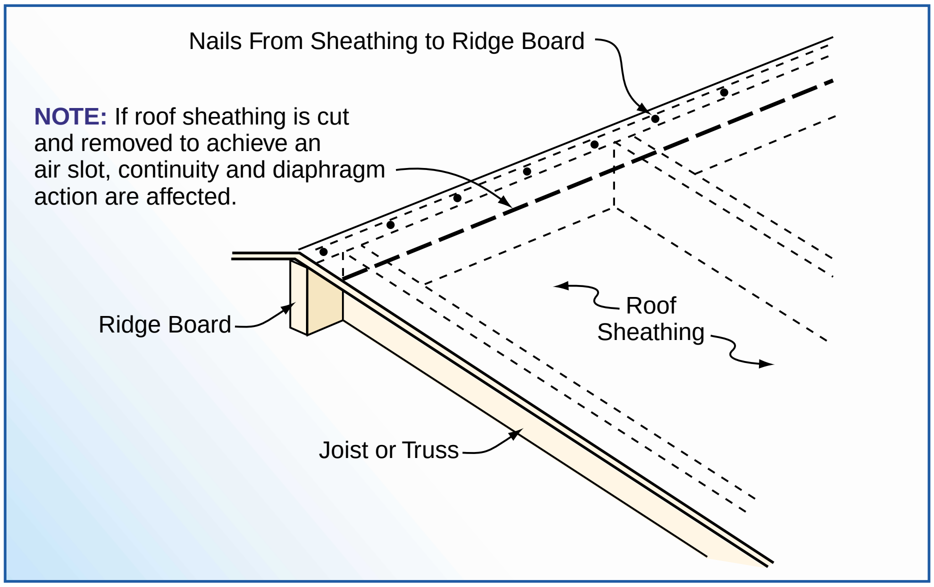
If the pavers are installed over a single-ply membrane, it is recommended that a layer of extruded polystyrene intended for protected membrane systems be specified over the membrane to provide protection in the event the pavers break. Alternatively, a stone protection mat intended for use with aggregate-ballasted systems could be specified.

## 12.7.6 Roof Ridge Vents

Continuous ridge vent installations, primarily used on roofs with asphalt shingles, have typically not addressed the issue of maintaining structural integrity of the roof sheathing. When the roof sheathing is used as a structural diaphragm as it is in high-wind and seismic hazard areas, the structural integrity of the roof can be compromised by the continuous vent.

The roof sheathing, usually plywood or oriented strand board (OSB), is intended to act as a diaphragm. The purpose of the diaphragm is to resist lateral forces. To properly function, the diaphragm must have the capability of transferring the load at its boundaries from one side of the roof to the other; it normally does this through the ridge board. The continuity, or load transfer is accomplished with nails. This approach is illustrated by Figure 12-111.

**Figure 12-111**
Method for maintaining a continuous load path at the roof ridge by nailing roof sheathing.



The problem with the continuous ridge vent application is the need to develop openings through the diaphragm to allow air to flow from the attic space up to and through the ridge vent. For existing buildings not equipped with ridge vents, this requires cutting slots or holes in the sheathing. If a saw is used to cut off 1–2 inches along either side of the ridge, the integrity of the diaphragm is affected. This method of providing roof ventilation should not be used without taking steps to ensure proper load transfer.

There are methods, however, of providing the proper ventilation while maintaining the continuity of the roof diaphragm. They include the following:

1. Drilling 2- to 3-inch-diameter holes in the sheathing between each truss or rafter approximately 1-1/2 inches down from the ridge. The holes should be equally spaced and should remove no more than one-half of the total amount of sheathing area between the rafters. For example, if the rafters are spaced 24 inches o.c. and 2-inch-diameter holes are drilled, space them at 6 inches o.c., which will allow about 12 in$^2$ of vent area per linear foot when holes are placed along either side of the ridge. This concept is illustrated in Figure 12-112.
2. Install two ridge boards separated by an air space of at least 3 inches, with solid blocking between the ridge boards at each rafter or truss. Stop the sheathing at the ridge board and fully nail the sheathing as required. The ridge vent must be wide enough to cover the 3-inch gap between the ridge boards. The ridge board and blocking must be nailed to resist the calculated shear force.

**NOTE**

**Refer to Chapter 12 of the IBC 2000 or Chapter 8 of the IRC 2000 for additional information about the required vent area for roof and attic spaces. Ventilation may be provided by any of the following methods:**

1. **mechanical**
2. **turbine vents**
3. **gable end vents**
4. **soffit vents**
5. **ridge vents**

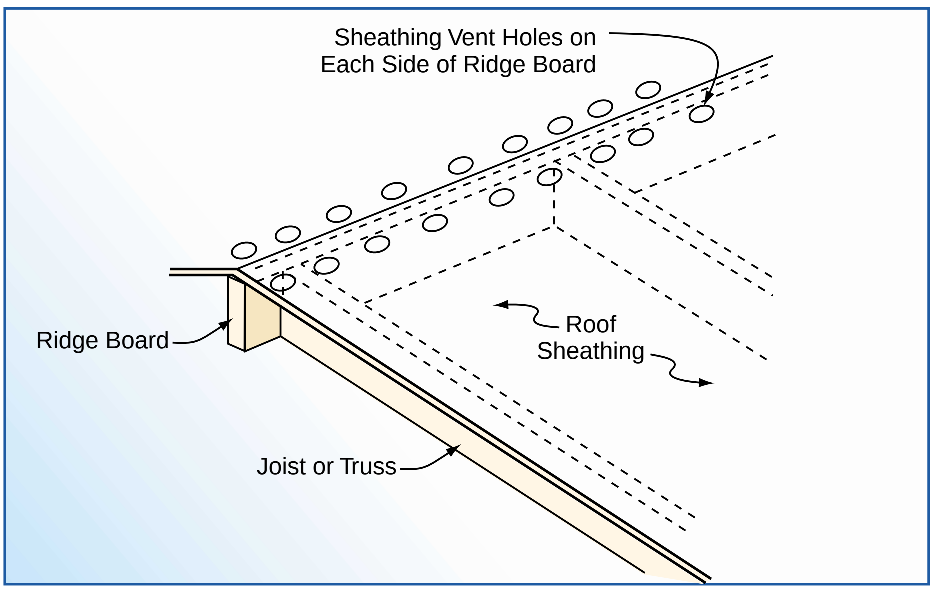



**Figure 12-112**
Holes drilled in roof sheathing for ventilation—roof diaphragm action is maintained. (For clarity, sheathing nails are not shown.)

**WARNING**

Designers and builders are cautioned not to inadvertently weaken the lateral load path by cutting back the sheathing for a ridge vent without providing for an alternative method of transferring the loads.

For new construction, the designer should detail the ridge vent installation with the proper consideration for the load transfer requirement. Where high-diaphragm loads may occur, a design professional should be consulted regarding the amount of sheathing that can be removed or other methods of providing ventilation while still transferring lateral loads. The need to meet these requirements may become a significant problem in large or complex residential buildings where numerous ventilation openings are required. In these instances, ridge vents may need to be augmented with other ventilating devices (e.g., static or gable-end vents).

Many ridge vent products are not very wide. When these products are used, it may be difficult to provide sufficiently large openings through the sheathing and still maintain diaphragm integrity if holes are drilled through the sheathing. Manufacturers' literature often illustrates large openings at the ridge with little or no consideration for the transfer of lateral loads.

**NOTE**

For additional information, see *Protecting Building Utilities From Flood Damage – Principles and Practices for the Design and Construction of Flood-Resistant Building Utility Systems*, FEMA 348 (FEMA 1999).

**DEFINITION**

DFE = Design Flood Elevation

See Chapter 11, Section 11.6.2, for additional information.

## 12.8 Utilities/Mechanical Equipment

### 12.8.1 Elevators

Elevators are being installed with increasing frequency in elevated, single-family homes in coastal areas. These elevators are used for primarily small numbers of people, but are large enough to provide handicap accessibility and accommodate small pieces of household equipment.

Small personal-size elevators are almost always designed with a shaft that is installed away from an outside wall. The elevator shaft must have a landing, usually at the very bottom level (usually ground level), and a cab platform near the top. The bottoms of elevators that have a landing at the lower level are almost always below the DFE.

FEMA's NFIP Technical Bulletin 4 (see Appendix H) discusses the installation of elevator equipment in the floodplain. As explained in the bulletin, elevator accessory equipment should be installed above the BFE to prevent damage. It is important to note that one of the safety features of elevators causes them to descend to the lowest level during power outages so that occupants will not be trapped in the cab. During flooding, this feature may expose the cab to flood waters. For elevator installation in coastal buildings, the designer must ensure the elevator stops at a level above the DFE when the power is lost. One method is to install a system of interlocking controls and float switches, as described in NFIP Technical Bulletin 4.

### 12.8.2 Design of Exterior-Mounted Mechanical Equipment

High winds, flooding, and seismic events are the natural hazards that can cause the greatest damage to exterior-mounted mechanical and electrical equipment.

#### 12.8.2.1 High Winds

Blowoff of exhaust fans, fan cowlings, and vent hoods commonly occurs during high winds. The resulting windblown debris can cause damage to other buildings, and water can enter the building that lost the equipment. Tearing away of the equipment typically occurs because of inadequate anchorage of the equipment to the roof, inadequate strength of the equipment itself (i.e., loss of fan cowlings), and corrosion.

Considering the small size of most exhaust fans, vent hoods, and air-conditioning units used on residential buildings, the following prescriptive attachment recommendations should be sufficient for most residences:

- For curb-mounted units, specify #14 screws with gasketed washers.
- For curbs with sides less than 12 inches, specify one screw at each side of the curb.
- For curbs between 12 and 24 inches, specify two screws per side.
- For curbs between 24 and 36 inches, specify three screws per side.
- For buildings within 3,000 feet of the ocean, stainless steel screws are recommended.
- For units that have flanges attached directly to the roof, attachment with #14 pan-head screws is recommended. A minimum of two screws per side, with a maximum spacing of 12 inches o.c., is recommended.

If the equipment is more than 30 inches above the curb, the attachment design should be based on calculated wind loads. ASCE 7-98 does not provide adequate guidance for determining equipment loads. Until such criteria are provided, the following approach is recommended:

- Assume a negative (i.e., uplift) load on the top of the equipment, a negative (i.e., suction) load on one side of the equipment, and a positive load on the opposite side of the equipment.
- Apply the loads to the longest side of the equipment.
- Consider the equipment as partially enclosed.
- Use component and cladding coefficients (consider the top of the equipment as the roof, and the sides as walls).
- Select coefficients for the field of the roof and field of the wall (i.e., do not use perimeter or corner coefficients).

Until equipment manufacturers produce more wind-resistant equipment, job-site strengthening of fan cowlings and vent hoods is recommended. One approach is to use 1/8-inch-diameter stainless steel cables, as shown in Figure 12-113. Two or four cables are recommended, depending on design wind conditions. Alternatively, additional, heavy straps could be screwed to the cowling and curb**.**

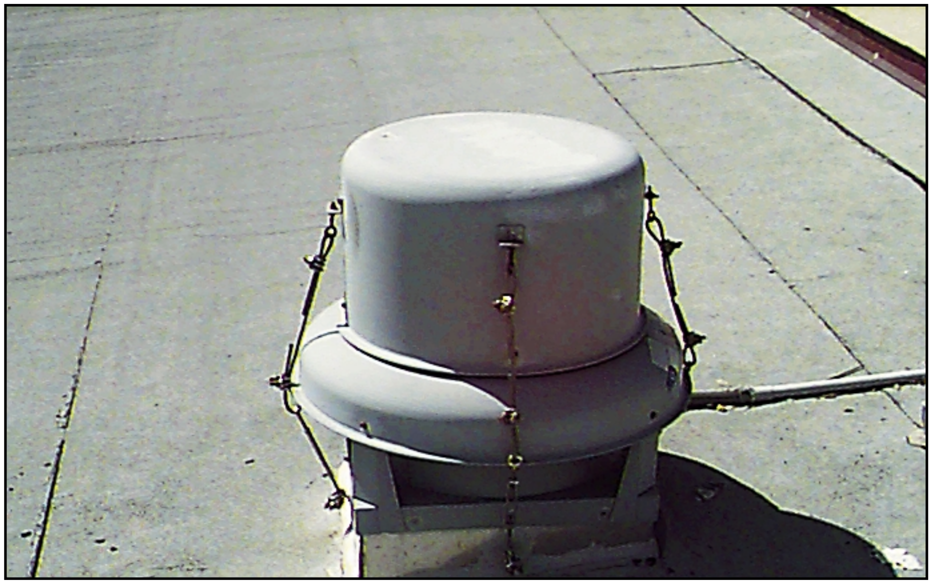


**Figure 12-113**
Typhoon Paka (1997), Guam. Stainless steel cables for strengthening fan cowlings and vent hoods.

To avoid corrosion problems, nonferrous metal, stainless steel, or steel with minimum G-90 hot-dip galvanized coating is recommended for the equipment itself, equipment stands, and equipment anchors when the equipment is on buildings located within 3,000 feet of the ocean. Stainless steel fasteners are also recommended.

### 12.8.2.2 Flooding

Flood damage to mechanical equipment is typically caused by failure to sufficiently elevate equipment as shown in Figure 12-114. Figure 12-115 shows proper elevation of an air-conditioning condenser in a floodprone area.

**Figure 12-114**
Hurricane Georges (1998), U.S. Gulf Coast. Mechanical equipment damaged as a result of insufficient elevation.

**Figure 12-115**
Proper elevation of mechanical equipment in a floodprone area.

Outdoor or exposed mechanical equipment for one-to-four family residential buildings will normally be limited to the following:

- air-conditioning condensers
- ductwork (air supply and return)
- exhaust fans
- well pumps

Flood waters can force mechanical equipment from its supports and sever its connection to mechanical or electrical systems. Mechanical equipment can also be damaged or destroyed by inundation in flood waters, especially salt water. A very short period of inundation may not destroy some types of mechanical equipment, but any inundation of electrical equipment will, at a minimum, cause significant damage to wiring and other components.

Minimizing flood damage to mechanical equipment requires elevating it above the DFE. Because of the uncertainty of wave heights and the probability of waves splashing, the designer should consider additional elevation for this equipment to help minimize damage.

In V zones, equipment must be installed either on a cantilevered platform (see Section 13.6.2) supported by the first floor framing system or on an open foundation like that used for the primary building. It is strongly recommended that any open foundation used to support mechanical equipment be of the same size, depth, and structural integrity as the main building foundation.

In A zones, mechanical equipment must be elevated to the DFE on either an open or solid foundation or otherwise protected from flood waters entering or accumulating in the system components. For houses constructed over crawlspaces, some HVAC systems are installed in such a way that the ductwork is routed through the crawlspace. This ductwork must be installed above the DFE or be made watertight in order to minimize flood damage. Many ductwork systems today are constructed with insulated board and thus would not withstand flood inundation without being destroyed.

#### 12.8.2.3 Seismic

Residential mechanical equipment units are normally not heavy. Therefore, with some care in attachment design for resistance to shear and overturning forces, these units should perform well during seismic events. Air-conditioning units that are mounted on tall elevated platforms will experience higher accelerations than ground-mounted units; therefore, extra attention should be given to attachment in areas prone to large ground motion accelerations.

### 12.8.3 Design of Interior Mechanical Equipment

High winds will normally not affect the operation of indoor equipment; however, flood waters can cause substantial damage to furnaces, boilers, water heaters, and distribution ductwork. Flood waters can extinguish a flame, short-circuit the equipment’s electrical system, and inundate equipment and ductwork with sediment.

There are only two primary methods of reducing flood damage to interior equipment:

1. Elevate the equipment and the ductwork above the DFE. This elevation may be accomplished by hanging the equipment from the existing first floor or placing the equipment in the attic or some other location above the DFE.
2. Build a waterproof structure around the equipment, allowing access for maintenance and replacement of some of the equipment parts.

**NOTE**

Although the IBC and IRC specify that flood-resistant materials be used below the BFE, this manual recommends that flood-resistant materials be used below the DFE.

### 12.8.4 Electrical, Telephone, and Cable TV Systems

Electric utilities serving residential buildings in coastal areas are frequently in a harsh and corrosive environment where increased maintenance and a shorter life can be expected. Common electrical components on residential buildings that might be exposed to severe wind or flood events consist of electric meters, electric panels, electric feeds from the utility company, receptacles, lights, security system connections, and telephone connections.

The design of an electric utility system (and any other utility service) must consider that the success of the system depends on numerous components operating successfully—all of which are at some risk of failure. For instance, if there are 10 components in the system that must operate for the system to be successful and the reliability of each component is 95 percent, the reliability of the entire system is $(0.95)^{10}$ = 60 percent. A system of this type has a high probability of failing during a natural hazard event.

The primary method of protection is to elevate all of these components above the DFE; however, it frequently is not possible to accomplish this — in fact, there are conflicts between the concepts embodied in floodplain management requirements and other building code regulations. Depending on the DFE, the two most likely conflicts that are difficult to resolve involve the placement of the electric meter and the location of a light switch at the base of entry/exit stairs required for safe egress by the building code.

Meters are normally required to be no higher than eye level for easy reading by utility company employees; however, this height is often below the expected flood level for coastal homes. Figure 12-116 shows an electric meter that is easily accessible by the utility company, but that is below the DFE. Figure 12-117 shows a bank of meters and electric feeds that failed during Hurricane Opal.

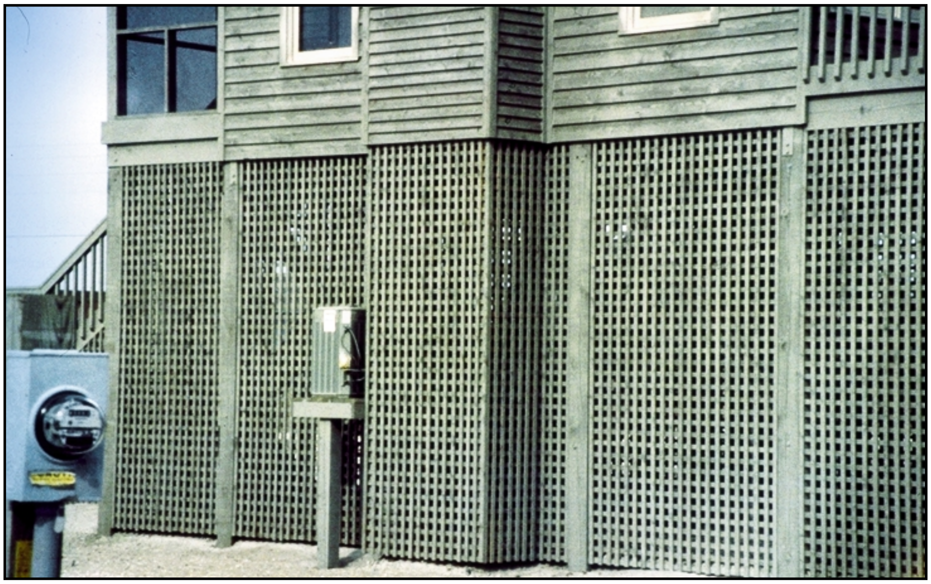


**Figure 12-116**
Elevated electric meter.

**Figure 12-117**
Electric meters and feed lines that failed during Hurricane Opal (1995).

Traditionally, the utility company is responsible for the service to a house to the point that is metered. The service to the house is provided either underground or overhead. Underground service must be designed for flood conditions and must be buried deep enough for protection from scour and erosion. The length of the service connection from grade to the meter is usually a few feet. This short connection should be designed so that the conduit or electric cable will not be contaminated by flood water and so that debris will not strike the conduit or cable and disable the power.

Wind damage to exterior-mounted electrical equipment is infrequent, in part because of the small size of most equipment (e.g., disconnect switches and conduit). Exceptions are satellite dishes and electrical service penetrations through the roof, as shown in Figure 12-118. This problem can be avoided with underground service. It can also be avoided by terminating the overhead lines at a service mast mounted on a freestanding concrete pylon, and providing underground lines between the pylon and residence. A concrete pylon is shown in Figure 12-119.

**Figure 12-118**
Hurricane Marilyn (1995), U.S. Virgin Islands. Damage caused by dropped overhead service.

**Figure 12-119**
Hurricane Georges (1998), Puerto Rico. Concrete pylon used for overhead service mast.

Other recommendations for protecting electrical system components include the following:

- Install panel feeds and conduits on the landward side of piles or other types of foundation elements to protect them from floating debris and frontal flood forces.
- Install the minimum number of switches below the DFE that will provide compliance with the electrical code. The feeds to the switch receptacles should come from above the DFE to minimize flood inundation.
- Install lights that are energized by a motion detector on the level below the DFE instead of lights operated by a switch.
- Install the feed from the utility company to the house below the ground so that damage to this connection is minimized.
- Install the electric panel above the DFE in a location that is easily accessible from the interior of the house.

Do not install electrical components behind or through breakaway walls. Components installed in this way are likely to be damaged when the breakaway wall fails. In addition, they may prevent complete failure of the wall and may transfer loads into the supporting structure of the house.

Satellite dish failures are typically caused by the designer's failure to perform wind load calculations and provide for adequate anchorage.

**NOTE**

**See Section VI-D of *Engineering Principles and Practices for Retrofitting Flood Prone Residential Buildings,* FEMA 259 (FEMA 1995), for guidance on determining the proper size of an emergency generator.**

Because a severe wind event will certainly interrupt electric service, designers and homeowners need to make decisions about the need for temporary power. If a decision is made to install a generator in a new or existing building, this manual recommends the following:

- Locate the generator above the DFE.
- Locate the generator so that engine exhaust fumes are vented to the outside.
- Locate the fuel source above the DFE, and store an amount of fuel adequate for the expected use and length of time the generator will operate.
- Install the generator where its noise and vibration will cause the least disruption.
- Size the generator for the expected load, including the starting and running load. Determine what the expected load will be (e.g., does the load include heat, refrigeration, lights, sump pumps, sewer ejector pumps). Heating and cooling require very large generators and should probably be provided only for safety and freeze protection.

- Provide an "emergency load" subpanel to supply critical circuits. Do not rely on extension cords. Supply the emergency panel from the "load side" of a manual or automatic transfer switch.
- Determine whether operation of the generator will be manual or automatic. (Manual operation is simpler and less expensive.)
- Decide whether to (1) install a quick connection at the building's electric panel so that the generator can feed the building's electric system or (2) run extension cords from the generator to the selected equipment. Protection must be provided at the panel so that generator power will not "back feed" into the supply line and endanger electric workers when they are reconnecting power to the building.

**WARNING**

**Do not "backfeed" emergency power through the service panel. Utility workers can be killed!**

## 12.8.5 Water and Wastewater Systems

These systems include the following:

- wells
- septic systems
- sanitary systems
- municipal water connections
- fire sprinkler systems

### 12.8.5.1 Wells

Protection of well systems from a severe event (primarily a flood) must consider the following, at a minimum:

- Flood waters that enter aquifers or that saturate the soil can contaminate the water supply.
- Non-submersible well pumps must be above the DFE.
- If water is to be available following disasters, a alternative power source must be provided.
- The water supply line riser must be protected from water and debris flow damage; this means the supply line must be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.
- Backflow valves must be installed to prevent flood waters from flowing into the water supply when water pressure in the supply system is lost.

**WARNING**

In some areas, high groundwater levels may preclude the installation of septic tanks below the level of expected erosion and scour.

### 12.8.5.2 Septic Systems

Protection of septic systems must consider the following, at a minimum:

- If the septic tank is dislodged from its position in the ground, the piping will come disconnected, releasing sewage into floodwaters, and the tank could damage the nearest structure. Therefore, the system must be buried below the expected depth of erosion and scour, and the tank must be anchored to prevent a buoyancy failure
- The sewage riser lines and septic tank risers must be protected from water and debris flow damage; this means risers should be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.

### 12.8.5.3 Sanitary Systems

Protection of sanitary systems must consider the following, at a minimum:

- Sanitary riser lines must be protected from water and debris flow damage; this means risers should be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.
- When the line breaks at the connection of the building line and main sewer line, a check valve in the line may help prevent raw sewage from flowing back out of the line and contaminating the soil near the building.

### 12.8.5.4 Municipal Water Connections

Protection of municipal water connections is accomplished primarily by protecting the water riser into the building from damage by debris. If water risers are severed during a coastal event, damage to the water supply system can include the following:

- waste matter from flooded sewer or septic systems intruding into the water system
- sediment filling some portion of the pipes
- breaks in the pipes at multiple locations

### 12.8.5.5 Fire Sprinkler Systems

Protection of this system is similar to the others - the primary issue is to locate the sprinkler riser such that the location provides shielding from damage. In addition, there must be consideration to the location of shutoff valves, etc., so that, if there is damage to an unprotected portion of the fire water supply line, this damage is not unnecessarily added to the damage caused by the natural hazard event.

## 12.9 Appurtenant Structures

### 12.9.1 Decks, Gazebos, and Covered Porches Attached to Buildings

Many decks and other exterior attached structures have failed during hurricanes. For decks and other structures without roofs, the primary cause of failure has been inadequate support — the pilings have either not been embedded deep enough to prevent failure or have been too small to carry the large forces from natural hazards.

The following are recommendations for the design of decks and other exterior attached structures:

- Where possible, the deck should be supported with the same type of foundation and structural system as the primary building. If this is not possible, care should be taken to ensure that the main building and attachment have similar stiffness.
- Either decks and other structures should be structurally independent of the main structure, or the additional forces they will generate should be expressly considered in the design of the main structure and the attachment to the main structure. If the attachment relies on the lateral-force-resisting system of the main structure, its anchorage must not be allowed to fail. Unless the deck is structurally independent, any attachment method that is based on the "breakaway deck" concept will create debris. Note that any construction seaward of mean high tide must be carried out in such a way that damage to the insured structure is minimized.
- If the deck surface is constructed at floor level, the deck surface/floor level joint provides a point of entry for wind-driven rain. Eliminate this problem by lowering the deck surface below the floor level.
- Cantilevering a deck from a building eliminates the need for piles altogether and should be considered when the deck dimensions can be accommodated with this structural technique. Caution must be exercised with this method to keep water out of the main house framing. Chapter 13 discusses construction techniques for flashing cantilever decks that will minimize water penetration into the house.
- Exposure to the coastal environment is severe for decks and other exterior appurtenant structures. Wood must be preservative-treated or naturally decay resistant, and fasteners must be corrosion resistant.

#### 12.9.1.1 Handrails

To minimize the effects of wind pressure, flood forces, and wave impacts, handrails for decks should be open, with slender vertical or horizontal

**WARNING**

The NFIP regulations define "appurtenant structure" as "a structure which is on the same parcel of property as the principal structure to be insured and the use of which is incidental to the use of the principal structure." In this manual, appurtenant structure means any other building or constructed element on the same property as the primary building.

**WARNING**

Decks should not cantilever over bulkheads or retaining walls where waves can run up the vertical wall and under the deck.

**NOTE**

See FEMA NFIP Technical Bulletin 5, *Free of Obstruction Requirements* (Appendix H), for additional information about the types of construction allowed in Coastal High Hazard Areas.

members spaced in accordance with the locally adopted building code. Many deck designs include solid panels (some made of plexiglass) between the top of the deck handrail and the deck. These solid panels must resist the design wind and flood loads (below the DFE); otherwise, they will become debris.

#### 12.9.1.2 Stairways

Many coastal homes have stairways leading to ground level. During flooding, flood forces often move the stairs and frequently separate them from the point of attachment. When this occurs, the stairs become debris and can cause damage to nearby houses and other buildings.

Recommendations for stairs that descend below the BFE include the following:

- To the extent permitted by code, use open-riser stairs to let flood water through the stair stringers, and anchor the stringers to a permanent foundation such as piles driven to a depth sufficient to prevent failure from scour.
- Extend the bottom of the stair carriages several feet below grade to account for possible scour. Stairs constructed in this fashion are more likely to remain in place during a coastal hazard event and therefore more likely to be usable for access after the event. In addition, by decreasing the likelihood of damage, this approach will reduce the likelihood of the stairs becoming debris.
- Construct the stairs so they are retractable. This requires a winch, cable, and hinged stair so that when a storm is expected, the winch can be used to raise the stairs out of harm's way (check local building codes for construction requirements of these systems).

**NOTE**

See Appendix I for guidance concerning the design and construction of dune walkovers.

### 12.9.2 Walkways, Sidewalks, and Other Ground-Level Structures

The walkway and sidewalk least susceptible to damage is the one that does not exist, so eliminating ground-level structures is one way a designer can reduce potential damage. In most instances, these ground-level accessories become debris during a flood event and can cause damage to neighboring buildings.

The designer should first consult state or local officials concerning regulations that govern ground-level structures. Some states have specific guidelines that must be followed for these types of structures. Ground-level structures include the following:

- wood walkways
- wood dune walkovers
- wood ground-level or near-ground-level decks

- planters
- concrete walkways
- concrete driveways

For wood structures, one of the following is recommended:

- Secure the structure with a foundation that will resist both the expected scour and the flood force that will come up from under the structure and lift it out of the soil. Figure 12-120 illustrates this concept.
- Construct the structure in such a way that it can be removed and stored prior to the storm event.

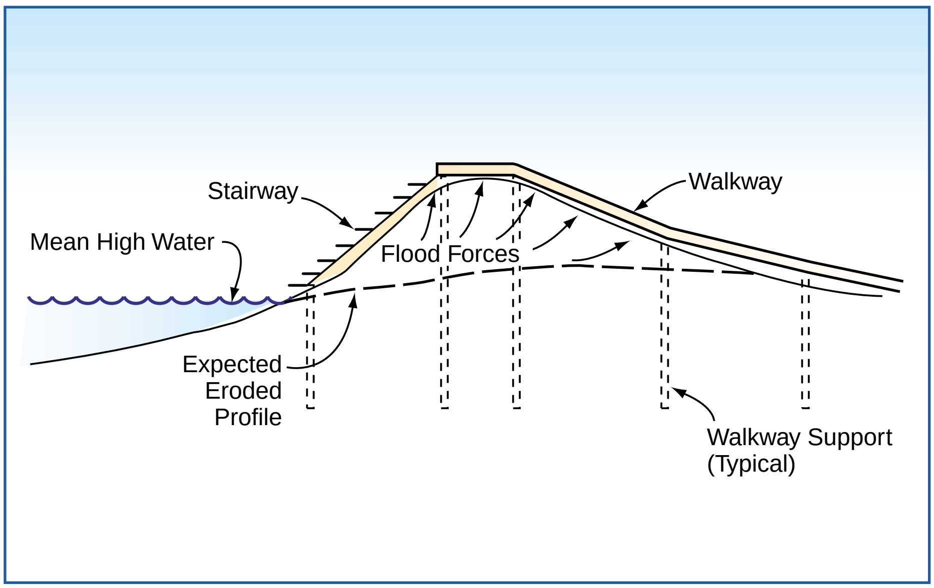


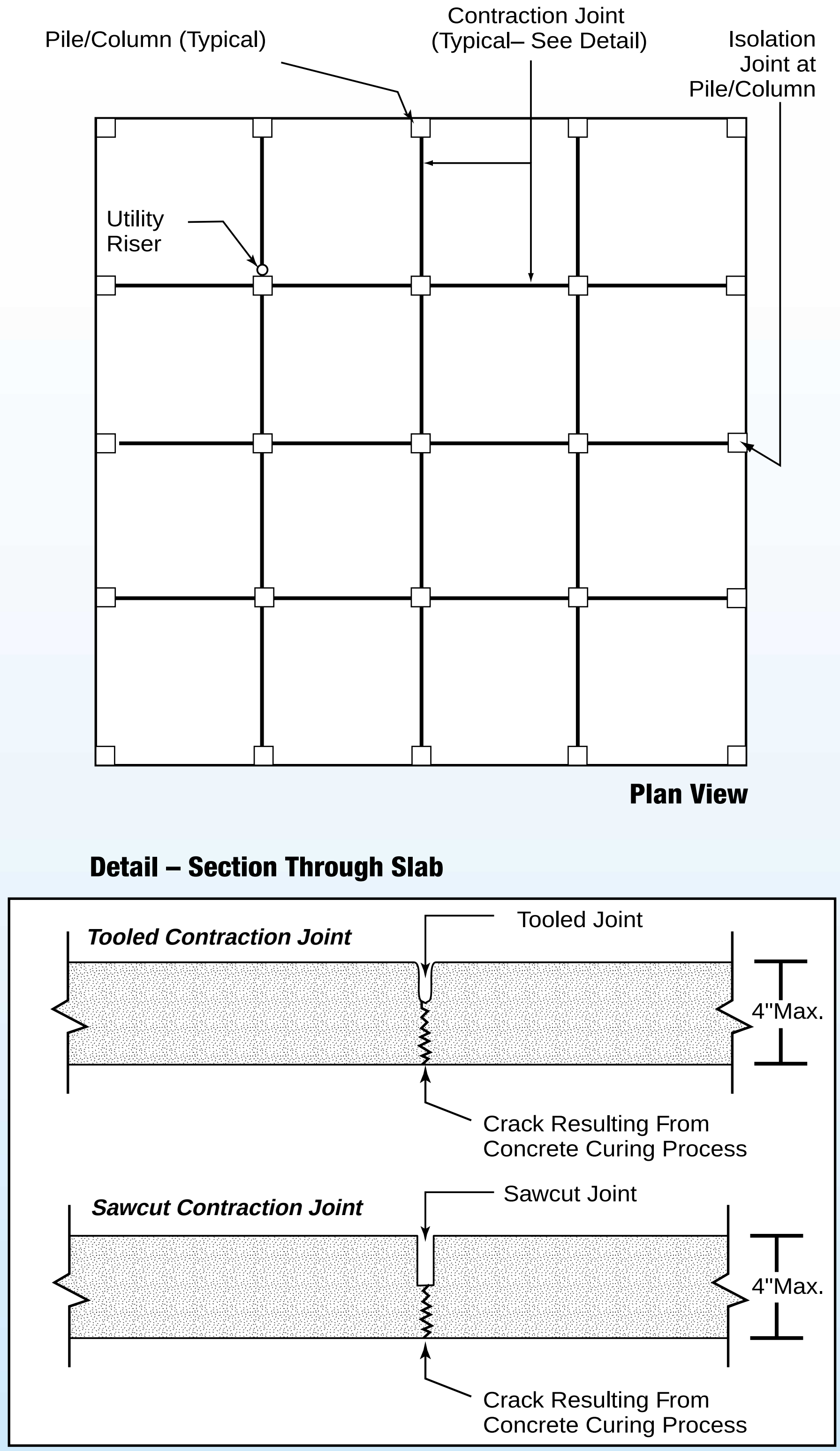

**Figure 12-120**
Wood walkover construction.

For concrete structures, the following is recommended:

- Do not install any reinforcement in walkways or driveways located below the building
- Do not secure the concrete to any structural element of the building such as a pile or column
- Install the concrete in small segments (approximately 4 feet x 4 feet) that will easily break up during a severe storm event
- Install the concrete with a maximum thickness of 3–4 inches

Figure 12-121 (FEMA 1997) illustrates these recommendations.

**Figure 12-121**
Recommended contraction joint layout for frangible slab-on-grade below elevated building.




**NOTE:** INSTALL EXPANSION AND ISOLATION JOINTS AS APPROPRIATE IN ACCORDANCE WITH STANDARD PRACTICE OR AS REQUIRED BY STATE AND LOCAL CODES.

### 12.9.3 Access to Elevated Buildings

The first floor of buildings in the Special Flood Hazard Area will be elevated a few feet to many feet above the exterior grade in order to protect the building and its contents from flood damage. Buildings in A zones may be only a few feet above grade; buildings in V zones may be 8 feet to more than 12 feet above grade. Access to these elevate buildings must be provided by one or more of the following:

- stairs
- ramps
- elevator

Stairs must be constructed in accordance with the local building code so that the run and rise of the stairs conform to the requirements. Chapter 10 of the IBC and IRC require a minimum run of 11 inches per stair tread and a maximum rise of 7 inches per tread. An 8-foot elevation difference requires 11 treads or almost 12 feet of horizontal space for the stairs. Local codes will also have requirements concerning other stair characteristics, such as stair width and handrail height.

Ramps that comply with regulations for access by persons with disabilities must have a maximum slope of 1:12 with a maximum rise of 30 inches and a maximum run of 30 feet without a level landing. The landing length must be a minimum of 60 inches. As a result, access ramps generally will not be practical for buildings elevated more than a few feet above grade, and then only when adequate space is available.

**NOTE**

For more information about elevator installation in buildings located in SFHAs, see FEMA NFIP Technical Bulletin 4, in Appendix H.

Elevators are being installed in many 1- to 4-family residential structures and provide an easy way to gain access to elevated floors of a building (including the first floor). There must an elevator entrance on the lowest floor; therefore, in flood hazard areas, some of the elevator equipment will be below the BFE. FEMA's NFIP Technical Bulletin 4 provides guidance on how to install elevators so that damage to elevator components is minimized during a flood.

**NOTE**

Check with local floodplain management officials for information about regulations governing the disturbance of primary frontal dunes. Such regulations can affect various types of coastal construction, including the installation of appurtenant structures such as swimming pools.

### 12.9.4 Pools and Hot Tubs

Many homes at or near the coast have a swimming pool or hot tub as an accessory. Some of these pools are fiberglass and are installed on a pile-supported structural frame. Others are in-ground concrete pools. The designer should consider the following when a pool is to be installed at a coastal home:

- Only an in-ground pool may be constructed beneath an elevated V-zone building. In addition, the top of the pool and the accompanying deck or walkway must be flush with the existing grade, and the area below the lowest floor must remain unenclosed.

**CROSS-REFERENCE**

Refer to Appendix L for design guidance regarding frangible pools.

**NOTE**

The construction of pools below or adjacent to buildings in coastal high hazard areas must meet the requirements presented in FEMA's NFIP Technical Bulletin 5, *Free of Obstruction Requirements for Buildings Located in Coastal High Hazard Areas* (see Appendix H). In general, pools must either be (1) elevated above the BFE on an open foundation or (2) constructed in the ground in such a way as to minimize the effects of scour and the potential for the creation of debris.

- Enclosures around pools beneath elevated buildings constitute recreational use and are therefore not allowed, even if constructed to breakaway standards. Lattice and insect screening are allowed, because they do not create an enclosure under a community's NFIP-compliant floodplain management ordinance or law.
- A pool adjacent to an elevated V-zone building may be either constructed at grade or elevated. Elevated pools must be constructed on an open foundation and the bottom of the lowest horizontal structural member must be at or above the DFE so that the pool will not act as an obstruction.
- The design professional must assure community officials that a pool beneath or adjacent to an elevated V-zone building will not be subject to breaking up or floating out of the ground during a coastal flood and will therefore not increase the potential for damage to the foundations and elevated portions of any nearby buildings. If an in-ground pool is constructed in an area that can be inundated by flood waters, the elevation of the pool must account for the potential buoyancy of the pool. If a buoyancy check is necessary, it should be made with the pool empty. In addition, the design professional must design and site the pool so that any increased wave or debris impact forces will not affect any nearby buildings.
- Pools and hot tubs have water pumps, piping, heaters, filters, and other equipment that is expensive and that can be damaged by flood waters and sediment. All such equipment should be placed above the DFE where practical.
- Equipment required for fueling the heater, such as electric meters or gas tanks, should be placed above the DFE. It may also be necessary to anchor the gas tank to prevent a buoyancy failure. If buried, the tank must not be susceptible to erosion and scour and thus failure of the anchoring system.

For concrete pools, buoyancy failure is also possible when flood waters cover the pool. In addition, flood flows can scour the soil surrounding a buried pool and tear the pool from its anchors. When this happens, the pieces of the pool become large waterborne debris.

### 12.9.5 Boat Houses

In some coastal communities that have access to waterways where boats can be used, covered boat storage areas are sometimes constructed adjacent to the house. When appurtenant structures are constructed in V zones seaward of mean high tide, these structures must be **detached from the main structure** and built to minimize damage to that insured structure.

The important design considerations for an appurtenant structure such as a boat house include the following:

- The roof of the appurtenant structure must be sufficiently anchored to a adequate foundation so that it will not lift off in a high-wind event.
- The appurtenant structure must be detached from the main building so that damage to the appurtenant structure does not result in damage to the main building.
- The structure must be adequate to carry the load of a boat being lifted from the water and stored during a high-wind event.

### 12.9.6 Storage Buildings

In order for storage buildings to survive severe events, they must be:

- elevated above the expected flood level and anchored to resist flotation, collapse, and lateral movement, or
- anchored to resist lateral movement and wet floodproofed.

In wet floodproofing, flood waters are allowed to enter a building. When this method is used, the portion of the building below the expected flood level must be constructed of flood-resistant materials so that inundation does not cause significant damage (i.e., damage requiring more than cleanup and low-cost repair, such as painting).

**NOTE**

**Check with local officials to determine whether wet floodproofing is allowed by local floodplain management ordinances or laws.**

Regardless of the method used, the building and/or its foundation must be protected from erosion and scour.

Without sufficient anchorage, storage buildings exposed to flood forces will become large debris that can cause significant damage to nearby structures. In V zones and coastal A zones, anchoring these lightweight buildings adequately is virtually impossible; therefore, it is recommended that storage buildings not be placed in V zones and coastal A zones. When a storage building is anchored to a foundation that resists uplift, overturning, and sliding, it is still possible for portions of the building to come apart in high winds that cause pressures greater than the failed parts of the building were designed to carry. This suggests that storage buildings will fail just like houses unless an adequate load path is provided. A minimal anchoring technique for storage buildings is illustrated in Figure 12-122.

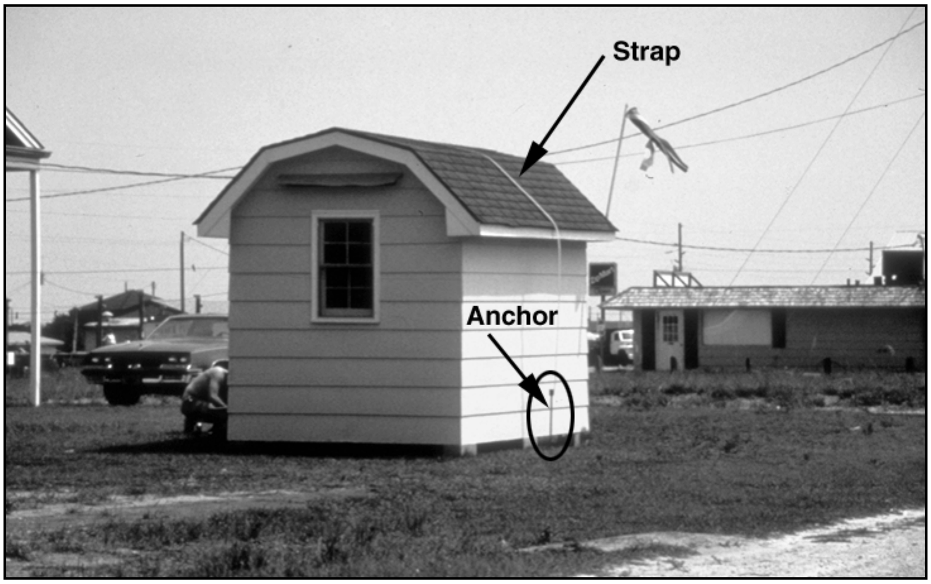



**Figure 12-122**
Storage building anchored with strap and ground anchor. This anchoring system will resist only low-level flooding and moderate winds.

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# Chapter 13: Constructing the Building

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# Constructing the Building

## 13.1 Introduction

Construction of residential buildings in coastal zones presents challenges to the builder not usually found in more inland locations. For all coastal residential buildings, these challenges may include the following:

- connection details require additional inspections
- the need for careful surveying to place the building within property line setbacks and above the Design Flood Elevation (DFE)
- the additional care required to ensure that all elements of the building will withstand the large forces associated with high wind speeds and coastal flooding
- the additional care that must be taken in constructing a building envelope that will withstand the intrusion of air and moisture under the effects of high wind speeds
- the difficulty of providing durable exterior construction in a moist, sometimes salt-laden, environment
- the requirement to protect and, usually, place utilities above the DFE

In constructing coastal residential buildings on elevated pile foundations, builders face additional challenges:

- the difficulty of constructing a driven pile foundation to accepted construction plan tolerances
- the difficulty of building on an elevated post-and-beam foundation, compared to building on continuous wall foundations

This chapter discusses construction aspects of the above challenges, as well as other aspects of the coastal construction process. Individual sections cover construction items that will probably require the most care or attention on the part of the builder in order for the design intent to be achieved.

While much of the discussion concerns constructing the building to meet the architect's and engineer's design intent for current and future conditions, it is also important that the building elements be durable. Wood decay and termite infestation, metal corrosion, and concrete and masonry deterioration can

**NOTE**

The National Flood Insurance Program (NFIP) regulations state that for buildings in V zones, "a registered professional engineer or architect shall develop or review the structural design, specifications and plans for the construction, and shall certify that the design and methods of construction to be used are in accordance with accepted standards of practice" for meeting the provisions of the NFIP regulations regarding buildings in V zones (see Chapter 6 of this manual).

**NOTE**

Sections of this chapter refer to specific requirements of the 2000 *International Building Code* (referred to as the IBC 2000), prepared by the International Code Council (ICC 2000a). The ICC also prepared the 2000 *International Residential Code for One- and Two-Family Dwellings* (ICC 2000b), referred to as the IRC 2000. Designers should refer to pertinent sections of the IRC 2000, in addition to those of the IBC 2000 cited here.

**NOTE**

If there is a conflict between design drawings and standard code practice, the most conservative should apply.

weaken the building significantly so that it is hazardous to occupy under any conditions and more likely to fail in a severe natural hazard event.

Builders may find that the permitting and inspection procedures in coastal areas are more involved than those in inland areas. Not only must all Federal, state, and local Coastal Zone Management and other regulatory requirements be met, the design plans and specifications may need to be sealed by a design professional. Building permit submittals often must include detailed drawings and information for all the elements of the wind-resisting load path, including sheathing material, sheathing nailing, strap and tiedown descriptions, bolted connections, and pile description and placement. The placement of utilities above the DFE, breakaway walls, and flood equalization openings must be clearly shown. Site inspections will likely focus on the approved plans, and building officials may be less tolerant of deviations from these approved construction documents. Several sections of this chapter identify points for possible inspections.

## 13.2 Foundation Construction

### 13.2.1 Layout

After the permit submittal and approval process is completed, the construction site must be made ready for the foundation construction. Surveying and staking must be done accurately to establish the building setback locations, the DFE, and the house plan and pile locations. Figure 13-1, a site layout illustration that shows pile locations, batter boards, and setbacks, is intended to show the possible constraints a contractor may face in actually laying out a pile-supported structure on a narrow coastal lot. There may be conflicts between what the contractor would like to do to prepare the site and what environmental controls dictate can be done at that site. Leveling of the site, especially altering dunes, and removal of existing vegetation may be restricted. These restrictions may constrict access by pile drivers and other heavy equipment.

In an elevated building with a pile foundation, the layout of the horizontal girders and beams should anticipate the fact that the final plan locations of the tops of the piles will likely not be precise. Irregularities in the piles and the soil will often prevent the piles from being driven perfectly plumb. The use of thick shims or overnotching for alignment at bolted pile-girder connections will often have a significant adverse effect on the connection capacity.

**NOTE**

Foundation design is discussed in detail in Section 12.4.3, in Chapter 12 of this manual.

Figure 13-2 shows the typical process of pile-notching. The use of a chain saw for this process can lead to inaccuracies at this early stage of construction. Figure 13-3 shows a wood pile that is overnotched; Figure 13-4 shows a pile properly notched to support the floor girder, and cut so that there is plenty of wood remaining at the top of the pile.

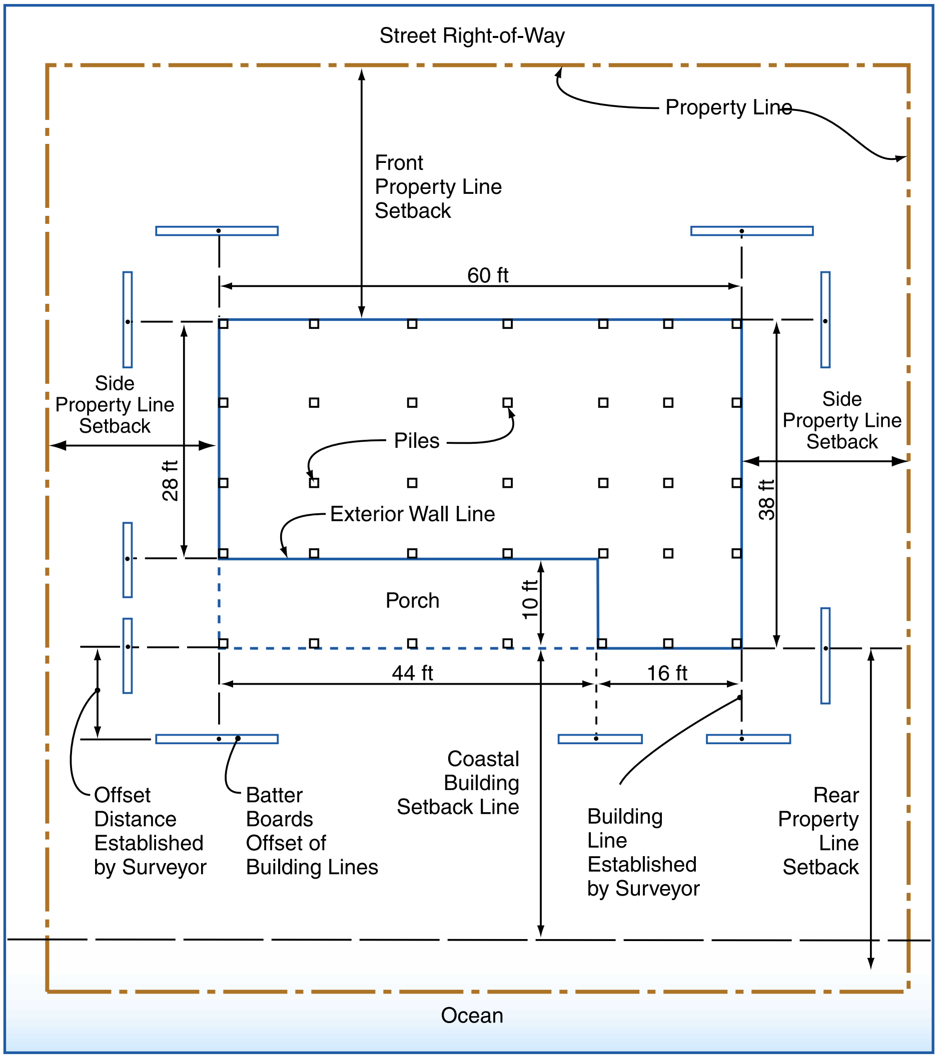



**Figure 13-1**
Site layout.

A rule of thumb regarding notching is to notch no more than 50 percent of the pile cross-sectional area. Notching more than this area will require reinforcing the pile with a steel plate (or material of similar strength). Section 13.3 presents additional information concerning the reinforcement of overnotched and misaligned piles.

The primary floor girders spanning between pile or foundation supports should preferably be oriented parallel to the primary flow of potential flood water and wave action. This orientation (normally at right angles to the shoreline) allows the lowest horizontal structural member perpendicular to flow to be the floor joists. Thus, in an extreme flood, the girders would not likely be subjected to the full force of the storm water and debris along their more exposed surfaces.

**Figure 13-2**
Typical pile notching process. Photograph by Patty McDaniel.

**Figure 13-3**
Overnotched pile. Also note mislocated bolt.

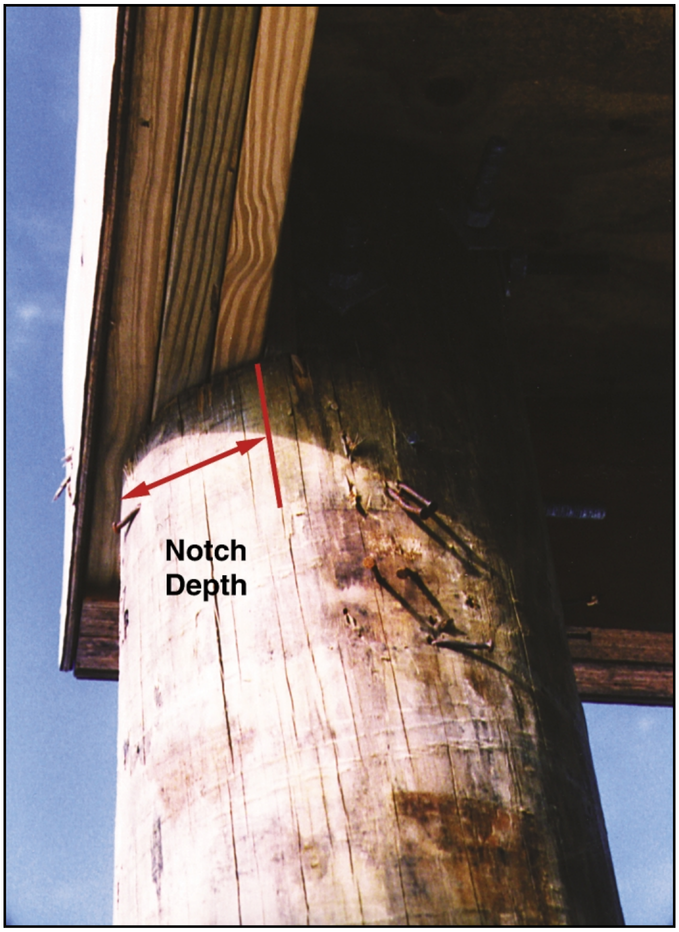
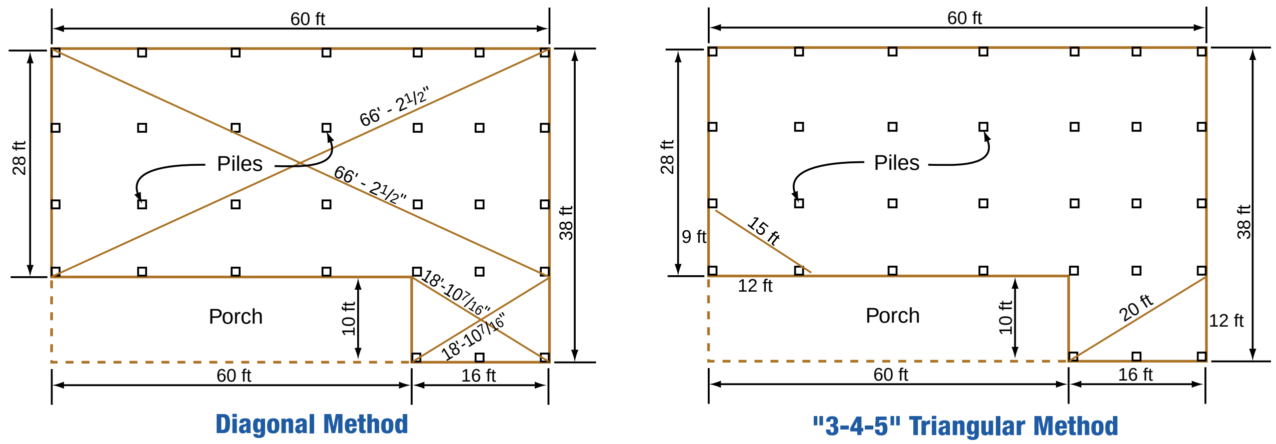



**Figure 13-4**
Properly notched pile. Note that the outer member of this three-member beam is supported by the through-bolt rather than the beam seat.

One of the most important layout steps is to “square” the first-floor framing. The entire structure is built upon the first floor; therefore, it is imperative that the first floor be level and square. The “squaring” process normally involves taking diagonal measurements across the outer corners and shifting either or both sides until the diagonal measurements are the same, at which point the house is square. An alternative is to take the measurements of a “3-4-5” triangle and shift the floor framing until the “3-4-5” triangular measurement is achieved. See Figure 13-5 for an illustration of these squaring methods.

Figure 13-5
Plan view of foundation showing techniques for squaring a building.




**WARNING**

**The amount of long-term and storm-induced erosion expected to occur at the site (see Section 7.5, in Chapter 7 of this manual) must be determined before any assumptions about, or analyses of, the soils are made. Only those soils that will remain after erosion can be relied on to support the foundation members.**

## 13.2.2 Soils

The soils on any site can vary between solid rock to loose sand. The foundation design will be based on soil assumptions derived from sources that include the following:

- soil borings
- a review of borings from nearby sites
- a test pit dug at or near one of the pilings or foundation corners
- information from the local office of the Natural Resource Conservation Service (formerly Soil Conservation Service) and Soil Surveys published for each county
- test piles

Designs are frequently prepared in which the bearing capacity of the soil is assumed and it is the builder's responsibility to verify that design assumption. In pile-supported structures, where the building support relies upon friction between the pile and soil, two important soil parameters must be known or determined:

- for cohesionless soils, the angle of internal friction
- for cohesive soils, the cohesion value in lb/ft$^2$

The United States Department of Agriculture (USDA) has developed the Unified Soil Classification system, which categorizes and describes soil types (see Table 13.1). General engineering guidelines about the properties of these soil types are listed in Table 13.2.

**Table 13.1** Soil Type Definitions Based on USDA Unified Soil Classification System

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

**Table 13.2** Engineering Properties of Soil Types Classified by USDA

| Soil Type | Symbol | Bearing Capacity ($lb/ft^2$) | Cohesion ($lb/ft^2$) | Angle of Internal Fiction° $\phi$ |
|---|---|---|---|---|
| Gravels | GW | 2700 - 3000 | | 38 - 46 |
| | GP | 2700 - 3000 | | 38 - 46 |
| | GM | 2700 - 3000 | | 38 - 46 |
| | GC | 2700 - 3000 | | 38 - 46 |
| Sands | SW – loose | 800 - 1600 | | 34 - 42 |
| | SP – loose | 800 - 1600 | | 34 - 42 |
| | SM - firm | 1600 - 3500 | | 28 - 40 |
| | SC - firm | 1600 - 3500 | | 38 - 46 |
| Fine-Grain Silt and Clays | CL - soft | 600 - 1200 | 0 - 250 | |
| | CH - soft | 600 - 1200 | 250 - 500 | |
| | CL – firm | 1500 – 2500 | 500 - 1000 | |
| | CH – firm | 1500 – 2500 | 500 - 1000 | |
| | CL – stiff | 3000 – 4500 | 1000 – 2000 | |
| | CH - stiff | 3000 - 4500 | 2000 - 4000 | |

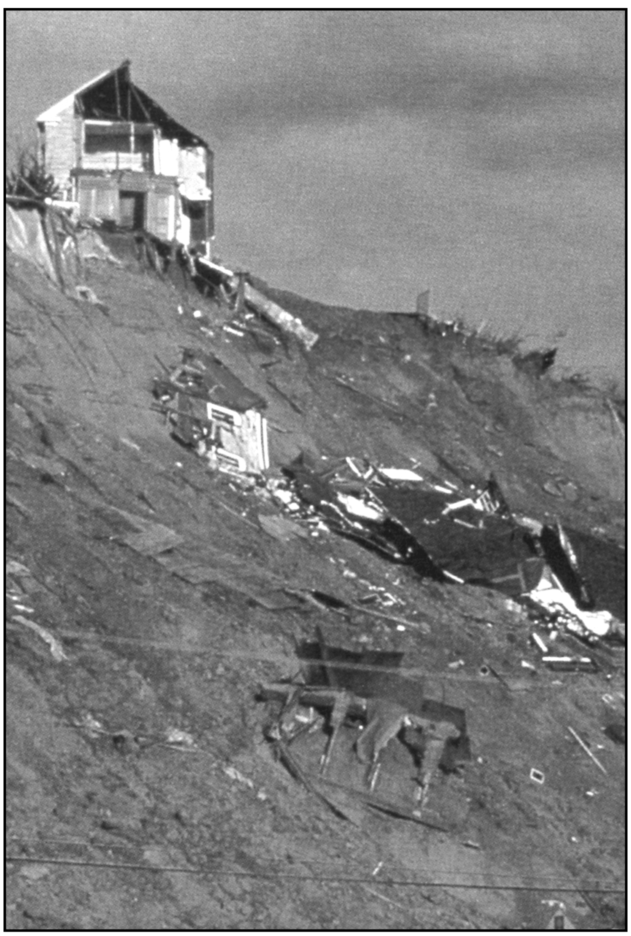
**NOTE**

Permafrost conditions (e.g., as in coastal areas of Alaska) are not addressed in this manual. "Permafrost" refers to subsoil that remains permanently frozen.

The soil bearing capacities listed in Table 13.2 are intended to provide a suggested range of values that can be used when other data are not available. However, soils can vary significantly in bearing capacity from one site to the next. **This manual recommends that a geotechnical engineer be consulted when any unusual or unknown soil condition is encountered.**

Slope stability is often difficult to predict unless there is a history of slope failures at or near the site, or unless soil borings taken at the site indicate that failures are possible. An experienced geotechnical engineer can predict from the steepness of the slope, the drainage of the site above the slope, the soil type, and the angle of internal friction of the soil whether slope failure is likely at a particular site. The International Building Code 2000, hereafter referred to as the IBC 2000 (ICC 2000a), provides some guidance on the placement of footings near slopes. Figure 13-6 shows what can happen to buildings placed near the location of a slope failure.

**Figure 13-6**
Damage caused by slope failure.



### 13.2.3 Pile Foundations

Pile foundations are the most common type of coastal building foundation in V zones and should be used in coastal A zones where flood depths exceed about 4 feet (see Figure 13-7). The most common type of pile foundation is the elevated wood pile foundation, in which the tops of the piles extend above grade to about the level of the DFE. Horizontal framing girders connected to the tops of the piles form a platform on which the house is built. This section concentrates on the construction of an elevated wood pile foundation.

**NOTE**

See Chapter 12 for a discussion of pile capacities compared to the installation method.

**Figure 13-7**
Typical wood-pile foundation.

Precautions must be taken in the handling and storage of pressure-preservative-treated round or square wood piles. They should not be dragged along the ground or dropped. They should be stored well-supported on skids so that there is air space beneath the piles and they are not in standing water. Additional direction and precautions for pile handling, storage, and construction are found in Section 13.2.6 of this manual and in Section M4-91 of the American Wood-Preservers' Association (AWPA) Standards (AWPA 1994).

A major consideration in the effectiveness of pile foundations is the method for inserting the piles into the ground, which can determine the pile load capacity. The best method is to use a pile driver, which uses leads to hold the pile in position while a single- or double-acting diesel- or air-powered hammer drives the pile into the ground.

The pile driver method, while cost-effective for a development with a number of houses being constructed at one time, can be expensive for a single building. The drop hammer method is a lower-cost alternative to the pile driver. A drop hammer consists of a heavy weight that is raised by a cable attached to a power-driven winch and then dropped onto the end of the pile.

It is common practice to estimate the ultimate capacity of a single pile on the basis of the driving resistance. Several formulas are available for making such estimates. However, the results are not always reliable and may over-predict or under-predict the capacity, so the formulas should be used with caution. One method for testing the recommended capacity developed from a formula is to load test at least one pile at each location of known soil variation.

One formula for determining pile capacity is shown as Formula 13.1, which is for drop hammer pile drivers. Formulas for other pile driver configurations are provided in *Foundation and Earth Structures Design*, U.S. Department of the Navy Manual 7.2 (USDN 1982).

formula

**Pile Driving Resistance for Drop Hammer Pile Drivers**

**Formula 13.1**
**Pile Driving Resistance for Drop Hammer Piledrivers**

$$Q_{all} = 2WH/\ (S + 1)$$

where: $\mathbf{Q_{all}}$ = allowable pile capacity in lb

**W** = weight of the striking parts of the hammer in lb

**H** = effective height of the fall in ft

**S** = average net penetration, given as in per blow for the last 6 in of driving

The advantage of driving piles, compared to using the other methods discussed below, is that the driving operation forces the soil outward around the pile, densifying the soil and causing increased friction along the sides of the pile, which provides greater pile load resistance. A disadvantage of pile driving, particularly with light equipment, is that it allows the piles "to wander" away from their intended locations. The resulting variation in the final locations of the pile tops can complicate subsequent construction of floor beams and bracing. The problem is worsened by piles with considerable warp, nonuniform soil conditions, and material buried below the surface of the ground such as logs, gravel bars, and abandoned foundations. It is prudent to inquire about subsurface conditions at the site of a proposed structure before committing to the type of pile or the installation method. A thorough investigation of site conditions can help prevent costly installation errors.

The soils investigation should include determinations of the following:

- the type of foundations that have been installed in the area in the past
- the type of soil that might be expected, from past soil borings and soil surveys
- whether the proposed site has been used for any other purpose, which would indicate whether there are any buried materials on the site
- how the site may have been used in the past, from a search of land records for past ownership

A less desirable but frequently used method of inserting piles into sandy soil is "jetting." Jetting involves forcing a high-pressure stream of water through a pipe advanced along the side of the pile. The water blows a hole in the sand into which the pile is continuously pushed or dropped until the required depth is reached. Unfortunately, jetting loosens the soil around the pile and the soil below the tip, resulting in a lower load capacity.

Holes for piles may be excavated by an auger if the soil is sufficiently clayey or silty. In addition, some sands may contain enough clay or silt to permit augering. This method can be used by itself or in conjunction with pile driving. If the hole is full-sized, the pile is dropped in and the void backfilled. Alternatively, an undersized hole can be excavated and a pile driven into it. When the soil conditions are appropriate, the hole will stay open long enough to drop or drive in a pile. In general, piles dropped or driven into augered holes may not have as much capacity as those driven without augering.

If precast concrete piles or steel piles are used, only a regular pile driver with leads and a single- or double-acting hammer should be used. For any pile driving, the building jurisdiction or the engineer-of-record will probably require that a driving log be kept for each pile. The log will show the number of inches per blow as the driving progresses—a factor used in determining the pile capacity, as shown in Formula 13.1. As noted in Section 12.4.3, the two primary determinants of pile capacity are the depth of embedment in the soil and the soil properties.

#### 13.2.3.1 Diagonal Bracing of Piles

The building design may include diagonal bracing of the piles in one or both plan directions. Figure 13-8 illustrates diagonal wood bracing. Diagonal bracing strengthens and stiffens the pile foundation at the cost of greater exposure to wave and debris impact. For most pile spacings and heights, diagonal bracing is designed as a tension-only brace. This means that the brace is too slender to be stable in resisting a compressive force. In a tension-only bolted brace connection, there must be an end distance of 7 bolt diameters in the brace (as illustrated in Figure 12-69, in Chapter 12) and a side distance of 4 bolt diameters in the pile. These clearances may be difficult to achieve if two adjacent braces end on the same side of a pile.

**Figure 13-8**
Diagonal wood bracing in a wood-pile foundation.

With tension-only braces, the design intent can be met only when all of the following conditions are met:

- The horizontal floor beams or girders just above the diagonals must serve as stiff, strong, and stable compression struts that span between the pile tops. These members allow forces to the piles that are not diagonally braced in the direction of the force to be transmitted to a pile that is braced in that direction.
- Solid connections, usually achieved with bolts, must be provided that transmit forces from the brace to the pile or floor system
- Bracing members must have sufficient strength to resist failure in tension throughout their life. This life will be shortened if the connections or the bracing members corrode, split, twist, bend, or otherwise change in such a way that their structural integrity is compromised.

The placement of the lower bolted connection of the diagonal to the pile requires some judgment. If the connection is placed too high above grade, the pile length below the connection is unbraced and the overall bracing is less strong and stiff.  If the connection is placed too close to grade, the bolt hole is more likely to be flooded or infested with termites. Because the bolt hole passes through the untreated part of the pile, flooding and subsequent decay or termite infestation will weaken the pile at a vulnerable location. The bolt hole should be treated with preservative as discussed in Section 13.2.8 of this manual and in Section M4-91 of the AWPA Standards (AWPA 1994).

### 13.2.3.2 Knee Bracing of Piles

Knee braces can be effective in supporting the pile against the lateral forces of wind and water. Figure 13-9 illustrates knee bracing. Knee bracing increases the strength and stiffness of an elevated pile foundation by restraining rotation at the top of the pile and reducing the pile bending length. Knee bracing is not as stiff or as strong as diagonal bracing. Knee braces have an advantage over diagonal braces in that they present less obstruction to waves and debris. Knee braces are shorter than diagonal braces and are usually designed for both tension and compression loads.

**Figure 13-9**
Knee bracing in a wood-pile foundation.

The entire load path into and through the knee brace must be designed with sufficient capacity. The girder or beam to which the knee brace is connected must have the bending strength to resist the axial force in the knee brace. The connections at each end of the knee brace must have sufficient capacity in both tension and compression.

#### 13.2.3.3 Grade Beams in Pile Foundations

The pile foundation design may or may not include grade beams. When used, grade beams tie the piles together, usually in both horizontal directions. Grade beams are usually made of wood or reinforced concrete. Their exposure to ground contact and possible wave forces requires highly durable construction. Wood grade beams must be of treated wood and be field treated where cuts and bores are made. For concrete grade beams, the concrete mix design, cover thickness, and curing must optimize durability. In addition, in areas subject to erosion or scour, grade beams must be designed to be self-supporting. Durability of concrete and wood construction is discussed in Section 13.2.7.

In V zones, grade beams must be used only for lateral support of the piles. If a floor is poured so that it is attached to or is monolithic with the grade beams, the bottom of the grade beam becomes the bottom of lowest horizontal structural member, which, as noted in Chapter 6, Section 6.4.3.3, must be at or above the Base Flood Elevation in order for the structure to be in compliance with the NFIP regulations.

If grade beams are used with wood piles, it is important that the connection of the grade beam to the pile does not encourage water retention. The maximum bending moment in the piles occurs at the grade beams. Decay caused by water retention at that critical point in the piles would likely induce failure under high-wind or wave forces.

#### 13.2.3.4 Wood-Pile-to-Wood-Girder Connections

Piles are often notched to provide a bearing surface for a girder. The notching should not remove more than 50 percent of the pile cross section. Section 12.5.9 describes the load calculation that must be performed at this connection to resist failure in shear. The designer can state just how much bearing is needed for sufficient bearing strength and acceptable eccentricity. The girder must be truly bearing on the surface to be effective. If the bolts are not placed low in the girder, it can shrink away from the bearing and the load will be carried in the bolts. Where the side and bottom of a girder are in contact with a notched pile, the wood of the pile at the notch is mostly untreated because the pressure preservative does not fully penetrate the wood. The surfaces of the pile exposed by notching should be treated with a field preservative.

The pile-to-girder connection may be subjected to large uplift forces in strong winds. The bolted connection must withstand these forces. The bottom bolt must be at least 4 bolt diameters from the bottom of the girder. The top bolt must be at least 7 bolt diameters from the top of the pile. The bolts should not be too widely spaced vertically across a sawn wood deep girder, as shrinkage will cause the girder to split between the bolts. A glue-laminated or parallel strand girder will be drier at installation and much less susceptible to this problem. Bolted connections should always be installed with washers on both the head and threaded ends of the bolt.

Vertical lag bolt, spike, or nail connections into the top of a wood pile should be avoided. Such connections have no building-code-allowed capacity in withdrawal and a reduced capacity in shear. In addition, the penetration invites water intrusion and subsequent decay that would further weaken the connection. For connections made with spikes driven into the side grain of a member, the spikes should be driven into drilled guide holes so that the wood will not be split.

**NOTE**

**For more information about protecting metal connectors from corrosion, refer to FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas*, in Appendix H.**

Exposed wood should be assumed to have a moisture content greater than 19 percent in coastal areas. There is evidence that fasteners embedded in treated wood or naturally durable wood with a moisture content of more than 19 percent are prone to corrosion, because of the treatment chemicals and the natural wood extractive. The IBC 2000 (ICC 2000a) requires that fasteners for pressure-preservative-treated wood be of hot-dipped galvanized steel, stainless steel, silicon bronze, or copper. The California Redwood Association recommends the use of hot-dip galvanized nails in redwood.

### 13.2.4 Masonry Foundation Construction

The combination of high winds and moist (sometimes salt-laden) air can have a damaging effect on masonry construction by forcing moisture into the smallest of cracks or openings in the masonry joints. The entry of moisture into reinforced masonry construction can lead to corrosion of the reinforcement and subsequent cracking and spalling of the masonry. Moisture resistance is highly influenced by the quality of the materials and the quality of the masonry construction at the site. Masonry material selection is discussed in Section 12.6, in Chapter 12 of this manual.

**WARNING**

**Open masonry foundations In earthquake hazard areas require special reinforcement detailing and pier proportions to meet the requirement for increased ductility.**

The quality of masonry construction depends on many considerations. Masonry units and packaged mortar and grout materials should be stored off the ground and covered. Mortar and grouts must be carefully batched and mixed. As the masonry units are placed, head and bed joints must be well mortared and tooled. Masonry work in progress must be well protected. Concave joints and V-joints provide the best moisture resistance.

Moisture penetration or retention must be carefully controlled where masonry construction adjoins other materials. As in any construction in the coastal building envelope, flashing at masonry must be continuous, durable, and of sufficient height and extent to impede the penetration of expected wind-driven precipitation. Because most residential buildings with masonry foundations have other materials (e.g., wood, concrete, steel, vinyl) attached to the foundation, allowance must be made for shrinkage of materials as they dry out

## WARNING

Figure 13-10 shows an open masonry foundation with only two rows of piers. It is unlikely that this foundation system could resist overturning caused by the forces described in Chapter 11.

and for differential movement between the materials. Expansion and contraction joints must be placed so that the materials can easily move against each other.

Masonry is used for piers, columns, and foundation walls. As explained in Chapter 6, the NFIP regulations require open foundations (e.g., piles, piers, posts, and columns) for buildings constructed in V zones. Buildings in A zones may be constructed on any foundation system. **However, because of the history of observed damage in coastal A zones, and the magnitude of the flood and wind forces that can occur in these areas, this manual recommends that only open foundation systems be constructed in coastal A zones.** Figure 13-10 shows an open masonry foundation.

**Figure 13-10**
Open masonry foundation.

Reinforced masonry has much more strength and ductility for resisting large wind, water, and earthquake forces than does unreinforced masonry. **This manual also recommends that permanent masonry construction in and near coastal flood hazard areas (both A zones and V zones) be fully or partially reinforced and grouted solid regardless of the purpose of the construction and the design loads. Grout should be in conformance with**

**the requirements of the IBC 2000 (ICC 2000a).** Knockouts should be placed at the bottom of fully grouted cells to ensure that the grout completely fills the cells from top to bottom.

For concrete masonry units, choosing Type I "moisture controlled" units and keeping them dry in transit and on the job will minimize shrinkage cracking. Usually, for optimum crack control, Type S mortar should be used for belowgrade applications and Type N mortar used for above-grade applications. The IBC 2000 specifies grout proportions by volume for masonry construction.

**NOTE**

In areas not subject to earthquake hazards, breakaway walls below elevated buildings may be of unreinforced, ungrouted masonry construction.

### 13.2.5 Concrete Foundation Construction

Concrete foundation or superstructure elements in coastal construction will almost always require steel reinforcement. Figure 13-11 shows a concrete foundation, and Figure 13-12 shows a house being constructed with concrete. Completed cast-in-place exterior concrete elements should provide 1-1/2 inch or more of concrete cover over the reinforcing bars. This thickness of cover concrete serves to protect the reinforcing bars from corrosion. An epoxy coating is often used to protect the bars from corrosion. The bars are also protected by the natural alkalinity of the concrete. However, if salt water penetrates the cover concrete and reaches the reinforcing steel, the concrete alkalinity is reduced by the salt chloride and the steel can corrode if it is not otherwise protected. As the corrosion forms, it expands and cracks the concrete, allowing the additional entry of water and further corrosion. Eventually, the corrosion of the reinforcement and the cracking of the concrete weakens the concrete structural element, making it less able to resist loads caused by natural hazards.

**NOTE**

Section 7.7 of the ACI Building Code Requirements for Structural Concrete, ACI 318-95 (ACI 1995), specifies minimum amounts of concrete cover for various construction applications.

**Figure 13-11**
Concrete foundation.

**Figure 13-12**
Concrete house.

During placement, concrete will normally require vibration to eliminate air pockets and voids in the finished surface. The vibration must be sufficient to eliminate the air, but not separate the concrete or water from the mix.

To ensure durability and long life, it is especially important in coastal, salt-water-affected locations that concrete construction be carried out carefully in a fashion that promotes durability. Appendix J, in Volume III of this manual, describes the IBC 2000 requirements for more durable concrete mixes with lower water-cement ratios and higher compressive strengths (5,000 psi) to be used in a salt water environment. The IBC 2000 also requires that additional cover thickness be provided. Proper placement, consolidation, and curing is also essential for durable concrete. The concrete mix water-cement ratio required by the IBC 2000 or by the design should not be exceeded by the addition of water at the site.

It is likely that concrete will have to be pumped at many sites because of access limitations or elevation differences between the top of the forms and the concrete mix truck chute. Pumping concrete will require some minor changes in the mix so that the concrete will flow smoothly through the pump and hoses. Plasticizers should be used to make the mix pumpable; do not use water to improve the flow of the mix. Concrete suitable for pumping generally must have a slump of at least 2 inches and a maximum aggregate size of 33-40 percent of the pump pipeline diameter. Pumping will also increase the temperature of the concrete, thus changing the curing time and characteristics of the concrete (depending on the outdoor temperature).

Freeze protection may be needed, particularly for columns and slabs, if pouring is done in cold temperatures. Concrete placed in cold weather takes longer to cure, and the uncured concrete may freeze, which will adversely affect its final strength. Methods of preventing concrete from freezing during curing include the following:

- heating adjacent soil before pouring on-grade concrete
- warming the mix ingredients before batching
- warming the concrete with heaters after pouring (avoid overheating)
- placing insulating blankets over and around the forms after pouring
- selecting a cement mix that will shorten curing time (e.g., hi-early)

Like masonry, concrete is used for piers, columns, and walls. However, the recommendation made in Section 13.2.4 of this manual regarding open foundations in coastal A zones applies to concrete foundations as well. In addition, because the environmental impact of salt-laden air and moisture make the damage potential significant for concrete, **this manual recommends that all concrete construction in and near coastal flood hazard areas (both V zones and A zones) be constructed with the more durable 5,000-psi minimum compressive strength concrete regardless of the purpose of the construction and the design loads.**

Coastal A

V

### 13.2.6 Wood Foundation Construction

All of the wood used in the foundation piles, girders, beams, and braces must be pressure-preservative-treated wood or, when allowed, naturally decay-resistant wood. Section 12.6 (in Chapter 12) and Appendix J discuss the selection of materials for these wood elements. Piles must be treated with waterborne arsenicals, creosote, or both. Girders and braces may be treated with waterborne arsenicals, pentachlorophenol, or creosote. Certain precautions apply to working with any of these treated wood products, and additional precautions apply for pentachlorophenol- and creosote-treated wood. Additional information is available from Consumer Information Sheets where the products are sold.

When working with all treated wood, avoid frequent or prolonged inhalation of the sawdust. When sawing and boring, wear goggles and a dust mask. Only treated wood that is visibly clean and free of surface residue should be used for patios, decks, and walkways. Before eating or drinking, wash all exposed skin areas thoroughly. If preservatives or sawdust accumulate on clothes, wash the clothes (separately from other household clothing) before wearing them again. Dispose of the cuttings by ordinary trash collection or burial. The cuttings should not be burned in open fires or in stoves, fireplaces, or residential boilers because toxic chemicals may be produced as part of the smoke and ashes. The cuttings may be burned only in commercial or industrial incinerators or boilers in accordance with state and Federal regulations.

Avoid frequent or prolonged skin contact with pentachlorophenol or creosote-treated wood; when handling it, wear long-sleeved shirts and long pants and use gloves impervious to the chemicals (e.g., vinyl-coated gloves).

Pentachlorophenol-pressure-treated wood should not be used in residential interiors except for laminated beams or for building components that are in ground contact and are subject to decay or insect infestation and where two coats of an appropriate sealer are applied. Sealers may be applied at the installation site. Urethane, shellac, latex epoxy enamel, and varnish are acceptable sealers.

Creosote-treated wood should not be used in residential interiors. Coal tar pitch and coal tar pitch emulsion are effective sealers for outdoor creosote-treated wood-block flooring. Urethane, epoxy, and shellac are acceptable sealers for all creosote-treated wood.

Wood foundations are being constructed in some parts of the country as part of a basement or crawlspace. These foundation elements have walls constructed with pressure-preservative-treated plywood and footings constructed with wide pressure- preservative-treated wood boards such as 2x10's or 2x12's. Because the NFIP regulations allow continuous foundation walls (with the required openings) in coastal A zones, continuous wood foundations might seem to be acceptable in these areas. However, because of the potential forces from waves less than 3 feet high (as discussed in Chapters 11 and 12), a wood foundation supported on a wood footing is not recommended in coastal A zones.

Coastal A

### 13.2.7 Foundation Material Durability

Ideally, all of the pile-and-beam foundation framing of a coastal building would be protected from rain by the overhead structure, even though all of the exposed materials should be resistant to decay and corrosion. In practice, the overhead structure includes both enclosed spaces (such as the main house) and outside decks. The spaces between the floor boards on an outside deck allow water to pass through and fall on the framing below. A worst case for potential rain and moisture penetration exists when less permeable decks collect water and channel it to fall as a stream onto framing below. In addition, wind driven rain and ocean spray will penetrate into many small spaces; protection of the wood in these spaces is important to long-term durability of the structure.

The durability of the exposed wood frame will be improved if it is detailed to shed water during wetting and to dry readily afterward. Decay will occur in those wetted locations where the moisture content of the exposed untreated interior core of treated wood elements remains above the fiber saturation point—about 30 percent. The moisture content of seasoned (S-DRY) 2x wood when it arrives at the job site can be as high as 19 percent, but this moisture content is quickly reduced as the wood dries in the finished building. (The moisture content of the large members (i.e., greater than 3x) will be much more than 19 percent when they arrive at the job site, and it will take months to drop below 19 percent.)

The potential for deterioration is greatest at end grain surfaces. Water is most easily absorbed along the grain, allowing it to penetrate deep into the member where it does not readily dry. Figure 13-13 illustrates deterioration in the end of a post installed on a concrete base. This is a typical place for wood deterioration to occur. Even when the end grain is more exposed to drying, the absorptive nature of the end grain creates an exaggerated shrink/swell cycling, resulting in checks and splits, which in turn allow increased water penetration.

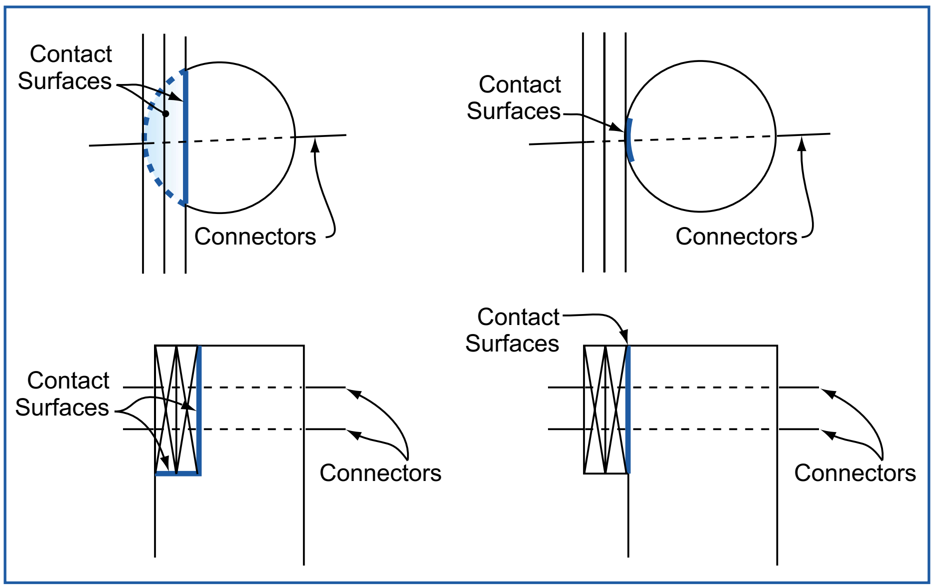
**Figure 13-13**
**Wood decay at the base of a post supported by concrete.**

Exposed pile tops present the vulnerable horizontal end grain cut to the weather. Cutting the exposed top of a pile at a slant will not prevent decay, and may even channel water into checks. Water will enter checks and splits in the top and side surfaces of beams and girders. It can then penetrate into the untreated core and cause decay. These checks and splits occur naturally in large sawn timbers as the wood dries and shrinks over time. They are less common in glue-laminated timbers and built-up sections. It is generally, but not universally, agreed that caulking the checks and splits is unwise because caulking is likely to promote water retention more than keep water out. The best deterrent is to try to keep the water from reaching the checks and splits.

Framing construction that readily collects and retains moisture, such as pile tops, pile-beam connections, and horizontal girder and beam top surfaces, can be covered with flashing or plywood. However, there should always be an air gap between the protected wood and the flashing so that water vapor passing out of the wood is not condensed at the wood surface. For example, a close-fitting cap of sheet metal on a pile top can cause water vapor coming out of the pile top to condense and cause decay. The cap can also funnel water into the end grain penetrations of the vertical fasteners.

When two flat wood surfaces are in contact in a connection, the contact surface will tend to retain any water directed to it. The wider the connection's least dimension, the longer the water will be retained, and the higher the likelihood of decay. Treated wood in this contact surface will be more resistant to decay, but only at an uncut surface. Make the least dimension of the contact surface as small as possible. When the contact surfaces are for structural bearing, provide only as much bearing surface as needed, considering both perpendicular-to-grain and parallel-to-grain bearing design stresses. For example, deck boards on 2x joists have a smaller contact surface least dimension than deck boards on 4x joists. A beam bolted alongside an unnotched round wood pile has a small least dimension of the contact surface. Figure 13-14 illustrates this least dimension concept.

**Figure 13-14**
**Minimizing the least dimension of wood contact surfaces.**




Poor durability performance has been observed in exposed "sistered" members. Where sistered members must be used in exposed conditions, they should be of ground-contact-rated treated wood and the top surface should be covered with a self-adhering modified bitumen ("peel and stick") flashing membrane. This material is available in rolls as narrow as 3 inches wide. These membranes seal around nail penetrations to keep water out. In contrast, sheet-metal flashings over sistered members, when penetrated by nails, can channel water into the space between the members.

Other methods of improving exposed structural frame durability include the following:

- Use drip cuts to avoid horizontal water movement along the bottom surface of a member. Figure 13-15 shows this type of cut.
- Avoid assemblies that form “buckets” and retain water adjacent to wood.
- Avoid designs that result in ledges below a vertical or sloped surface. Ledges collect water quite readily, and the resulting ponding due to rain or condensation alternating with solar radiation will cause shrink-swell cycling, resulting in checks, which allow increased water penetration.
- To the extent possible, minimize the number of vertical holes in exposed horizontal surfaces from nails, lags, and bolts.
- Where possible, avoid the use of stair stringers that are notched for each stair. Notching exposes the end grain, which is then covered by the stair. As a result the stair will tend to retain moisture at the notch, right where the bending stress is greatest at the minimum depth section. Figure 13-16 illustrates this stair stringer exposure. Figure 13-17 shows the type of deterioration that can result.
- An alternative stair stringer installation is shown in Figure 13-18 where the stair treads are either nailed onto a cleat, or the stringer is routed out so the tread fits into the routed-out area. Even these alternatives allow water retention at end grain surfaces; therefore these surfaces should be field-treated with wood preservative.
- Caulk joints at wood connections to keep water out. Caulk only the top joints in the connection. Recaulk after the wood has shrunk (which can take up to a year for larger members).
- When structurally possible, consider using spacers or shims to separate contact surfaces. A space of about 1/16 inch will discourage water retention by capillary action, but can easily fill with dirt and debris. A 1/4-inch–1/2-inch space is sufficient to allow water and debris to clear from the interface. This spacing has structural limitations; a bolted connection with an unsupported shim will have much less shear capacity than an unspaced connection, because of increased bolt bending and unfavorable bearing stress distribution in the wood.

**Figure 13-15**
Drip cut minimizes horizontal water movement along the bottom surface of a wood member.

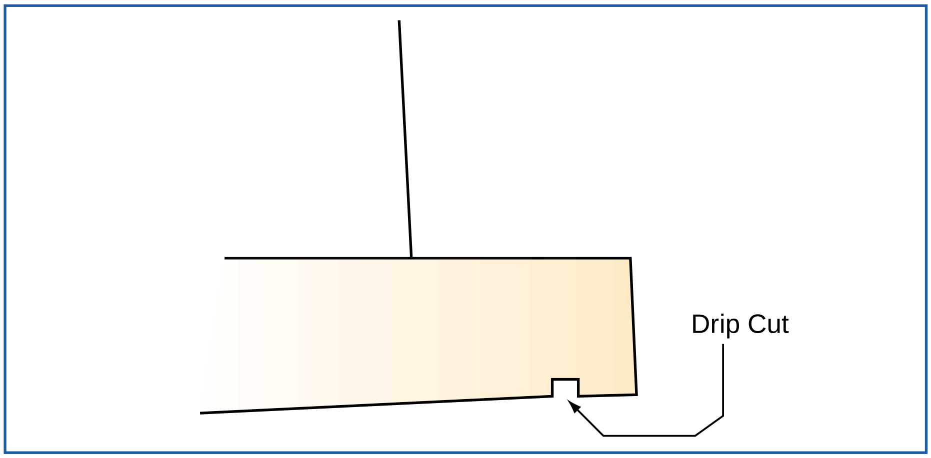



**Figure 13-16**
Exposure of end grain in stair stringer cuts.

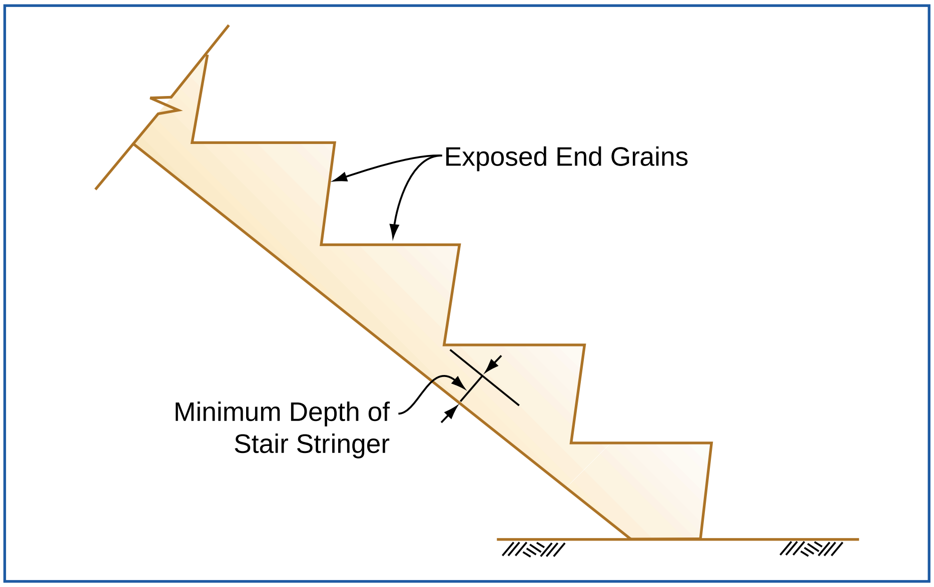



**Figure 13-17**
Deterioration in a notched stair stringer.

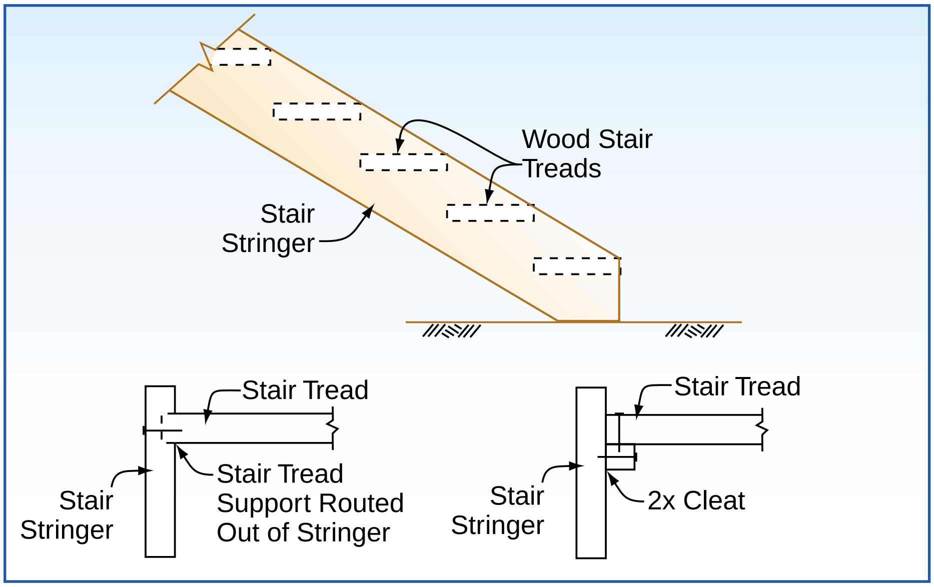



**Figure 13-18**
Alternative method of installing stair treads.

## 13.2.8 Field Preservative Treatment

Field cuts and bores of pressure-preservative-treated piles, timbers, and lumber are inevitable in coastal construction. Unfortunately, these cuts expose the inner untreated part of the wood member to possible decay and infestation. Although field preservative treatments are much less effective than pressure-preservative treatment, the decay and infestation potential can be minimized with treatment of the cuts and bolt holes with field-applied preservative. The AWPA standard *Care of Pressure-Treated Wood Products* (AWPA 1991) describes field treatment procedures and field cutting restrictions for poles, piles, and sawn lumber.

Field application of preservatives should always be done in accordance with instructions on the label. The most thorough application is by dip soaking for at least 3 minutes. When this is impractical, treatment may be done by thorough brushing or spraying. End grain is much more absorptive to field-applied preservatives than side grain. Bored holes should be poured full of *preservative.* If the hole passes through a check, then brushing of the hole will be necessary, because the preservative would otherwise run into the check instead of remaining in and saturating the hole.

The preservatives commonly used in pressure treating wood (waterborne arsenicals, pentachlorophenol, and creosote) are not acceptable for field application. Copper napthenate is the most widely used field-applied preservative. Its deep green color may be objectionable, but wood treated with it can be painted with alkyd paints in dark colors after extended drying. Zinc napthenate is less effective than copper napthenate, especially in preventing insect infestation. It is clear and therefore can be used where the color of copper napthenate is objectionable. It should not be painted with latex paints.

Tributyltin oxide (TBTO) is available, but should not be used in or near marine environments, because the leachates are toxic to aquatic organisms. Sodium borate is also available, but it does not readily penetrate dry wood, and it rapidly leaches out of wood when water is present; therefore, it is not recommended.

**NOTE**

**When substitutions are proposed, the designer's approval should be obtained before the substitution is made. The ramifications of the change must be evaluated, including the effects on the building components, constructability, and long-term durability. Code and regulatory ramifications must also be considered.**

### 13.2.9 Substitutions

During construction, a contractor may find that materials called for in the construction plans or specifications are not available or that the delivery time for those materials will be too long and will delay the completion of the building. These conflicts will require that decisions be made about substituting one type of construction material for another. Because of the high natural hazard forces imposed on a building near the coast, and the effects of the severe year-round environment in coastal areas, some substitutions must be made only after approval by a design professional and, if necessary, the local building official.

### 13.2.10 The Top Foundation Issues for Builders

1. Piles, piers, or columns must be properly aligned.
2. The piles, piers, or columns must be driven or placed at the proper elevation to resist failure and must extend below the expected depth of scour and erosion.
3. Foundation materials must be flood damage-resistant (pressure treated wood, masonry, concrete).
4. Provide adequate support at the top of the foundation element to properly attach the floor framing system. Do not notch a wood foundation element more than 50 percent of its cross-sectional area.
5. Breakaway walls are intended to fail; do not overnail these walls to the foundations; do not install utilities or other obstructions behind these walls; do not finish inside these walls.
6. Where foundation elements are masonry or concrete (except slabs-on-grade), place the proper size of reinforcing, the proper number of steel bars, and provide the proper concrete cover over the steel.
7. Exposed steel in the foundation will corrode; plan for it by installing hot-dipped galvanized or stainless steel.
8. Areas of pressure-treated wood that have been cut or drilled will retain water and will decay; treat these cut areas in the field.

### 13.2.11 Inspection Points

There are many construction details in the foundation that, if not completed properly, can cause failure during a severe natural hazard event or cause premature failure because of deterioration caused by the harsh coastal environment. Improperly constructed foundations are frequently covered up, so any deficiency in the load-carrying or distributing capacity of one member will not easily be detected until failure occurs. It is therefore very important to inspect the foundation while construction is in progress to ensure that the design is completed as intended. Table 13.3 suggests critical inspection points for the foundation.

**Table 13.3**
Foundation Inspection Points

| Inspection Point | Reason |
|---|---|
| 1. Pile-to-girder connection | Ensure that pile is not overnotched, that it is field-treated, and that bolts are properly installed with washers and proper end and edge distance |
| 2. Joist-to-girder connection | Verify presence of positive connection with properly nailed, corrosion-resistant connector |
| 3. Joist blocking | Ensure that the bottom of the joist is prevented from bending/buckling |
| 4. Sheathing nailing – number, spacing, depth | Sheathing must act as shear diaphragm |
| 5. Material storage – protection from elements prior to installation | Ensure that the wood does not absorb too much moisture prior to installation – exposure promotes checks and splits in wood, warp and separation in plywood |
| 6. Joist and beam material – excessive crown or lateral warping, large splits | Install new floors level and eliminate need to repair large splits in new material |

## 13.3 Structural Frame

One of the most critical aspects of building in a coastal area is the method of connecting the structural members. A substantial difference usually exists between connections acceptable in inland construction and those required to withstand the natural hazard forces and environmental conditions in coastal areas. Construction in non-coastal, non-seismic areas usually must support only vertical dead and live loads and modest wind loads. In most coastal areas, large forces are applied by wind, velocity flooding, wave impact, and floating debris. The calculated forces along the complete load path usually require that the builder provide considerable lateral and uplift capacity in and between the roof, walls, floors, girders, and piles. Consequently, builders should be sure to use the specified connectors or approved substitutes. Connectors that look alike may not have the same capacity, and a connector designed for gravity loads may have little uplift resistance.

**WARNING**

**It is important to note that the connections described in this manual are designed to "hold" the building together in a "design event." Builders who have not experienced such an event may underestimate the importance of installing connectors according to manufacturers' recommendations. It is extremely important that connectors be installed properly.**

The nails required for the connection hardware may not be regularly found on the job site. Full-diameter 8d to 20d short nails are commonly specified for specific hurricane/seismic connection hardware. To develop their full strength, these connections require that all of the holes in the hardware be nailed with the proper nails. In the aftermath of recent hurricanes, failed connector straps and other hardware were often found to have been attached with **too few nails**, **nails of insufficient diameter**, or **the wrong type of nail**. Figure 13-19 shows a connector that failed because of insufficient nailing.

**Figure 13-19**
Hurricane Iniki (1992), Hawaii. Connector failure caused by insufficient nailing.

**WARNING**

Proper nail selection and installation are critical. Contractors should not substitute different nails or nailing patterns without approval from the designer.

**NOTE**

Additional information about pneumatic nail guns can be obtained from the International Staple, Nail and Tool Association, 512 West Burlington Ave., Suite 203, LaGrange, IL 60525-2245. A report prepared by National Evaluation Service, Inc., titled *Power-Driven Staples and Nails for Use in All Types of Building Construction* (NES 1997), presents information about the performance of pneumatic nail guns and includes prescriptive nailing schedules.

As mentioned previously, connection hardware must be corrosion-resistant. If galvanized connectors are used, additional care must be taken during nailing. When a hammer strikes the connector and the nail during installation, some of the galvanizing protection is knocked off. One way to avoid this problem is to use corrosion-resistant connectors that do not depend on a galvanized coating, such as stainless steel or wood (see Section 12.6.6, in Chapter 12 of this manual). Stainless steel nails should only be used with stainless steel connectors. An alternative to hand-nailing is to use one of the pneumatic hammers now available that "shoot" nails into connector holes.

All connections between members in a wood-frame building are made with nails, bolts, screws, or a similar fastener. Each of these fasteners is installed by hand. The predominant method of installing nails is by pneumatic nail gun. Many nail guns use nails commonly referred to as "sinkers." Sinkers are slightly smaller in diameter and thus have lower withdrawal and shear capacities than those of the same size common nail. Nail penetration is governed by air pressure for pneumatic nailers, and this is an **important quality control issue** for builders. Many prescriptive codes have nailing schedules for various building elements such as shearwalls and diaphragms.

Toenailing should not be used to make a structural connection. Toenailing reduces the withdrawal capacity of the nails and frequently splits the wood, reducing the capacity even further.

Pile alignment and notching are critical not only to successful floor construction, but also to the structural adequacy during a natural hazard event. Construction problems related to these issues are also inevitable, so solutions to pile misalignment and overnotching must be developed. Figure 13-20 illustrates a method of reinforcing an overnotched pile, including one that is placed on a corner. The most appropriate solution to pile misalignment is to re-drive a pile in the correct location. An alternative is illustrated in Figure 13-21, which shows a method of supporting a beam at a pile that has been driven "outside the layout" of the pile foundation. Figure 13-22 illustrates proper pile notching for both two-member and four-member beams.

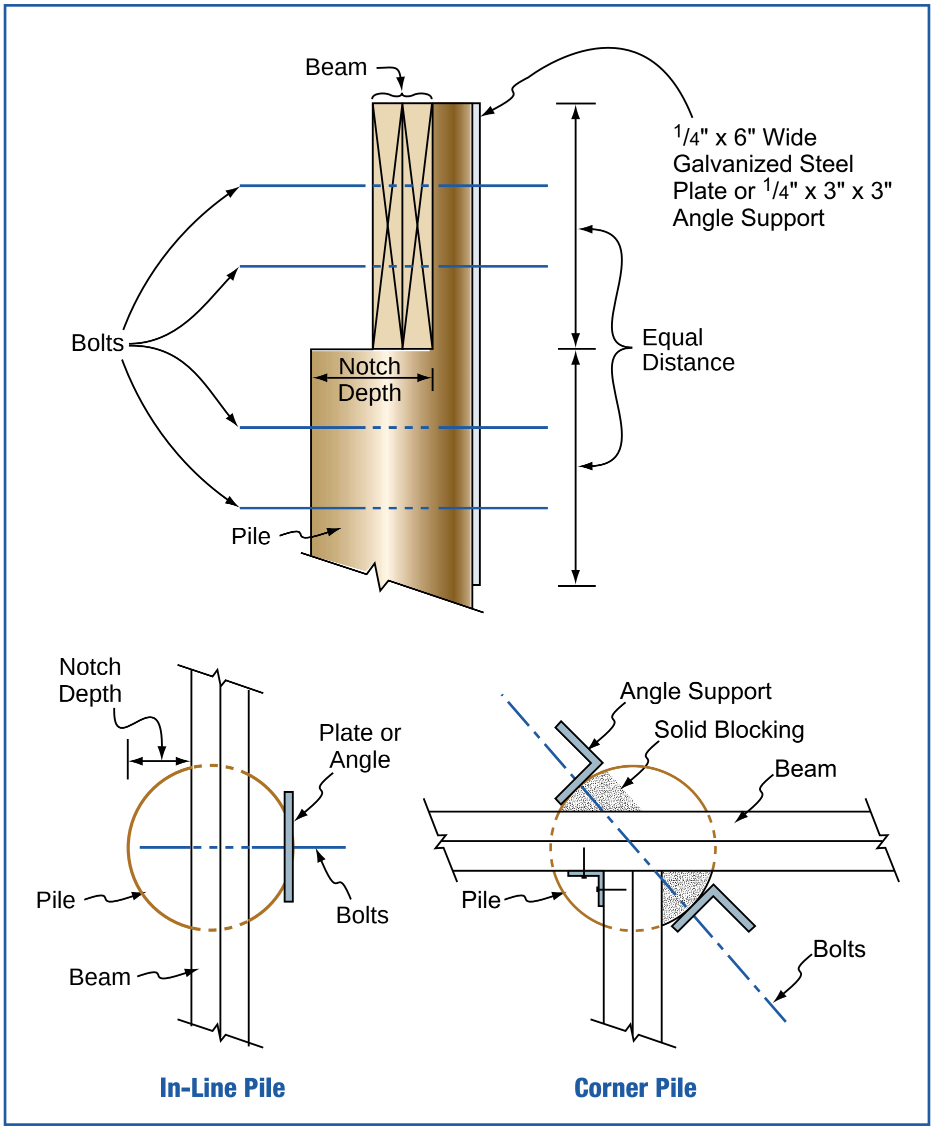



**Figure 13-20**
**Reinforcement of overnotched piles.**

**Figure 13-21**
Beam support at misaligned piles.

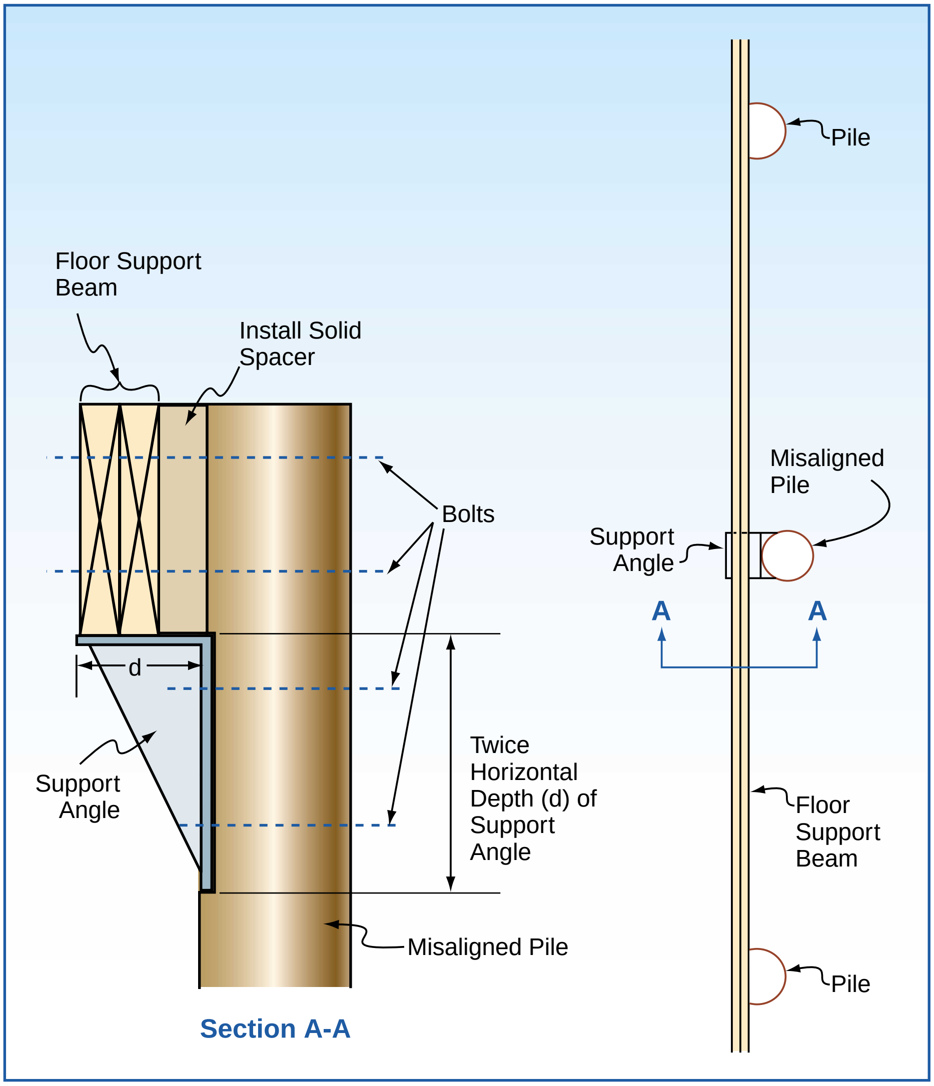
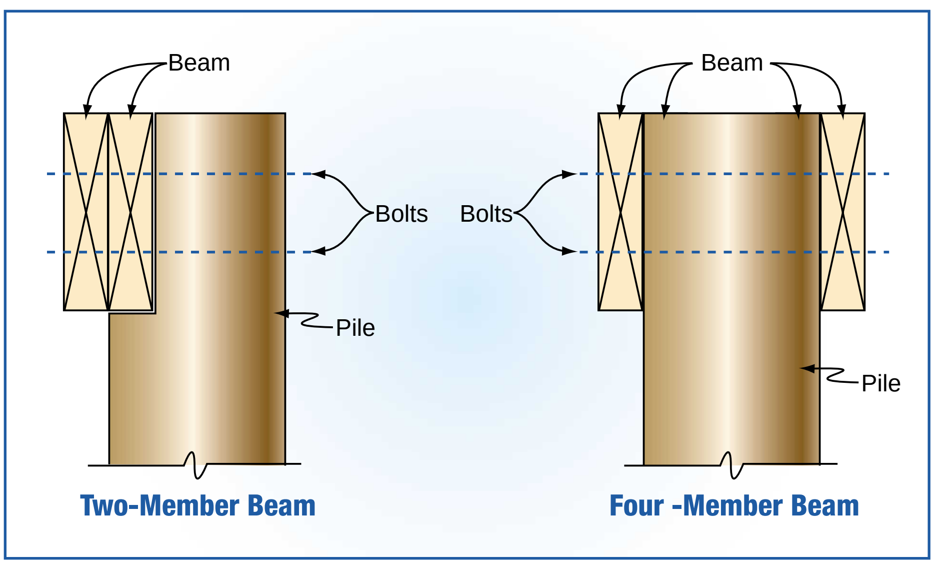



**Figure 13-22**
Proper pile notching for two-member and four-member beams.



After the "square" foundation has been built, the primary layout concerns regarding how the building will perform under loads are confined to other building elements being properly located so that load transfer paths are complete.

### 13.3.1 Floor Framing

The connection between wood floor joists and the supporting beams and girders is usually a bearing connection for gravity forces with a twist strap tie for uplift forces. Figure 13-23 shows a twist tie connection. This connection is subjected to large uplift forces from high winds. In addition, the undersides of elevated structures, where these connectors are located, are particularly vulnerable to salt spray; the exposed surfaces are not washed by rain, and they stay damp longer because of their sheltered location. Consequently, the twist straps and the nails used to secure them must be hot-dipped galvanized or stainless steel. One way to reduce the corrosion potential for metal connectors located under the building is to cover the connectors with a plywood bottom attached to the undersides of the floor joists. (The bottom half of the joist-to-girder twist straps will still be exposed, however.) This covering will help keep insulation in the floor joist space as well as protect the metal connectors.

**NOTE**

See FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas*, in Appendix H.

Because the undersides of V-zone buildings are exposed, the first floor is more vulnerable to uplift wind and wave forces, as well as to the lateral forces of moving water, wave impact, and floating debris. These loads cause compressive and lateral forces in the normally unbraced lower flange of the joist. Solid blocking or 1x3 cross-bridging at 8-foot centers is recommended for at least the first floor joists unless substantial sheathing (at least 1/2 inch thick) has been well-nailed to the bottom of these joists. Figure 13-23 also shows solid blocking between floor joists.

**Figure 13-23**
Metal twist strap ties (circled). Also, note solid blocking between floor joists.

Floor framing materials other than 2x sawn lumber are becoming popular in many parts of the country. These materials include wood floor trusses and wood I-beams. Depending on the shape of the joist and the manufacturer, the proper installation of these materials may require some additional steps. For instance, some wood I-beams require solid blocking at the end of the joist where it is supported so that the plywood web does not crush. Figure 13-24 illustrates the use of plywood web I-beams as joists. As shown in the figure, the bottom flanges of the joists are braced with a small metal strip that helps keep the flange from twisting. Solid wood blocking is a corrosion-resistant alternative to the metal braces.

**Figure 13-24**
**Plywood web I-beams used as floor joists with metal brace used to keep the bottoms of the joists from twisting. Also note glue-laminated beam.**

Floor surfaces in high-wind, flood, or seismic hazard areas are required to act as a diaphragm, as discussed in Chapter 12. For the builder, this means that the floor joists and sheathing are an important structural component. Therefore, the following installation features may require added attention:

- Joints in the sheathing should fully bear on top of a joist, not a scabbed-on board used as floor support.
- Nailing must be done in accordance with a shear diaphragm plan.
- Construction adhesive is important for preventing “squeaky” floors, but the adhesive must not be relied upon for shear resistance in the floor.

Joints in the sheathing across the joists must be fully blocked with a full-joist-height block. (Horizontal floor diaphragms with lower shear capacities can be unblocked if tongue-and-groove sheathing is used.)

#### 13.3.1.1 Horizontal Beams and Girders

As discussed in Appendix J, girders and beams can be solid sawn timbers, glue-laminated timbers (see Figure 13-24), or built-up sections. The girders span between the piles and support the beams and joists. The piles are usually notched to receive the girders. To meet the design intent, girders, beams, and joists must be square and level, girders must be secured to the piles, and beams and joists must be secured to the girders.

The layout process involves careful surveying , notching, sawing, and boring. The bottom of the notch provides the bearing surface for downward vertical loads. The bolted connection between the girder and the vertical notch surface provides capacity for uplift loads and stability. Girder splices are made as required at these connections. **Splices in multiple-member girders may be made away from the pile, but must be engineered so that the splices occur at points of zero bending moment**. This concept is illustrated in Figure 13-25.

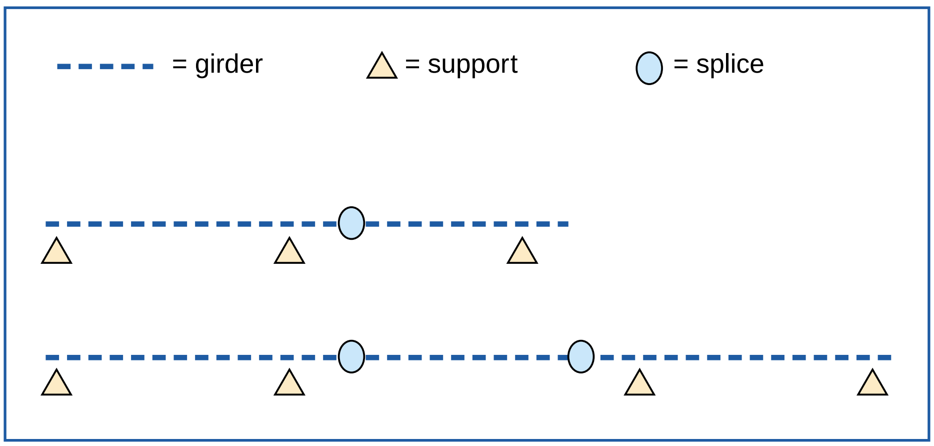



**Figure 13-25**
**Acceptable locations for splices in multiple-member girders.**

#### 13.3.1.2 Substitution of Floor Framing Materials

The considerations discussed in Section 13.2.9 for substitution of foundation materials also apply to substitutions of floor framing materials.

#### 13.3.1.3 Floor Framing Inspection Points

As a guide for floor framing inspections, Table 13.4 suggests critical inspection points.

**Table 13.4**
Floor Framing Inspection Points

| Inspection Point | Reason |
|---|---|
| 1. Pile-to-girder connection | Ensure that pile is not overnotched, that it is field-treated, and that bolts are properly installed with washers and proper end and edge distance |
| 2. Joist-to-girder connection | Verify presence of positive connection with properly nailed, corrosion-resistant connector |
| 3. Joist blocking | Ensure that the bottom of the joist is prevented from bending/buckling |
| 4. Sheathing nailing – number, spacing, depth | Sheathing must act as shear diaphragm |
| 5. Material storage – protection from elements prior to installation | Ensure that the wood does not absorb too much moisture prior to installation – exposure promotes checks and splits in wood, warp and separation in plywood |
| 6. Joist and beam material – excessive crown or lateral warping, large splits | Install new floors level and eliminate need to repair large splits in new material |

### 13.3.2 Wall Framing

The exterior walls and designated interior shear walls are an important part of the building's vertical and lateral force-resisting system. All exterior walls must withstand in-plane (i.e., parallel to the wall surface), gravity, and wind uplift tensile forces, and out-of-plane (i.e., normal or perpendicular to the wall surface) wind forces. Designated exterior and interior shear walls must withstand shear and overturning forces transferred through the walls to and from the adjacent roof and floor diaphragms and framing.

The framing of the walls must be of the specified material and must be fastened in accordance with the design drawings and standard code practice. Exterior wall and designated shear wall sheathing panels must be of the specified material and must be fastened with accurately placed nails whose size, spacing, and durability are in accordance with the design. Horizontal sheathing joints in shear walls must be solidly blocked. Shear transfer can be better accomplished if the sheathing extends the full height from the bottom of the floor joist to the top plate (see Figure 13-26), but sheathing this long is frequently not available.

The design drawings may show tiedown connections between large shearwall vertical posts and main girders. Especially in larger, taller buildings, these connections must resist thousands of pounds of overturning forces during high winds. See Section 12.4.2 for an example of the magnitude of these forces. The connections must be accomplished with careful layout, boring, and assembly. Shear transfer nailing at the top plates and sills must be in accordance with the design. Proper nailing and attachment of the framing material around openings is very important. Section 12.4.2 also highlights the difficulty of transferring large shear loads when there are large openings in the shearwall.

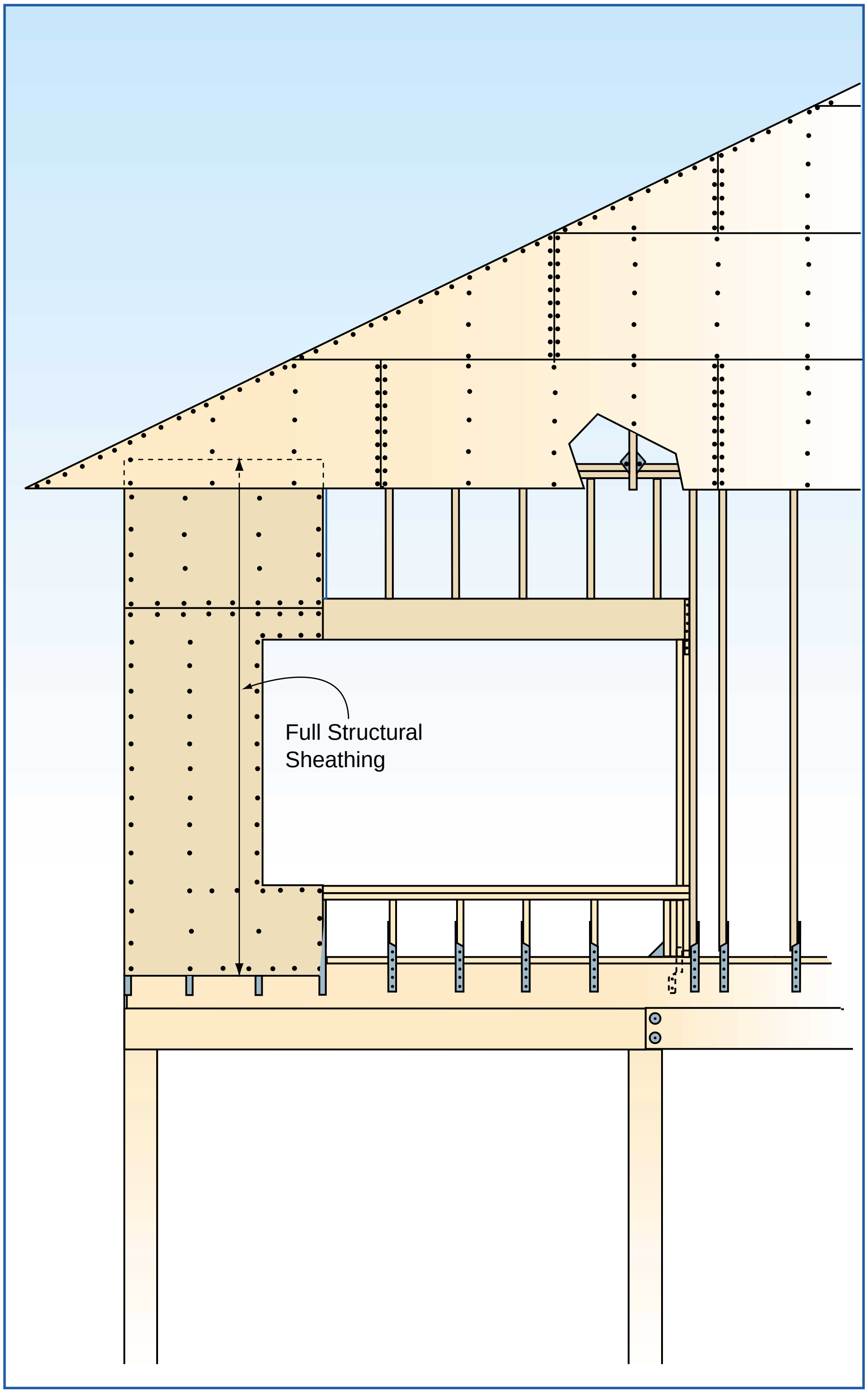



**Figure 13-26**
Using full-height sheathing improves transfer of shear.

It is very important that shearwall sheathing (e.g., plywood, OSB) with an exterior exposure be finished appropriately, with pigmented finishes such as paint (which last longer than unpigmented finishes) or semitransparent penetrating stains. It is also important that these finishes be properly maintained. Salt crystal buildup in surface checks in siding can cause damage to the siding. This damage is typically worse in siding that is sheltered from precipitation, because the salt crystals are made larger from salt spray but never washed off with fresh rainwater.

To meet the design intent, walls:

- must be plumb and square to each other and to the floor,
- must be lined up over solid support such as a beam, floor joists, or a perimeter band joist,
- must not have any more openings than designated by the plans,
- must not have openings located in places other than designated on the plans,
- must consist of material expected to resist corrosion and deterioration, and
- must be properly attached to the floors above and below the wall, including holddown brackets required to transfer overturning forces.

In addition, all portions of walls designed as shearwalls must be covered with sheathing nailed in accordance with either the plans or a specified prescriptive standard.

#### 13.3.2.1 Interior Steel Frames

In coastal buildings with large openings or cathedral ceilings, the design may include steel moment-resisting frames for wind and/or seismic lateral forces. These frames are necessary when even double-sheathed plywood walls have insufficient capacity. Figure 12-49, in Chapter 12, shows a steel moment frame installed in the wall of a coastal residential building.

The fabrication of steel moment frames will usually be done by a steel specialty subcontractor, who will first prepare a set of shop drawings from the design drawings. The contractor and designer should both check the accuracy of the shop drawings. Most frames will have to be transported in sections and assembled on site with field bolting and/or welding. The building code or designer may require special inspection or shop certification for the shop and field frame welding. If the frame is not exposed, a finish of shop primer will be adequate. Exposed parts of the frame will require hot-dip galvanizing or some other finish suitable for exterior exposure.

Alignment of the frame in the building will be critical. It is also important that the connections that transfer forces to the frame be properly accomplished so that the frame can effectively brace the structure. These steel-to-wood connections are usually made with bolts, threaded rod welded to the steel for connecting to the wood, or with powder-actuated fasteners "shot" into the steel. The ability of these powder-actuated devices to transfer the shear and tension forces must be verified (and certified) by the supplier.

#### 13.3.2.2 Substitution of Wall Framing Materials

The considerations discussed in Section 13.2.9 for substitution of foundation materials also apply to substitutions of wall framing materials.

#### 13.3.2.3 Wall Framing Inspection Points

As a guide for wall framing inspections, Table 13.5 suggests critical inspection points.

**Table 13.5**
Wall Inspection Points

| Inspection Point | Reason |
|---|---|
| 1. Wall framing attachment to floors | Ensure that nails used are of sufficient size, type, and number |
| 2. Size and location of openings | Critical to performance of shear wall |
| 3. Wall stud blocking | Ensure that there is support for edges of sheathing material |
| 4. Sheathing nailing – number, spacing, depth of nails | Sheathing must act as shear diaphragm |
| 5. Material storage – protection from elements prior to installation | Ensure that the wood does not absorb too much moisture prior to installation (Exposure promotes checks and splits in wood, warp and separation in plywood.) |
| 6. Stud material – excessive crown (crook) or lateral warping (bow) | Maintain plumb walls and eliminate eccentricities in vertical loading |
| 7. Header support over openings | Ensure that vertical and lateral loads will be transferred along the continuous load path |

### 13.3.3 Roof Framing

Proper roof construction is very important in high-wind and earthquake hazard areas. Reviews of wind damage to coastal buildings reveal that most damage starts with the failure of roof elements. The structural integrity of the roof depends on a complete load path, including the resistance to uplift of porch and roof overhangs, gable end overhangs, roof sheathing nailing, roof framing nailing and strapping, roof member-to-wall strapping, and gable end-wall bracing.

All of this construction must be done with the specified wood materials, straps, and nails. The appropriate nails must be used in all of the holes in the straps so that the straps will develop their full strength. Sheathing nails must be of the specified length, diameter, and head, and the sheathing must be nailed at the correct spacing. In addition, sheathing nails must penetrate the underlying roof framing members and **must not be overdriven**, which frequently occurs when pneumatic nail guns are used. When prefabricated roof trusses are used, handling precautions must be observed, and the trusses must be laterally braced as specified by the designer or manufacturer.

**IMPORTANT**

**The most common roof structure failure is the uplift failure of porch, eave, and gable end overhangs. The next most common is roof sheathing "peeling" away from the framing. The nailing of the sheathing at the leading edge of the roof, the gable edge, and the joints at the hip rafter or ridge are all very important, as is securing the roof framing to prevent uplift. This failure point is also the most likely place that progressive failure of the entire structure could begin.**

Field investigations indicate failure of houses with wood-framed roofs occurs first at the roof, often at improper fastening between the roof sheathing and building frame. Figure 13-27 shows an example of what happened to plywood roof sheathing during Hurricane Andrew when fasteners had not been properly embedded into the top chord of the roof truss. This potential failure mode persists in new housing; Figure 13-28 shows nails that missed the roof rafters on a house being constructed in 1998.

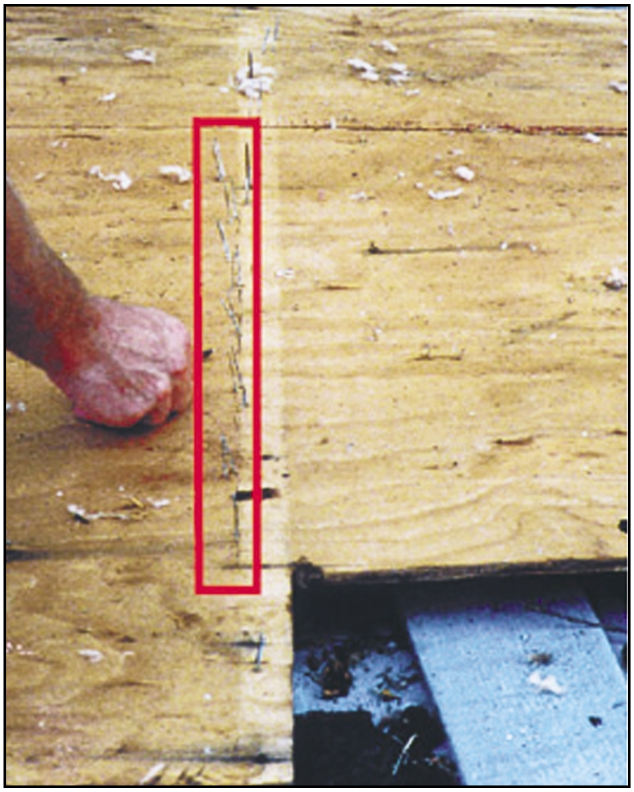


**Figure 13-27**
Hurricane Andrew (1992), Florida. Roof sheathing found in debris. Staples were off-line and therefore not connected to top chord of supporting truss. Note light area on the underside of sheathing (highlighted) where top chord of truss was in contact with sheathing.

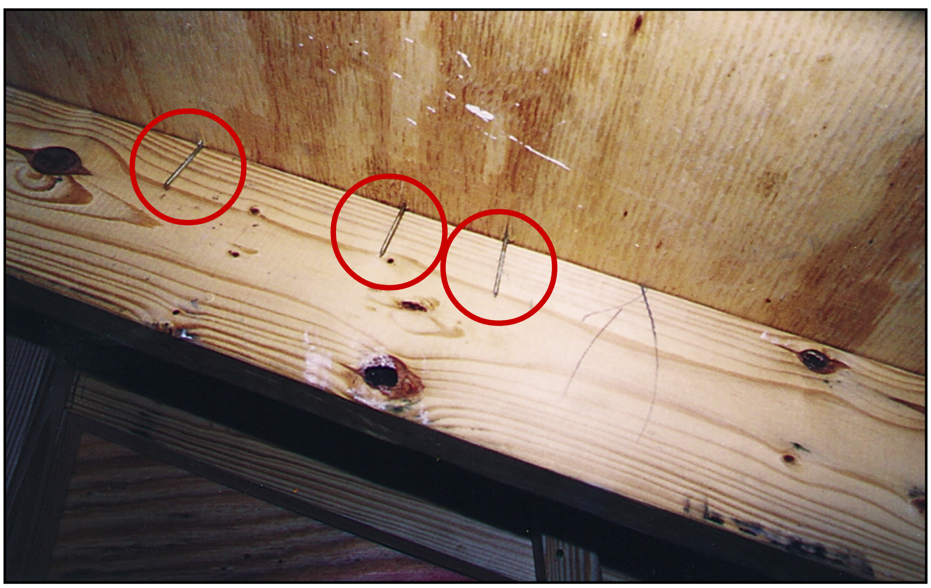


**Figure 13-28**
Sheathing nails (circled) missed roof rafter in new construction.

To meet the design intent, roofs must meet the following requirements:

- Roof trusses and rafters must be properly attached to the walls.
- Roof sheathing must be nailed according to either the construction plans or a specified prescriptive standard.
- Roofs must consist of materials expected to resist corrosion and deterioration, particularly the connectors.

#### 13.3.3.1 Substitution of Roof Framing Materials

The considerations discussed in Section 13.2.9 for substitution of foundation materials also apply to substitutions of roof framing materials.

#### 13.3.3.2 Roof Frame Inspection Points

As a guide for roof framing inspections, Table 13.6 suggests critical inspection points.

**Table 13.6**
Roof Frame Inspection Points

| Inspection Point | Reason |
|---|---|
| 1. Roof framing attachment to walls | Ensure that sufficient number, size, and type of nails is used in the proper connector |
| 2. Size and location of openings | Critical to performance of roof as a diaphragm |
| 3. "H" clips or roof frame blocking | Ensure that there is support for edges of the sheathing material |
| 4. Sheathing nailing – number, spacing, depth of nails | Sheathing must act as shear diaphragm and resist uplift |
| 5. Material storage – protection from elements prior to installation | Ensure that the wood does not absorb too much moisture prior to installation (Exposure promotes checks and splits in wood, warp and separation in plywood.) |
| 6. Rafter or ceiling joist material – excessive crown or lateral warping | Maintain level ceiling |
| 7. Gable-end bracing | Ensure that bracing conforms to design requirements and specifications |

### 13.3.4 The Top Structural Frame Issues for Builders

1. Connections between structural elements (roofs to walls, etc.) must be made so that the full natural hazard forces are transferred along a continuous load path.
2. Carefully nail components so that the nails are fully embedded.
3. Comply with manufacturers recommendations on hardware use and load ratings.
4. Use only material rated and specified for the expected use and environmental conditions.
5. The weakest connections will fail first; the concept of continuous load path must be considered for every connection in the structure. It will be important to pay particular attention to these connections.
6. Exposed steel in the structural frame will corrode, even in places such as the attic space. Plan for it by installing hot-dipped galvanized or stainless steel hardware and nails.
7. Compliance with suggested nailing schedules for roof, wall, and floor sheathing is very important.

**WARNING**

Do not substitute nails, fasteners, or connectors without approval of the designer.

## 13.4 Building Envelope

The building envelope comprises the roof covering, exterior wall covering, and exterior doors and windows. The floor is also considered a part of the envelope for buildings elevated on open foundations. The keys to successful building envelope construction include the following:

- A suitable design must be provided that is sufficiently specified and detailed to allow the contractor to understand the design intent and to give the contractor adequate and clear guidance.

  Lack of sufficient and clear design guidance regarding the building envelope is common. In this situation, the contractor should seek additional guidance from the designer or be responsible for providing design services in addition to constructing the building.

- The building must be constructed as intended by the designer (i.e., the contractor must follow the drawings and specifications).

  Examples include installing flashings, building paper, or air infiltration barriers so that water is shed at laps; using the specified type and size of fasteners and spacing them as specified; eliminating dissimilar metal contact; using materials that are compatible with one another; installing components in a manner that accommodates thermal movements so that buckling or jacking out of fasteners is avoided; applying finishes to adequately cleaned, dried, and prepared substrates; installing backer rods or bond breaker tape at sealant joints; and tooling sealant joints.

- For products/systems specified by performance criteria, the contractor must exercise care in selecting those products/systems and in integrating them into the building envelope.

  For example, if the designer specifies a window by requiring that it be capable of resisting a specified wind pressure, the contractor should ensure that the type of window that is being considered can resist the pressure when tested in accordance with the specified test (or a suitable test if a test method is not specified). Furthermore, the contractor needs to ensure that the manufacturer, designer, or other qualified entity provides guidance on how to attach the window frame to the wall so that the frame will resist the design pressures.

- When the selection of accessory items is left to the discretion of the contractor, without prescriptive or performance guidance, the contractor must be aware of and consider special conditions at the site (e.g., termites, unusually severe corrosion, high earthquake or wind loads) that should influence the selection of the accessory items.

  For example, instead of using screws in plastic sleeves to anchor components to a concrete or masonry wall, a contractor can use metal expansion sleeves or steel spikes intended for anchoring to concrete, which should provide a stronger and more reliable connection. Or, the use of plastic shims at metal doors may be appropriate to avoid termite attack.

- Proposed substitutions of materials must be thoroughly evaluated and must be approved by the designer (see Section 13.2.9).

  The building envelope must be installed in a manner that will not compromise the building's structural integrity. For example, during construction, if a window larger than originally intended is to be installed because of delivery problems or other reasons, the contractor should obtain the designer's approval prior to installation. The larger window may unacceptably reduce the shear capacity of the wall, or different header or framing connection details may be necessary. Likewise, if a door is to be located in a different position, the designer should evaluate the change to determine whether it adversely affects the structure.

- Adequate quality control (i.e., inspection by the contractor's personnel) and adequate quality assurance (i.e., inspection by third parties such as the building official, the designer, or a test lab) must be provided.

  The amount of quality control/quality assurance will depend on the magnitude of the natural hazards being designed for, complexities of the building design, and the type of products/systems being used. For example, installation of windows that are very tall and wide and make up the majority of a wall deserves more inspection than isolated, relatively small windows. Inspection of roof coverings and windows is generally more critical than inspection of most wall coverings, because of the general susceptibility of roofing and glazing to wind and the resulting damage from water infiltration that commonly occurs when these elements fail.

### 13.4.1 Substitution of Building Envelope Materials

The considerations discussed in Section 13.2.9 for substitution of foundation materials also apply to substitutions of envelope materials.

### 13.4.2 Building Envelope Inspection Points

As a guide for building envelope inspections, Table 13.7 suggests critical inspection points.

**Table 13.7**
Building Envelope Inspection Points

| Inspection Point | Reason |
|---|---|
| 1. Siding attachment to wall framing | Ensure there are sufficient number, type, and spacing of nails |
| 2. Attachment of windows and doors to the wall framing | Ensure there are sufficient number, type, and spacing of either nails or screws |
| 3. Flashings around wall and roof openings, roof perimeters, and at changes in building shape | Prevent water penetration into building envelope |
| 4. Roof covering attachment to sheathing, including special connection details | Minimize potential for wind blowoff (In high-seismic-load areas, attention to attachment of heavy roof coverings, such as tile, is needed to avoid displacement of the covering.) |
| 5. Attachments of vents and fans at roofs and walls | Reduce chance that vents or fans will blow off and allow wind-driven rain into the building |

### 13.4.3 The Top Building Envelope Issues for Builders

1. Many manufacturers do not rate their products in a way such that it is easy to determine if the product will really be adequate for the coastal environment and the expected loads. Require suppliers to provide information about product reliability in this environment.
2. Wind-driven rain will find a way into the house if there is a path left open. Sealing openings and shedding water will play a significant part in building a successful coastal home.
3. Window and door products are particularly vulnerable to wind-driven rain leakage and air infiltration. These products should be tested and rated for the expected coastal conditions.
4. Use the current high-wind techniques of extra roof surface sealing or attachment at the eaves and gable end edges.
5. Coastal buildings do require more maintenance than inland structures. This maintenance requirement needs to be considered in the selection of materials and the care with which they are installed.

## 13.5 Appurtenant Structures

**WARNING**

**Cantilever decks should not be placed over bulkheads or retaining walls where wave runup on the vertical structure could damage the deck.**

### 13.5.1 Decks

Decks often form a significant area of the elevated building platform. They are usually of 2x material, spaced to allow water drainage. The material choices, discussed in detail in Appendix J, include pressure-preservative-treated wood, naturally durable wood, and wood-plastic composites. The deck boards should be placed with a 1/8-inch–1/4-inch spacing to allow for drainage and possible expansion when wetted. This gap is especially needed at the end grain. Many builders fasten deck boards with screws or ring-shank nails to minimize nail pops.

The question of whether to place flat-grained deck boards heart side up (rings concave up) or bark side up (rings concave down) has been a subject of continued debate for years. Both orientations have advantages and disadvantages. Wood will naturally shrink to be concave on the bark side. Treated southern pine lumber from small trees will tend to have more treatment on the bark (sapwood) side. Shakes (cracks along the growth rings) tend to form on the heart side. So a deck board with the heart side up will more likely dry with a convex top surface that sheds water, but it will be prone to having shakes on the top surface. With the bark side up, the board will more likely dry with a concave top surface that holds water, but the exposed surface will be the treated sapwood that will resist the retained water. For minimum warp, select the more costly vertical-grained lumber over flat-grained lumber. Choose dried wood over green, or use green wood with less tendency to warp. Green heart redwood and cedar are less prone to warp than green Douglas fir.

Wood railings around decks must be carefully designed for both strength and durability. The IBC 2000 (ICC 2000a) requires that handrails resist a horizontal distributed load of 20 lb/ft and a nonconcurrent point load of 200 lb. The designer and builder must achieve this capacity in the wood railing construction and the associated connections while considering stress reductions required by the code for wet use and ripped lumber. The completed railing construction should not retain water in its connections that will lead to decay. Open railing systems are preferable to solid railings because of the increased wind load induced on solid railings. Also, open railings below the DFE allow for improved flow-through of flood waters. The local building code will specify the maximum allowable size of the openings between the railings.

Post-storm investigations of building damage have provided substantial evidence that decks and other exterior structures create a significant amount of debris when they break apart under the forces of wind and water. It is important that the construction methods and materials used in the installation of these exterior structures meet the same requirements as those established for the primary building, because the failure modes are the same and the load mechanisms are the same. Indeed, sometimes the loads on appurtenant structures are even higher because of their exposure. In considering the potential loads on decks, the builder should keep in mind that many decks become screened-in porches with roofs and thus, in the event of high winds, experience even greater loads. Decks can be built to withstand severe events, as shown by Figure 13-29, a photograph of a deck that survived when the building did not.

**Figure 13-29**
Building damage from Hurricane Opal at Pensacola Beach, Florida. The deck, which was constructed to State of Florida Coastal Construction Control Line (CCCL) design requirements, survived; the building, which was not constructed to CCCL requirements, did not survive. See Appendix G for a discussion of the Florida CCCL.

Stairs leading from decks are frequently a source of storm-created debris. Stairs should have open risers and should be supported on posts or piles embedded below the expected depth of storm-induced scour if they are expected to survive a severe storm. Debris from stairs is minimized when the stairs are located on the landward side of the building rather than the seaward side.

An alternative is to connect the stairs to the deck in such a way that the stairs can be removed or elevated above the expected flood depth. Frequently, such a design involves the use of winches and cables so that the stairs can be elevated by a person on the deck level. In order for the stair section to be lifted, the stair/deck connection must be hinged. Figure 13-30 illustrates this stair elevation technique. Note that this system cannot be used for stairs that provide the only means of egress and access for the building.

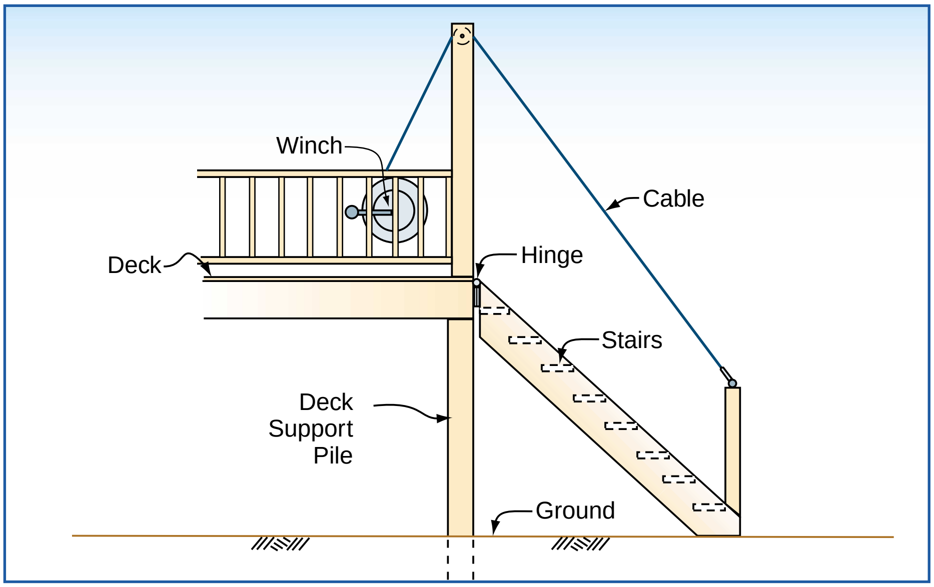



**Figure 13-30**
Stair elevation system. Note that this system cannot be used for stairs that provide the only means of egress and access for the building.

## 13.5.2 Storage Buildings

Storage buildings and any other building installed away from the primary building should be anchored so that they will not be susceptible to overturning or sliding into another building and causing collateral damage. Storage buildings will be exposed to similar loads as the main structure and can fail in the same way. The most significant problem is that the storage building can become debris and cause more severe damage and loss to adjacent buildings.

An effective way to reduce damage from the failure of storage buildings and other small buildings is to not install these buildings in coastal areas. If the building is considered sacrificial, it should be pointed out to the homeowner and neighbors that the sacrifice may cause significant damage to either the primary or adjacent structures.

## 13.5.3 Swimming Pools and Hot Tubs

Pools and hot tubs are normally made of one of the following:

- reinforced concrete
- fiberglass
- reinforced masonry

For one-piece units, such as those made of fiberglass, the installer will frequently use a crane to set the pool, so the building site will need to be accessible for this large piece of equipment. Locations for pumps, piping, a heater (if there is one), fuel supply, and other associated equipment must be found so that the following requirements are met:

- The equipment must be located in the proper place to supply the pool with water.
- The equipment must be elevated to or above the DFE so that the potential for flood damage is minimized.
- The equipment must be set so that wind, or seismic, and water forces (including inundation by salt water or sediment) will not displace or damage it. Unless specifically designed, most mechanical equipment is not intended to be inundated with salt water. Equipment installed in the corrosive coastal environment will normally require corrosion-resistant piping (usually PVC), stainless steel pump impellers, cast iron pump bodies, and totally enclosed electrical components.

The design considerations for one-piece pools are covered in Section 12.9.4 in Chapter 12. The builder, however, must be able to execute the design intent, which includes the following:

- A one-piece pool must be able to resist flood forces with minimal damage, whether the pool is full or empty. This means that an in-ground pool must resist failure from buoyancy as well as wall fracture.
- The pool equipment must be easily returned to service after a severe event. A flexible connection between the pump and piping may help achieve this objective so the equipment can move under forces from the severe event. This coupling will also reduce stress on the pipe from vibration.
- Pool accessories (e.g., cleaning nets, lane dividers) can become airborne debris and should have a secure storage location.

The design intent for concrete pools includes the following:

- Elevation of an in-ground pool should be such that scour will not permit the pool to fail from either normal internal loads of the filled pool or from exterior loads imposed by the flood forces.
- The pool should be located as far landward as possible and should be oriented in such a way that flood forces are minimized. This includes placing the pool with the narrowest dimension facing the direction of flow, orienting the pool so there is little to no angle of attack from flood water, and installing a pool with rounded instead of square corners. All of these design choices will reduce the amount of scour around the pool and thereby improve the chances the pool will survive the storm. These concepts are illustrated in Figure 13-31.

**NOTE**

**Two articles concerning swimming pool design standards in V zones are included in Appendix L. The articles—*Scour Impact of Coastal Swimming Pools on Beach Systems* and *Conceptual Breakaway Swimming Pool Design for Coastal Areas*—report the results of studies funded by FEMA.**

**CROSS-REFERENCE**

**See Section 12.9.4, in Chapter 12, for design guidance and regulatory requirements regarding swimming pools.**

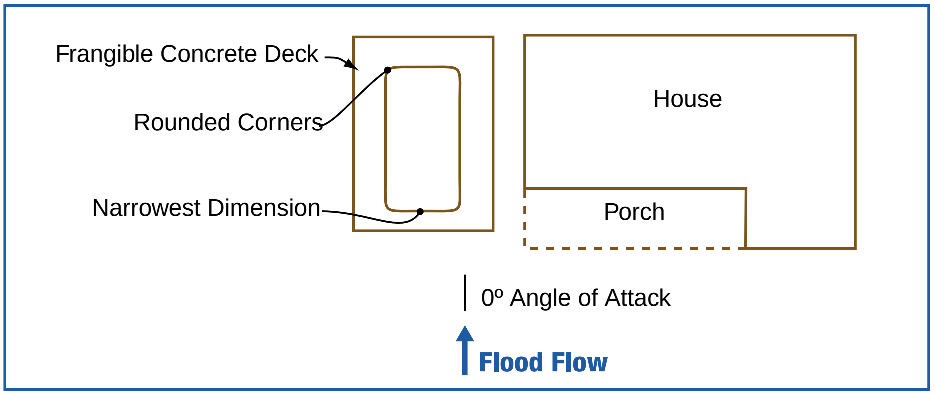



**Figure 13-31**
Recommendations for orientation of in-ground pools.

- A concrete pool "deck" should be frangible, so that flood forces will create concrete fragments that will help reduce scour. The concrete deck should be installed with no reinforcing and should have contraction joints placed at 4-foot squares to "encourage" failure. See Figure 12-121, in Chapter 12, for details on constructing a frangible concrete pad.
- Pools should not be installed on fill in or near a V zone. Otherwise, a pool failure may result from scour of the fill material

### 13.5.4 Walkways and Sidewalks

Walkways and sidewalks built adjacent to a coastal building will normally require permits from the local building official, and local regulations will usually require that these structures be shown on the plans. Walkways built over a dune or on, around, or over an erosion control structure will probably require a building permit from the state agency responsible for dune and beach protection, regardless of whether the walkway is for private or public beach access.

Walkways and sidewalks are usually built of concrete or wood. Concrete will normally be restricted to "flat" work or sidewalks, parking pads, and similar features. As recommended in other sections of this manual, this concrete should be installed with no reinforcing steel and it should have contraction joints at 4-foot squares so that it can be easily broken into 4-foot x 4-foot sections when subjected to flood forces.

Wood walkway members in ground contact should be ground-contact-rated pressure-preservative-treated wood. Wood walkway members not in ground contact should be aboveground-rated (or better) pressure-preservative-treated wood or naturally decay-resistant wood. The environmental conditions at or near the coast will be particularly hard on such walkways: salt air, rain, sun, and sand will work to alternately dry out, wet, and abrade the wood. Wood walkways at grade level should be anchored with posts buried several feet in the ground to prevent uplift from wind. During a severe flood event, scour may occur at the edges of the walkway and cause an uplift failure, so the posts should be buried a minimum of 8 feet below the expected level of scour (see Appendix I).

There are several state-initiated guidance documents on walkway/walkover construction. Appendix I includes a construction guidance document from the State of Florida, Bureau of Beaches and Coastal Systems, *Beach/Dune Walkover Guidelines* (January 1998) and a document from Florida Sea Grant titled *Beach Dune Walkover Structures* (December 1983).

## 13.6 Utility/Mechanical Equipment

This section presents guidance concerning the installation of elevators, building utility systems (heating, ventilating, and cooling [HVAC], electrical, water, and wastewater), and storage tanks. For detailed information about the design and construction of utility systems for buildings in flood hazard areas, refer to *Protecting Building Utilities From Flood Damage – Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems*, FEMA 348 (FEMA 1999).

### 13.6.1 Elevators

Elevators are becoming commonplace in many coastal homes. Normally, these elevators have only a one- to four-person capacity, and they almost always require installation in an area that will at least be partially below the DFE. To minimize flood damage to the elevator and its parts, the builder should look for locations above the DFE to mount elevator equipment (e.g., electrical controls, hydraulic pumps).

Normally, for fire safety reasons, elevators are equipped with a default device that sends the cab to the lowest floor when there is a power outage (which will always occur during a major storm). For the protection of the elevator equipment and the occupants, this manual recommends that a float switch be installed that activates when inundated by flood water and sends the elevator cab to a floor above the DFE. For additional information, see FEMA NFIP Technical Bulletin 4, *Elevator Installation for Buildings Located in Special Flood Hazard Areas*, in Appendix H.

**NOTE**

For additional information about the proper design and construction of utility system components for buildings in flood hazard areas, refer to *Protecting Building Utilities From Flood Damage – Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems*, FEMA 348 (FEMA 1999).

### 13.6.2 Heating, Ventilating, and Cooling (HVAC) Systems

HVAC systems include a number of components that must be installed so that they are protected from damage during a severe wind, flood, or seismic event. These components include the following:

- outdoor condensers
- air-handling units
- ductwork for supply and return air
- electrical components for power to the air-handlers and controls
- fuel storage (if not electric)

If the HVAC components inside the building are all installed above the DFE, many potential losses will be avoided. The single most expensive component often lost during a severe event is the outdoor condenser. The reason is that the condenser is usually placed in a position that, while perhaps minimizing expense for the HVAC system, leaves it exposed to the wind and water forces that accompany a severe storm.

To minimize damage to the outdoor condenser, do the following:

- Mount it on the side of the building that will be least affected by flood velocity flow and waves.
- Mount it above the DFE.
- Secure it so that it cannot move, vibrate, or be blown off its support.
- Protect it from damage by airborne debris.

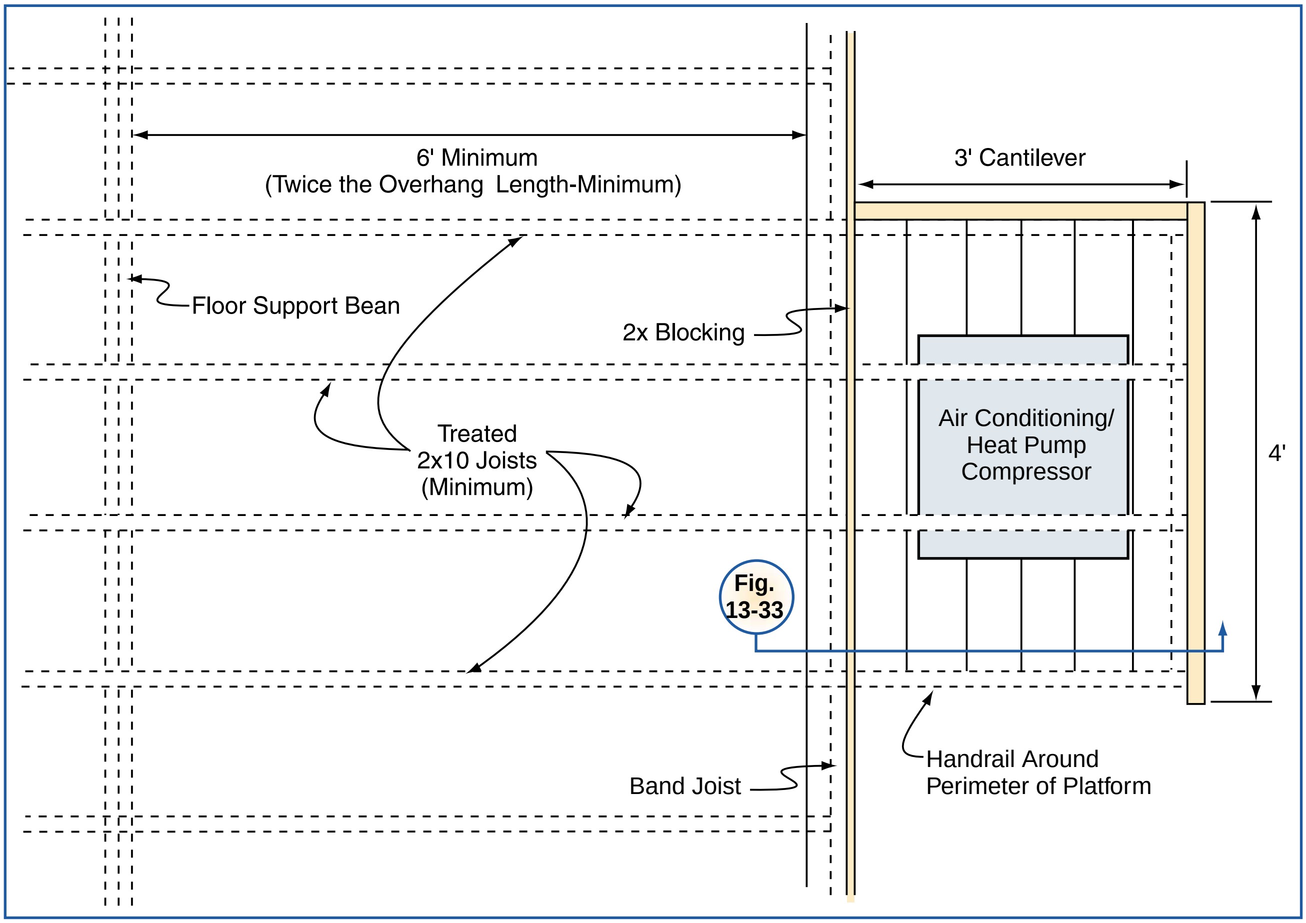
There are several ways to support an outdoor condenser with a connection to the floor framing. These include the following:

- cantilever floor framing (see Figures 13-32 and 13-33; this method requires careful detailing to prevent water penetration into the building floor and wall systems)
- pile-supported
- wood-brace-supported (see Figures 13-34 and 13-35)
- rooftop mount

**COST CONSIDERATION**

HVAC equipment that is not elevated above the DFE will probably need to be repaired or replaced after a flood.

**Figure 13-32**
Cantilever floor framing for air-conditioning/heat pump compressor platform – plan view.



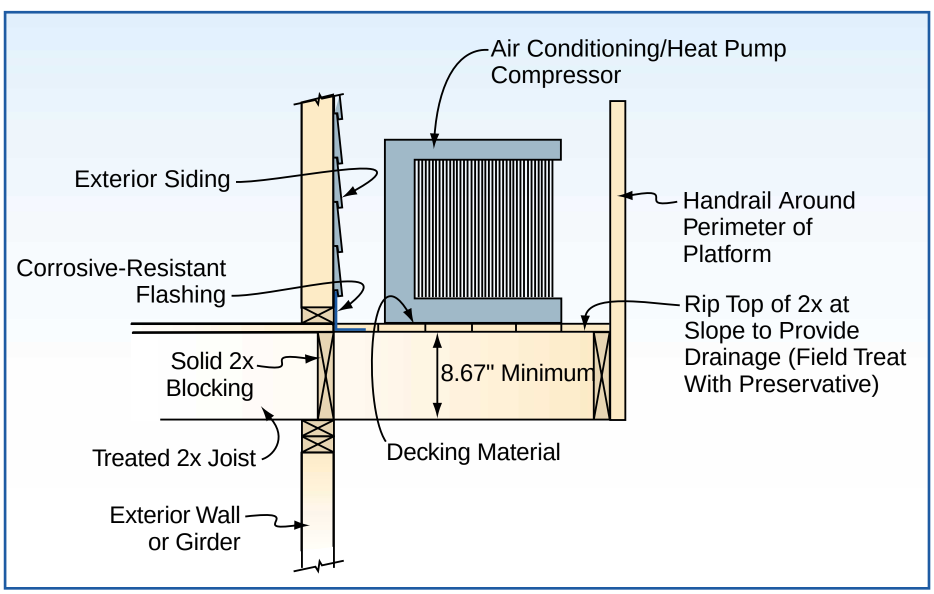




**Figure 13-33**
Cantilever floor framing for air-conditioning/heat pump compressor platform – elevation view.

**Figure 13-34**
Wood-brace-supported air-conditioning/heat pump compressor platform – plan view.

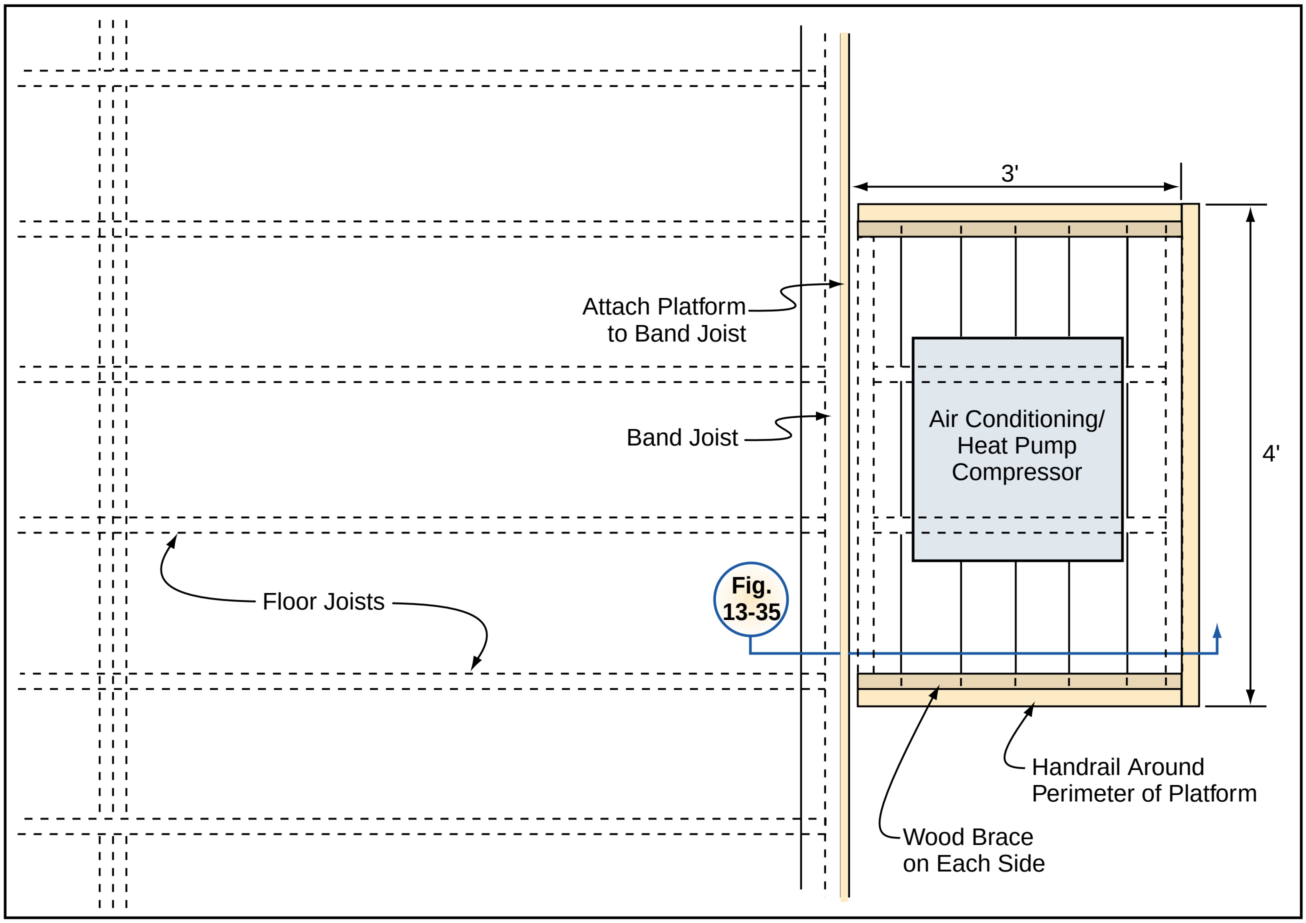
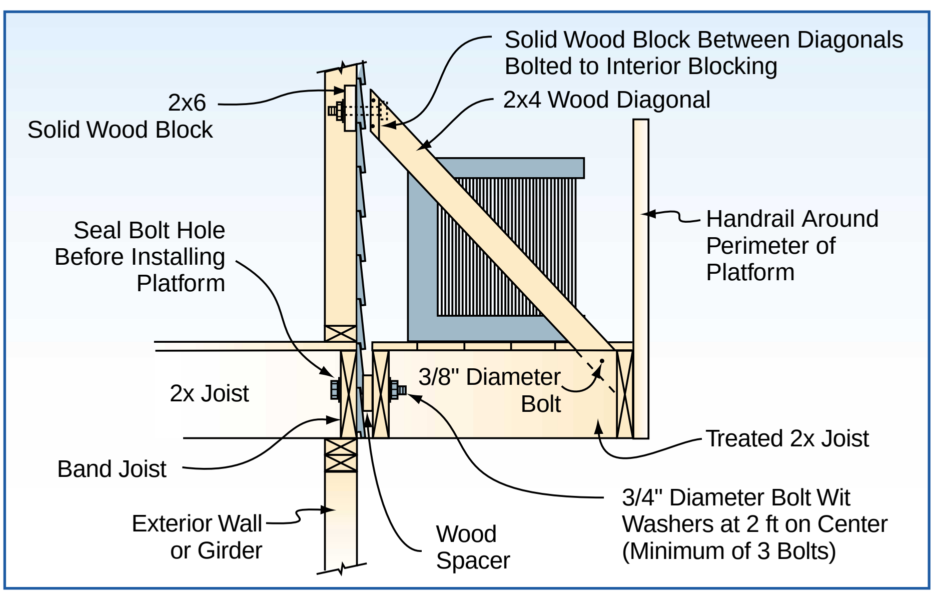



**Figure 13-35**
Wood-brace-supported air-conditioning/heat pump compressor platform – elevation view.



### 13.6.3 Electrical Systems

Installation of electrical components in coastal environments must consider the following:

- Any outlet, switch, or fixture below the DFE should be assumed to be at risk of damage and shorting out when inundated with water. For example, outlets installed below the DFE should be ground fault protected and equipped with moisture-resistant covers.
- Ferrous exterior fixtures and switches will corrode in the salt-laden environment.
- Any opening that wires are run through will leak water unless tightly sealed. Use drip loops to minimize water entry.
- Wires, conduits, and other system components should be installed on the landward side of piles or foundation elements, out of the path of flood forces.
- Many electrical companies will require that the electric meter be placed so that it can be read by utility company employees from the ground.
- Electrical components must not be attached to or penetrate breakaway walls.
- Electrical panels must be at or above the DFE.
- Electrical service below the DFE must be the minimum required for life safety.

**CROSS-REFERENCE**

Detailed information about the installation and protection of electrical system components is provided in *Protecting Building Utilities From Flood Damage – Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems*, FEMA 348 (FEMA 1999).

**WARNING**

Utility system components must not be connected to breakaway walls under elevated buildings in coastal flood hazard areas. The resistance provided by electrical and plumbing lines and other utility system components can prevent the walls from breaking away as intended under flood forces.

### 13.6.4 Water and Wastewater Systems

The builder should install water and waste water risers and runs in such a way that flood and wind damage to them will be minimized. This means that risers should be installed behind piles or other foundation elements to protect them from flood forces and the impact of debris. Risers should not be installed on breakaway walls, because the lines will be damaged when the breakaway wall fails. Pipe runs that are parallel to the floor joists should be installed as high as possible between the joists. Pipe runs that are perpendicular to the floor joists should also be installed as high as possible. This may be accomplished by notching the joists and attaching ceiling sheathing. Such notching should be kept to a minimum, however, so that joists are not significantly weakened. An alternative is to install wood spacers on the bottom of the joists to provide sufficient room for pipe runs before installing a ceiling.

**WARNING**

**Do not** anchor tanks to breakaway walls under elevated buildings.

### 13.6.5 Tanks

Small tanks used for fuel, such as propane tanks, should be elevated above the DFE; if a tank is detached from its location during a severe flood event, it can become a windborne or waterborne missile. In addition to being elevated, small tanks also need to be strapped or otherwise secured to the building so that they will not be detached by high winds or a seismic event. Any strapping used should be corrosion-resistant. This technique can be altered to accommodate an exterior location. Builders should keep in mind that the strapping must resist forces from all four plan directions.

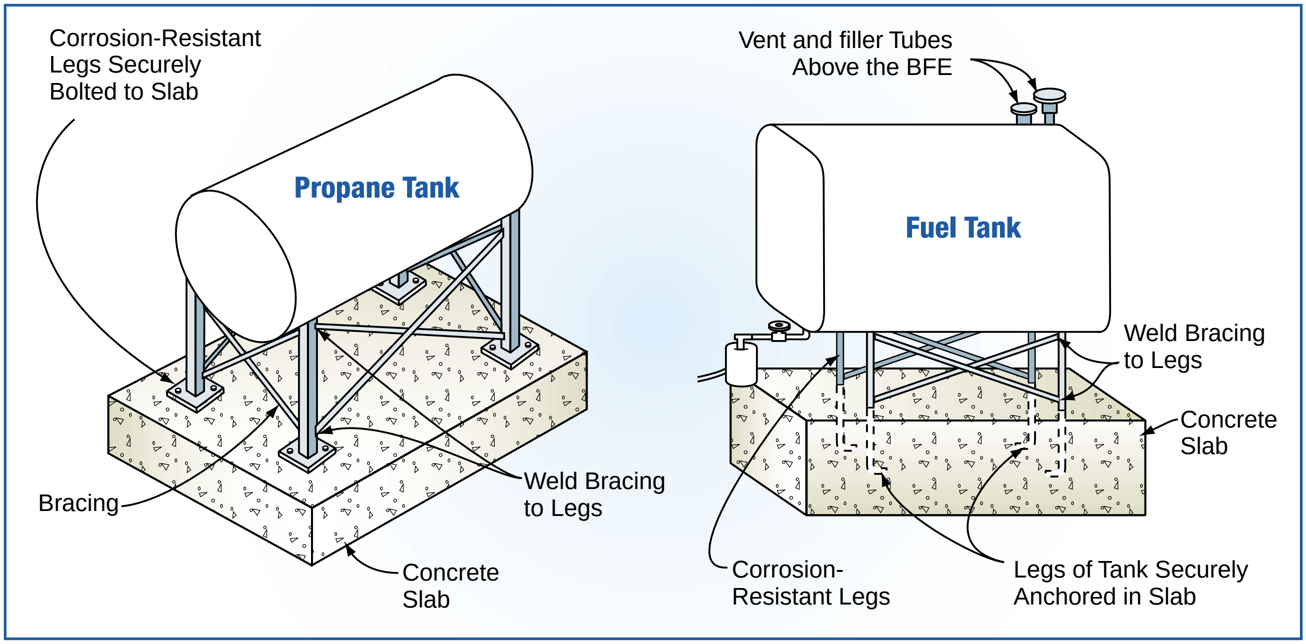
Larger tanks will normally be buried belowgrade or supported by and attached to a foundation. In a Special Flood Hazard Area, erosion and scour can expose the tank; the tank can fill with sediment/soil and salt water; or buoyancy forces on the tank can be sufficient to lift the tank out of the ground. Larger tanks may be used for septic tanks or for the storage of propane or fuel oil.

This manual recommends the following for minimizing damage to belowgrade tanks:

- Orient the tank so that scour is minimized. Do this by
    - orienting the tank with the narrowest dimension of the tank perpendicular to the flood flow,
    - orienting the tank so there is no angle of attack of the flood water, and
    - locating the tank so that flow diverted or channeled by nearby structures is not directed toward the tank.
- Secure the tank so that buoyancy forces will not lift the tank out of the ground. Do this by anchoring the tank to a concrete pad heavy enough to keep the tank in the ground.

Aboveground tanks can be anchored in at least two ways (see Figure 13-36). In addition, the tank must be located such that scour does not undermine the slab foundation, and the openings of fill lines and overflows must be above the DFE so that flood water will not enter the tank.

**Figure 13-36**
Anchoring techniques for aboveground tanks.

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# Chapter 14: Maintaining the Building

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# Maintaining the Building

## 14.1 Introduction

To maintain maximum building performance, a coastal building's structural system and envelope (i.e., exterior wall covering, doors, windows, and roof covering) must not be allowed to deteriorate. If the building is significantly degraded by corrosion, wood decay, termite attack, or weathering, its vulnerability to damage from natural hazards is increased. Figure 14-1 shows a decayed pile that appears from the outside to be in acceptable condition. Under the loads imposed by a natural hazard event, this pile could fail.

**Figure 14-1**
Pile that appears acceptable from the outside but that is decayed in the center.

The key to reduced long-term maintenance is the initial selection of appropriate materials and proper construction. **Maintenance and repair demands will be directly influenced by decisions made during building design and construction.** If less durable materials are installed, the frequency and costs of required maintenance and repair will be increased. Design and detailing of various building systems (e.g., exposed structural, window or roof system) also significantly influence maintenance and repair demands.

**COST CONSIDERATION**
Maintenance and repair costs are directly related to design decisions, materials selection, and construction methods.

To help ensure that a coastal building is properly maintained, this manual recommends that buildings be inspected annually (see Table 14.1) by persons knowledgeable of the systems and materials they are inspecting. The following building components should be inspected annually: building envelope, including wall sheathings, doors, windows, shutters, and roof coverings; foundation and structural frame; and exterior-mounted mechanical and electrical equipment. Items requiring maintenance, repair, or replacement should be documented and the required work scheduled.

Other items that should be inspected include cavities through which air can freely circulate (e.g., above soffits) and, depending on structural system characteristics and access, the structural system. For example, painted, light-gauge, cold-formed steel framing is vulnerable to corrosion, and the untreated cores of treated timber framing are vulnerable to decay and termite attack. Depending on visual findings, it may be prudent to determine the condition of concealed items through non-destructive or destructive tests (e.g., test cuts).

**Table 14.1** Maintenance Inspection Checklist

| Inspection Item | Condition | | | Repair/ Replace | |
|---|---|---|---|---|---|
| | GOOD | FAIR | POOR | YES | NO |
| **Foundation:** *Wood pile* – decay, termite infestation, severe splits, connection to framing | | | | | |
| *Sill plates* – deterioration, splits, lack of attachment to foundation | | | | | |
| *Masonry* – deteriorated mortar joints, cracked block, step cracks indicating foundation settlement | | | | | |
| *Concrete* – spalling, exposed reinforcing steel, ≥ 1/4-in vertical cracks or horizontal cracks with lateral shift in the concrete across the crack | | | | | |
| **Exterior walls:** *Siding* – deterioration, withdrawal of nails, discoloration, buckling, nails missing studs, caulking | | | | | |
| *Trim* – deterioration, discoloration, separation at joints | | | | | |
| **Porches/columns:** Condition of top and bottom connections to framing, deterioration at base of wood columns | | | | | |
| **Floors:** *Joists or beams* – decay, termite infestation, corrosion at tiedown connectors, splits, excessive holes or notching, excessive sagging | | | | | |
| *Sheathing* – deterioration, "squeaky" floor, excessive sagging, nails missing joists | | | | | |
| **Windows/doors:** *Glazing* – cracked panes, condensation between panes of insulated glass, nicks in glass surface, sealant cracked/dried out | | | | | |
| *Trim* – deterioration, discoloration, separation at joints, caulking dried out or separated | | | | | |
| **Roof:** *Asphalt shingles* – granule loss, shingles curled, nails withdrawing from sheathing | | | | | |
| *Wood shakes* – splits, nails withdrawing, discoloration, deterioration, moss growth | | | | | |
| *Metal* – corrosion, discoloration | | | | | |
| *Flashings* – corrosion, joints separated, nails withdrawing | | | | | |
| **Attic:** *Framing* – condition of truss plates, sagging or bowed rafters or truss chords, deterioration of roof sheathing, evidence of water leaks, adequate ventilation | | | | | |

## 14.2 Effects of Coastal Environment

### 14.2.1 Corrosion

The corrosive effect of salt-laden wind-driven moisture in coastal areas cannot be overstated. Salt-laden, moist air can corrode exposed metal surfaces and can penetrate any opening in the building. The need to protect metal surfaces through effective design and maintenance is very important in the long-term life of the individual building components and the life of the entire building. Corrosion of structural elements is particularly damaging to the ability of the building to withstand the forces from a natural hazard event.

### 14.2.2 Termites

The likelihood of termite infestation in coastal buildings can be reduced by maintenance that makes the building site drier and otherwise less hospitable to termites:

- Store firewood and other wood items, including wood mulch, on the ground, away from the building.
- Keep gutters and downspouts in good repair and positioned to direct water away from the building.
- Keep water pipes, water fixtures, and drainpipes in good repair.
- Avoid dampness in crawlspaces by providing adequate ventilation or installing impervious ground cover membranes.
- Avoid frequent plant watering adjacent to the house, and keep plants trimmed away from the walls.

### 14.2.3 Moi sture

There are many sources of exterior moisture in the coastal environment. Wherever this moisture is retained, wood decay, mildew, or other forms of deterioration can progress. For example, Figure 14-2 shows decay at the base of a wood post where moisture was retained, and Figure 14-3 shows decay behind a connection plate for a beam. Interior moisture must also be considered. Significant interior sources of water vapor, such as kitchens, baths, and clothes dryers, should be vented to the outside in such a way that condensation does not occur on interior or exterior surfaces.

**Figure 14-2**
Wood decay at the base of a post supported by concrete.

**Figure 14-3**
Wood decay behind a metal beam connector.

Decay of wood framing in crawlspaces is very likely in low-lying coastal areas. Moisture migration into the floor system can be reduced if the floor of the crawlspace is covered with a vapor barrier of at least 6 mil polyethylene. Also, in accordance with the local building code, wood framing in this space must be pressure-treated or naturally decay-resistant. In addition, the building code will prescribe ventilation requirements.

### 14.2.4 Weathering

The combined effects of sun and water on many building materials, particularly wood, cause weathering effects, which include the following:

- fading of finishes
- accelerated checking and splitting of wood
- gradual loss of thickness of wood (see Section 14.3.1)

In combination, the effects of weathering reduce the life of building materials unless they are naturally resistant to weathering or are protected from it, either naturally or by maintenance.

## 14.3 Building Elements That Require Frequent Maintenance

### 14.3.1 Siding

Solar ultraviolet (UV) degradation occurs at a rate of about 1/16 inch over 10 years on exposed wood. This is not significant for dimension lumber, but it is significant for plywood with 1/8-inch veneers. If the exterior plywood is the shearwall sheathing, this loss will be significant over time. Maintenance suggestions for siding materials include the following:

- Protect plywood from UV degradation with pigmented finishes rather than clear finishes. Pigmented finishes also are especially recommended for exposed shearwall sheathing.
- Wood siding must be protected with a protective sealant—usually a semi-transparent stain or paint.
- Keep siding surfaces and exterior equipment free of salt and mildew. Wash salt from siding surfaces and outdoor air-conditioning condensers not washed by rain, taking care to direct the water stream downward. As required, wash mildew from siding using commercially available products or the homemade solution of bleach and detergent described in *Finishes for Exterior Wood: Section, Application and Maintenance* (Williams et al. 1996).

- Caulk seams, joints, and building material discontinuities with a caulking compound intended for severe exterior exposures. Renew this caulking every 5 years at a minimum or when staining or painting the siding and trim. Caulking applied at large wood members should be renewed about 1 year later after the wood has shrunk away from the caulked joint.

- Re-nail siding when nails withdraw (pop out). Re-nail at a new location so the new nail does not go into the old nail hole.

### 14.3.2 Roofs

Roof coverings are typically the building envelope material most susceptible to deterioration. Also, depending on roof system design, minor punctures or tears in the roof covering can allow water infiltration, which can lead to serious damage to the roof system and other building components. Maintenance suggestions for roof materials include the following:

- Check the general condition of the roof covering. Granule loss from asphalt shingles is always a sign of some deterioration, although some loss is to be expected from new shingles. Dab roofing cement under the tabs of the first layer of shingles, including the base course, to help ensure that this layer stays down in high winds. Dab roofing cement under any shingle tabs that have lifted up from the existing tack strip. Check the nails that attach the shingles to the roof for corrosion or pullout. Check metal flashings and replace or repair them as necessary.

- Clean dirt, moss, leaves, vegetative matter, and mildew from wood shakes, and re-coat them with a clear wood preservative.

- Clean corroded surfaces of ferrous metal roofs, and apply an appropriate paint or sealer. Check the attachment of the roof surface to the deck. Screws and nails can work loose and may require tightening. Some roofing systems are attached to the underlayment with clips that can corrode – these clips should be inspected, and any corroded clips replaced. Even structural steel roof support members can become corroded as illustrated in Figure 14-4.

- Remove debris from the roof, and ensure that drains, scuppers, gutters, and downspouts are not clogged.

- Periodically re-coat single-ply asphaltic membranes where appropriate. Check mineral-surfaced cap sheets for granule coverage and re-coat if coverage is insufficient, provided there is sufficient service life remaining with the membrane.

- Remove old asphalt shingles before recovering. Installing an additional layer of shingles requires longer nails. In addition, it is more difficult to install the new layer flat enough, and with enough nails, that uplift will not occur, even in relatively low wind speeds. As a result, the new layer will be susceptible to wind uplift and damage.

**Figure 14-4**
Typhoon Paka (1997), Guam. Corrosion observed in the bottom flange of a structural steel roof support member. Photograph by Tom Smith.

### 14.3.3 Glazing

Glazing includes windows, doors, skylights, and shutters. Glazing is particularly vulnerable to damage in coastal areas because high winds create airborne debris that can strike the glazing. Very small objects such as sand grains, small stones, or roof gravel or granules can strike the glazing many times without ever actually breaking the glass. But these repeated impacts will weaken the glass until some object strikes it and causes a failure. The references listed in Section 14.7 include several articles that address this subject. Maintenance suggestions for glazing include the following:

- Check glazing for excessive scratches and chips and replace as needed.
- Check glazing gaskets/sealants for deterioration. Repair or replace as needed. Broken seals in insulated glass are not uncommon in coastal areas.
- Check wood frames for decay and termite attack, and check metal frames for corrosion. Frames should be periodically repainted (where appropriate), and damaged wood should be replaced. Maintaining the putty in older wood windows will minimize sash decay.
- Check for signs of water damage (e.g., water stains, rust streaks from joints). Check sealants for substrate bond and general condition. Repair or replace as needed.

- Check shutters for general integrity and attachment. Periodically repaint where appropriate. Replace or strengthen the attachment of the shutter system to the building as appropriate.
- Check locks and latches frequently for corrosion and proper operation. Lock mechanisms are vulnerable to attack by salt-laden air. Applying a lubricant or rust inhibitor will improve the operation of these mechanisms over the short term.

### 14.3.4 Outdoor Mechanical/Electrical Equipment

Most outdoor mechanical and electrical equipment includes metal parts, which will corrode in the coastal environment. The life expectancy will improve if the salt is washed off the outside of the equipment frequently. This will occur naturally if the equipment is fully exposed to rainwater; however, partially protected equipment is subject to greater corrosion because of the lack of this natural rinsing action.

Using alternative materials that do not include metal parts would also help reduce the problems caused by corrosion. However, building owners should expect some of the following types of problems simply because of the environment:

- Electrical contacts will malfunction and either short out or cause intermittent operation.
- Housings for electrical equipment, HVAC condensers, ductwork, and other components will deteriorate more rapidly in the coastal environment.
- Metal fasteners and clips used to secure equipment will deteriorate more rapidly in the coastal environment.

**CROSS-REFERENCE**

**Installation of horizontal 2x members with the cup (concave surface) down will minimize water retention and wood deterioration (see Section 13.5.1).**

### 14.3.5 Decks/Exterior Wood

The approach to the maintenance of exterior wood 2x members is different from that for thicker members. The formation of small checks and splits in 2x wood members from cyclical wetting and drying can be reduced by the use of water-repellent finishes. The formation of larger checks and splits in thicker wood members is caused more by long-term drying and shrinking and will not be as significantly reduced by the use of water-repellent finishes.

Cyclical wetting and drying, such as from dew or precipitation, causes the exterior of a wood member to swell and shrink more quickly than the interior. This causes stress in the surface, which leads to the formation of checks and splits. This shrink-swell cycling is worst on south and west exposures. Checks and splits, especially on horizontal surfaces, provide paths for water to reach the interior of a wood member and remain, where they eventually cause decay.

Maintaining a water-repellent finish, such as a pigmented paint, semi-transparent stain, or clear finish, on the wood surface can reduce the formation of checks and splits. These finishes are not completely water- or vapor- repellent, but they significantly slow cyclical wetting and drying. Of the available finishes, pigmented paints and semi-transparent stains have the longest lifetime; clear finishes must be reapplied frequently to remain effective. Matte clear finishes are available that are almost unnoticeable on bare wood. These finishes are therefore attractive for decking and other "natural" wood, but they must be renewed frequently, when water no longer beads on the finished surface.

Moisture-retaining debris tends to collect between deck boards and in the gaps in connections. Periodic cleaning of this debris from between wood members, especially at end grains, will allow drying to proceed and will inhibit decay.

The best way to maintain larger timbers is to keep water away from joints, end grain surfaces, checks, and splits. Much can be learned by standing under the house during a rain with the prevailing wind blowing to see where the water goes. Measures, such as those described in Section 13.2.7, can then be taken or renewed to minimize the effect of this water on the larger timbers.

### 14.3.6 Metal Connectors

Most sheet-metal connectors, such as tiedown straps, joist hangers, and truss plates, used in structural applications in the building should be specified to last the lifetime of the building without the need for maintenance. However, the use of corrosion-prone connectors is a common problem in existing coastal houses.

If galvanized connectors remain gray, the original strength is generally unaffected by corrosion. When most of the surface of the connector turns rust red, the sacrificial galvanizing has been consumed and the corrosion rate of the unprotected steel can be expected to accelerate by up to a factor of 50 times.

The thin steel used in sheet-metal connectors has little reserve strength to offset rapid corrosion. During routine inspections, any sheet-metal connectors found to have turned rust red or to show severe, localized rusting sufficient to compromise their structural capacity should be replaced immediately. Be advised, however, that the replacement of sheet metal connectors is usually difficult for a number of reasons: the connection may be under load, the nails or bolts used to secure connectors are usually hard to remove, and the location of a connector often makes removal awkward.

As indicated in Chapter 13, corrosion rates can be reduced if exposure to salt-laden air is minimized or reduced. Covering exposed connectors with a sheathing material reduces their exposure and therefore increases their life expectancy.

**CROSS-REFERENCE**

The selection of metal connectors for use within the building envelope and in exposed locations is addressed in Chapter 12 of this manual.

**WARNING**

Using corrosion-prone sheet metal connectors will increase maintenance requirements and potentially compromise structural integrity.

## 14.4 Maintenance Techniques Required for Natural Hazards

The maintenance practices described above for minimizing corrosion, wood decay, termite infestation, and UV degradation will improve the resistance of a coastal building to flood, wind, and seismic damage by maintaining the strength of the structural elements. The additional measures described in the following sections will further maintain the building's resistance to natural hazards.

### 14.4.1 Flooding

When designing for the lateral force capacity of an unbraced or braced pile foundation, the designer should have allowed for a certain amount of scour. Scour in excess of the amount allowed for will reduce the embedment of the piles and cause them to be overstressed in bending during the maximum design flood, wind, or earthquake. As allowed by local regulations and practicality, the grade level should be maintained at the original design elevation.

Scour and long-term beach erosion may affect pile maintenance requirements. If tidal wetting was not anticipated in the original design, the piles may have received the level of preservative treatment required only for ground contact and not the much higher marine treatment level that provides borer resistance. If the pile foundation is wetted by high tides or runup, borer infestation is possible. Wrapping treatments that minimize borer infestation are available for the portions of the piles above grade that are subject to wetting.

### 14.4.2 Seismic and Wind

Many seismic tiedowns at shearwall vertical chords use a vertical threaded rod as the tension member. Each end of the threaded rod engages the tiedown hardware or a structural member. Over time, cross-grain shrinkage in the horizontal wood members between the threaded rod connections will loosen the threaded rod, allowing more rocking movement and possible damage to the structure. Whenever there is an opportunity to access the tiedowns, the nuts on the rods should be tightened firmly. New proprietary tiedown systems are available that do this automatically.

Owners often want to remodel their buildings, and the remodeling plans often include making new openings in exterior or interior walls for doors or windows. Designers must be careful not to make openings in bearing walls or shearwalls without restoring the lost structural capacity. It is relatively easy to recognize a bearing wall in the field and to use a header or other means to accommodate an opening. It is much more difficult to identify a shearwall in the field without access to the original construction drawings. A shearwall will have structural panel sheathing on one or both sides. Exterior, interior, bearing, and non-bearing walls can all be shearwalls. Removing a portion of a shearwall not only takes away shear capacity, but also can disrupt the overturning force load path. The designer of the building should be consulted for the placement of any openings of significant size in structural-panel-sheathed walls.

Shearwall sill plates bearing directly on continuous footings or concrete slabs-on-grade, if used in coastal construction, are particularly susceptible to decay if moist conditions are present. Figure 14-5 shows a deteriorated sill plate. Even if the decay of the preservative-treated sill plate is retarded, the attached untreated plywood can easily decay. As a result, the shear wall will lose strength. Conditions that promote this sill and plywood decay include an outside soil grade above the sill, stucco without a weep screed at the sill plate, and sources of excessive interior water vapor. Correcting these conditions will help maintain the strength of the shear walls.

**WARNING**

**Enlarging existing openings in shearwalls or creating new openings can reduce the structural capacity of the wall and building.**

**Figure 14-5**
Deteriorated wood sill plate.

## 14.5 Retrofit Opportunities

Retrofit opportunities will present themselves every time maintenance work is required for a major element of the building. Improvements to the building that are made to increase resistance to the effects of natural hazards should focus on those items that will potentially return the largest benefit to the building owner. Retrofit improvements should be considered for the following building elements if the existing building is considered inadequate to resist natural hazard loads and opportunities for improvements present themselves:

- roof
- siding
- decks and porches
- exterior metal
- windows and doors
- foundation
- exterior equipment

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**DEFINITION**

**Retrofitting is the combination of adjustments or additions to existing building features that are intended to eliminate or reduce the possibility of flood, wind, or seismic damage.**

**NOTE**

In retrofitting, the most cost-effective techniques will normally involve fixing the weakest structural links and improving the water penetration resistance of the building envelope. To identify the weakest links, start at the top of the building and work down the load path.

Developing a maintenance schedule for a coastal building, based on normal life expectancies of building materials, will give the owner an idea about when certain retrofit projects could be contemplated (assuming a natural hazard event does not accelerate the schedule). A schedule can be developed from the life expectancies of various building elements presented in Table 14.2

**Table 14.2** Life Expectancies for Elements of Coastal Buildings

| Building Element | Range of Expected Life (Years) |
|---|---|
| **Roof:** | |
| ☐ asphalt shingles | 15 - 20 |
| ☐ wood shakes | 25 - 40 |
| ☐ metal | 40 - 60 |
| **Siding:** | |
| ☐ wood | 20 - 50 |
| ☐ vinyl | 15 - 25 |
| ☐ EIFS | 10 - 15 |
| ☐ Masonry/stucco | 40 - 50+ |
| **Decks/porches** | 10 - 20 |
| **Exterior metal** (handrails, connectors, etc.) | 5 – 10 |
| **Windows/doors:** | |
| ☐ glazing | 10 – 30 |
| ☐ frames | 10 – 40 |
| ☐ mechanisms | 10 – 15 |
| **Foundation:** | |
| ☐ wood piles | 25 – 40 |
| ☐ masonry piers | 30 – 80 |
| ☐ concrete piles or piers | 40 – 80 |
| **Exterior equipment** (HVAC, lights, etc.) | 8 - 15 |

Table 14.2 suggests that retrofit opportunities exist for the building elements listed below. It should be noted that opportunities for retrofitting may also be created by the need to repair damage caused by a natural hazard event.

- When the roof is replaced, the attachment of the sheathing to the trusses or rafters can be checked, and hurricane/seismic connectors can be installed at the rafter-to-wall or truss-to-wall connections.

- Gable ends can be braced in conjunction with other retrofits, or by themselves.
- If siding or roof sheathing has to be replaced, hurricane/seismic connectors can be installed at the rafter-to-wall or truss-to-wall connections, the exterior wall sheathing attachment can be checked, and structural sheathing can be added to shearwalls. Adding wall-to-foundation ties may also be possible.
- Exterior siding attachment can be improved with more fasteners at the time the exterior is re-coated.
- Windows, doors, skylights reinforcement and attachment can be improved whenever they are accessible.
- When windows and doors are replaced, glazing and framing can be used that is impact resistant and provides greater UV protection.
- Floor framing-to-beam connections can be improved whenever they are accessible.
- Beam-to-pile connections can be improved whenever they are accessible.
- At any time, deficient metal connectors that are accessible should be replaced with stainless steel or hot-dip-galvanized connectors.
- When HVAC equipment is replaced, the replacements should be more durable, so that they will last longer in a coastal environment, and they should be elevated to or above the Design Flood Elevation (DFE).
- Utility attachment can be improved when the outside equipment is replaced or relocated.
- At any time, in the attic space, straps should be added to rafters across the ridge beam, straps should be added from rafters to top wall plates, and gable wall framing should be braced. In addition, the uplift resistance of the roof sheathing can be increased through the application of adhesive at the roof sheathing-to-roof rafter joint (see Figure 14-6 for an illustration of the adhesive attachment).
- At any time, garage doors should be reinforced as shown in Figure 14-7, or replaced with new wind- and debris-resistant doors.
- At any time, metal light fixtures should be replaced with fixtures that have either wood or vinyl exteriors.
- At any time, carbon steel handrails should be replaced with vinyl coated, plastic, stainless steel, or wood handrails.

**Figure 14-6**
Typical 1/4-inch bead of adhesive between roof sheathing and roof rafter.

**Figure 14-7**
Garage door on south Florida house reinforced with 2x4 wood girts and metal mullions.

## 14.6 Retrofit Costs

Retrofit costs will vary widely from region to region because of one or more of the following:

- structural conditions in the building
- price variations of the material and labor
- familiarity with retrofit techniques on the part of the contractor
- building layout complexities
- age of the building

However, the cost to retrofit will normally fall within an expected range that is useful for estimating purposes. Before any work is performed, the contractor should be asked to provide a written detailed price proposal so that all costs can be verified. There are significant uncertainties in retrofitting that must be considered in estimating the cost and in defining the scope of a retrofitting project with a contractor. Table 14.3 presents expected ranges of costs for the retrofit opportunities listed in Section 14.5. **The costs are based on the assumption that each building element discussed has been exposed to provide easy access to the structure.**

**Table 14.3**
Cost Estimates for Retrofitting

| Retrofit Opportunity | Cost* |
|---|---|
| **Roof:** | |
| ☐ Reattach sheathing | \$0.05 - \$0.10/ft$^2$ |
| ☐ Install metal straps at ridge beam | \$5 – \$6/each |
| ☐ Brace gable wall framing | \$80 – \$100/gable end |
| ☐ Install adhesive between sheathing and rafters | \$0.10 - \$0.20/ft$^2$ |
| **Siding:** | |
| ☐ Reattach sheathing to shear wall | \$0.05 - \$0.10/ft$^2$ |
| ☐ Add sheathing to shear wall | \$0.65 - \$0.80/ft$^2$ |
| ☐ Add wall/foundation ties | \$6 – \$10/each |
| ☐ Install hurricane connector at truss/ wall connection | \$0.20 - \$0.30/ft$^2$ |
| **Replace glass with impact-resistant glazing** | \$15 - \$30/ft$^2$ |
| **Replace metal connectors** | \$5 – \$8/each |
| **Replace metal handrails** | \$10 – \$30/linear ft |
| **Replace metal light fixtures** | \$10 – \$30/fixture |
| **Reinforce garage doors** | \$150 – \$200/8-ft door |

* Estimates based on 1999 prices

## 14.7 References


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