Table 10-2. ASTM D2487-10 Soil Classifications (concluded)

Major Divisions		Group Symbol	Typical Names	Classification Criteria
Fine-grained soils: 50% or more passes No. 200 sieve	Silts and clay liquid limit 50% or less	ML	Inorganic silts, very fine sands, rock flout, silty or clayey fine sands	
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
Fine-grained soils: 50% or more passes No. 200 sieve	Silts and clay liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
		CH	Inorganic clays of high plasticity, fat clays	
		OH	Organic clays of medium to high plasticity	
Highly organic soils		PT	Peat, muck, and other highly organic soils	

Adapted, with permission, from ASTM D2487-10 *Standard Practice for Classification of Soils for Engineering Purposes* (Unified Soil Classification System), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. The complete standard is available at ASTM International, http://www.astm.org.

Angle of Internal Friction/Soil Friction Angle

The angle of internal friction is a measure of the soil's ability to resist shear forces without failure. Internal friction depends on soil grain size, grain size distribution, and mineralogy.

The angle of internal friction is used in the design of shallow and deep foundations. It is also used to determine the sliding resistance developed between the bottom of a footing and the foundation at the adjacent soil strata via Equation 10.1.

The following factors should be considered. The normal force includes only the weight of the building (dead load). Live loads should not be considered. Also, ASD load factors in ASCE 7-10 allow only 60 percent of the dead load of a structure to be considered when resisting sliding forces. Foundation materials exert less normal force on a foundation when submerged, so the submerged weight of all foundation materials below the design stillwater depth should be used.

Editions of the IBC contain presumptive coefficients of friction for various soil types (for example, coefficients of friction are contained in Table 1806.2 in the 2009 IBC). Those coefficients can be used in Equation 10.1 by substituting them for the term "tan (φ)."

Subgrade Modulus n_h

The subgrade modulus (n_h) is used primarily in the design of pile foundations. It, along with the pile properties, determines the depth below grade of the point of fixity (point of zero movement and rotation) of a pile under lateral loading.

EQUATION 10.1. SLIDING RESISTANCE

$$F = \tan(\varphi)(N)$$

where:

F = resistance to sliding (lb)

φ = angle of internal friction

N = normal force on the footing (lb)

The inflection point is critical in determining whether piles are strong enough to resist bending moments caused by lateral loads on the foundation and the elevated building. The point of fixity is deep for soft soils (low subgrade modulus) and stiff piles and shallow for stiff soils (high subgrade modulus) and flexible piles.

Subgrade moduli range from 6 to 150 pounds/cubic inch for soft clays to 800 to 1,400 pounds/cubic inch for dense sandy gravel. See Section 10.5.3 for more information on subgrade modulus.

10.4 Design Process

The following are the major steps in foundation analysis and design.

- Determine the flood zone that the building site is in. For a site that spans more than one flood zone (e.g., Zone V and Coastal A Zone, Coastal A Zone and Zone A), design the foundation for the most severe zone (see Chapter 3).
- Determine the design flood elevation and design stillwater elevation (see Chapter 8).
- Determine the projected long- and short-term erosion (see Chapter 8).
- Determine the site elevation and determine design stillwater depths (see Chapter 8).
- Determine flood loads including breaking wave loads, hydrodynamic loads, flood-borne debris loads, and hydrostatic loads. Buoyancy reduces the weight of all submerged materials, so hydrostatic loads need to be considered on all foundations (see Chapter 8).
- Obtain adequate soils data for the site (see Section 10.3.3).
- Determine maximum scour and erosion depths (see Chapter 8).
- Select foundation type (open/deep, open/shallow, closed/deep, or closed/shallow). Use open/deep foundations in Zone V and Coastal A Zone. Use open/shallow foundations in Coastal A Zone only when scour and erosion depths can be accurately predicted and when the foundation can extend beneath the erosion depths. See Sections 10.2 and 10.3.1.
- Determine the basic wind speed, exposure, and wind pressures (see Chapter 8). Determine live and dead loads and calculate all design loads on the elevated building and on the foundation elements (see Chapter 8).

- Determine forces and moments at the top of the foundation elements for all load cases specified in ASCE 7-10. Use load combinations specified in Section 2.3 for strength-based designs or Section 2.4 for stress-based designs. Apply forces and moments to the foundation.

- Design the foundation to resist all design loads and load combinations when exposed to maximum predicted scour and erosion.

10.5 Pile Foundations

Pile foundations are widely used in coastal environments and offer several benefits. Pile foundations are deep and, when properly imbedded, offer resistance to scour and erosion. Piles are often constructed of treated timber, concrete, or steel although other materials are also used.

Treated timber piles are readily available and because they are wood, they can be cut, sawn, and drilled with standard construction tools used for wood framing. ASTM D25-99 contains specifications on round timber piles including quality requirements, straightness, lengths and sizes (circumferences and diameters) as well as limitations on checks, shakes, and knots. The *National Design Specification for Wood Construction* (ANSI/AF&PA 2005) contains design values for timber piles that meet ASTM D25-99 specifications.

Pre-cast (and typically pre-stressed) concrete piles are not readily available in some areas but offer several benefits over treated timber piles. Generally, they can be fabricated in longer lengths than timber piles. For the same cross section, they are stronger than timber piles and are not vulnerable to rot or wood-destroying insects. The strength of concrete piles can allow them to be used without grade beams. Foundations without grade beams are less vulnerable to scour than foundations that rely on grade beams (See Section 10.5.6).

Steel piles are generally not used in residential construction but are common in commercial construction. Field connections are relatively straightforward, and since steel can be field drilled and welded, steel-to-wood and steel-to-concrete connections can be readily constructed. ASTM A36/A36M-08 contains specifications for mild (36 kip/square inch) steels in cast or rolled shapes. ASTM standards for other shapes and steels include:

- For steel pipe, ASTM A53/A53M-10, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless* (ASTM 2010c)

- For structural steel tubing, ASTM A500-10, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes* (ASTM 2010b); and ASTM A501-07, *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing* (ASTM 2007)

- For welded and seamless steel pipe piles, ASTM 252-10, *Standard Specification for Welded and Seamless Steel Pipe Piles* (ASTM 2010d)

Fiber-reinforced polymer (FRP) piles are becoming more commonplace in transportation and marine infrastructure but are rarely used in residential applications. However, the usage of FRP piles in residential applications is expected to increase. New construction materials can offer many benefits such as sustainability, durability, and longevity but like any new construction material, the appropriateness of FRP piles should be thoroughly investigated before being used in new applications. Although FRP is not discussed in the publication, Technical Fact Sheet 1.8, *Non-Traditional Building Materials and Systems,* in FEMA P-499 provides guidance on using new materials and new systems in coastal environments.

Table 10-3 is a summary of the advantages and special considerations for three of the more common pile materials.

Table 10-3. Advantages and Special Considerations of Three Types of Pile Materials

Material	Advantages	Special Considerations
Wood	• Comparatively low initial cost • Readily available in most areas • Easy to cut, saw and drill • Permanently submerged piles resistant to decay • Relatively easy to drive in soft soil • Suitable for friction and end bearing pile	• Difficult to splice • Subject to eventual decay when in soil or intermittently submerged in water • Vulnerable to damage from driving (splitting) • Comparatively low compressive load • Relatively low allowable bending stress
Concrete	• Available in longer lengths than wood piles • Corrosion resistant • Can be driven through some types of hard material • Suitable for friction and end-bearing piles • Reinforced piles have high bending strength • High bending strength allows taller or more heavily loaded pile foundations to be constructed without grade beams	• High initial cost • Not available in all areas • Difficult to make field adjustments for connections • Because of higher weight, require special consideration in high seismic areas
Steel	• High resistance to bending • Easy to splice • Available in many lengths, sections, and sizes • Can be driven through hard subsurface material • Suitable for friction and end-bearing piles • High bending strength, which allows taller or more heavily loaded pile foundations to be constructed without grade beams	• Vulnerable to corrosion • May be permanently deformed if struck by heavy object • High initial cost • Some difficulty with attaching wood framing

The critical aspects of pile foundations include the pile material and size and pile embedment depth. Pile foundations with inadequate embedment do not have the structural capacity to resist sliding and overturning (see Figure 10-2). Inadequate embedment and improperly sized piles greatly increase the probability for structural collapse. However, when properly sized, installed, and braced with adequate embedment into the soil (with consideration for erosion and scour effects), a building's pile foundation performance allows the building to remain standing and intact following a design flood event (see Figure 10-3).

10.5.1 Compression Capacity of Piles – Resistance to Gravity Loads

The compression capacity of piles determines their ability to resist gravity loads from the elevated structure they support. One source that provides an equation for the compression capacity of piles is the *Foundation and Earth Structures*, Design Manual 7.2 (USDN 1986). The manual contains Equation 10.2 for determining

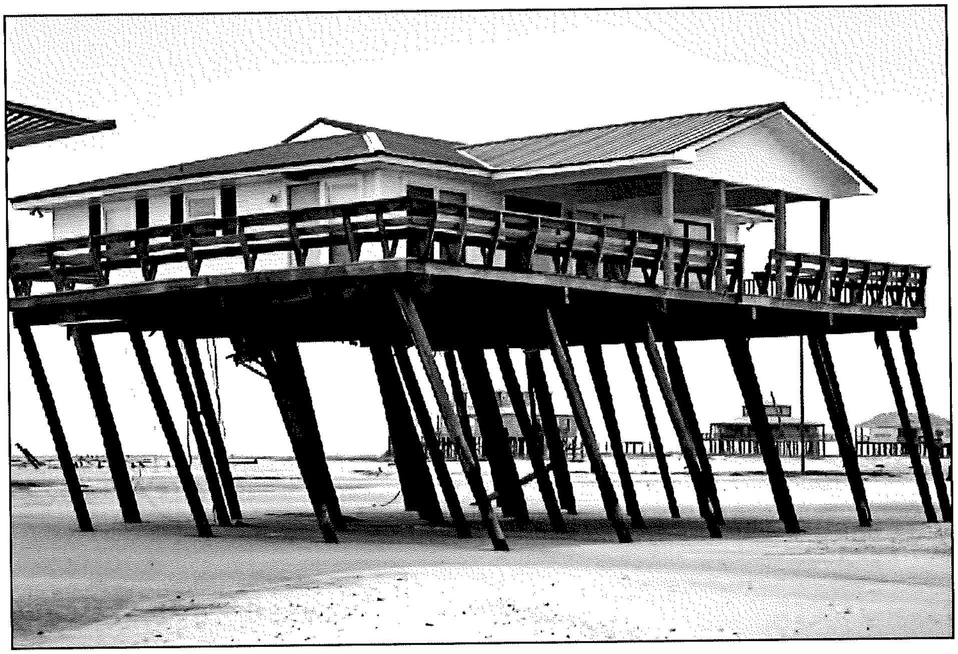


Figure 10-2.
Near collapse due to insufficient pile embedment, Hurricane Katrina (Dauphin Island, AL, 2005)

Figure 10-3.
Surviving pile foundation, Hurricane Katrina (Dauphin Island, AL, 2005)

the compression capacity of a single pile when placed in granular (non-cohesive) soils. Design Manual 7.2 also contains methods of determining compression capacity of a pile placed in cohesive soils.

The resistance of the pile is the sum of the capacity that results from end bearing and friction. The capacity from end bearing is the first term in Equation 10.2; the capacity from friction is given in the second term.

Equation 10.2 gives the ultimate compression capacity of a pile. The allowable capacity (Q_{allow}) used in ASD depends on a Factor of Safety applied to the ultimate capacity. For ASD, Design Manual 7.2 recommends a Factor of Safety of 3.0; thus, $Q_{allow} = Q_{ult}/3$.

EQUATION 10.2. ULTIMATE COMPRESSION CAPACITY OF A SINGLE PILE

$$Q_{ult} = P_T N_q A_T + \sum K_{HC} P_0 D s \tan\delta$$

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 (see Table 10-4)

A_T = area of pile tip (ft^2)

K_{HC} = earth pressure in compression (see Table 10-5)

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

δ = friction angle between pile and soil (see Table 10-6)

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

D = depth of embedment (ft)

Table 10-4. Bearing Capacity Factors (N_q)

Parameter	Pile Bearing Capacity Factors												
φ (degrees)[a]	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (driven pile displacement)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q (drilled piers)[b]	5	8	10	12	14	17	21	25	30	38	43	60	72

N_q = bearing capacity factor

φ = angle of internal friction

(a) Limit φ to 28° if jetting is used

(b) When a bailer or grab bucket is used below the groundwater table, calculate end bearing based on φ not exceeding 28 degrees. For piers larger than 24 inches in diameter, settlement rather than bearing capacity usually controls the design. For estimating settlement, take 50% of the settlement for an equivalent footing resting on the surface of comparable granular soils.

Table 10-5. Earth Pressure Coefficients

Pile Type	K_{HC}	K_{HT}
Driven single H-pile	0.5 – 1.0	0.3 – 0.5
Driven single displacement pile	1.0 – 1.5	0.6 – 1.0
Driven single displacement tapered pile	1.5 – 2.0	1.0 – 1.3
Driven jetted pile	0.4 – 0.9	0.3 – 0.6
Drilled pile (less than 24-inch diameter)	0.7	0.4

K_{HC} = earth pressure compression coefficient

K_{HT} = earth pressure tension coefficient

Table 10-6. Friction Angle Between Soil and Pile (δ)

Pile Type	δ
Timber	$\frac{3}{4}\varphi$
Concrete	$\frac{3}{4}\varphi$
Steel	20 degrees

φ = angle of internal friction

10.5.2 Tension Capacity of Piles

The tension capacity of piles determines their ability to resist uplift and overturning loads on the elevated structure. One source that provides pile capacity in tension load is the Design Manual 7.2, which is also a reference on compression capacity. Equation 10.3 determines the tension capacity in a single pile.

EQUATION 10.3. ULTIMATE TENSION CAPACITY OF A SINGLE PILE

$$T_{ult} = \sum K_{HT} P_0 D s \tan\delta$$

where:

T_{ult} = ultimate load capacity in tension (lb)

K_{HT} = earth pressure in tension (see Table 10-5)

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

δ = friction angle between pile and soil (see Table 10-6)

s = surface area of pile per unit length (ft^2/ft or ft)

D = depth of embedment (ft)

Note: *With the recommended Factor of Safety of 3.0, the allowable tension capacity,* $T_{allow} = T_{ult}/3$.

The Design Manual 7.2 provides tables to identify bearing capacity factors (N_q), earth pressure coefficients (K_{HC} and K_{HT}), and friction angle between pile and soil (δ) based on pile type and the angle of internal friction (φ) of the soil.

Example 10.1 illustrates compression and tension capacity calculations for a single pile not affected by scour or erosion.

Table 10-7 contains example calculations using Equations 10.2 and 10.3 for the allowable compression (gravity loading) and tension (uplift) capacities of wood piles for varying embedments, pile diameters, and installation methods. The table also illustrates the effect of scour around the pile on the allowable compression and tension loads. Scour (and erosion) reduces pile embedment and therefore pile capacity. For this table, a scour depth of twice the pile diameter ($2d$) with no generalized erosion is considered.

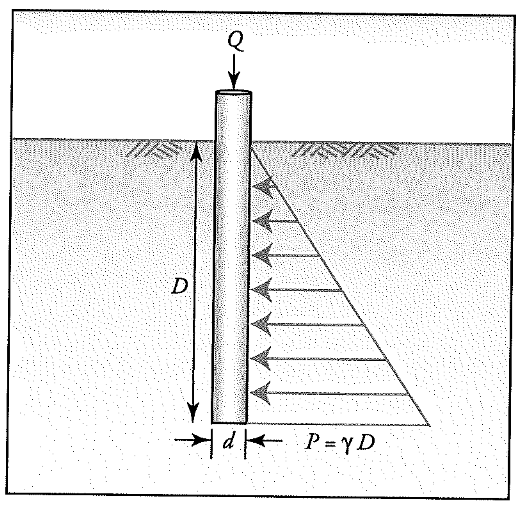
EXAMPLE 10.1. CALCULATION FOR ALLOWABLE CAPACITIES OF WOOD PILES

Given:

- Closed end, driven timber pile
- Diameter *(d)* = 1 ft
- Depth of embedment *(D)* = 15 ft
- Soil density (γ) = 65 lb/ft^3
- Angle of internal friction (φ) = 30 K_{HC}= 1.0 (applicable coefficient from Table 10-5)
- Earth pressure in tension (K_{HT}) = 0.6 (applicable coefficient from Table 10-5)
- Bearing capacity factor (N_q) = 21 (applicable coefficient from Table 10-4)
- Factor of Safety = 3.0

Find:

1. Allowable tension and compression capacities of wood piles embedded in soil



Q = load
D = length of pile
d = diameter of pile
P = pressure
γ = soil density

Illustration A.
Pile schematic and pressure diagram

EXAMPLE 10.1. CALCULATION FOR ALLOWABLE CAPACITIES OF WOOD PILES (concluded)

Solution for #1: Find the allowable tension and compression capacity of the wood pile embedded in soil as follows:

- To determine the resultant pressure from the soil on the pile:

$$\delta = \frac{3}{4}(\varphi) = \frac{3}{4}(30^\circ) = 22.5^\circ$$

$$P_0 = P_t = \gamma D = (65 \text{ lb/ft}^3)(15 \text{ ft}) = 975 \text{ lb/ft}^2$$

- Geometrical properties of the pile surfaces upon which pressure from the soil is applied to the pile are:

$$A_t = (\pi)(\frac{1}{2}d)^2 = (3.14)[(0.5)(1 \text{ ft})]^2 = 0.785 \text{ ft}^2$$

$$P_0 = P_t = \gamma D = (65 \text{ lb/ft}^3)(15 \text{ ft}) = 975 \text{ lb/ft}^2$$

Allowable compression capacity:

$$Q_{ult} = (975 \text{ lb / ft}^2)(21)(0.785 \text{ ft}^2) + (1.0)(975 \text{ lb/ft}^2)(\tan 22.5^\circ)(3.14 \text{ ft}^2\text{/ft})(15 \text{ ft})$$

$$Q_{ult} = 35{,}095 \text{ lb}$$

$$Q_{all} = \frac{Q_{ult}}{3} = \frac{35{,}095 \text{ lb}}{3} = \mathbf{11{,}698 \text{ lb}}$$

Allowable tension capacity:

$$T_{ult} = (0.6)(975 \text{ lb/ft}^2)(\tan 22.5^\circ)(3.14 \text{ ft}^2\text{/ft})(15 \text{ ft}) = 11{,}413 \text{ lb}$$

$$T_{all} = \frac{T_{ult}}{3} = \frac{11{,}413 \text{ lb}}{3} = \mathbf{3{,}804 \text{ lb}}$$

The purpose of Table 10-7 is to illustrate the effects of varying diameters, depths of embedment, and installation methods on allowable capacities. See Section 10.5.4 for information of installation methods. Example calculations used to determine the values in Table 10-7 are used in Example 10.1. The values in Table 10-7 are not intended to be used for design purposes.

Table 10-7. Allowable Compression and Tension of Wood Piles Based on Varying Diameters, Embedments, and Installation Methods

Diameter and Embedment	Installation Method	Compression (pounds) No Scour	Compression (pounds) 2d Scour	Tension (pounds) No Scour	Tension (pounds) 2d Scour
d = 12 inches D = 15 feet	Driven	11,698	9,406	3,804	2,857
	Jetted	7,894	6,548	1,902	1,429
	Augered	6,990	5,545	2,536	1,905
d = 12 inches D = 20 feet	Driven	18,416	15,560	6,763	5,478
	Jetted	11,652	10,081	3,382	2,739
	Augered	11,292	9,453	4,509	3,652
d =10 inches D = 15 feet	Driven	9,004	7,482	3,170	2,505
	Jetted	5,834	4,977	1,585	1,252
	Augered	5,470	4,497	2,114	1,670

d = diameter
D = depth of embedment

10.5.3 Lateral Capacity of Piles

The lateral capacity of piles is dictated by the piles and the pile/soil interface. The ability of the pile to resist lateral loads depends on the pile size and material, the soil properties, and on presence or absence of pile bracing.

One of the critical aspects of pile design is the distance between the lateral load application point and the point of fixity of the pile. That distance constitutes a moment arm and governs how much bending moment develops when a pile is exposed to lateral loads. For a foundation to perform adequately, that moment must be resisted by the pile without pile failure.

Equation 10.4 determines the distance between the point where the lateral load is applied and the point of fixity for an unbraced pile. Note that in Equation 10.4, "d" is the depth below grade of the point of fixity, not the diameter of the pile. Also, see Figure 10-4 for the deflected shape of a laterally loaded pile.

Table 10-8 lists recommended values for n_h, modulus of subgrade reaction, for a variety of soils (Bowles 1996). For wood pilings, the depths to points of fixity range from approximately 1 foot in stiff soils to approximately 5 feet in soft soils.

The ability of site soils to resist lateral loads is a function of the soil characteristics, their location on the site, and their compressive strength. Chapter 7 of the *Timber Pile Design and Construction Manual* (Collin 2002) contains methods of determining the lateral resistance of timber piles for both fixed pile head conditions (i.e., piles used with grade beams or pile caps) and free pile head conditions (i.e., piles free to rotate at their top). The manual also contains methods of approximating lateral capacity and predicting pile capacity when detailed soils data are known.

EQUATION 10.4. LOAD APPLICATION DISTANCE FOR AN UNBRACED PILE

$$L = H + \frac{d}{12}$$

where:

L = distance between the location where the lateral force in applied and the point of fixity (i.e., moment arm) (ft)

d = depth from grade to inflection point (inches); $d = 1.8\left(\frac{EI}{n_h}\right)^{\frac{1}{5}}$

H = distance above eroded ground surface (including localized scour) where lateral load is applied (ft)

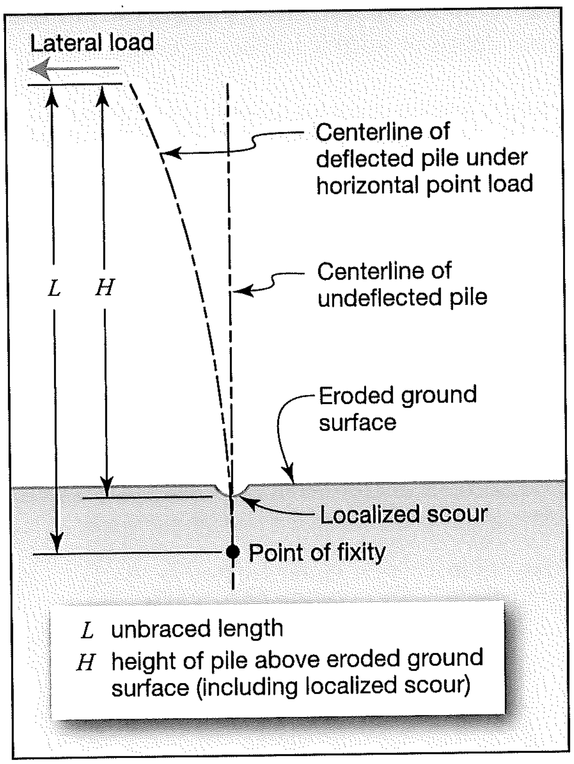


Figure 10-4.
Deflected pile shape for an unbraced pile

Table 10-8. Values of n_h, Modulus of Subgrade Reaction

Soil Type	n_h, Modulus of Subgrade Reaction (pound/cubic inch)
Dense sandy gravel	800 to 1,400
Medium dense coarse sand	600 to 1,200
Medium sand	400 to 1,000
Fine to silty fine sand	290 to 700
Medium clay (wet)	150 to 500
Soft clay	6 to 150

10.5.4 Pile Installation

Methods for installing piles include driving, augering, and jetting. A combination of methods may also be used. For example, piles may be placed in augered holes and then driven to their final depth. Combining installation methods can increase the achievable embedment depth. With increased depths, a pile's resistance to lateral and vertical loads can be increased, and its vulnerability to scour and erosion will be reduced.

- ***Driving*** involves hitting the top of the pile with a pile driver or hammer until the pile reaches the desired depth or it is driven to refusal. Piles can be driven with vibratory hammers. Vibratory hammers generate vertical oscillating movements that reduce the soil stress against the pile and which makes 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 equations to determine capacity.

- ***Augering*** involves placing the pile into a pre-drilled hole typically made with an auger. The augured hole can be the full diameter of the pile or a smaller diameter than the pile. Pre-drilling is completed to a predetermined depth, which often is adjusted for the soils found on the site. After placing the pile into the pre-drilled hole, the pile is then driven to its final desired depth or until it reaches refusal.

- ***Jetting*** is similar to augering but instead of using a soils auger, jetting involves using a jet of water (or air) to remove soils beneath and around the pile. Like augering, jetting is used in conjunction with pile driving.

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 10-5 illustrates the three pile installation methods. Table 10-9 lists advantages and special considerations for each method.

Figure 10-5.
Pier installation methods

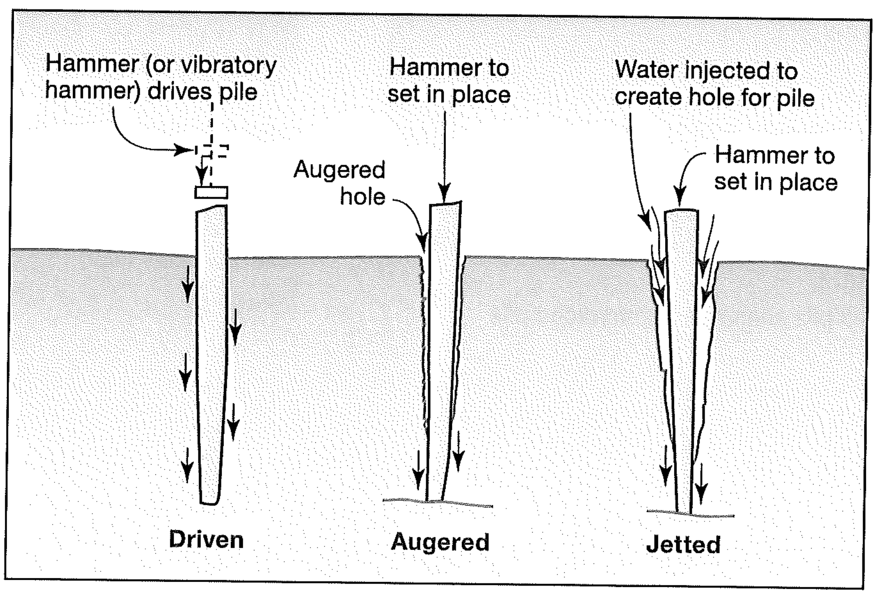


Table 10-9. Advantages and Special Considerations of Pile Installation Methods

Installation Method	Advantages	Special Considerations
Driving	• Well-suited for friction piles • Common construction practice • Pile capacity can be determined empirically	• Requires subsurface investigation • May be difficult to reach terminating soil strata if piles are only driven • Difficult to maintain plumb during driving and thus maintain column lines
Augering	• Economical • Minimal driving vibration to adjacent structures • Well-suited for end bearing • Visual inspection of some soil stratum possible • Convenient for low headroom situations • Easier to maintain column lines	• Requires subsurface investigation • Not suitable for highly compressed material • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients K_{HC} and K_{HT} to 40 percent of that driven for piles • Capacity must be determined by engineering judgment or load test
Jetting	• Minimal driving vibration to adjacent structures • Well-suited for end bearing piles • Easier to maintain column lines	• Requires subsurface investigation • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients K_{HC} and K_{HT} to 40 percent of that driven for piles • Capacity must be determined by engineering judgment or load test

K_{HC} = earth pressure compression coefficient
K_{HT} = earth pressure tension coefficient

10.5.5 Scour and Erosion Effects on Pile Foundations

Coastal homes are often exposed to scour and erosion, and because moving floodwaters cause both scour and erosion, it is rare for an event to produce one and not the other. As Figure 10-6 illustrates, scour and erosion have a cumulative effect on pile foundations. They both reduce piling embedment.

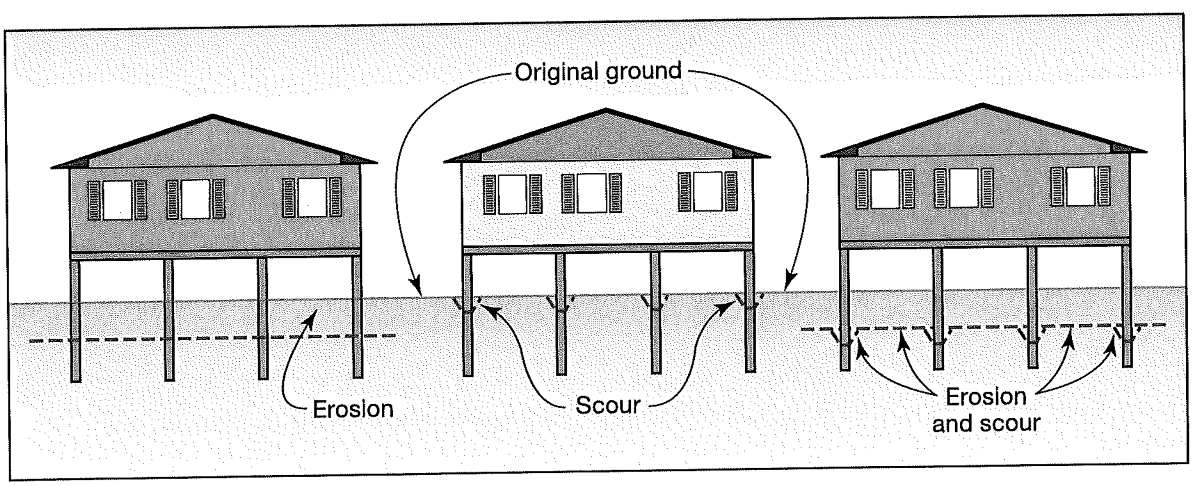


Figure 10-6.
Scour and erosion effects on piling embedment

A properly designed pile foundation must include a consideration of the effects of scour and erosion on the foundation system. Scour washes away soils around the piling, reducing pile embedment, and increases stresses within the pile when the pile is loaded. The reduced embedment can cause the foundation to fail at the pile/soil interface. The increased stresses can cause the pile itself to fracture and fail.

Erosion is even more damaging. In addition to reducing pile embedment depths and increasing stresses on piles, erosion increases the flood forces the foundation must resist by increasing the stillwater depth at the foundation that the flood produces. Pile foundations that are adequate to resist flood and wind forces without being undermined by scour and erosion can fail when exposed to even minor amounts of scour and erosion.

An example analysis of the effects of scour and erosion on a foundation is provided in *Erosion, Scour, and Foundation Design* (FEMA 2009a), published as part of Hurricane Ike Recovery Advisories and available at http://www.fema.gov/library/viewRecord.do?id=3539.

The structure in the example is a two-story house with 10-foot story heights and a 32-foot by 32-foot foundation. The house is away from the shoreline and elevated 8 feet above grade on 25 square timber piles spaced 8 feet apart. Soils are medium dense sands. The house is subjected to a design wind event with a 130-mph (3-second gust) wind speed and a 4-foot stillwater depth above the uneroded grade, with storm surge and broken waves passing under the elevated building.

Lateral wind and flood loads were calculated in accordance with ASCE 7-05. Although the wind loads in ASCE 7-10 vary from ASCE 7-05 somewhat, the results of the analyses do not change significantly. Piles were analyzed under lateral wind and flood loads only; dead, live, and wind uplift loads were neglected. If the neglected loads are included, deeper pile embedment and possibly larger piles than the results of the analysis indicated may be needed. Three timber pile sizes (8-inch square, 10-inch square, and 12-inch square) were evaluated using pre-storm embedment depths of 10 feet, 15 feet, and 20 feet and five erosion and scour conditions (erosion = 0 or 1 foot; scour = 2.0 times the pile diameter to 4.0 times the pile diameter).

The results of the analysis are shown in Table 10-10. A shaded cell indicates that the combination of pile size, pre-storm embedment, and erosion/scour does not provide the bending resistance and/or embedment required to resist lateral loads. The reason for foundation failure is indicated in each shaded cell ("P" for failure due to bending and overstress within the pile and "E" for an embedment failure from the pile/soil interaction). "OK" indicates that the bending and foundation embedment criteria are both satisfied by the particular pile size/pile embedment/erosion-scour combination.

The key points from the example analysis are as follows:

- Scour and erosion can cause pile foundations to fail and must be considered when designing pile foundations.
- Failures can result from either overloading the pile itself or from overloading at the pile/soil interface.
- Increasing a pile's embedment depth does not offset a pile with a cross section that is too small or pile material that is too weak.
- Increasing a pile's cross section (or its material strength) does not compensate for inadequate pile embedment.

Table 10-10. Example Analysis of the Effects of Scour and Erosion on a Foundation

Pile Embedment Before Erosion and Scour	Erosion and Scour Conditions	Pile Diameter (a)		
		8 inches	10 inches	12 inches
		Reason for Failure		
10 feet	Erosion = 0, Scour = 0	P, E	E	OK
	Erosion = 1 foot, Scour = 2.0a	P, E	E	E
	Erosion = 1 foot, Scour = 2.5a	P, E	E	E
	Erosion = 1 foot, Scour = 3.0a	P, E	E	E
	Erosion = 1 foot, Scour = 4.0a	P, E	P, E	E
15 feet	Erosion = 0, Scour = 0	P	OK	OK
	Erosion = 1 foot, Scour = 2.0a	P	OK	OK
	Erosion = 1 foot, Scour = 2.5a	P	OK	OK
	Erosion = 1 foot, Scour = 3.0a	P	OK	OK
	Erosion = 1 foot, Scour = 4.0a	P, E	P, E	E
20 feet	Erosion = 0, Scour = 0	P	OK	OK
	Erosion = 1 foot, Scour = 2.0a	P	OK	OK
	Erosion = 1 foot, Scour = 2.5a	P	OK	OK
	Erosion = 1 foot, Scour = 3.0a	P	OK	OK
	Erosion = 1 foot, Scour = 4.0a	P	P	OK

Two-story house supported on square timber piles and located away from the shoreline, storm surge and broken waves passing under the building, 130-mph wind zone, soil = medium dense sand.

a = pile diameter

E = foundation fails to meet embedment requirements

OK = bending and foundation embedment criteria are both satisfied by the particular pile size/pile embedment/erosion-scour combination

P = foundation fails to meet bending

10.5.6 Grade Beams for Pile Foundations

Piles can be used with or without grade beams or pile caps. Grade beams create resistance to rotation (also called "fixity") at the top of the piles and provide a method to accommodate misalignment in piling placement. When used with grade beams, the piles and foundation elements above the grade beams work together to elevate the structure, provide vertical and lateral support for the elevated home, and transfer loads imposed on the elevated home and the foundation to the ground below.

Pile and grade beam foundations should be designed and constructed so that the grade beams act only to provide fixity to the foundation system and not to support the lowest elevated floor. If grade beams support the lowest elevated floor of the home, they become the lowest horizontal structural member and significantly higher flood insurance premiums would result. Grade beams must also be designed to span between adjacent piles, and the piles must be capable of resisting both the weight of the grade beams when undermined by erosion and scour and the loads imposed on them by forces acting on the structure.

Pile foundations with grade beams must be constructed with adequate strength to resist all lateral and vertical loads. Failures during Hurricane Katrina often resulted from inadequate connections between the columns and footings or grade beams below (see Figure 10-7).

If grade beams are used with wood piles, the potential for rot must be considered when designing the connection between the grade beam and the pile. The connection must not encourage water retention. The maximum bending moment in the piles occurs at the grade beams, and decay caused by water retention at critical points in the piles could induce failure under high-wind or flood forces.

While offering some advantages, grade beams can become exposed by moving floodwaters if they are not placed deeply enough. Once exposed, the grade beams create large horizontal obstructions in the flood path that significantly increase scour. Extensive scour was observed after Hurricane Ike in 2008 around scores of homes constructed with grade beams (see Figure 10-8).

Although not possible for all piling materials, foundations should be constructed without grade beams whenever possible. For treated timber piles, this can limit elevations to approximately 8 feet above grade. The actual limit depends greatly on flood forces, number of piles, availability of piles long enough to be driven to the required depth and extend above grade enough to adequately elevate the home, and wind speed and geometry of the elevated structure. For steel and concrete piles, foundations without grade beams are practical in many instances, even for taller foundations. Without grade beams to account for pile placement, additional attention is needed for piling alignment, and soils test are needed for design because pile performance depends on the soils present, and presumptive piling capacities may not adequately predict pile performance.

Figure 10-7.
Column connection failure, Hurricane Katrina (Belle Fontaine Point, Jackson County, MS, 2005)

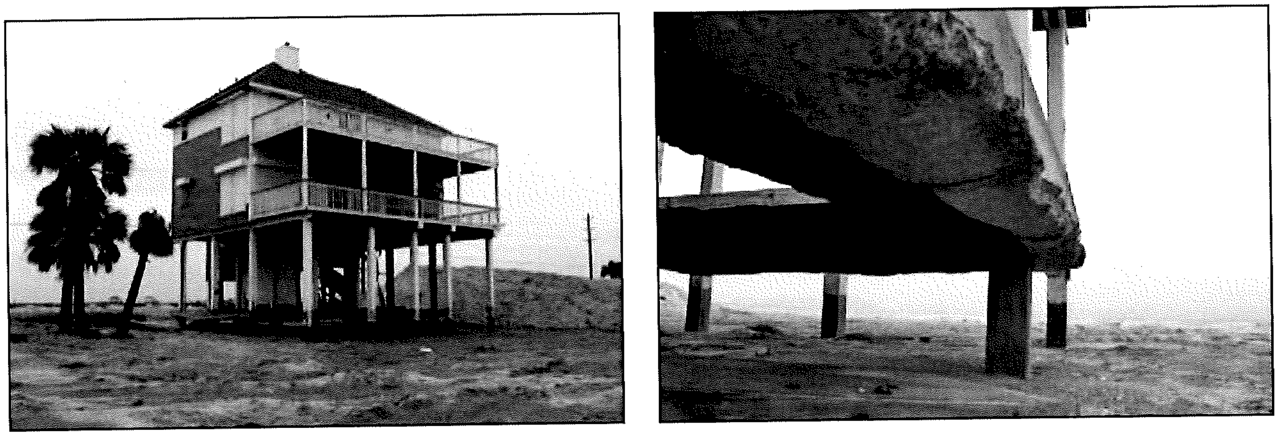


Figure 10-8.
Scour around grade beam, Hurricane Ike (Galveston Island, TX, 2008)

10.6 Open/Deep Foundations

In this section, some of the more common types of open/deep foundation styles are discussed. Treated timber pile foundations are discussed in Section 10.6.1, and other types of open/deep pile foundations are discussed in Section 10.6.2.

10.6.1 Treated Timber Pile Foundations

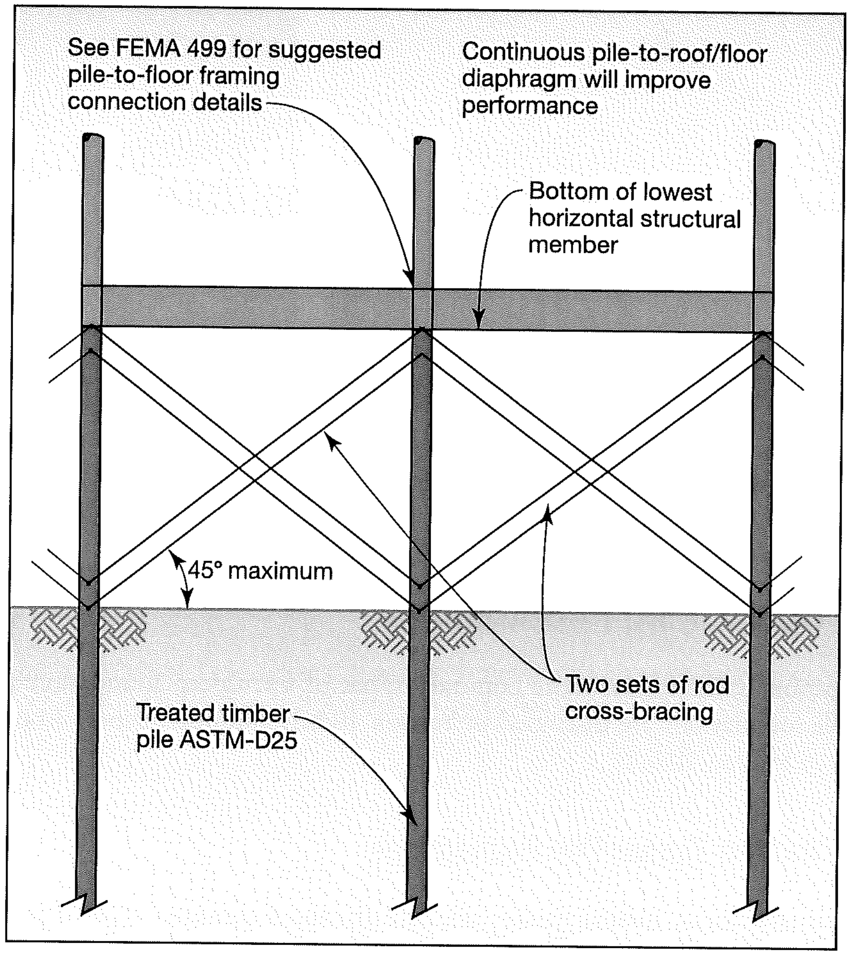
In many coastal areas, treated timber piles are the most common type of an open/deep style foundation. Timber piles are the first choice of many builders because they are relatively inexpensive, readily available, and relatively easy to install. The driven timber pile system (see Figure 10-9) is suitable for moderate elevations. Home elevations greater than 10 feet may not be practical because of pile length availability, the pile strength required to resist lateral forces (particularly when considering erosion and scour), and the pile embedment required to resist lateral loads after being undermined by scour and erosion.

When used without grade beams, timber piles typically extend from the pile tip to the lowest floor of the elevated structure. With timber piles and wood floor framing, the connection of the elevated structure to the piling is essentially a pinned connection because moment resisting connections in wood framing are difficult to achieve. Pinned connections do not provide fixity and require stronger piles to resist the same loads as piles that benefit from moment resisting connections at their tops.

Improved performance can be achieved if the piles extend beyond the lowest floor to the roof (or an upper floor level). Doing so provides resistance to rotation where the pile passes through the first floor. This not only reduces stresses within the piles but also increases the stiffness of the pile foundation and reduces movement under lateral forces. Extending piles in this fashion improves survivability of the building.

The timber pile system is vulnerable to flood-borne debris. During a hurricane event, individual piles can be damaged or destroyed by large, floating debris. Two ways of reducing this vulnerability are (1) using piles with diameters that are larger than those called for in the foundation design and (2) using more piles and continuous beams that can redistribute loads around a damaged pile. Using more piles and continuous beams increases structural redundancy and can improve building performance.

Figure 10-9.
Profile of timber pile foundation type



FEMA P-550, *Recommended Residential Construction for Coastal Areas* (FEMA 2006), contains a foundation design using driven timber piles. The foundation design is based on presumptive piling capacities that should be verified prior to construction. Also, the design is intended to support an elevated building with a wide range of widths and roof slopes and as such contains some inherent conservatism in the design. Design professionals who develop foundation designs for specific buildings and have site information on subsurface conditions can augment the FEMA P-550 design to provide more efficient designs that reduce construction costs.

10.6.1.1 Wood Pile-to-Beam Connections

In pile foundations that support wood-framed structures, systems of perimeter and interior beams are needed to support the floors and walls above. Beams must be sized to support gravity loads and, in segmented shear wall construction, resist reactions from shear wall segments. To transfer those loads to the foundation, wood piles are often notched to provide a bearing surface for the beams. Notches should not reduce the pile cross section by more than 50 percent (such information is typically provided by a design professional on contract documents). For proper transfer of gravity loads, beams should bear on the surface of the pile notch.

Although connections play an integral role in the design of structures, they are typically regarded as the weakest link. Guidance for typical wood-pile to wood-girder connections can be found in Fact Sheet 3.3, *Wood Pile to Beam Connections*, in FEMA P-499.

10.6.1.2 Pile Bracing

When timber piles with a sufficiently large cross section are not available, timber piles may require bracing to resist lateral loads. Bracing increases the lateral stiffness of a pile foundation system so that less sway is felt under normal service loads. Bracing also lowers the location where lateral forces are applied to individual piles and reduces bending stresses in the pile. When bracing is used, the forces from moving floodwaters and from flood-borne debris that impacts the braces should be considered.

NOTE

Fact Sheet 3.2, *Pile Installation,* in FEMA P-499 recommends that pile bracing be used only for reducing the structure's sway and vibration for comfort. In other words, bracing should be used to address serviceability issues and not strength issues. The foundation design should consider the piles as being unbraced as the condition that may occur when floating debris removes or damages the bracing. If the pile foundation is not able to provide the desired strength performance without bracing, the designer should consider increasing the pile size.

Bracing is typically provided by diagonal bracing or knee bracing. Diagonal bracing is more effective from a structural standpoint, but because diagonal bracing extends lower into floodwaters, it is more likely to be damaged by flood-borne debris. It can also trap flood-borne debris, and trapped flood-borne debris increases flood forces on the foundation.

Knee bracing does not extend as deeply into floodwaters as cross bracing and is less likely to be affected by flood-borne debris but is less effective at reducing stresses in the pile and also typically requires much stronger connections to achieve similar structural performance as full-length cross bracing.

Diagonal Bracing

Diagonal bracing often consists of dimensional lumber that is nailed or bolted to the wood piles. Steel rod bracing and wire rope (cable) bracing can also be used. Steel rod bracing and cable bracing have the benefit of being able to use tensioning devices, such as turnbuckles, which allow the tension of the bracing to be maintained. Cable bracing has an additional benefit in that the cables can be wrapped around pilings without having to rely on bolted connections, and wrapped connections can transfer greater loads than bolted connections. Figure 10-10 shows an example of diagonal bracing using dimensional lumber.

Diagonal braces tend to be slender, and slender braces are vulnerable to compression buckling. Most bracing is therefore considered tension-only bracing. Because wind and flood loads can act in opposite directions, tension-only bracing must be installed in pairs. One set of braces resists loads from one direction, and the second set resists loads from the opposite direction. Figure 10-11 shows how tension-only bracing pairs resist lateral loads on a home.

The placement of the lower bolted connection of the diagonal brace to the pile requires some judgment. If the connection is too far above grade, the pile length below the connection is not braced and the overall foundation system is less strong and stiff.

Figure 10-10.
Diagonal bracing using dimensional lumber

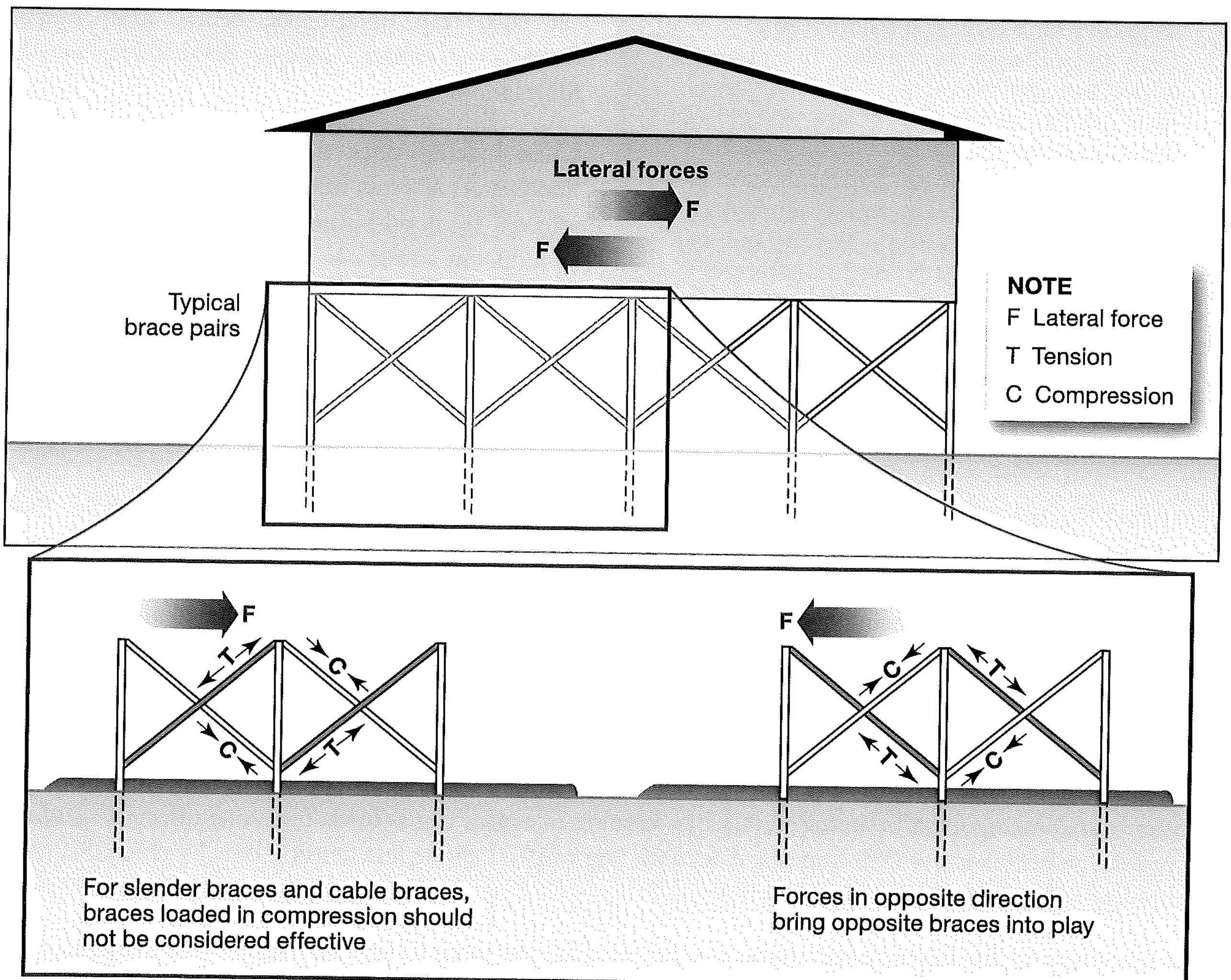


Figure 10-11.
Diagonal bracing schematic

EXAMPLE 10.2. DIAGONAL BRACE FORCE

Given:

- Lateral load = 989 lb
- Brace angle = 45

Find:

1. Tension force in the diagonal brace in Illustration A.

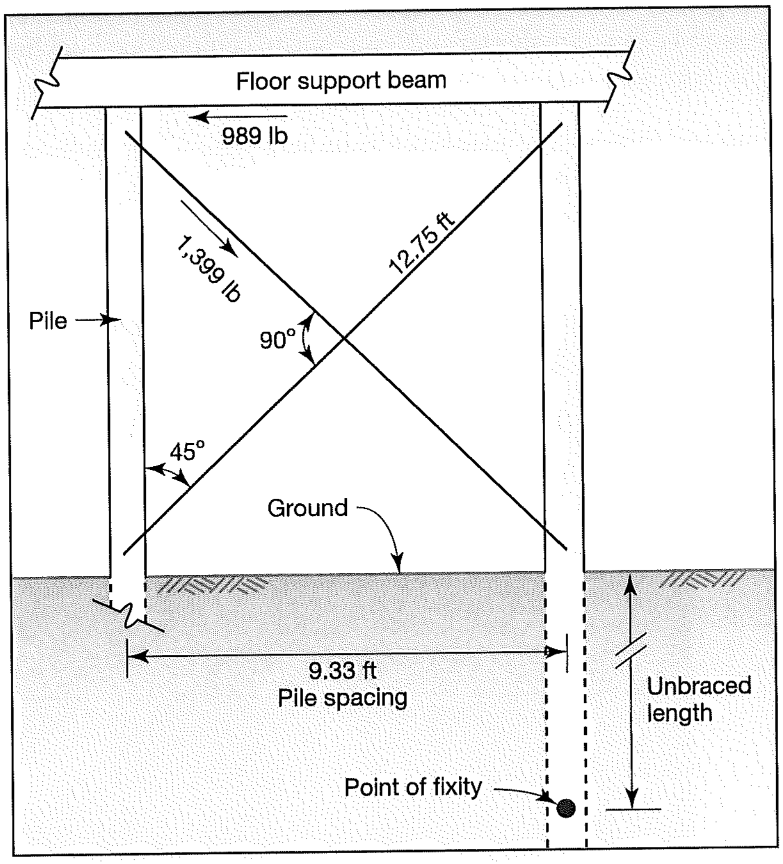


Illustration A. Force diagram for diagonal bracing

Solution for #1: The tension force in the diagonal brace can be found as follows:

Rod bracing is used and assumed to act in tension only because of the rigidity of the rod brace in tension and lack of stiffness of the rod in compression.

- The tension brace force is calculated as follows:

$$T_{diagonal} = \frac{989 \text{ lb}}{\cos 45°} = \mathbf{1{,}399\ lb}$$

Interaction of the soil and the pile should be checked to ensure that the uplift component of the brace force can be resisted.

For timber piles, if the connection is too close to grade, the bolt hole is more likely to be flooded and subject to decay or termite infestation, which can weaken the pile at a vulnerable location. All bolt holes should be treated with preservative after drilling and prior to bolt placement.

NOTE

Bolt holes in timber piles should be field-treated (see Chapter 11).

Knee Bracing

Knee braces involve installing short diagonal braces between the upper portions of the pilings and the floor system of the elevated structure (see Figure 10-12). The braces increase the stiffness of an elevated pile foundation and can contribute to resisting lateral forces. Although knee braces do not stiffen a foundation as much as diagonal bracing, they offer some advantages over diagonal braces. For example, knee braces present less obstruction to waves and debris, are shorter and less prone to compression buckling than diagonal braces, and may be designed for both tension and compression loads.

The entire load path into and through the knee brace must be designed. The connections at each end of each knee brace must have sufficient capacity to handle both tension and compression and to resist axial loads in the brace. The brace itself must have sufficient cross-sectional area to resist compression and tensile loads.

Figure 10-12.
Knee bracing

The feasibility of knee bracing is often governed by the ability to construct strong connections in the braces that connect the wood piles to the elevated structure.

10.6.1.3 Timber Pile Treatment

Although timber piles are chemically treated to resist rot and damage from insects, they can be vulnerable to wood-destroying organisms such as fungi and insects if the piles are subject to both wetted and dry conditions. If the piles are constantly submerged, fungal growth and insect colonies cannot be sustained; if only periodically submerged, conditions exist that are sufficient to sustain wood-destroying organisms. Local design professionals familiar with the performance of driven, treated timber piles can help quantify the risk. Grade beams can be constructed at greater depths or alternative pile materials can be selected if damage from wood-destroying organisms is a major concern.

Cutting, drilling, and notching treated timber piles disturb portions of the piles that have been treated for rot and insect damage. Because pressure-preservative-treated piles, timbers, and lumber are used for many purposes in coastal construction, the interior, untreated parts of the wood can be exposed to possible decay and infestation. Although treatments applied in the field are much less effective than factory treatments, the potential for decay can be minimized with field treatments. AWPA M4-06 describes field treatment procedures and field cutting restrictions for poles, piles, and sawn lumber.

Field application of preservatives should be done in accordance with the instructions on the label, but if instructions are not provided, dip soaking for at least 3 minutes is considered effective. When dip soaking for 3 minutes is impractical, treatment can be accomplished by thoroughly brushing or spraying the exposed area. The preservative is absorbed better at the end of a member or end grains than on the sides or side grains. To safeguard against decay in bored holes, the preservative should be poured into the holes. If the hole passes through a check (such as a shrinkage crack caused by drying), the hole should be brushed; otherwise, the preservative will run into the check instead of saturating the hole.

Copper naphthenate is the most widely used preservative for field treatment. Its color (deep green) may be objectionable aesthetically, but the wood can be painted with alkyd paints after extended drying. Zinc naphthenate is a clear alternative to copper naphthenate but is not as effective in preventing insect infestation and should not be painted with latex paints. Tributyltin oxide 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 rapidly leaches out when water is present. Sodium borate is therefore not recommended. Waterborne arsenicals, pentachlorophenol, and creosote are unacceptable for field applications.

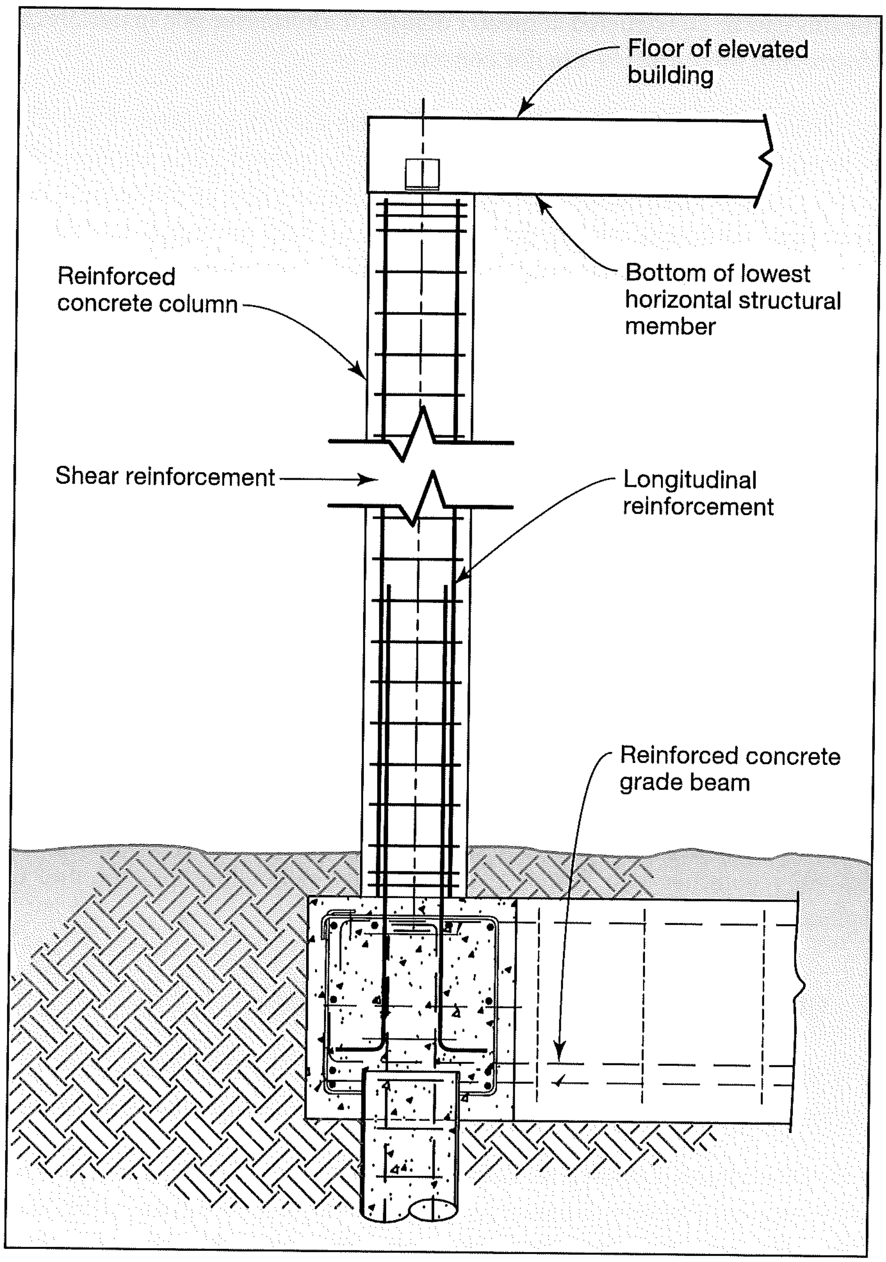
10.6.2 Other Open/Deep Pile Foundation Styles

Several other styles of pile foundations, in addition to treated timber piles, are used although their use often varies geographically depending on the availability of materials and trained contractors.

FEMA P-550 contains foundation designs that use deep, driven steel and treated timber piles and grade beams that support a system of concrete columns. The second edition of FEMA P-550 (FEMA 2006) added a new design for treated timber piles that incorporates elevated reinforced beams constructed on the concrete columns. In the new design, the elevated beams work with the columns and grade beams to create reinforced concrete portal frames that assist in resisting lateral loads. The elevated beams also create a suitable platform that can support a home designed to a prescriptive standard such as *Wood Framed Construction Manual for One- and Two-Family Dwellings* (AF&PA 2012) or ICC 600-2008.

Figure 10-13 shows one of the deep pile foundation systems that uses treated timber piles and grade beams. The steel pipe pile and grade beam foundation system contained in FEMA P-550 is similar but requires fewer piles because the higher presumptive strength of the steel piles compared to the timber piles. Figure 10-14 shows the foundation system added in the Second Edition of FEMA P-550 (FEMA 2009b), which incorporates an elevated concrete beam.

Figure 10-13. Section view of a steel pipe pile with concrete column and grade beam foundation type
DEVELOPED FROM FEMA P-550, CASE B



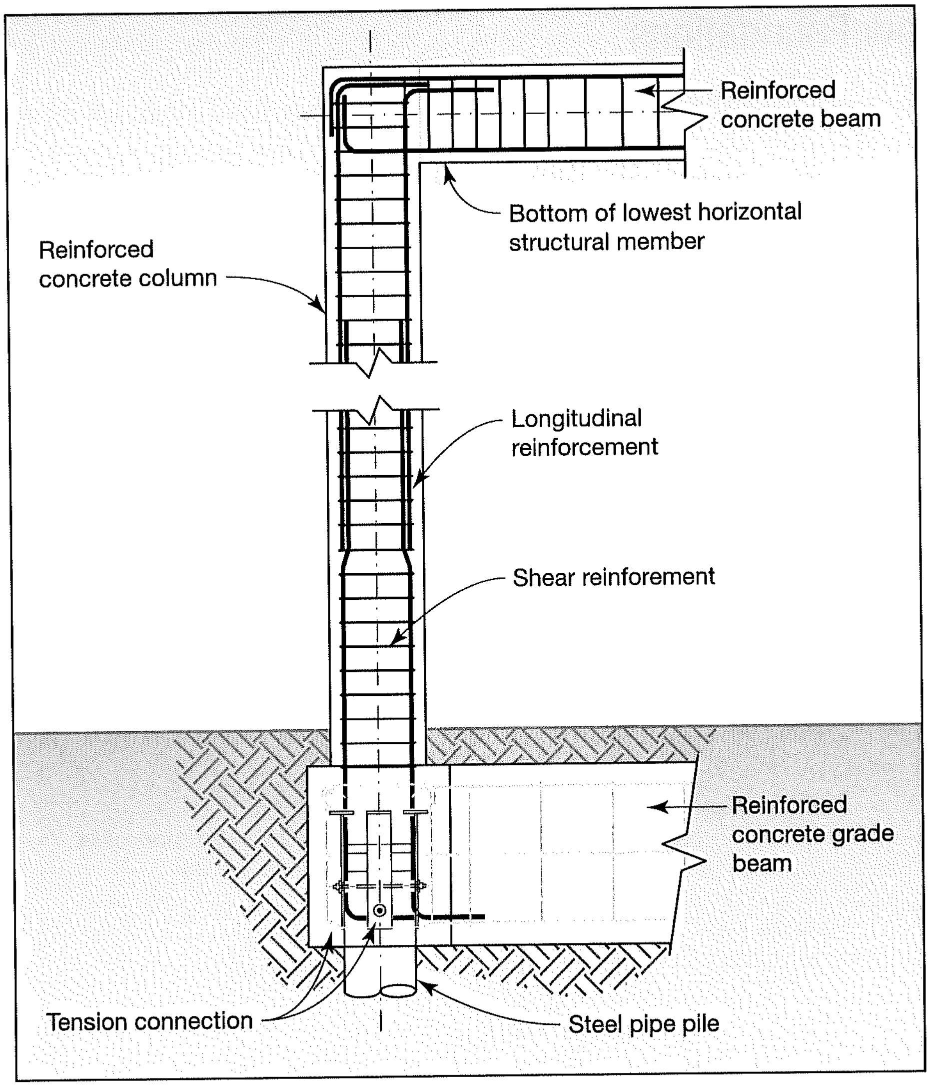


Figure 10-14.
Section view of a foundation constructed with reinforced concrete beams and columns to create portal frames
SOURCE: ADAPTED FROM FEMA P-550, SECOND EDITION, CASE H

The grade beams that are shown in Figures 10-13 and 10-14 should not be used as structural support for a concrete slab that is below an elevated building in Zone V. Although a concrete slab may serve as the floor of a ground-level enclosure (usable only for parking, storage, or building access), the slab must be independent of the building foundation. If a grade beam is used to support the slab, the slab becomes the 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. For buildings in Zone V, the NFIP, IBC and IRC require that the lowest floor elevated to or above the BFE be supported by the bottom of the lowest horizontal structural member. Keeping the slab from being considered the lowest floor requires keeping the slab and grade beams separate, which means the slab and grade beams cannot be monolithic or connected by reinforcing steel or other means.

Like the driven, treated pile foundation discussed in Section 10.6.1, the foundation designs discussed in this section are based on presumptive piling capacities that should be verified prior to construction. Also, design professionals who develop foundations designs for specific buildings and have site information on subsurface conditions can augment the FEMA P-550 design to provide more efficient designs that reduce construction costs.

10.7 Open/Shallow Foundations

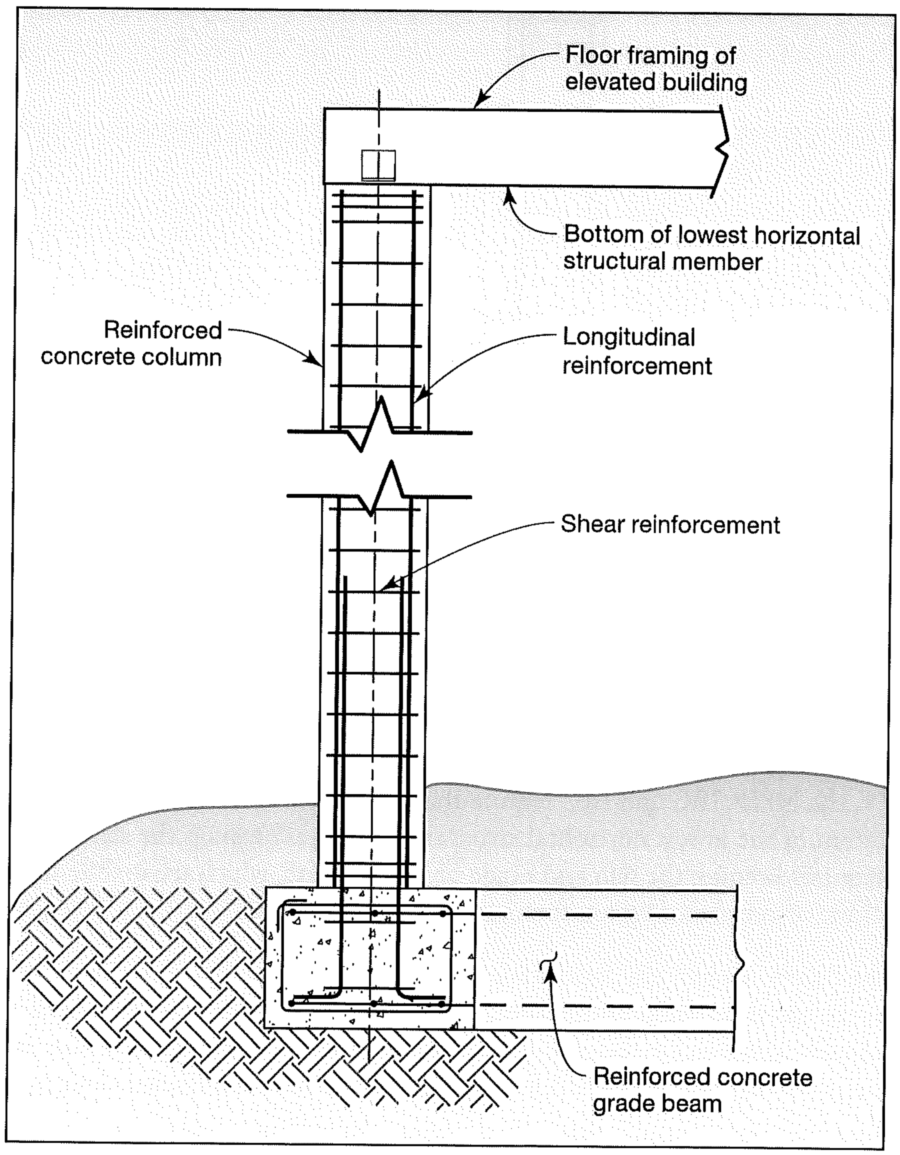
Open/shallow foundations are recommended for areas that are exposed to moving floodwaters and moderate wave actions but are not exposed to scour and erosion, which can undermine shallow footings. Open/shallow foundations are recommended for some riverine areas where an open foundation style is desirable and for buildings in Coastal A Zone where scour and erosion is limited.

In Coastal A Zones where the predicted scour and erosion depths extend below the achievable depth of shallow footings and in Coastal A Zone where scour and erosion potential is unknown or cannot be accurately predicted, open/deep foundations should be installed.

FEMA P-550 contains designs for open shallow foundations. The foundations are resistant to moving floodwaters and wave action, but because they are founded on shallow soils, they can be vulnerable to scour and erosion.

Figure 10-15. Profile of an open/ shallow foundation

SOURCE: ADAPTED FROM FEMA P-550, CASE D



The FEMA P-550 designs make use of a rigid mat to resist lateral forces and overturning moment. Frictional resistance between the grade beams and the supporting soils resist lateral loads. The weight of the foundation and the elevated structure resist uplift forces. Because the foundation lacks the uplift resistance provided by piles, foundation elements often need to be relatively large to provide sufficient dead load to resist uplift, particularly when they are submerged. Grade beams need to be continuous because, as is shown in Section 10.9, discrete foundations that have sufficient capacity to resist lateral and uplift forces without overturning are difficult to design.

FEMA P-550 contains two types of open/shallow foundations. The foundation type shown in Figure 10-15 uses a matrix of grade beams and concrete columns to elevate the building. The grade beam shown in Figure 10-15 should not be used as structural support for a concrete slab that is below an elevated building in Zone V. If the grade beam is used to support the slab, the slab will be considered the lowest floor of the building, which will lead to the insurance ramifications described in Section 10.6.2.

When used to support wood framing, the columns of open/shallow foundations are typically designed as cantilevered beam/columns subjected to lateral forces, gravity forces and uplift forces from the elevated structure and flood forces on the foundation columns. Because of the inherent difficulty of creating moment connections with wood framing, the connections between the top of the columns and the bottom of the elevated structure are typically considered pinned. Maximum shear and moment occurs at the bottom of the columns, and proper reinforcement and detailing is needed in these areas. Also, because there are typically construction joints between the tops of the grade beams and the bases of the columns where salt-laden water can seep into the joints, special detailing is needed to prevent corrosion.

Designing an open/shallow foundation that uses concrete columns and elevated concrete beams can create a frame action that increases the foundation's ability to resist lateral loads. This design accomplishes two things. First, the frame action reduces the size of the columns and in turn reduces flood loads on them, and second, when properly designed, the elevated beams act like the tops of a perimeter foundation wall. Homes constructed to one of the designs contained in prescriptive codes can be attached to the elevated concrete beams with minimal custom design.

Unlike deep, driven-pile foundations, both types of open/shallow foundations can be undermined by erosion and scour. Neither foundation type should be used where erosion or scour is anticipated to expose the grade beam.

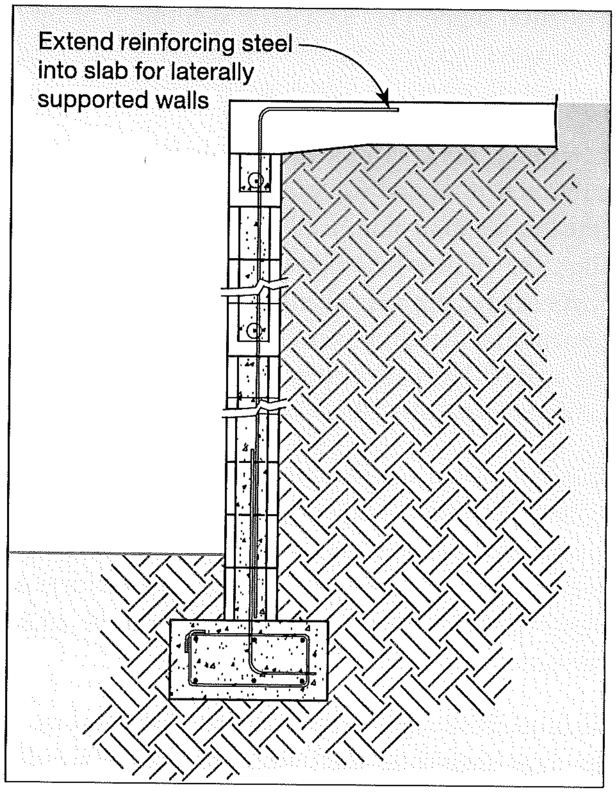
10.8 Closed/Shallow Foundations

Closed/shallow foundations are similar to the foundations that are used in non-coastal areas where flood forces are limited to slowly rising floods with no wave action and only limited flood velocities. In those areas, conventional foundation designs, many of which are included for residential construction in prescriptive codes and standards such as the 2012 IRC and ICC 600-2008, may be used. However, these codes and standards do not take into account forces from moving floodwaters and short breaking waves that can exist inland of Coastal A Zones. Therefore, caution should be used when using prescribed foundation designs in areas exposed to moving floodwaters and breaking waves.

FEMA P-550 contains two foundation designs for closed/shallow foundations: a stem wall foundation and a crawlspace foundation. Crawlspace foundation walls in SFHAs must be equipped with flood vents to equalize hydrostatic pressures on either side of the wall. See FEMA Technical Bulletin 1, *Openings in Foundation Walls and Walls of Enclosures* (FEMA 2008c). However, the flood vents do not significantly reduce hydrodynamic loads or breaking wave loads, and even with flood vents, flood forces in Coastal A Zones can damage or destroy these foundation styles.

Both closed/shallow foundations contained in FEMA P-550 are similar to foundations found in prescriptive codes but contain the additional reinforcement requirement to resist moving floodwaters and short (approximately 1.5-foot) breaking waves. Figure 10-16 shows the stem wall foundation design in FEMA P-550.

Figure 10-16.
Stem wall foundation design
SOURCE: ADAPTED FROM FEMA P-550, CASE F



10.9 Pier Foundations

Properly designed pier foundations offer the following benefits: (1) their open nature reduces the loads they must resist from moving floodwaters, (2) taller piers can often be constructed to provide additional protection without requiring a lot more reinforcement, and (3) the piers can be constructed with reinforced concrete and masonry materials commonly used in residential construction.

Pier foundations, however, can have drawbacks. If not properly designed and constructed, pier foundations lack the required strength and stability to resist loads from flood, wind or seismic events. Many pier foundation failures occurred when Hurricane Katrina struck the Gulf Coast in 2005.

The type of footing used in pier foundations greatly affects the foundation's performance (see Figure 10-17). When exposed to lateral loads, discrete footings can rotate so piers placed on discrete footings are suitable

Figure 10-17. Performance comparison of pier foundations: piers on discrete footings (foreground) failed by rotating and overturning while piers on more substantial footings (in this case a concrete mat) survived Hurricane Katrina (Pass Christian, MS, 2005)

only when wind and flood loads are relatively low. Piers placed on continuous concrete grade beams or concrete footings provide much greater resistance to lateral loads and are much less prone to failure. Footings and grade beams must be reinforced to resist the moment forces that develop at the base of the piers from the lateral loads on the foundation and the elevated home.

Like other open/shallow foundations, pier foundations are appropriate only where there is limited potential for erosion or scour. The maximum estimated depth for long- and short-term erosion and localized scour should not extend below the bottom of the footing or grade beam. In addition, adequate resistance to lateral loads is often difficult to achieve for common pier sizes on continuous footings. Even for relatively small lateral loads, larger piers designed as shear walls are often necessary to provide adequate resistance.

The following section provides an analysis of a pier foundation on discrete concrete footings. The analysis shows that discrete pier footings that must resist lateral loads are typically not practical.

10.9.1 Pier Foundation Design Examples

The following three examples discuss pier foundation design. Example 10.3 provides an analysis of the pier footing under gravity loads only (see Figure 10-18) and the footing size required to ensure that the allowable soil bearing pressure is not exceeded. Example 10.4 provides a consideration of uplift forces that many footings (see Figure 10-19) must resist to prevent failure during a design wind event. The analysis in Example 10.4 assumes that other foundation elements are in place to resist the lateral loads that must accompany uplift forces. Example 10.5 adds lateral loads to the pier and footing (see Figure 10-20) to model buildings that lack continuous foundation walls or other lateral load resisting features. The lateral loads can result from wind, seismic or moving floodwaters.

Figure 10-17. Performance comparison of pier foundations: piers on discrete footings (foreground) failed by rotating and overturning while piers on more substantial footings (in this case a concrete mat) survived Hurricane Katrina (Pass Christian, MS, 2005)

only when wind and flood loads are relatively low. Piers placed on continuous concrete grade beams or concrete footings provide much greater resistance to lateral loads and are much less prone to failure. Footings and grade beams must be reinforced to resist the moment forces that develop at the base of the piers from the lateral loads on the foundation and the elevated home.

Like other open/shallow foundations, pier foundations are appropriate only where there is limited potential for erosion or scour. The maximum estimated depth for long- and short-term erosion and localized scour should not extend below the bottom of the footing or grade beam. In addition, adequate resistance to lateral loads is often difficult to achieve for common pier sizes on continuous footings. Even for relatively small lateral loads, larger piers designed as shear walls are often necessary to provide adequate resistance.

The following section provides an analysis of a pier foundation on discrete concrete footings. The analysis shows that discrete pier footings that must resist lateral loads are typically not practical.

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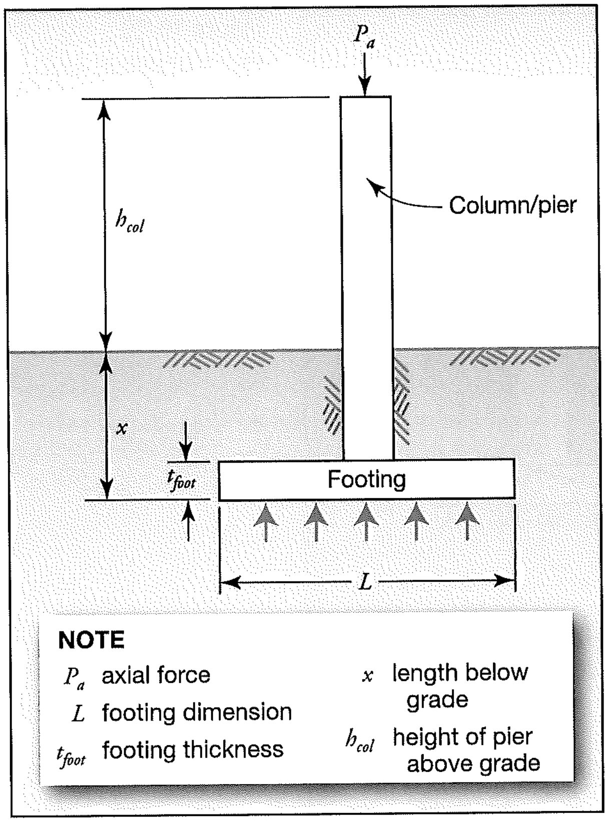

Figure 10-18.
Pier foundation and spread footing under gravity loading

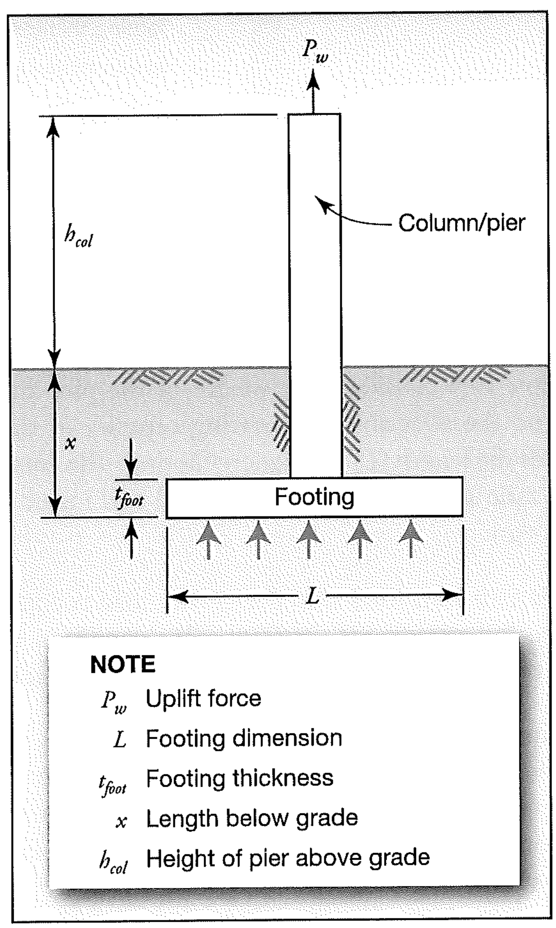

Figure 10-19.
Pier foundation and spread footing exposed to uplift forces

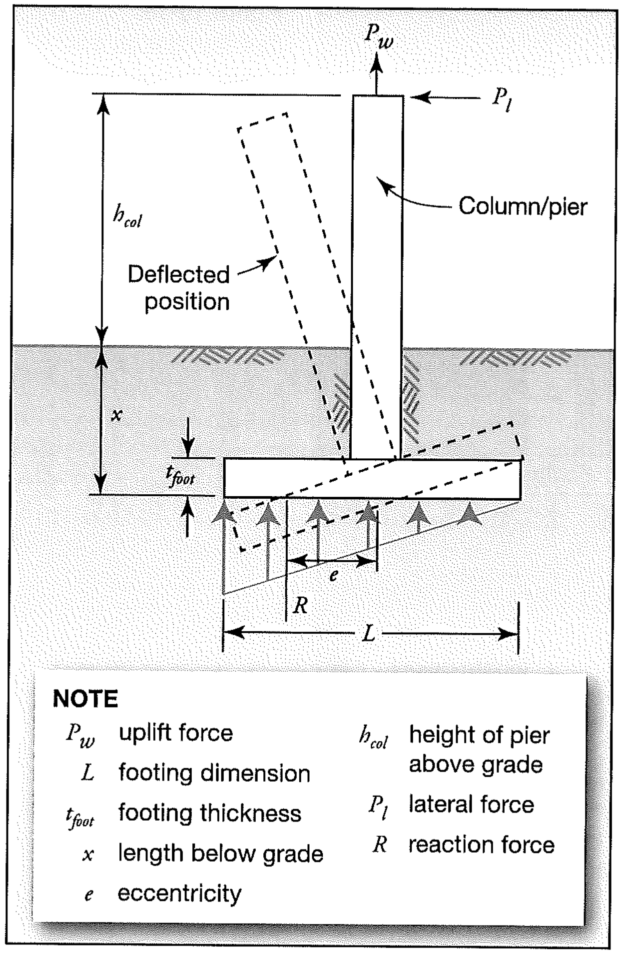

Figure 10-20.
Pier foundation and spread footing exposed to uplift and lateral forces

Several equations exist for designing discrete footings exposed to gravity loads only. Equation 10.5, which models the weight of the footing by reducing the allowable bearing capacity of the soils by the weight of the footing, is used for Example 10-3.

Equation 10.5 considers the weight of the pier and footing, the gravity load imposed on the top of the pier, and the allowable soil bearing capacity of the soils to determine footing dimensions. The equation provides the length (L) of a square footing. The equation can be modified for rectangular footings of a given aspect ratio β (ratio of width to length) and including β in the denominator of the term to the right of the equals sign.

Equation 10.5 assumes that the gravity load is equally distributed across the bottom surface of the footing and the soil stresses are constant. This condition is appropriate when the gravity loads are applied at the center of the pier (and the pier is centered on the footing) and when no lateral loads are applied.

The foundation system must have sufficient weight to prevent failure when uplift loads are applied. ASCE 7-10 requires the designer to consider only 60 percent of the dead load when designing for uplift (ASD load combination #7). If the foundation is located in an SFHA, portions of it will be located below the stillwater elevation and will be submerged during a design event. The dead load of a material is less when submerged so the submerged weight must be considered (see Section 8.5.7). In Example 10.4, it is assumed that the stillwater depth at the site is 2 feet.

EQUATION 10.5. DETERMINATION OF SQUARE FOOTING SIZE FOR GRAVITY LOADS

$$L = \left[\frac{P_a + (h_{col} + x - t_{foot}) W_{col} t_{col} w_c}{q - t_{foot} w_c} \right]^{0.5}$$

where:

L = square footing dimension (ft)

P_a = gravity load on pier (lb)

h_{col} = height of pier above grade (ft)

x = distance from grade to bottom of footing (ft)

W_{col} = column width (ft)

t_{col} = column thickness (ft)

w_c = unit weight of column and footing material (lb/ft^3)

q = soil bearing pressure (psf)

t_{foot} = footing thickness (ft)

EXAMPLE 10.3. PIER FOOTING UNDER GRAVITY LOAD

Given:

- Figure 10-18
- Gravity load on pier (P_a) = 2,880 lb (includes roof live load, live load, and dead load)
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^3
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Illustration A)
- Piers spaced 6 ft o.c. (see Illustration A)

EXAMPLE 10.3. PIER FOOTING UNDER GRAVITY LOAD (concluded)

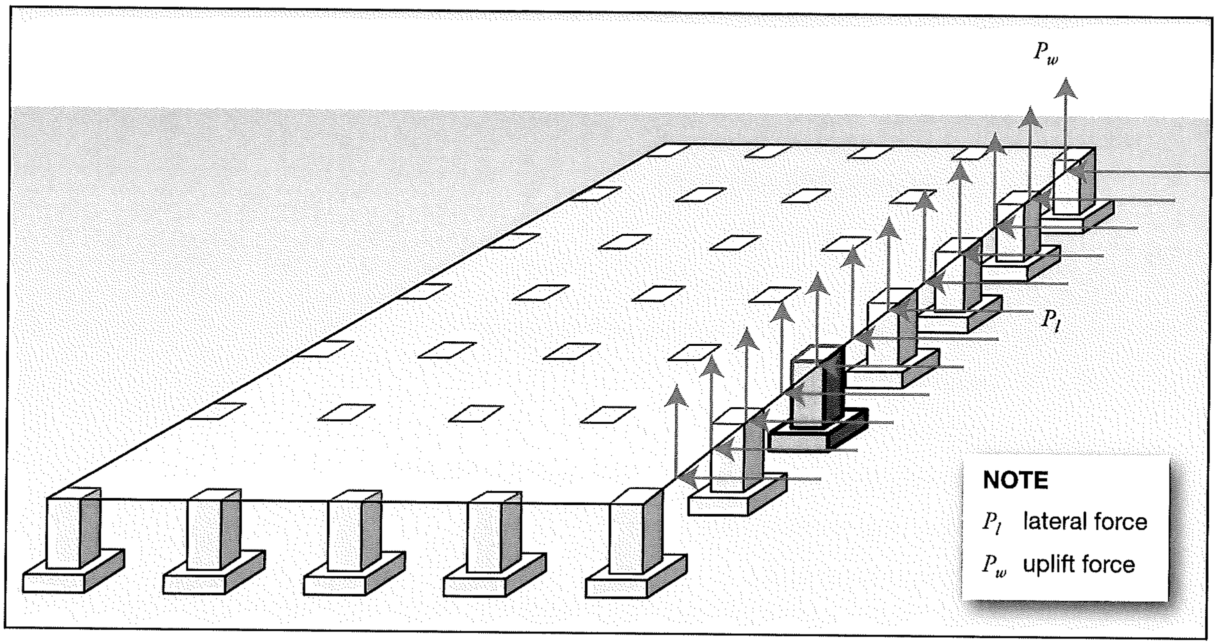

Illustration A. Site layout

Find: The appropriate square footing size for the given gravity load.

Solution: The square footing size can be found using Equation 10.5:

$$L = \left[\frac{P_a + (h_{col} + x - t_{foot}) W_{col} t_{col} w_c}{q - t_{foot} w_c} \right]^{0.5}$$

$$L = \left[\frac{2{,}880 \text{ lb} + (4 \text{ ft} + 2 \text{ ft} - 1 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft})(150 \text{ lb/ft}^3)}{2{,}000 \text{ psf} - (1 \text{ ft})(150 \text{ lb/ft}^3)} \right]^{0.5}$$

$$L = 1.5 \text{ ft}$$

The IRC requires a minimum of 2-in. projection for spread footing. Moving to the next minimum standard footing size, a **24-in. x 24-in. x 12-in. square footing** to resist the gravity loads should be used.

Example 10.3 and Example 10.4 model the conditions where the pier and footing only resist axial loads that create no moment on the footing. In those states, the soils are equally loaded across the footing. When a pier and footing foundation must resist lateral loads (or must resist gravity loads applied at some distance Δ from the centroid of the pier), the footing must resist applied moments, and soils below the footing are no longer stressed equally. Soils on one side of the footing experience compressive stresses that are greater than the average compressive stress; soils on the opposite side of the footing experience stresses lower than the average.

EXAMPLE 10.4. PIER FOOTING UNDER UPLIFT LOAD

Given:

- Figure 10-19
- Stillwater flood depth (d_s) = 2 ft
- Density of water (ρ_{water}) = 64 lb/ft^3
- Uplift load on pier (P_w) = 2,514 lb
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^3
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Example 10.4, Illustration A)
- Piers spaced 6 ft. on center (see Illustration A)

Find: The appropriate square footing size for the given uplift loads.

Solution: The square footing size can be found as follows:

First consider the dead load of submerged portion of column

$$DL_{submerged} = (w_c - \rho_{water})(x + d_s - t_{foot})(W_{col})(t_{col})$$

$$DL_{submerged} = (150 \text{ lb/ft}^3 - 64 \text{ lb/ft}^3)(2 \text{ ft} + 2 \text{ ft} - 1 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft}) = 459 \text{ lb}$$

Then consider the dead load of portion of column above the stillwater level

$$DL_{above} = (w_c)(h_{col} - d_x)(W_{col})(t_{col})$$

$$DL_{above} = (150 \text{ lb/ft}^3)(4 \text{ ft} - 2 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft}) = 533 \text{ lb}$$

Total column dead load can then be found

$$DL_{Total} = DL_{submerged} + DL_{above} = 992 \text{ lb}$$

The footing, when submerged, must provide sufficient weight to resist the deficit of the column dead load. The submerged footing dead load required is given by the following equation:

EXAMPLE 10.4. PIER FOOTING UNDER UPLIFT LOAD (concluded)

Submerged footing dead load =

$$\frac{1}{0.6}\left[P_W - 0.6(DL_{Total})\right] = \frac{1}{0.6}\left[2{,}514 \text{ lb} - 0.6(992 \text{ lb})\right] = 3{,}198 \text{ lb}$$

Footing volume required =

$$\frac{3{,}198 \text{ lb}}{(150 \text{ lb/ft}^3 - 64 \text{ lb/ft}^3)} = 37.0 \text{ ft}^3$$

For a 12-inch-thick footing, the footing area = 37 ft^2

The analysis shows that **a square, 6 ft by 6 ft by 12 in., submerged concrete footing** and a 5-ft tall, 16-in. square, partially submerged concrete column are required to resist 2,514 lb of uplift. Increasing the footing thickness to 2 ft would allow the footing dimensions to be reduced to 4 ft 6 in.

At some value of lateral load or eccentricity, the compressive stresses on one side of the footing go to zero. Because there are no tensile connections between the footing and the supporting soils, the footing becomes unstable at that point and can fail by rotation. Failure can also occur when the bearing strength on the other side of the footing is exceeded.

Equation 10.6 relates soil bearing pressure to axial load, lateral load, and footing dimension. For a given axial load, lateral load, and footing dimension, the equation can be used to solve for the maximum and minimum soil bearing pressures, q on each edge of the footing. The maximum can be compared to the allowable soil bearing pressure to determine whether the soils will be overstressed. The minimum stress determines whether instability occurs. Both maximum and minimum stresses are used to determine footing size. Alternatively, for a given allowable soil bearing pressure, axial load, and lateral load, the equation can be solved for the minimum footing size.

EQUATION 10.6. DETERMINATION OF SOIL PRESSURE

$$q = \frac{P_t}{L^2} \pm 6\frac{M}{L^3}$$

where:

q = minimum and maximum soil bearing pressures at the edges of the footing (lb/ft2)

P_t = total vertical load for the load combination being analyzed

M = applied moment $P_l\,(h_{col} + x)$ (ft lbs) where x and h_{col} are as defined previously and P_l is the lateral load applied at the top of the column

When designing a pier and footing, P_t and P_l depend on the load combination being analyzed.

EXAMPLE 10.5. PIER FOOTING UNDER UPLIFT AND LATERAL LOADS

Given:

- Figure 10-20
- Stillwater flood depth (d_s) = 2 ft
- Lateral load on pier (P_l) = 246 lb (from design example in Chapter 9: (205 plf)/6 ft times 5 piers assumed to be resisting this force)
- Uplift load on pier (P_w) = 2,514 lb (derived from 419 psf from Chapter 9 times 6 ft)
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^2
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Example 10.3, Illustration A)
- Piers spaced 6 ft o.c. (see Illustration A)

Find: The appropriate square footing size for the given uplift and lateral loads.

Solution: The square footing size can be found using Equation 10.6:

For simplicity, this example assumes the pier is partially submerged and exposed to uplift forces (as in Example 10.4) but that there are no loads from moving floodwaters or wave action. In an actual design, those forces would need to be considered. Also, if the vertical load is applied at an eccentricity "Δ", the moment $P_t\Delta$ must be combined with $P_l(H + x)$ (by vector addition) to determine the total moment applied to the footing.[2]

The total induced moment at the footing can be modeled by considering an effective reaction R numerically equal to the total vertical load P_t but applied at an eccentricity e from the centroid of the footing. The lateral load is modeled at the centroid of the footing where it contributes only to sliding. The equivalent eccentricity e is given by the following formula:

2 Unless the eccentricity from the lateral loads is collinear with the eccentricity from the vertical loads, the footing will be exposed to biaxial bending. For biaxial bending, soil stresses must be checked in both directions.

EXAMPLE 10.5. PIER FOOTING UNDER UPLIFT AND LATERAL LOADS (concluded)

EQUATION A

$$e = \frac{M}{P_t} \text{ (see Figure 10-20)}$$

where:

e = eccentricity

P_t = total vertical load for the load combination being analyzed

M = applied moment $P_l(H + x)$ (ft-lbs) where x and H are as defined previously

P_l is the lateral load applied at the top of the column. For equilibrium, R must be applied within the "kern" of the footing (for a square footing, the kern is a square with dimension of $L/3$ centered about the centroid of the footing). Mathematically, e cannot exceed $L/6$. Ensuring that the reaction R is applied within the kern of the footing prevents tensile stresses from forming on the edge of the footing.

Calculating the minimum soils stress for various footing widths (using a recursive solution) shows that the **footing would need to be 11 ft 4 in. wide to prevent overturning.** Increasing the footing thickness to 2 feet would allow the footing size to be reduced to approximately 8 ft 9 in. Either design is not practical to construct.

10.9.2 Pier Foundation Summary

These analyses indicate that piers with discrete footings are practical to construct when they are required to resist gravity loads only but are not practical when they must resist uplift forces or lateral loads. Although prescriptive designs for pier foundations are available in some codes and standards, users of the codes and standards should ensure that the designs take into account all of the loads the foundations must resist. Prescriptive designs should only be used to resist lateral and uplift loads after they have been confirmed to be adequate.

Constructing piers on continuous footings makes pier foundations much more resistant to coastal hazards, but prescriptive designs for piers on continuous footings are not present in widely adopted codes such as the IRC and IBC. Until prescriptive designs using piers are developed, these styles of foundations should be engineered. Continuous footings are discussed in Section 11.1.5 of FEMA 549, *Hurricane Katrina in the Gulf Coast* (FEMA 2006), and continuous footing designs that can be used for the basis of engineered foundations are contained in FEMA P-550.

10.10 References

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Designing the Building Envelope

This chapter provides guidance on the design of the building envelope in the coastal environment.[1] The building envelope comprises exterior doors, windows, skylights, exterior wall coverings, soffits, roof systems, and attic vents. In buildings elevated on open foundations, the floor is also considered a part of the envelope.

CROSS REFERENCE

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

High wind is the predominant natural hazard in the coastal environment that can cause damage to the building envelope. Other natural hazards also exist in some localities. These may include wind-driven rain, salt-laden air, seismic events, hail, and wildfire. The vulnerabilities of the building envelope to these hazards are discussed in this chapter, and recommendations on mitigating them are provided.

Good structural system performance is critical to avoiding injury and minimizing damage to a building and its contents during natural hazard events but does not ensure occupant or building protection. Good

1 The guidance in this chapter is based on a literature review and field investigations of a large number of houses that were struck by hurricanes, tornadoes, or straight-line winds. Some of the houses were exposed to extremely high wind speeds while others experienced moderately high wind speeds. Notable investigations include Hurricane Hugo (South Carolina, 1989) (McDonald and Smith, 1990); Hurricane Andrew (Florida, 1992) (FEMA FIA 22; Smith, 1994); Hurricane Iniki (Hawaii, 1992) (FEMA FIA 23); Hurricane Marilyn (U.S. Virgin Islands, 1995) (FEMA unpublished); Typhoon Paka (Guam, 1997) (FEMA-1193-DR-GU); Hurricane Georges (Puerto Rico, 1998) (FEMA 339); Hurricane Charley (Florida, 2004) (FEMA 488); Hurricane Ivan (Alabama and Florida, 2004) (FEMA 489); Hurricane Katrina (Louisiana and Mississippi, 2005) (FEMA 549); and Hurricane Ike (Texas, 2008) (FEMA P-757).

performance of the building envelope is also necessary. Good building envelope performance is critical for buildings exposed to high winds and wildfire.

Good performance depends on good design, materials, installation, maintenance, and repair. A significant shortcoming in any of these five elements could jeopardize the performance of the building. Good design, however, is the key element to achieving good performance. Good design can compensate to some extent for inadequacies in the other elements, but the other elements frequently cannot compensate for inadequacies in design.

The predominant cause of damage to buildings and their contents during high-wind events has been shown to be breaching of the building envelope, as shown in Figure 11-1, and subsequent water infiltration. Breaching includes catastrophic failure (e.g., loss of the roof covering or windows) and is often followed by wind-driven water infiltration through small openings at doors, windows, and walls. The loss of roof and wall coverings and soffits on the house in Figure 11-1 resulted in significant interior water damage. Recommendations for avoiding breaching are provided in this chapter.

For buildings that are in a Special Wind Region (see Figure 3-7) or in an area where the basic (design) wind speed is greater than 115 mph,[2] it is particularly important to consider the building envelope design and construction recommendations in this chapter in order to avoid wind and wind-driven water damage. In wind-borne debris regions (as defined in ASCE 7), building envelope elements from damaged buildings are often the predominant source of wind-borne debris. The wall shown in Figure 11-2 has numerous wind-borne debris scars. Asphalt shingles from nearby residences were the primary source of debris. Following the design and construction recommendations in this chapter will minimize the generation of wind-borne debris from residences.

Figure 11-1.
Good structural system performance but the loss of shingles, underlayment, siding, housewrap, and soffits resulted in significant interior water damage. Estimated wind speed: 125 mph.[3] Hurricane Katrina (Louisiana, 2005)

2 The 115-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05, or an earlier version is used, the equivalent wind speed trigger is 90 mph.

3 The estimated wind speeds given in this chapter are for a 3-second gust at a 33-foot elevation for Exposure C (as defined in ASCE 7).Most of the buildings for which estimated speeds are given in this chapter are located in Exposure B, and some are in Exposure D. For buildings in Exposure B, the actual wind speed is less than the wind speed for Exposure C conditions. For example, a 130-mph Exposure C speed is equivalent to 110 mph in Exposure B.

Building integrity in earthquakes is partly dependent on the performance of the building envelope. Residential building envelopes have historically performed well during seismic events because most envelope elements are relatively lightweight. Exceptions have been inadequately attached heavy elements such as roof tile. This chapter provides recommendations for envelope elements that are susceptible to damage in earthquakes.

A building's susceptibility to wildfire depends largely on the presence of nearby vegetation and the characteristics of the building envelope, as illustrated in Figure 11-3. See FEMA P-737, *Home Builder's Guide to Construction in Wildfire Zones* (FEMA 2008), for guidance on materials and construction techniques to reduce risks associated with wildfire.

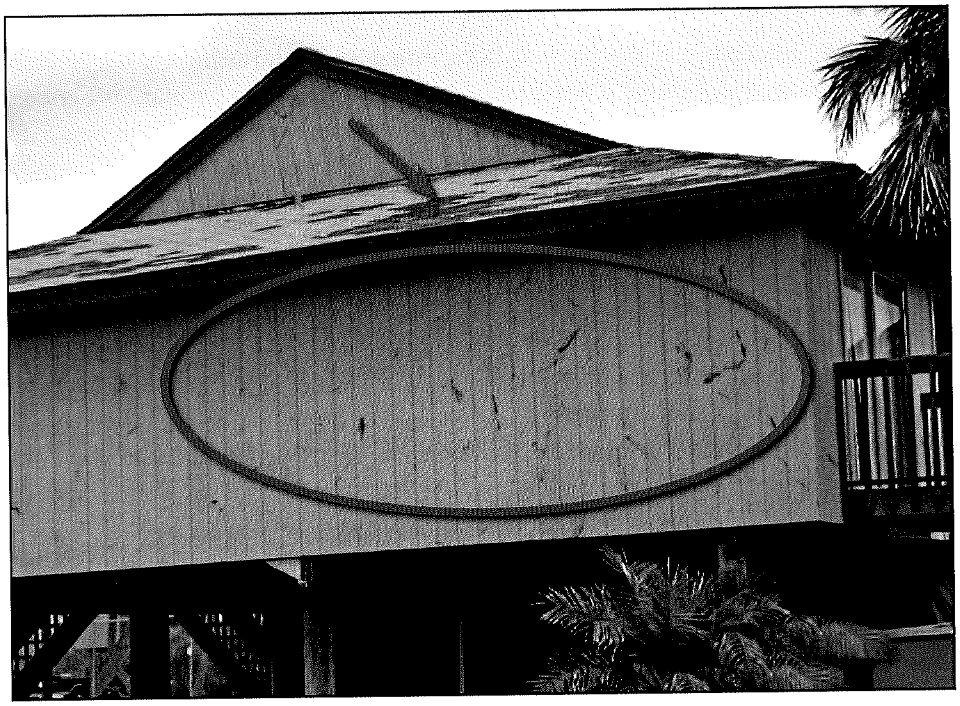


Figure 11-2.
Numerous wind-borne debris scars on the wall of this house and several missing asphalt shingles. Estimated wind speed: 140 to 150 mph. Hurricane Charley (Florida, 2004)

Figure 11-3.
House that survived a wildfire due in part to fire-resistant walls and roof while surrounding houses were destroyed
SOURCE: DECRA ROOFING SYSTEMS, USED WITH PERMISSION

This chapter does not address basic design issues or the general good practices that are applicable to residential design. Rather, the chapter builds on the basics by addressing the special design and construction considerations of the building envelope for buildings that are susceptible to natural hazards in the coastal environment. Flooding effects on the building envelope are not addressed because of the assumption that the envelope will not be inundated by floodwater, but envelope resistance to wind-driven rain is addressed. The recommended measures for protection against wind-driven rain should also be adequate to protect against wave spray.

11.1 Floors in Elevated Buildings

Sheathing is commonly applied to the underside of the bottom floor framing of a building that is elevated on an open foundation. The sheathing provides the following protection: (1) it protects insulation between joists or trusses from wave spray, (2) it helps minimize corrosion of framing connectors and fasteners, and (3) it protects the floor framing from being knocked out of alignment by flood-borne debris passing under the building.

A variety of sheathing materials have been used to sheath the framing, including cement-fiber panels, gypsum board, metal panels, plywood, and vinyl siding. Damage investigations have revealed that plywood offers the most reliable performance in high winds. However, as shown in Figure 11-4, even though plywood has been used, a sufficient number of fasteners are needed to avoid blow-off. Since ASCE 7 does not provide guidance for load determination, professional judgment in specifying the attachment schedule is needed. As a conservative approach, loads can be calculated by using the C&C coefficients for a roof with the slope of 7 degrees or less. However, the roof corner load is likely overly conservative for the underside of elevated floors. Applying the perimeter load to the corner area is likely sufficiently conservative.

To achieve good long-term performance, exterior grade plywood attached with stainless steel or hot-dip galvanized nails or screws is recommended (see the corroded nails in Figure 11-4).

11.2 Exterior Doors

This section addresses exterior personnel doors and garage doors. The most common problems are entrance of wind-driven rain and breakage of glass vision panels and sliding glass doors by wind-borne debris. Blow-off of personnel doors is uncommon but as shown in Figure 11-5, it can occur. Personnel door blow-off is typically caused by inadequate attachment of the door frame to the wall. Garage door failure via negative (suction) or positive pressure was common before doors with high-wind resistance became available (see Figure 11-6). Garage door failure is typically caused by the use of door and track assemblies that have insufficient wind resistance or by inadequate attachment of the tracks to nailers or to the wall. Failures such as those shown in Figures 11-5 and 11-6 can result in a substantial increase in internal pressure and can allow entrance of a significant amount of wind-driven rain.

CROSS REFERENCE

For information regarding garage doors in breakaway walls, see Fact Sheet 8.1, *Enclosures and Breakaway Walls*, in FEMA P-499, *Home Builder's Guide to Coastal Construction Technical Fact Sheet Series* (FEMA 2010b).

Figure 11-4.
Plywood panels on the underside of a house that blew away because of excessive nail spacing. Note the corroded nails (inset). Estimated wind speed: 105 to 115 mph. Hurricane Ivan (Alabama, 2004)

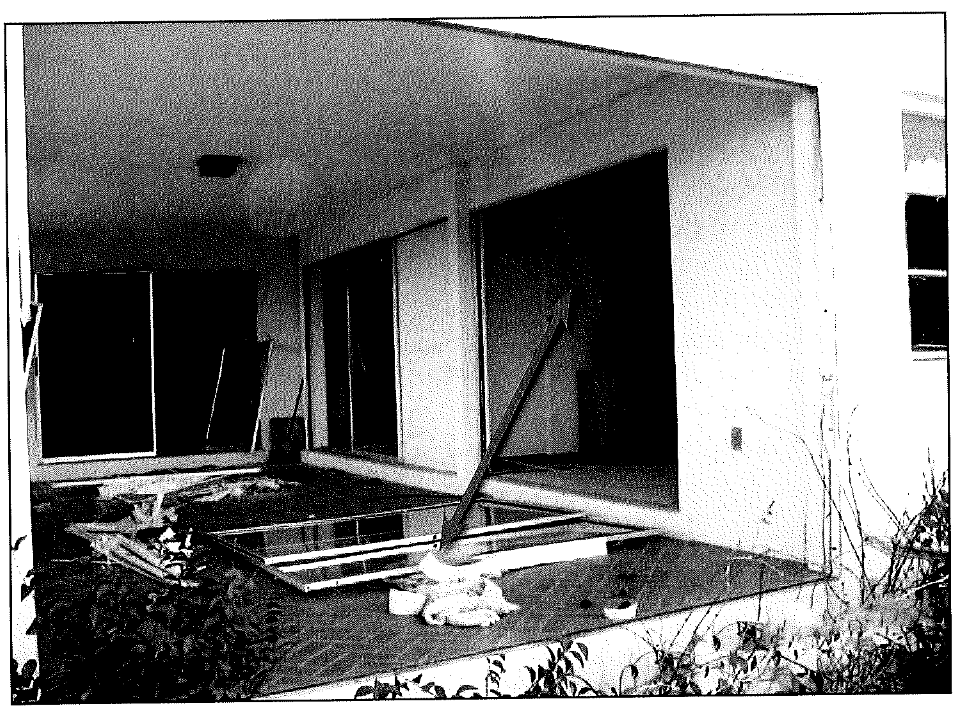


Figure 11-5.
Sliding glass doors pulled out of their tracks by wind suction. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

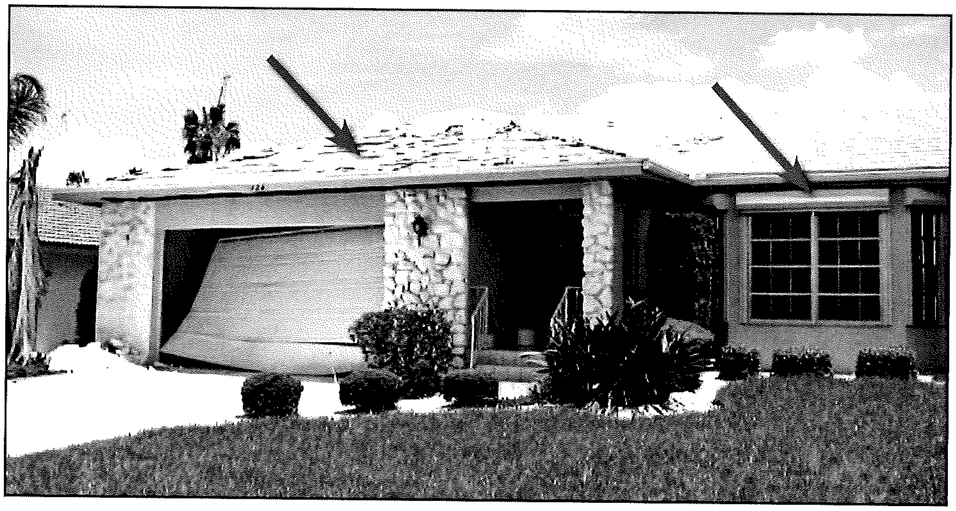


Figure 11-6.
Garage door blown from its track as a result of positive pressure. Note the damage to the adhesive-set tiles (left arrow; see Section 11.5.4.1). This house was equipped with roll-up shutters (right arrow; see Section 11.3.1.2). Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

11.2.1 High Winds

Exterior door assemblies (i.e., door, hardware, frame, and frame attachment to the wall) should be designed to resist high winds and wind-driven rain.

CROSS REFERENCE

For design guidance on the attachment of door frames, see AAMA TIR-A-14.

For a methodology to confirm an anchorage system provides load resistance with an appropriate safety factor to meet project requirements, see AAMA 2501.

Both documents are available for purchase from the American Architectural Manufacturers Association (http://aamanet.org).

11.2.1.1 Loads and Resistance

The IBC and IRC require door assemblies to have sufficient strength to resist the positive and negative design wind pressure. Personnel doors are normally specified to comply with AAMA/WDMA/CSA 101/I.S.2/A440, which references ASTM E330 for wind load testing. However, where the basic wind speed is greater than 150 mph,[4] it is recommended that design professionals specify that personnel doors comply with wind load testing in accordance with ASTM E1233. ASTM E1233 is the recommended test method in high-wind areas because it is a cyclic test method, whereas ASTM E330 is a static test. The cyclical test method is more representative of loading conditions in high-wind areas than ASTM E330. Design professionals should also specify the attachment of the door frame to the wall (e.g., type, size, spacing, edge distance of frame fasteners).

It is recommended that design professionals specify that garage doors comply with wind load testing in accordance with ANSI/DASMA 108. For garage doors attached to wood nailers, design professionals should also specify the attachment of the nailer to the wall.

CROSS REFERENCE

For design guidance on the attachment of garage door frames, see Technical Data Sheet #161, *Connecting Garage Door Jambs to Building Framing* (DASMA 2010). Available at http://www.dasma.com/PubTechData.asp.

4 The 150-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 120 mph.

11.2.1.2 Wind-Borne Debris

If a solid door is hit with wind-borne debris, the debris may penetrate the door, but in most cases, the debris opening will not be large enough to result in significant water infiltration or in a substantial increase in internal pressure. Therefore, in wind-borne debris regions, except for glazed vision panels and glass doors, ASCE 7, IBC, and IRC do not require doors to resist wind-borne debris. However, the 2007 FBC requires all exterior doors in the High-Velocity Hurricane Zone (as defined in the FBC) to be tested for wind-borne debris resistance.

CROSS REFERENCE

For more information about wind-borne debris and glazing in doors, see Section 11.3.1.2.

It is possible for wind-borne debris to cause door latch or hinge failure, resulting in the door being pushed open, an increase in internal pressure, and potentially the entrance of a significant amount of wind-driven rain. As a conservative measure in wind-borne debris regions, solid personnel door assemblies could be specified that resist the test missile load specified in ASTM E1996. Test Missile C is applicable where the basic wind speed is less than 164 mph. Test Missile D is applicable where the basic wind speed is 164 mph or greater.[5] See Section 11.3.1.2 regarding wind-borne debris testing. If wind-borne debris-resistant garage doors are desired, the designer should specify testing in accordance with ANSI/DASMA 115.

11.2.1.3 Durability

For door assemblies to achieve good wind performance, it is necessary to avoid strength degradation caused by corrosion and termites. To avoid corrosion problems with metal doors or frames, anodized aluminum or galvanized doors and frames and stainless steel frame anchors and hardware are recommended for buildings within 3,000 feet of an ocean shoreline (including sounds and back bays). Galvanized steel doors and frames should be painted for additional protection. Fiberglass doors may also be used with wood frames.

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 "Material Durability in Coastal Environments," a resource document available on the Residential Coastal Construction Web site, for information on wood treatment methods.

11.2.1.4 Water Infiltration

Heavy rain that accompanies high winds can cause significant wind-driven water infiltration. The magnitude of the problem increases with the wind speed. Leakage can occur between the door and its frame, the frame and the wall, and the threshold and the door. When wind speeds approach 150 mph, some leakage should be anticipated because of the high-wind pressures and numerous opportunities for leakage path development.[6]

5 The 164-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 130 mph.

6 The 150-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 120 mph.

The following elements can minimize infiltration around exterior doors:

- **Vestibule.** Adding a vestibule allows both the inner and outer doors to be equipped with weatherstripping. The vestibule can be designed with water-resistant finishes (e.g., tile), and the floor can be equipped with a drain. In addition, installing exterior threshold trench drains can be helpful (openings must be small enough to avoid trapping high-heeled shoes). Trench drains do not eliminate the problem because water can penetrate at door edges.
- **Door swing.** Out-swinging doors have weatherstripping on the interior side where it is less susceptible to degradation, which is an advantage to in-swinging doors. Some interlocking weatherstripping assemblies are available for out-swinging doors.
- **Pan flashing.** Adding flashing under the door threshold helps prevent penetration of water into the subflooring, a common place for water entry and subsequent wood decay. More information is available in Fact Sheet 6.1, *Window and Door Installation*, in FEMA P-499, *Home Builder's Guide to Coastal Construction Technical Fact Sheet Series* (FEMA 2010b).
- **Door/wall integration.** Successfully integrating the door frame and wall is a special challenge when designing and installing doors to resist wind-driven rain. More information is available in Fact Sheet 6.1 in FEMA P-499.
- **Weatherstripping.** A variety of pre-manufactured weatherstripping elements are available, including drips, door shoes and bottoms, thresholds, and jamb/ head weatherstripping. More information is available in Fact Sheet 6.1 in FEMA P-499.

Figure 11-7 shows a pair of doors that successfully resisted winds that were estimated at between 140 and 160 mph. However, as shown in the inset, a gap of about 3/8 inch between the threshold and the bottom of the door allowed a significant amount of water to be blown into the house. The weatherstripping and thresholds shown in Fact Sheet 6.1 in FEMA P-499 can minimize water entry.

Figure 11-7.
A 3/8-inch gap between the threshold and door (illustrated by the spatula handle), which allowed wind-driven rain to enter the house. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

11.3 Windows and Sklylights

This section addresses exterior windows (including door vision panels) and skylights. The most common problems in the coastal environment are entrance of wind-driven rain and glazing breakage by wind-borne debris. It is uncommon for windows to be blown-in or blown-out, but it does occur (see Figure 11-8). The type of damage shown in Figure 11-8 is typically caused by inadequate attachment of the window frame to the wall, but occasionally the glazing itself is blown out of the frame. Breakage of glazing from over-pressurization sometimes occurs with windows that were manufactured before windows with high-wind resistance became available. Strong seismic events can also damage windows although it is uncommon in residential construction. Hail can cause significant damage to skylights and occasionally cause window breakage.

11.3.1 High Winds

Window and skylight assemblies (i.e., glazing, hardware for operable units, frame, and frame attachment to the wall or roof curb) should be designed to resist high winds and wind-driven rain. In wind-borne debris regions, the assemblies should also be designed to resist wind-borne debris or be equipped with shutters, as discussed below.

11.3.1.1 Loads and Resistance

The IBC and IRC require that window and skylight assemblies have sufficient strength to resist the positive and negative design wind pressures. Windows and skylights are normally specified to comply with AAMA/WDMA/CSA 101/I.S.2/A440, which references ASTM E330 for wind load testing. However, where the basic wind speed is greater than 150 mph,[7] it is recommended that design professionals specify that

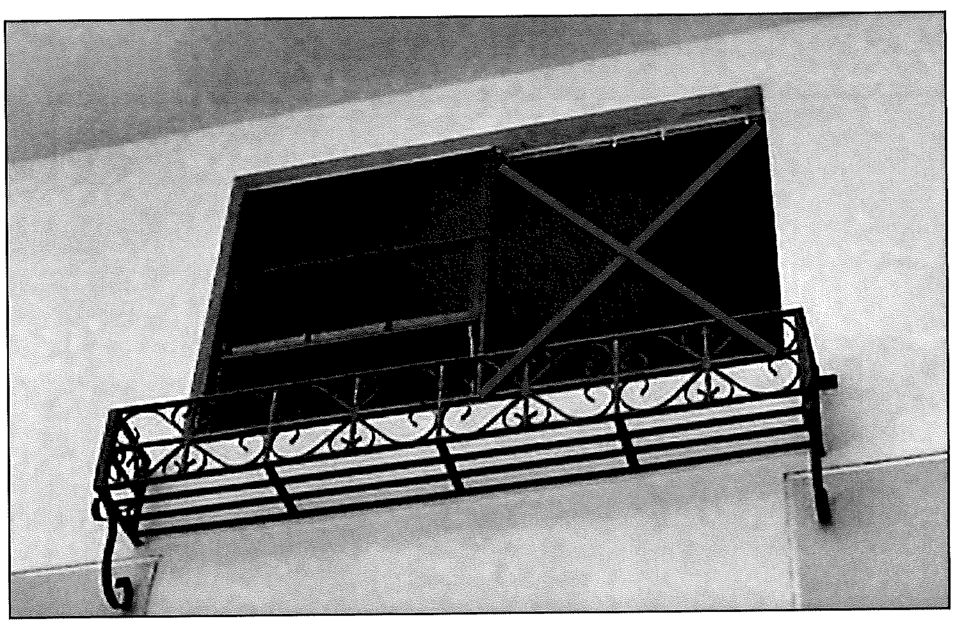


Figure 11-8.
Window frame pulled out of the wall because of inadequate window frame attachment. Hurricane Georges (Puerto Rico, 1998)

7 The 150-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 120 mph.

windows and skylights comply with wind load testing in accordance with ASTM E1233. ASTM E1233 is the recommended test method in high-wind areas because it is a cyclic test method, whereas ASTM E330 is a static test. The cyclical test method is more representative of loading conditions in high-wind areas than ASTM E330. Design professionals should also specify the attachment of the window and skylight frames to the wall and roof curb (e.g., type, size, spacing, edge distance of frame fasteners). Curb attachment to the roof deck should also be specified.

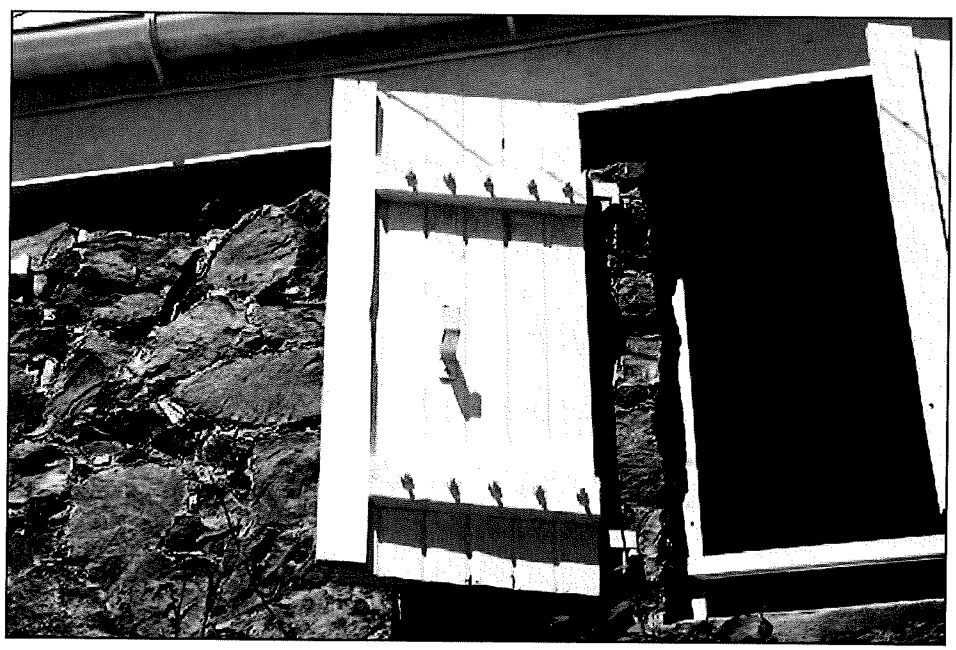
For design guidance on the attachment of frames, see AAMA TIR-A14 and AAMA 2501.

11.3.1.2 Wind-Borne Debris

When wind-borne debris penetrates most materials, only a small opening results, but when debris penetrates most glazing materials, a very large opening can result. Exterior glazing that is not impact-resistant (such as annealed, heat-strengthened, or tempered glass) or not protected by shutters is extremely susceptible to breaking if struck by debris. Even small, low-momentum debris can easily break glazing that is not protected. Broken windows can allow a substantial amount of water to be blown into a building and the internal air pressure to increase greatly, both of which can damage interior partitions and ceilings.

In windstorms other than hurricanes and tornadoes, the probability of a window or skylight being struck by debris is extremely low, but in hurricane-prone regions, the probability is higher. Although the debris issue was recognized decades ago, as illustrated by Figure 11-9, wind-borne debris protection was not incorporated into U.S. codes and standards until the 1990s. In order to minimize interior damage, the IBC and IRC, through ASCE 7, prescribe that exterior glazing in wind-borne debris regions be impact-resistant (i.e., laminated glass or polycarbonate) or protected with an impact-resistant covering (shutters). ASCE 7 refers to ASTM E1996 for missile (debris) loads and to ASTM E1886 for the test method to be used to demonstrate compliance with the ASTM E1996 load criteria. Regardless of whether the glazing is laminated glass, polycarbonate, or protected by shutters, glazing is required to meet the positive and negative design air pressures.

Figure 11-9. Very old building with robust shutters constructed of 2x4 lumber, bolted connections, and heavy metal hinges. Hurricane Marilyn (U.S. Virgin Islands, 1995)



Wind-borne debris also occurs in the portions of hurricane-prone regions that are inland of wind-borne debris regions, but the quantity and momentum of debris are typically lower outside the wind-borne debris region. As a conservative measure, impact-resistant glazing or shutters could be specified inland of the wind-borne debris region. If the building is located where the basic wind is 125 mph[8] or greater and is within a few hundred feet of a building with an aggregate surface roof or other buildings that have limited wind resistance, it is prudent to consider impact-resistant glazing or shutters.

With the advent of building codes requiring glazing protection in wind-borne debris regions, a variety of shutter designs have entered the market. Shutters typically have a lower initial cost than laminated glass. However, unless the shutter is permanently anchored to the building (e.g., accordion shutter, roll-up shutter), storage space is needed. Also, when a hurricane is forecast, the shutters need to be deployed. The difficulty of shutter deployment and demobilization on upper-level glazing can be avoided by using motorized shutters, although laminated glass may be a more economical solution.

Because hurricane winds can approach from any direction, when debris protection is specified, it is important to specify that all exterior glazing be protected, including glazing that faces open water. At the house shown in Figure 11-10, all of the windows were protected with roll-up shutters except for those in the cupola. One of the cupola windows was broken. Although the window opening was relatively small, a substantial amount of interior water damage likely occurred.

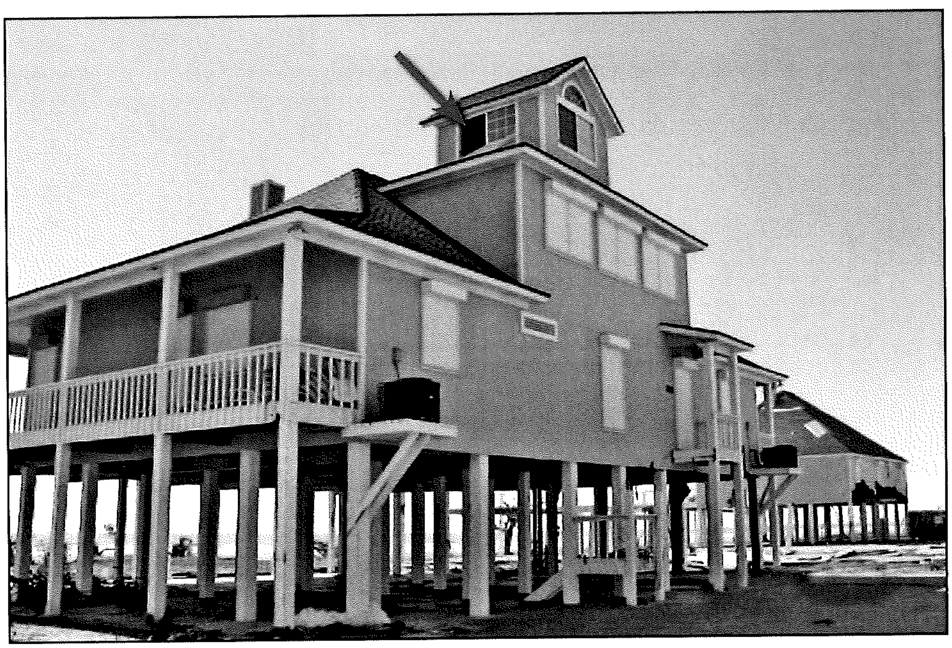


Figure 11-10. Unprotected cupola window that was broken. Estimated wind speed: 110 mph. Hurricane Ike (Texas, 2008)

The FBC requires exterior windows and sliding glass doors to have a permanent label or marking, indicating information such as the positive and negative design pressure rating and impact-resistant rating (if applicable). Impact-resistant shutters are also required to be labeled. Figure 11-11 is an example of a permanent label on a window assembly. This label provides the positive and negative design pressure rating, test missile rating,

8 The 125-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 100 mph.

and test standards that were used to evaluate the pressure and impact resistance. Without a label, ascertaining whether a window or shutter has sufficient strength to meet pressure and wind-borne debris loads is difficult (see Figure 11-12). It is therefore recommended that design professionals specify that windows and shutters have permanently mounted labels that contain the type of information shown in Figure 11-11.

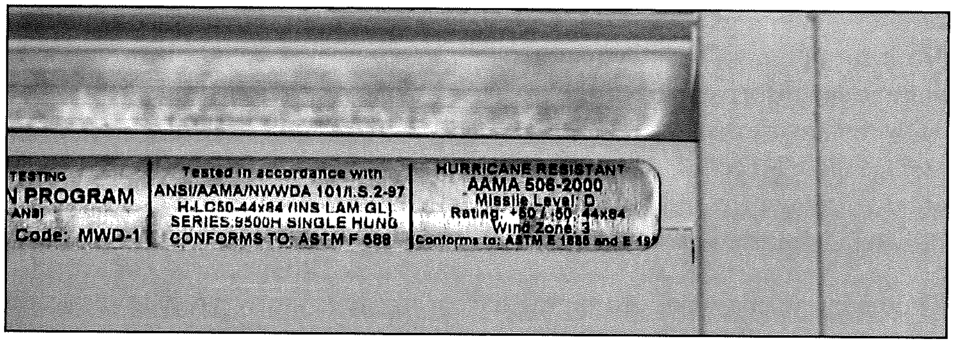


Figure 11-11.
Design pressure and impact-resistance information in a permanent window label. Hurricane Ike (Texas, 2008)

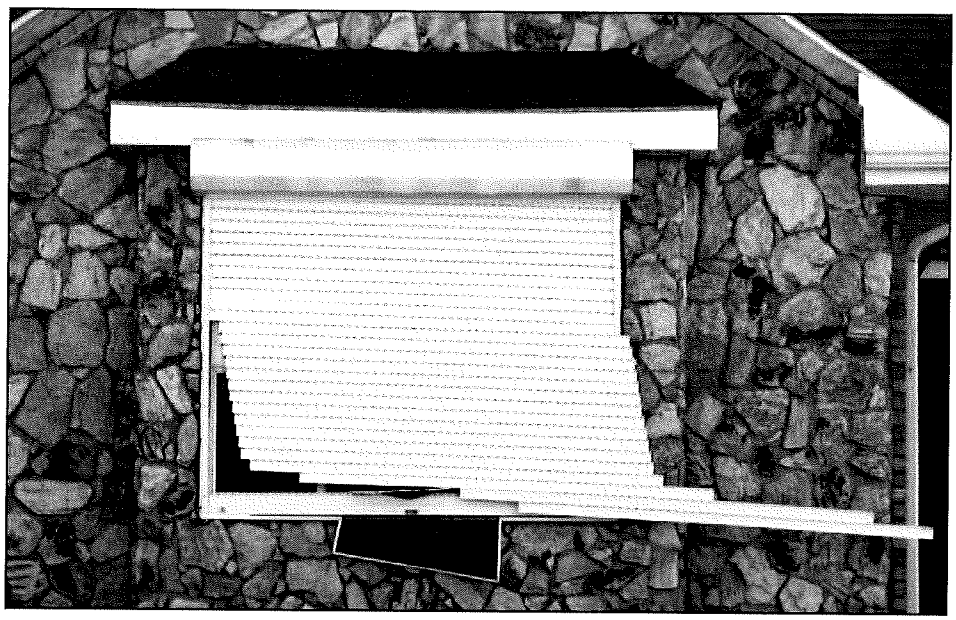


Figure 11-12.
Roll-up shutter slats that detached from the tracks. The lack of a label makes it unclear whether the shutter was tested in accordance with a recognized method. Estimated wind speed: 110 mph. Hurricane Katrina (Louisiana, 2005)

Glazing Protection from Tile Debris

Residential glazing in wind-borne debris regions is required to resist the test missile C or D, depending on the basic wind speed. However, field investigations have shown that roof tile can penetrate shutters that comply with test missile D (see Figure 11-13). Laboratory research conducted at the University of Florida indicates that test missile D compliant shutters do not provide adequate protection against tile debris (Fernandez et al. 2010). Accordingly, if tile roofs occur within 100 to 200 feet (depending on basic wind speed), it is recommended that shutters complying with test missile E be specified.

CROSS REFERENCE

More information, including a discussion of various types of shutters and recommendations pertaining to them, is available in Fact Sheet 6.2, *Protection of Openings – Shutters and Glazing,* in FEMA P-499.

Figure 11-13.
Shutter punctured by roof tile. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Jalousie Louvers

In tropical climates such as Puerto Rico, some houses have metal jalousie louvers in lieu of glazed window openings (see Figure 11-14). Metal jalousies have the appearance of a debris-resistant shutter, but they typically offer little debris resistance. Neither the UBC nor IRC require openings equipped with metal jalousie louvers to be debris resistant because glazing does not occur. However, the louvers are required to meet the design wind pressure.

Because the louvers are not tightly sealed, the building should be evaluated to determine whether it is enclosed or partially enclosed (which depends on the distribution and size of the jalousie windows). Jalousie louvers are susceptible to significant water infiltration during high winds.

11.3.1.3 Durability

Achieving good wind performance in window assemblies requires avoiding strength degradation caused by corrosion and termites. To avoid corrosion, wood or vinyl frames are recommended for buildings within 3,000 feet of an ocean shoreline (including sounds and back bays). 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. 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 "Material Durability in Coastal Environments," a resource document available on the Residential Coastal Construction Web site, for information on wood treatment methods.

Figure 11-14.
House in Puerto Rico with metal jalousie louvers

11.3.1.4 Water Infiltration

Heavy rain accompanied by high winds can cause wind-driven water infiltration. The magnitude of the problem increases with wind speed. Leakage can occur at the glazing/frame interface, the frame itself, or between the frame and wall. When the basic wind speed is greater than 150 mph,[9] because of the very high design wind pressures and numerous opportunities for leakage path development, some leakage should be anticipated when the design wind speed conditions are approached.

A design option that partially addresses this problem is to specify a strip of water-resistant material, such as tile, along walls that have a large amount of glazing instead of extending the carpeting to the wall. During a storm, towels can be placed along the strip to absorb water infiltration. These actions can help protect carpets from water damage.

It is recommended that design professionals specify that window and skylight assemblies comply with AAMA 520. AAMA 520 has 10 performance levels. The level that is commensurate with the project location should be specified.

The successful integration of windows into exterior walls to protect against water infiltration is a challenge. To the extent possible, when detailing the interface between the wall and

NOTE

Laboratory research at the University of Florida indicates that windows with compression seals (i.e., awning and casement windows) are generally more resistant to wind-driven water infiltration than windows with sliding seals (i.e., hung and horizontal sliding windows) (Lopez et al. 2011).

CROSS REFERENCE

For guidance on window installation, see:

- FMA/AAMA 100
- FMA/AAMA 200

9 The 150-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 120 mph.

the window, design professionals should rely on sealants as the secondary line of defense against water infiltration rather than making the sealant the primary protection. If a sealant joint is the first line of defense, a second line of defense should be designed to intercept and drain water that drives past the sealant joint.

CROSS REFERENCE

For a comparison of wind-driven rain resistance as a function of window installation in accordance with ASTM E2112 (as referenced in Fact Sheet 6.1 in FEMA P-499), FMA/AAMA 100, and FMA/AAMA 200, see Salzano et al. (2010).

When designing joints between walls and windows, the design professional should consider the shape of the sealant joint (i.e., , hour-glass shape with a width-to-depth ratio of at least 2:1) and the type of sealant to be specified. The sealant joint should be designed to enable the sealant to bond on only two opposing surfaces (i.e., a backer rod or bond-breaker tape should be specified). Butyl is recommended as a sealant for concealed joints and polyurethane for exposed joints. During installation, cleanliness of the sealant substrate is important, particularly if polyurethane or silicone sealants are specified, as is the tooling of the sealant.

Sealant joints can be protected with a removable stop (as illustrated in Figure 2 of Fact Sheet 6.1 of FEMA P-499). The stop protects the sealant from direct exposure to the weather and reduces the possibility of wind-driven rain penetration.

Where water infiltration protection is particularly demanding and important, onsite water infiltration testing in accordance with AAMA 502 can be specified. AAMA 502 provides pass/fail criteria based on testing in accordance with either of two ASTM water infiltration test methods. ASTM E1105 is the recommended test method.

11.3.2 Seismic

Glass breakage due to in-plane wall deflection is unlikely, but special consideration should be given to walls with 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 does not exceed the limits prescribed in the building code.

11.3.3 Hail

A test method has not been developed for testing skylights for hail resistance, but ASTM E822 for testing hail resistance of solar collectors could be used for assessing the hail resistance of skylights.

11.4 Non-Load-Bearing Walls, Wall Coverings, and Soffits

This section addresses exterior non-load-bearing walls, wall coverings, and soffits. The most common problems in the coastal environment are soffit blow-off with subsequent entrance of wind-driven rain into attics and wall covering blow-off with subsequent entrance of wind-driven rain into wall cavities. Seismic events can also damage heavy wall systems including coverings. Although hail can damage walls, significant damage is not common.

A variety of exterior wall systems can be used in the coastal environment. The following wall coverings are commonly used over wood-frame construction: aluminum siding, brick veneer, fiber cement siding, exterior insulation finish systems (EIFS), stucco, vinyl siding, and wood siding (boards, panels, or shakes). Concrete or concrete masonry unit (CMU) wall construction can also be used, with or without a wall covering.

11.4.1 High Winds

Exterior non-load-bearing walls, wall coverings, and soffits should be designed to resist high winds and wind-driven rain. The IBC and IRC require that exterior non-load-bearing walls, wall coverings, and soffits have sufficient strength to resist the positive and negative design wind pressures.

NOTE

ASCE 7, IBC, and IRC do not require exterior walls or soffits to resist wind-borne debris. However, the FBC requires exterior wall assemblies in the High-Velocity Hurricane Zone (as defined in the FBC) to be tested for wind-borne debris or to be deemed to comply with the wind-borne debris provisions that are stipulated in the FBC.

11.4.1.1 Exterior Walls

It is recommended that the exterior face of studs be fully clad with plywood or oriented strand board (OSB) sheathing so the sheathing can withstand design wind pressures that produce both in-plane and out-of-plane loads because a house that is fully sheathed with plywood or OSB is more resistant to wind-borne debris and water infiltration if the wall cladding is lost.[10] The disadvantage of not fully cladding the studs with plywood or OSB is illustrated by Figure 11-15. At this residence, OSB was installed at the corner areas to provide shear resistance, but foam insulation was used in lieu of OSB in the field of the wall. In some wall areas, the vinyl siding and foam insulation on the exterior side of the studs and the gypsum board on the interior side of the studs were blown off. Also, although required by building codes, this wall system did not have a moisture barrier between the siding and OSB/foam sheathing. In addition to the wall covering damage, OSB roof sheathing was also blown off.

Wood siding and panels (e.g., textured plywood) and stucco over CMU or concrete typically perform well during high winds. However, blow-off of stucco applied directly to concrete walls (i.e., wire mesh is not applied over the concrete) has occurred during high winds. This problem can be avoided by leaving the concrete exposed or by painting it. More blow-off problems have been experienced with vinyl siding than with

NOTE

Almost all wall coverings permit the passage of some water past the exterior surface of the covering, particularly when the rain is wind-driven. For this reason, most wall coverings should be considered water-shedding rather than waterproofing. A secondary line of protection with a moisture barrier is recommended to avoid moisture-related problems. Asphalt-saturated felt is the traditional moisture barrier, but housewrap is now the predominate moisture barrier. Housewrap is more resistant to air flow than asphalt-saturated felt and therefore offers improved energy performance.

Fact Sheet 1.9, *Moisture Barrier Systems*, and Fact Sheet 5.1, *Housewrap*, in FEMA P-499 address key issues regarding selecting and installing moisture barriers as secondary protection in exterior walls.

10 This recommendation is based on FEMA P-757, *Mitigation Assessment Team Report: Hurricane Ike in Texas and Louisiana* (FEMA 2009).

other siding or panel materials (see Figure 11-15). Problems with aluminum and fiber cement siding have also occurred (see Figure 11-16).

Siding

A key to the successful performance of siding and panel systems is attachment with a sufficient number of proper fasteners (based on design loads and tested resistance) that are correctly located. Fact Sheet 5.3, *Siding Installation and Connectors*, in FEMA P-499 provides guidance on specifying and installing vinyl, wood siding, and fiber cement siding in high-wind regions.

NOTE

In areas that experience frequent wind-driven rain and in areas that are susceptible to high winds, a pressure-equalized rain screen design should be considered when specifying wood or fiber cement siding. A rain screen design is accomplished by installing suitable vertical furring strips between the moisture barrier and siding material. The cavity facilitates drainage of water from the space between the moisture barrier and backside of the siding and facilitates drying of the siding and moisture barrier.

For more information, see Fact Sheet 5.3, *Siding Installation in High-Wind Regions*, in FEMA P-499.

Figure 11-15.
Blown-off vinyl siding and foam sheathing; some blow-off of interior gypsum board (circle). Estimated wind speed: 130 mph. Hurricane Katrina (Mississippi, 2006)

Brick Veneer

Blow-off of brick veneer has occurred often during high winds. Common failure modes include tie (anchor corrosion), tie fastener pull-out, failure of masons to embed ties into the mortar, and poor bonding between ties and mortar, and poor-quality mortar. Four of these failure modes occurred at the house shown in Figure 11-17. The lower bricks were attached to CMU and the upper bricks were attached to wood studs. In addition to the wall covering damage, roof sheathing was blown off along the eave.

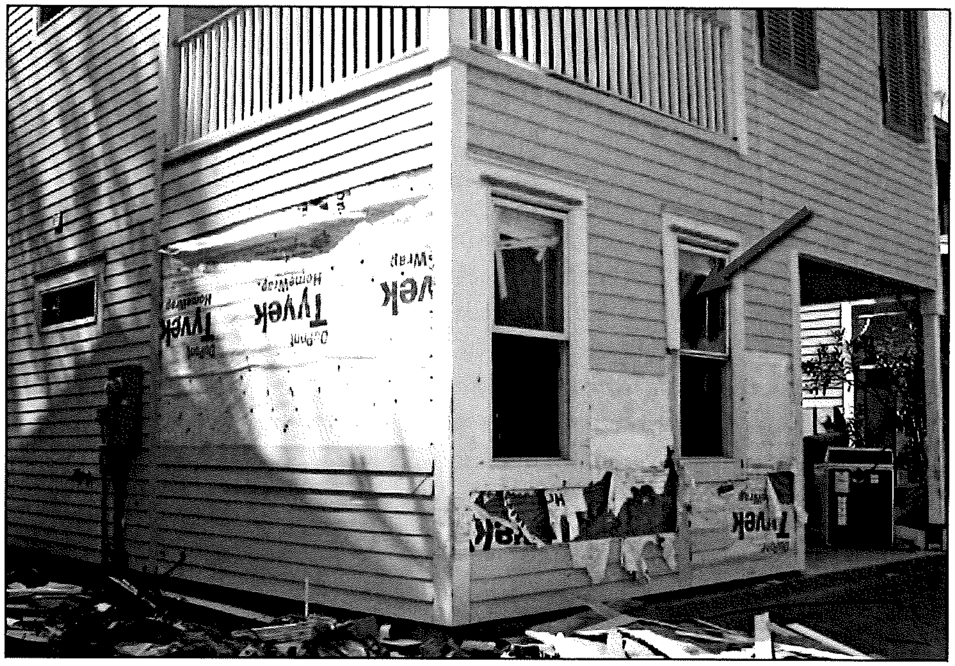


Figure 11-16.
Blown-off fiber cement siding; broken window (arrow). Estimated wind speed: 125 mph. Hurricane Katrina (Mississippi, 2006)

Figure 11-17.
Four brick veneer failure modes; five corrugated ties that were not embedded in the mortar joints (inset). Hurricane Ivan (Florida, 2004)

A key to the successful performance of brick veneer is attachment with a sufficient number of properly located ties and proper tie fasteners (based on design loads and tested resistance). Fact Sheet 5.4, *Attachment of Brick Veneer in High-Wind Regions,* in FEMA P-499 provides guidance on specifying and installing brick veneer in high-wind regions.

Exterior Insulating Finishing System

EIFS can be applied over steel-frame, wood-frame, concrete, or CMU construction. An EIFS assembly is composed of several types of materials, as illustrated in Figure 11-18. 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 size, and of sufficient number (based on design loads and tested resistance). Most EIFS failures are caused by an inadequate number of fasteners or an inadequate amount of adhesive.

> **NOTE**
>
> When a window or door assembly is installed in an EIFS wall assembly, sealant between the window or door frame and the EIFS should be applied to the EIFS base coat. After sealant application, the top coat is then applied. The top coat is somewhat porous; if sealant is applied to it, water can migrate between the top and base coats and escape past the sealant.

At the residence shown in Figure 11-19, the synthetic stucco was installed over molded expanded polystyrene (MEPS) insulation that was adhered to gypsum board that was mechanically attached to wood studs. Essentially all of the gypsum board blew off (the boards typically pulled over the fasteners). The failure was initiated by detachment of the gypsum board or by stud blow off. Some of the gypsum board on the interior side of the studs was also blown off. Also, two windows were broken by debris.

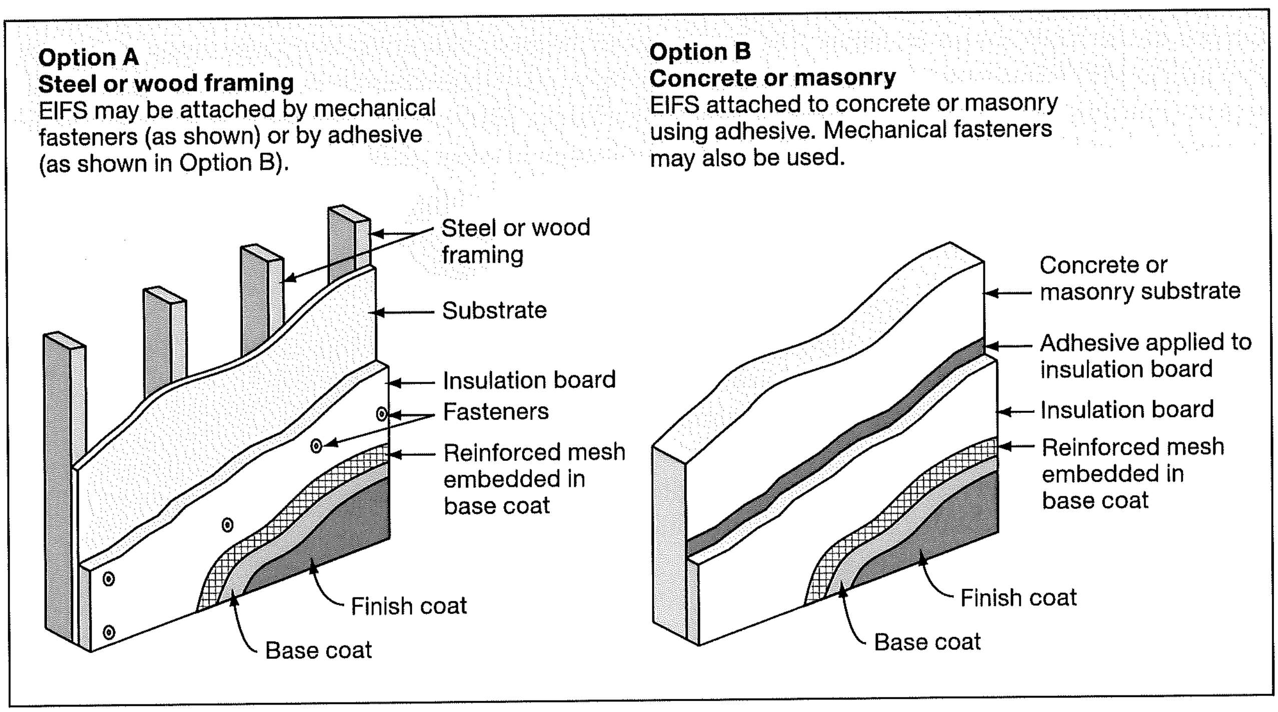


Figure 11-18.
Typical EIFS assemblies

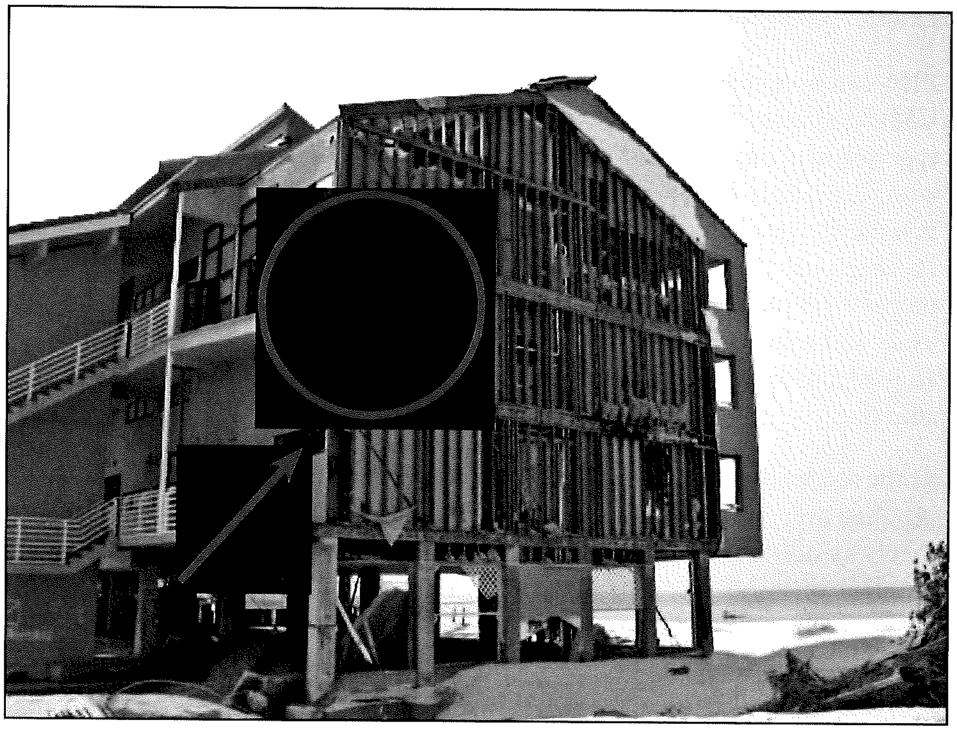


Figure 11-19.
Blown-off EIFS, resulting in extensive interior water damage; detachment of the gypsum board or stud blow off (circle); two windows broken by debris (arrow). Estimated wind speed: 105 to 115 mph. Hurricane Ivan (Florida, 2004)

Several of the studs shown in Figure 11-19 were severely rotted, indicating long-term moisture intrusion behind the MEPS insulation. The residence shown in Figure 11-19 had a barrier EIFS design, rather than the newer drainable EIFS design (for another example of a barrier EIFS design, see Figure 11-21). EIFS should be designed with a drainage system that allows for dissipation of water leaks.

Concrete and Concrete Masonry Unit

Properly designed and constructed concrete and CMU walls are capable of providing resistance to high-wind loads and wind-borne debris. When concrete and CMU walls are exposed to sustained periods of rain and high wind, it is possible for water to be driven through these walls. While both the IBC and IRC allow concrete and CMU walls to be installed without water-resistive barriers, the design professional should consider water-penetration-resistance treatments.

> **NOTE**
>
> Insulated versions of flood-opening devices can be used when enclosures are insulated. Flood openings are recommended in breakaway walls in Zone V and required in foundation walls and walls of enclosures in Zone A and Coastal A Zones.

Breakaway Walls

Breakaway walls (enclosures) are designed to fail under base flood conditions without jeopardizing the elevated building. Breakaway walls should also be designed and constructed so that when they break away, they do so without damaging the wall above the line of separation.

> **CROSS REFERENCE**
>
> For information on breakaway walls, see Fact Sheet 8.1, *Enclosures and Breakaway Walls,* in FEMA P-499.

At the house shown in Figure 11-20, floodwater collapsed the breakaway wall and initiated progressive peeling of the EIFS wall covering. A suitable flashing at the top of the breakaway wall would have avoided the progressive failure. When a wall covering progressively fails above the top of a breakaway wall, wave spray and/or wind-driven water may cause interior damage.

Figure 11-20.
Collapse of the breakaway wall, resulting in EIFS peeling. A suitable transition detail at the top of breakaway walls avoids the type of peeling damage shown by the arrows. Estimated wind speed: 105 to 115 mph. Hurricane Ivan (Alabama, 2004)

11.4.1.2 Flashings

Water infiltration at wall openings and wall transitions due to poor flashing design and/or installation is a common problem in many coastal homes (see Figure 11-21). In areas that experience frequent wind-driven rain and areas susceptible to high winds, enhanced flashing details and attention to their execution are recommended. Enhancements include flashings that have extra-long flanges, use of sealant, and use of self-adhering modified bitumen tape.

When designing flashing, the design professional should recognize that wind-driven rain 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) unless there is sealant between the layers.

> **NOTE**
>
> Some housewrap manufacturers have comprehensive, illustrated installation guides that address integrating housewrap and flashings at openings.

A key to successful water diversion is installing layers of building materials correctly to avoid water getting behind any one layer and leaking into the building. General guidance is offered below, design professionals should also attempt to determine the type of flashing details that have been used successfully in the area.

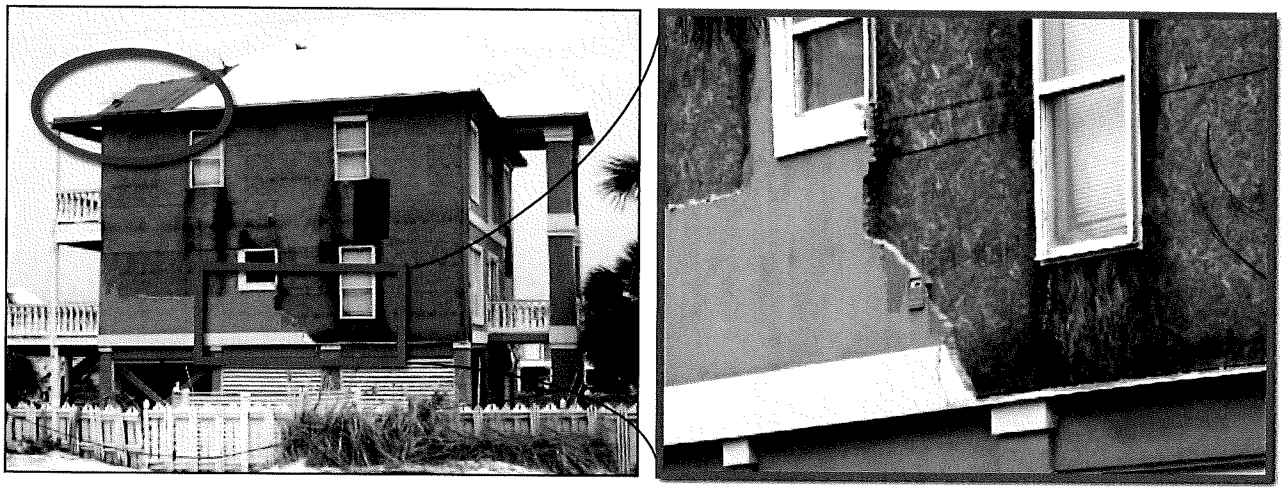


Figure 11-21.
EIFS with a barrier design: blown-off roof decking (top circle); severely rotted OSB due to leakage at windows (inset). Hurricane Ivan (2004)

Door and Window Flashings

An important aspect of flashing design and application is the integration of the door and window flashings with the moisture barrier. See the recommendations in FMA/AAMA 100, FMA/AAMA 200, and Salzano et al. (2010), as described in Section 11.3.1.4, regarding installation of doors and windows, as well as the recommendations given in Fact Sheet 5.1, *Housewrap*, in FEMA P-499. Applying self-adhering modified bitumen flashing tape at doors and windows is also recommended.

Roof-to-Wall and Deck-to-Wall Flashing

Where enhanced protection at roof-to-wall intersections is desired, step flashing with a vertical leg that is 2 to 4 inches longer than normal is recommended. For a more conservative design, in addition to the long leg, the top of the vertical flashing can be taped to the wall sheathing with 4-inch-wide self-adhering modified bitumen tape (approximately 1 inch of tape on the metal flashing and 3 inches on the sheathing). The housewrap should be extended over the flashing in the normal fashion. The housewrap should not be sealed to the flashing—if water reaches the backside of the housewrap farther up the wall, it needs to be able to drain out at the bottom of the wall. This detail and a deck-to-wall flashing detail are illustrated in Fact Sheet No. 5.2, *Roof-to-Wall and Deck-to-Wall Flashing*, in FEMA P-499.

11.4.1.3 Soffits

Depending on the wind direction, soffits can be subjected to either positive or negative pressure. Failed soffits may provide a convenient path for wind-driven rain to enter the building, as illustrated by Figure 11-22. This house had a steep-slope roof with a ventilated attic space. The exterior CMU/stucco wall stopped just above the vinyl soffit. Wind-driven rain entered the attic space where the soffit had blown away. This example and other storm-damage research have shown that water blown into attic spaces after the loss of soffits can cause significant damage and the collapse of ceilings. Even when soffits remain in place, water can penetrate through soffit vents and cause damage (see Section 11.6).

Figure 11-22. Blown-away soffit (arrow), which allowed wind-driven rain to enter the attic. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Loading criteria for soffits were added in ASCE 7-10. At this time, the only known test standard pertaining to soffit wind and wind-driven rain resistance is the FBC *Testing Application Standard (TAS) No. 100(A)-95* (ICC 2008). Wind-pressure testing is conducted to a maximum test speed of 140 mph, and wind-driven rain testing is conducted to a maximum test speed of 110 mph. Laboratory research has shown the need for an improved test method to evaluate the wind pressure and wind-driven rain resistance of soffits.

Plywood or wood soffits are generally adequately anchored to wood framing attached to the roof structure or walls. However, it has been common practice for vinyl and aluminum soffit panels to be installed in tracks that are frequently poorly connected to the walls and fascia at the edge of the roof overhang. Properly installed vinyl and aluminum soffit panels should be fastened to the building structure or to nailing strips placed at intervals specified by the manufacturer. Key elements of soffit installation are illustrated in Fact Sheet 7.5, *Minimizing Water Intrusion Through Roof Vents in High-Wind Regions*, in FEMA P-499.

11.4.1.4 Durability

For buildings within 3,000 feet of an ocean shoreline (including sounds and back bays), stainless steel fasteners are recommended for wall and soffit systems. For other components (e.g., furring, blocking, struts, hangers), nonferrous components (such as wood), stainless steel, or steel with a minimum of G-90 hot-dipped galvanized coating are recommended. Additionally, access panels are recommended so components within soffit cavities can be inspected periodically for corrosion or wood decay.

See "Material Durability in Coastal Environments," a resource document located on the Residential Coastal Construction Web site, for information on wood treatment if wood is specified in areas with severe termite problems.

11.4.2 Seismic

Concrete and CMU walls need to be designed for the seismic load. When a heavy covering such as brick veneer or stucco is specified, the seismic design should account for the added weight of the covering. Inadequate connection of veneer material to the base substrate has been a problem in earthquakes and can result in a life-safety hazard. For more information on the seismic design of brick veneer, see Fact Sheet 5.4, *Attachment of Brick Veneer in High-Wind Regions*, in FEMA P-499.

Some non-ductile coverings such as stucco can be cracked or spalled during seismic events. If these coverings are specified in areas prone to large ground-motion accelerations, the structure should be designed with additional stiffness to minimize damage to the wall covering.

11.5 Roof Systems

This section addresses roof systems. High winds, seismic events, and hail are the natural hazards that can cause the greatest damage to roof systems in the coastal environment. When high winds damage the roof covering, water infiltration commonly occurs and can cause 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 tile or slate may be dislodged and fall from the roof and present a hazard. A roof system that is not highly resistant to fire exposure can result in the destruction of the building during a wildfire.

NOTE

When reroofing in high-wind areas, the existing roof covering should be removed rather than re-covered so that the roof deck can be checked for deterioration and adequate attachment. See Figure 11-23. Also see Chapter 14 in this Manual.

NOTE

Historically, damage to roof systems has been the leading cause of building performance problems during high winds.

Residential buildings typically have steep-slope roofs (i.e., a slope greater than 3:12), but some have low-slope roofs. Low-slope roof systems are discussed in Section 11.5.8.

A variety of products can be used for coverings on steep-slope roofs. The following commonly used products are discussed in this section: asphalt shingles, cement-fiber shingles, liquid-applied membranes, tiles, metal panels, metal shingles, slate, and wood shingles and shakes. The liquid-applied membrane and metal panel systems are air-impermeable, and the other systems are air-permeable.[11]

At the residence shown in Figure 11-23, new asphalt shingles had been installed on top of old shingles. Several of the newer shingles blew off. Re-covering over old shingles causes more substrate irregularity, which can interfere with the bonding of the self-seal adhesive of the new shingles.

11 Air permeability of the roof system affects the magnitude of air pressure that is applied to the system during a wind storm.

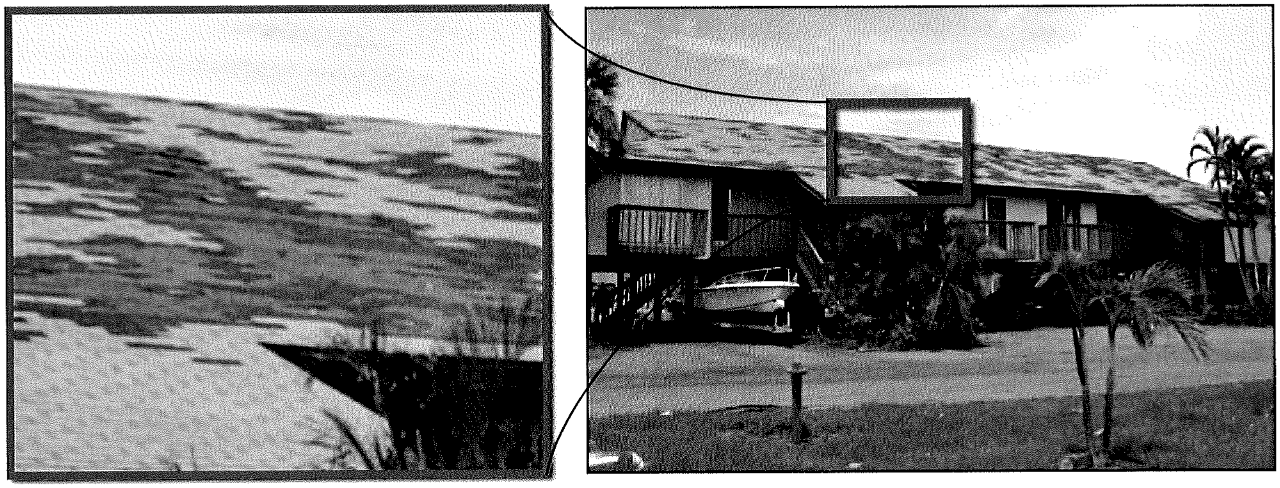


Figure 11-23.
Blow-off of several newer shingles on a roof that had been re-covered by installing new asphalt shingles on top of old shingles (newer shingles are lighter and older shingles are darker). Hurricane Charley (Florida, 2004)

11.5.1 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.

11.5.1.1 High Winds

The key elements to the successful wind performance of asphalt shingles are the bond strength of the self-sealing adhesive; mechanical properties of the shingle; correct installation of the shingle fasteners; and enhanced attachment along the eave, hip, ridge, and rakes. In addition to the tab lifts, the number and/or location of fasteners used to attach the shingles may influence whether shingles are blown off.

Underlayment

If shingles blow off, water infiltration damage can be avoided if the underlayment remains attached and is adequately sealed at penetrations. Figures 11-24 and 11-25 show houses with underlayment that was not effective in avoiding water leakage. Reliable

NOTE

Neither ASCE 7, IBC, or IRC require roof assemblies to resist wind-borne debris. However, the FBC requires roof assemblies located in the High-Velocity Hurricane Zone (as defined by the FBC) to be tested for wind-borne debris or be deemed to comply with the wind-borne debris provisions as stipulated in the FBC.

NOTE

Storm damage investigations have revealed that gutters are often susceptible to blow-off. ANSI/SPRI GD-1, *Structural Design Standard for Gutter Systems Used with Low-Slope Roofs* (ANSI/SPRI 2010) provides information on gutter wind and water and ice loads and includes methods for testing gutter resistance to these loads. Although the standard is intended for low-slope roofs, it should be considered when designing and specifying gutters used with steep-slope roofs.

ANSI/SPRI GD-1 specifies a minimum safety factor of 1.67, but a safety factor of 2 is recommended.

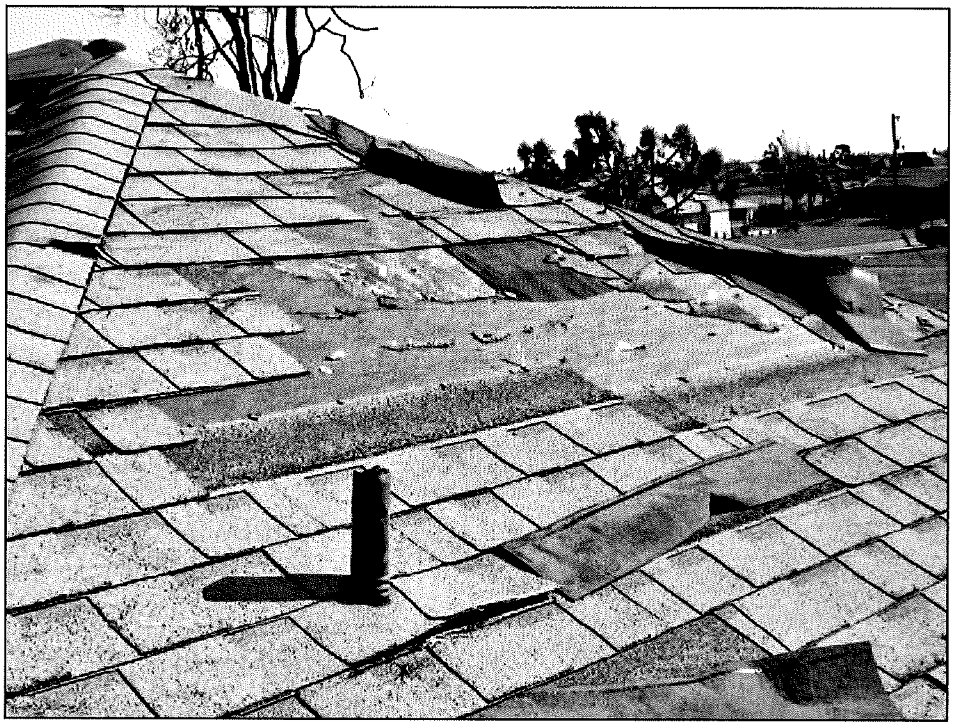


Figure 11-24.
Small area of sheathing that was exposed after loss of a few shingles and some underlayment. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Figure 11-25.
Typical underlayment attachment; underlayment blow-off is common if the shingles are blown off, as shown. Estimated wind speed: 115 mph. Hurricane Katrina (Louisiana, 2005)

secondary protection requires an enhanced underlayment design. Design enhancements include increased blow-off resistance of the underlayment, increased resistance to water infiltration (primarily at penetrations), and increased resistance to extended weather exposure.

If shingles are blown off, the underlayment may be exposed for only 1 or 2 weeks before a new roof covering is installed, but many roofs damaged by hurricanes are not repaired for several weeks. If a hurricane strikes a heavily populated area, roof covering damage is typically extensive. Because of the heavy workload, large numbers of roofs may not be repaired for several months. It is not uncommon for some roofs to be left for as long as a year before they are reroofed.

The longer an underlayment is exposed to weather, the more durable it must be to provide adequate water infiltration protection for the residence. Fact Sheet 7.2, *Roof Underlayment for Asphalt Shingle Roofs*, in FEMA P-499 provides three primary options for enhancing the performance of underlayment if shingles are blown off. The options in the fact sheet are listed in order of decreasing resistance to long-term weather exposure. The fact sheet provides guidance for option selection, based on the design wind speed and population of the area. The following is a summary of the enhanced underlayment options:

- **Enhanced Underlayment Option 1.** Option 1 provides the greatest reliability for long-term exposure. This option includes a layer of self-adhering modified bitumen. Option 1 has two variations. The first variation is shown in Figure 11-26. In this variation, the self-adhering sheet is applied to the sheathing, and a layer of #15 felt is tacked over the self-adhering sheet before the shingles are installed. The purpose of the felt is to facilitate future tear-off of the shingles. This variation is recommended in southern climates (e.g., south of the border between North and South Carolina). If a house is located in moderate or cold climates or has a high interior humidity (such as from an indoor swimming pool), the second variation, shown in Figure 11-27, is recommended.

NOTE

Some OSB has a factory-applied wax that interferes with the bonding of self-adhering modified bitumen. To facilitate bonding to waxed sheathing, a field-applied primer is needed. If self-adhering modified bitumen sheet or tape is applied to OSB, the OSB manufacturer should be contacted to determine whether a primer needs to be applied to the OSB.

In the second variation (Figure 11-27), the sheathing joints are taped with self-adhering modified bitumen. A #30 felt is then nailed to the sheathing, and a self-adhering modified bitumen sheet is applied to the felt before the shingles are installed. The second variation costs more than the first variation because the second variation requires sheathing tape, many more felt fasteners, and heavier felt. The purpose of taping the joints

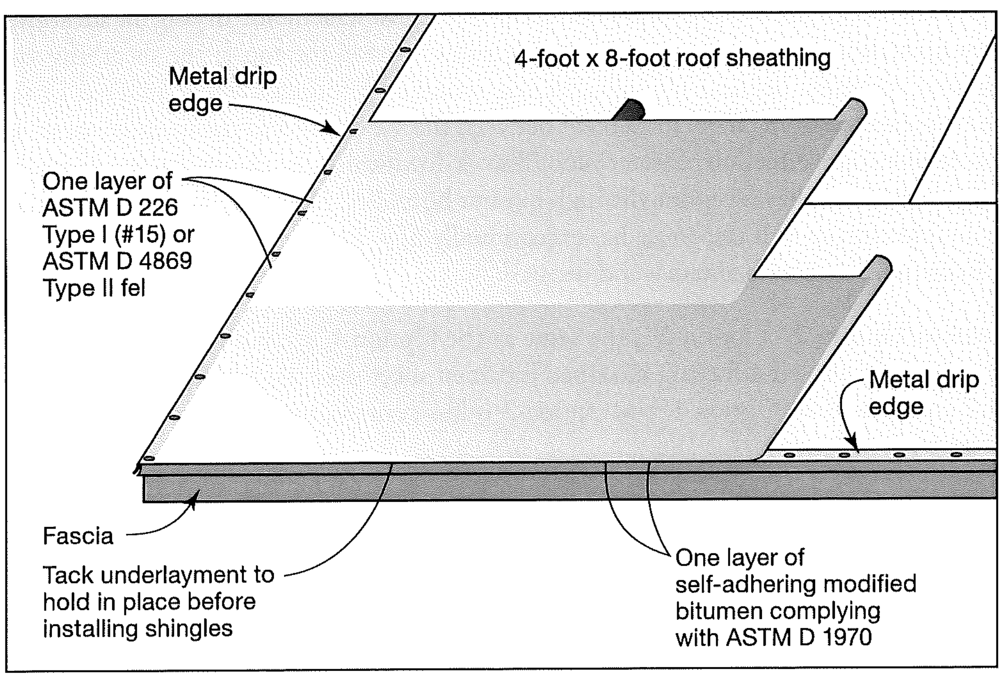
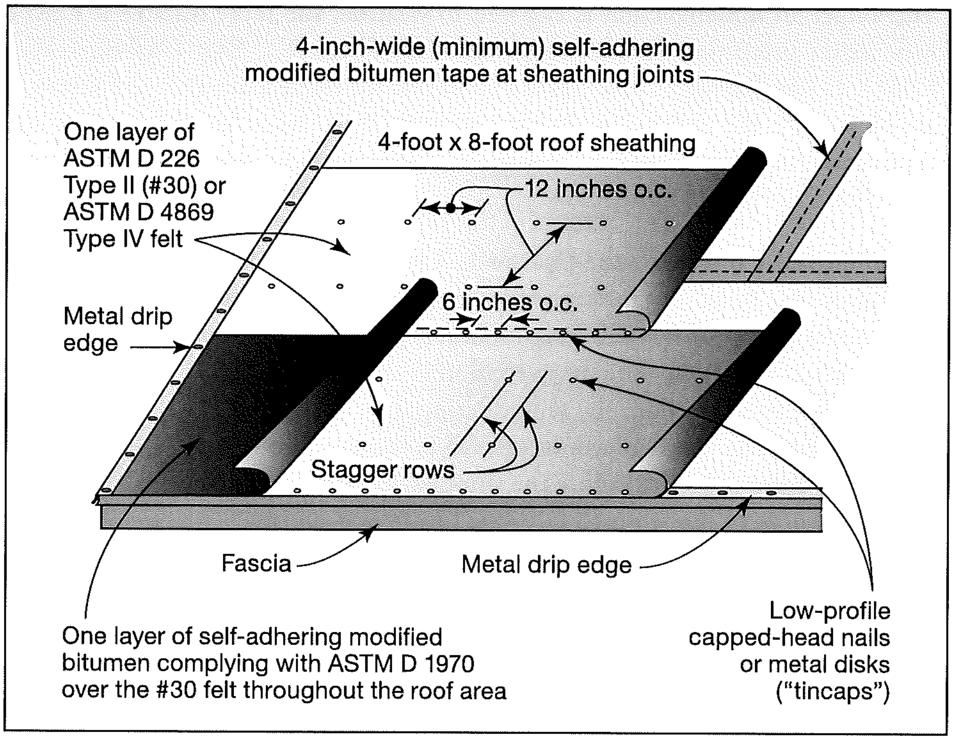


Figure 11-26. Enhanced underlayment Option 1, first variation: self-adhering modified bitumen over the sheathing

Figure 11-27.
Enhanced underlayment Option 1, second variation: self-adhering modified bitumen over the felt



is to avoid leakage into the residence if the felt blows off or is torn by wind-borne debris. (Taping the joints is not included in the first variation, shown in Figure 11-26, because with the self-adhering modified bitumen sheet applied directly to the sheathing, sheet blow-off is unlikely, as is water leakage caused by tearing of the sheet by debris.)

The second variation is recommended in moderate and cold climates because it facilitates drying the sheathing because water vapor escaping from the sheathing can move laterally between the top of the sheathing and the nailed felt. In the first variation, because the self-adhering modified bitumen sheet is adhered to the sheathing, water vapor is prevented from lateral movement between the sheathing and the underlayment. In hot climates where the predominate direction of water vapor flow is downward, the sheathing should not be susceptible to decay unless the house has exceptionally high interior humidity. However, if the first variation is used in a moderate or cold climate or if the house has exceptionally high interior humidity, the sheathing may gain enough moisture over time to facilitate wood decay.[12]

Enhanced Underlayment Option 2. Option 2 is the same as the Option 1, second variation, except that Option 2 does not include the self-adhering modified bitumen sheet over the felt and uses two layers of felt. Option 2 costs less than Option 1, but Option 2 is less conservative. Option 2 is illustrated in Fact Sheet 7.2 in FEMA P-499.

12 Where self-adhering modified bitumen is applied to the sheathing to provide water leakage protection from ice dams along the eave, long-term experience in the roofing industry has shown little potential for development of sheathing decay. However, sheathing decay has occurred when the self-adhering sheet is applied over all of the sheathing in cold climate areas.

- **Enhanced Underlayment Option 3.** Option 3 is the typical underlayment scheme (i.e., a single layer of #15 felt tacked to the sheathing, as shown in Figure 11-25) with the added enhancement of self-adhering modified bitumen tape. This option provides limited protection against water infiltration if the shingles blow off. However, this option provides more protection than the typical underlayment scheme. Option 3 is illustrated in Fact Sheet 7.2 in FEMA P-499.

Figure 11-28 shows a house that used Option 3. The self-adhering modified bitumen tape at the sheathing joints was intended to be a third line of defense against water leakage (with the shingles the first line and the felt the second line). However, as shown in the inset at Figure 11-28, the tape did not provide a watertight seal. A post-storm investigation revealed application problems with the tape. Staples (arrow, inset) were used to attach the tape because bonding problems were experienced during application. Apparently, the applicator did not realize the tape was intended to prevent water from leaking through the sheathing joints. With the tape in an unbonded and wrinkled condition, it was incapable of fulfilling its intended purpose.

Self-adhering modified bitumen sheet and tape normally bond quite well to sheathing. Bonding problems are commonly attributed to dust on the sheathing, wet sheathing, or a surfacing (wax) on the sheathing that interfered with the bonding.

In addition to taping the sheathing joints in the field of the roof, the hip and ridge lines should also be taped unless there is a continuous ridge vent, and the underlayment should be lapped over the hip and ridge. By doing so, leakage will be avoided if the hip or ridge shingles blow off (see Figure 11-29). See Section 11.6 for recommendations regarding leakage avoidance at ridge vents.

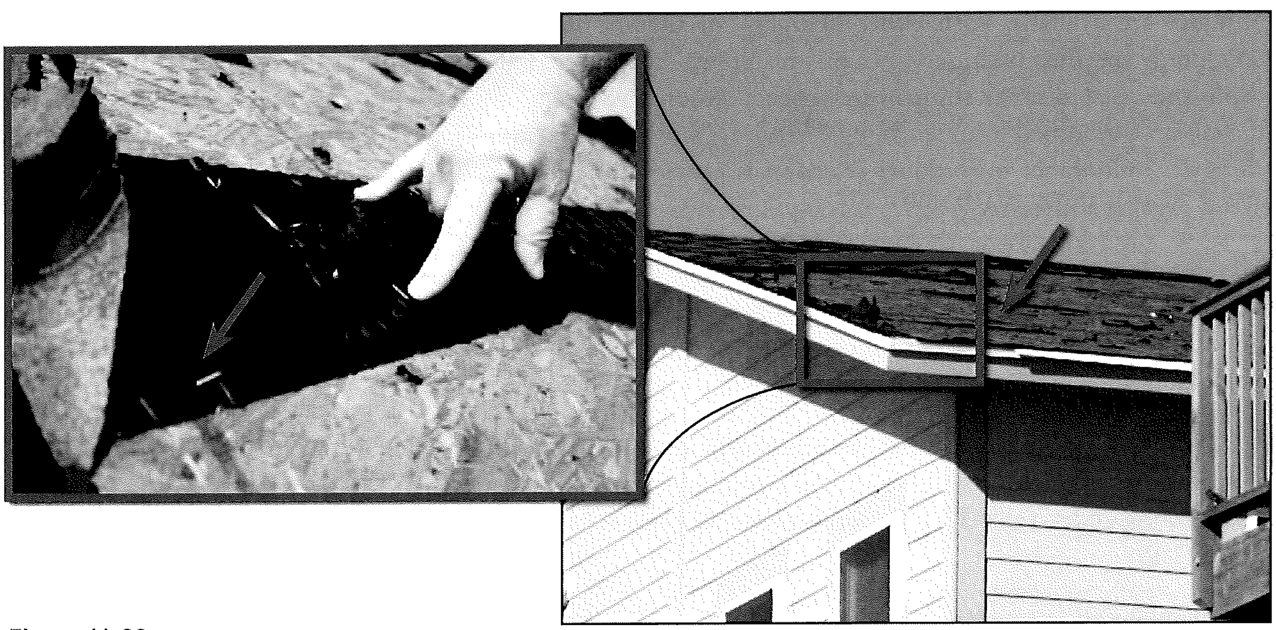


Figure 11-28.
House that used enhanced underlayment Option 3 with taped sheathing joints (arrow). The self-adhering modified bitumen tape (inset) was stapled because of bonding problems. Estimated wind speed: 110 mph. Hurricane Ike (Texas, 2008)
SOURCE: IBHS, USED WITH PERMISSION

Figure 11-29. Underlayment that was not lapped over the hip; water entry possible at the sheathing joint (arrow). Estimated wind speed: 130 mph. Hurricane Katrina (Mississippi, 2005)

Shingle Products, Enhancement Details, and Application

Shingles are available with either fiberglass or organic reinforcement. Fiberglass-reinforced shingles are commonly specified because they have greater fire resistance. Fiberglass-reinforced styrene-butadiene-styrene (SBS)-modified bitumen shingles are another option. Because of the flexibility imparted by the SBS polymers, if a tab on a modified bitumen shingle lifts, it is less likely to tear or blow off compared to traditional asphalt shingles.[13] Guidance on product selection is provided in Fact Sheet 7.3, *Asphalt Shingle Roofing for High-Wind Regions*, in FEMA P-499.

The shingle product standards referenced in Fact Sheet 7.3 specify a minimum fastener (nail) pull-through resistance. However, if the basic wind speed is greater than 115 mph,[14] the Fact Sheet 7.3 recommends minimum pull-through values as a function of wind speed. If a fastener pull-through resistance is desired that is greater than the minimum value given in the product standards, the desired value needs to be specified.

ASTM D7158 addresses wind resistance of asphalt shingles.[15] ASTM D7158 has three classes: Class D, G, and H. Select shingles that have a class rating equal to or greater than the basic wind speed prescribed in the building code. Table 11-1 gives the allowable basic wind speed for each class, based on ASCE 7-05 and ASCE 7-10.

Shingle blow-off is commonly initiated at eaves (see Figure 11-30) and rakes (see Figure 11-31). Blow-off of ridge and hip shingles, as shown in Figure 11-29, is also common. For another example of blow-off of ridge

13 Tab lifting is undesirable. However, lifting may occur for a variety of reasons. If lifting occurs, a product that is not likely to be torn or blown off is preferable to a product that is more susceptible to tearing and blowing off.

14 The 115-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05, or an earlier version is used, the equivalent wind speed trigger is 90 mph.

15 Fact Sheet 7.3 in FEMA P-499 references Underwriters Laboratories (UL) 2390. ASTM D7158 supersedes UL 2390.

Table 11-1. Allowable Basic Wind Speed as a Function of Class

ASTM D7158 Class[(a)]	Allowable Basic Wind Speed	
	Based on ASCE 7-05	Based on ASCE 7-10
D	90 mph	115 mph
G	120 mph	152 mph
H	150 mph	190 mph

(a) Classes are based on a building sited in Exposure C. They are also based on a building sited where there is no abrupt change in topography. If the residence is in Exposure D and/or where there is an abrupt change in topography (as defined in ASCE 7), the design professional should consult the shingle manufacturer.

Figure 11-30.
Loss of shingles and underlayment along the eave and loss of a few hip shingles. Estimated wind speed: 115 mph. Hurricane Katrina (Louisiana, 2005)

Figure 11-31.
Loss of shingles and underlayment along the rake. Estimated wind speed: 110 mph. Hurricane Ike (Texas, 2008)

and hip shingles, see Figure 11-35. Fact Sheet 7.3 in FEMA P-499 provides enhanced eave, rake, and hip/ridge information that can be used to avoid failure in these areas.

Storm damage investigations have shown that when eave damage occurs, the starter strip was typically incorrectly installed, as shown in Figure 11-32. Rather than cutting off the tabs of the starter, the starter was rotated 180 degrees (right arrow). The exposed portion of the first course of shingles (left arrow) was unbounded because the self-seal adhesive (dashed line) on the starter was not near the eave. Even when the starter is correctly installed (as shown on shingle bundle wrappers), the first course may not bond to the starter because of substrate variation. Fact Sheet 7.3 in FEMA P-499 provides information about enhanced attachment along the eave, including special recommendations regarding nailing, use of asphalt roof cement, and overhang of the shingle at the eave.

Figure 11-32. Incorrect installation of the starter course (incorrectly rotated starter, right arrow, resulted in self-seal adhesive not near the eave, dashed line). Estimated wind speed: 130 mph. Hurricane Katrina (Mississippi, 2005)

Storm damage investigations have shown that metal drip edges (edge flashings) with vertical flanges that are less than 2 inches typically do not initiate eave or rake damage. However, the longer the flange, the greater the potential for flange rotation and initiation of damage. If the vertical flange exceeds 2 inches, it is recommended that the drip edge be in compliance with ANSI/SPRI ES-1.

As with eaves, lifting and peeling failure often initiates at rakes and propagates into the field of the roof, as shown in Figure 11-33. Rakes are susceptible to failure because of the additional load exerted on the overhanging shingles and the configuration of the self-sealing adhesive. Along the long dimension of the shingle (i.e., parallel to the eave), the tab is sealed with self-sealing adhesive that is either continuous or nearly so. However, along the rake, the ends of the tab are only sealed at the self-seal lines, and the tabs are therefore typically sealed at about 5 inches on center. The result is that under high-wind loading, the adhesive at the rake end is stressed more than the adhesive farther down along the tab. With sufficient wind loading, the corner tab of the rake can begin to lift up and progressively peel, as illustrated in Figure 11-33.

Fact Sheet 7.3 in FEMA P-499 provides information about enhanced attachment along the rake, including recommendations regarding the use of asphalt roof cement along the rake. Adding dabs of cement, as shown in the Fact Sheet 7.3 in FEMA P-499 and Figure 11-33, distributes the uplift load across the ends of the rake shingles to the cement and self-seal adhesive, thus minimizing the possibility of tab uplift and progressive peeling failure.

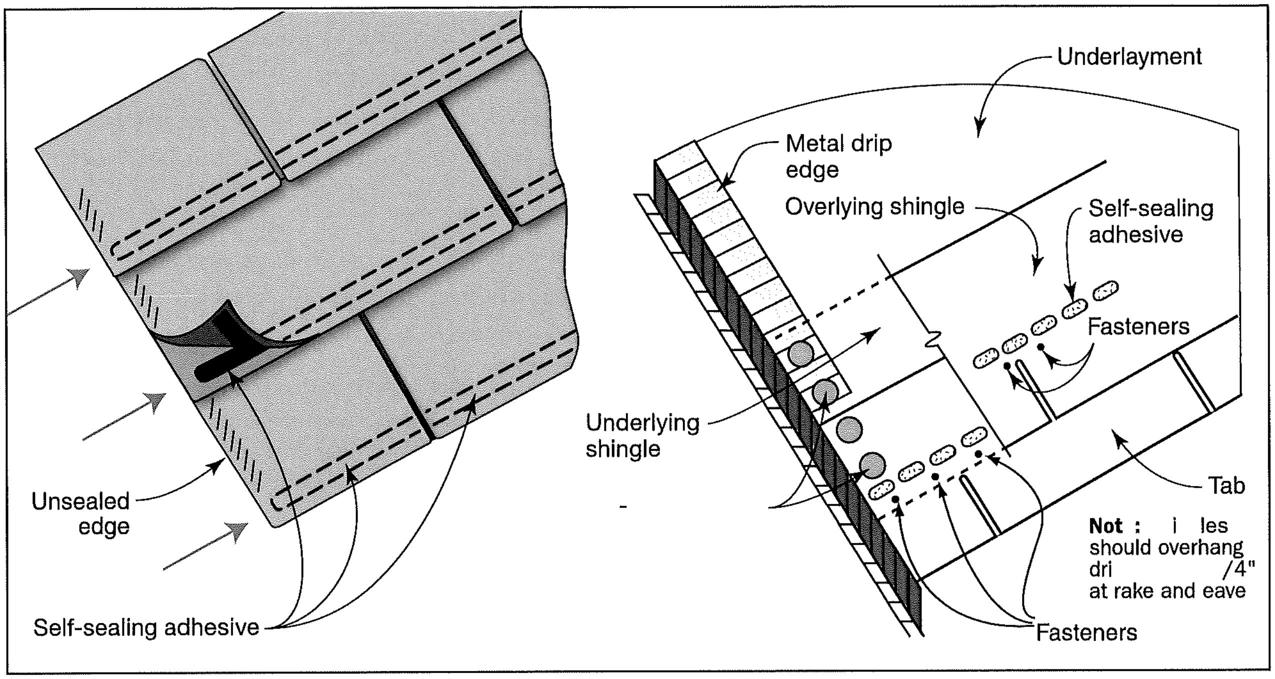


Figure 11-33.
Uplift loads along the rake that are transferred (illustrated by arrows) to the ends of the rows of self-sealing adhesive. When loads exceed resistance of the adhesive, the tabs lift and peel. The dabs of cement adhere the unsealed area shown by the hatched lines in the drawing on the left

Storm damage investigations have shown that on several damaged roofs, bleeder strips had been installed. Bleeder strips are shingles that are applied along the rake, similar to the starter course at the eave, as shown at Figure 11-34. A bleeder provides an extended straight edge that can be used as a guide for terminating the rake shingles. At first glance, it might be believed that a bleeder enhances wind resistance along the rake. However, a bleeder does not significantly enhance resistance because the concealed portion of the overlying rake shingle is the only portion that makes contact with the self-seal adhesive on the bleeder. As can be seen in Figure 11-34, the tab does not make contact with the bleeder. Hence, if the tab lifts, the shingle is placed in peel mode, which can easily break the bond with the bleeder. Also, if the tabs are not cut from the bleeder and the cut edge is placed along the rake edge, the bleeder's adhesive is too far inward to be of value.

If bleeder strips are installed for alignment purposes, the bleeder should be placed over the drip edge and attached with six nails per strip. The nails should be located 1 inch to 2 1/2 inches from the outer edge of the bleeder (1 inch is preferred if framing conditions permit). Dabs of asphalt roof cement are applied, similar to what is shown in Fact Sheet 7.3 in FEMA P-499. Dabs of asphalt roof cement are applied between the bleeder and underlying shingle, and dabs of cement are applied between the underlying and overlying shingles.

Storm damage investigations have shown that when hip and ridge shingles are blown off, there was a lack of bonding of the self-seal adhesive. Sometimes some bonding occurred, but frequently none of the adhesive had bonded. At the hip shown in Figure 11-35, the self-seal adhesive made contact only at a small area on the right side of the hip (circle). Also, at this hip, the nails were above, rather than below, the adhesive line. Lack of bonding of the hip and ridge shingles is common and is caused by substrate irregularity along the hip/ridge line. Fact Sheet 7.3 in FEMA P-499 provides recommendations regarding the use of asphalt roof cement to ensure bonding in order to enhance the attachment of hip and ridge shingles.

Figure 11-34.
A bleeder strip (double-arrow) that was used at a rake blow-off; lack of contact between the tab of the overlying shingle and the bleeder's self-seal adhesive (upper arrow). Estimated wind speed: 125 mph. Hurricane Katrina (Mississippi, 2005)

Figure 11-35.
Inadequate sealing of the self-sealing adhesive at a hip as a result of the typical hip installation procedure. Estimated wind speed: 105 mph. Hurricane Katrina (Mississippi, 2005)

Four fasteners per shingle are normally used where the basic wind speed is less than 115 mph.16 Where the basic wind speed is greater than 115 mph, six fasteners per shingle are recommended. Fact Sheet 7.3 in FEMA P-499 provides additional guidance on shingle fasteners. Storm damage investigations have shown that significant fastener mislocation is common on damaged roofs. When nails are too high above the nail line, they can miss the underlying shingle headlap or have inadequate edge distance, as illustrated

16 The 115-mph basic wind speed is based on ASCE 7-10, Risk Category II buildings. If ASCE 7-05 or an earlier version is used, the equivalent wind speed trigger is 90 mph.

in Figure 11-36. When laminated shingles are used, high nailing may miss the overlap of the laminated shingles; if the overlap is missed, the nail pull-through resistance is reduced (see Figure 11-37). High nailing may also influence the integrity of the self-seal adhesive bond by allowing excessive deformation (ballooning) in the vicinity of the adhesive.

The number of nails (i.e., four versus six) and their location likely play little role in wind performance as long at the shingles remain bonded. However, if they are unbounded prior to a storm, or debonded during a storm, the number and location of the nails and the shingles' nail pull-through resistance likely play an important role in the magnitude of progressive damage.

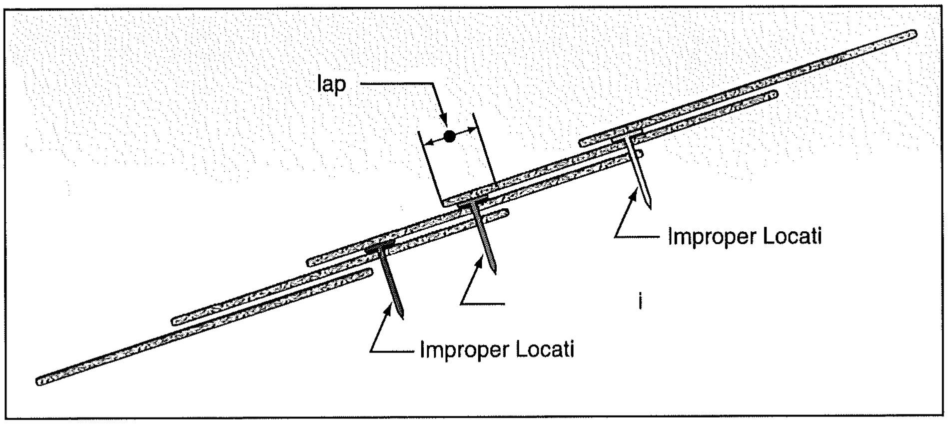


Figure 11-36.
Proper and improper location of shingle fasteners (nails). When properly located, the nail engages the underlying shingle in the headlap area (center nail). When too high, the nail misses the underlying shingle (left nail) or is too close to the edge of the underlying shingle (right nail)

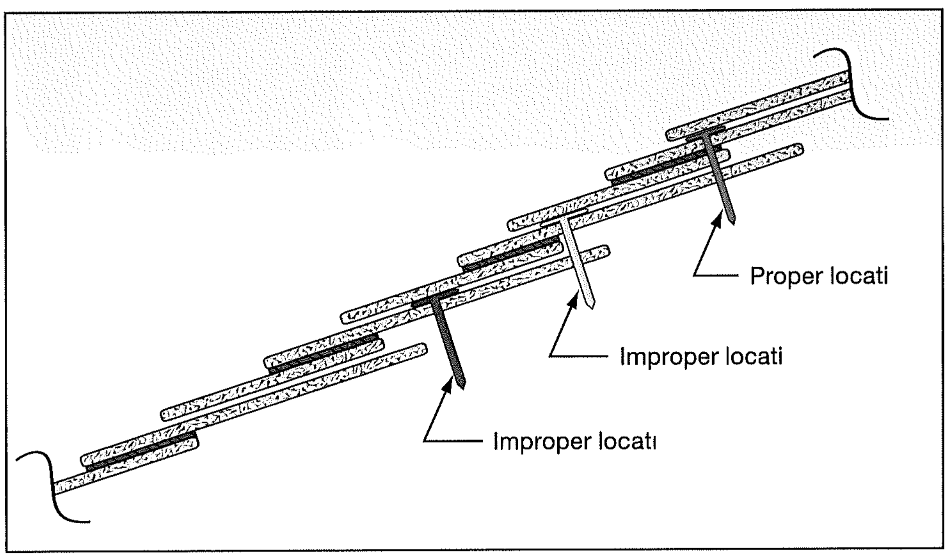
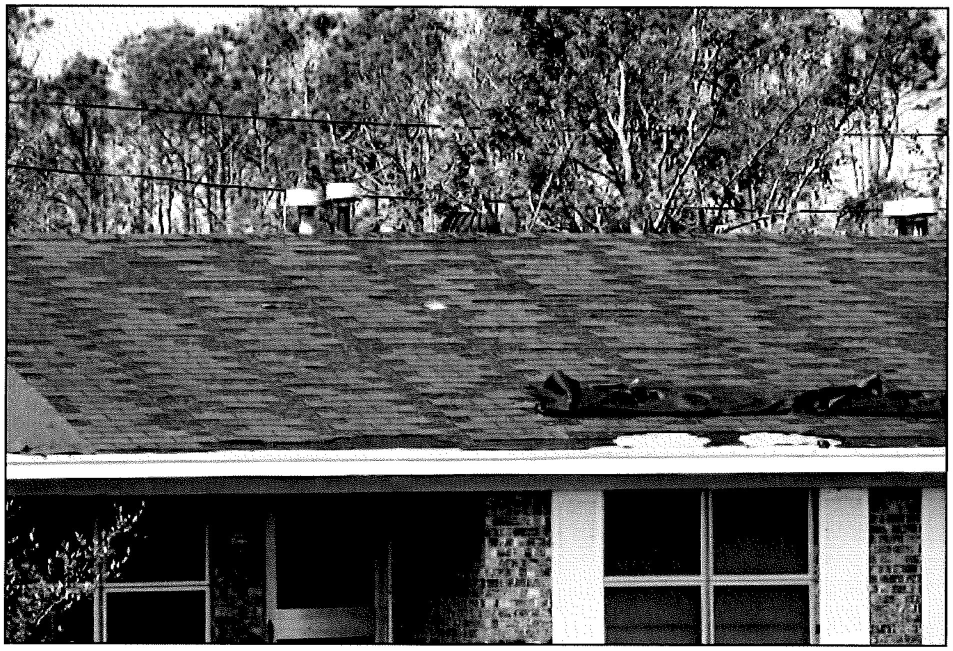


Figure 11-37.
Proper and improper location of laminated shingle fasteners (nails). With laminated shingles, properly located nails engage the underlying laminated portion of the shingle, as well as the headlap of the shingle below (right nail). When too high, the nail can miss the underlying laminated portion of the shingle but engage the headlap portion of the shingle (center nail), or the nail can miss both the underlying laminated portion of the shingle and the headlap of the underlying shingle (left nail)

Shingles manufactured with a wide nailing zone provide roofing mechanics with much greater opportunity to apply fasteners in the appropriate locations.

Shingle damage is also sometimes caused by installing shingles via the raking method. With this method, shingles are installed from eave to ridge in bands about 6 feet wide. Where the bands join one another, at every other course, a shingle from the previous row needs to be lifted up to install the end nail of the new band shingle. Sometimes installers do not install the end nail, and when that happens, the shingles are vulnerable to unzipping at the band lines, as shown in Figure 11-38. Raking is not recommended by the National Roofing Contractors Association or the Asphalt Roofing Manufacturers Association.

Figure 11-38.
Shingles that unzipped at the band lines because the raking method was used to install them. Estimated wind speed: 135 mph. Hurricane Katrina (Mississippi, 2005)



11.5.1.2 Hail

Underwriters Laboratories (UL) 2218 is a method of assessing simulated hail resistance of roofing systems. The test yields four ratings (Classes 1 to 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 in areas vulnerable to hail be specified to pass UL 2218 with a class rating that is commensurate with the hail load. Hail resistance of asphalt shingles depends partly on the condition of the shingles when they are exposed to hail. Shingle condition is likely to decline with roof age.

11.5.2 Fiber-Cement Shingles

Fiber-cement roofing products are manufactured to simulate the appearance of slate, tile, wood shingles, or wood shakes. The properties of various fiber-cement products vary because of differences in material composition and manufacturing processes.

11.5.2.1 High Winds

Because of the limited market share of fiber-cement shingles in areas where research has been conducted after high-wind events, few data are available on the wind performance of these products. Methods to calculate uplift loads and evaluate load resistance for fiber-cement products have not been incorporated into the IBC or IRC. Depending on the size and shape of the fiber-cement product, the uplift coefficient that is used for tile in the IBC may or may not be applicable to fiber-cement. If the fiber-cement manufacturer has determined that the tile coefficient is applicable to the product, Fact Sheet 7.4, *Tile Roofing for High-Wind Areas,* in FEMA P-499 is applicable for uplift loads and resistance. If the tile coefficient is not applicable, demonstrating compliance with ASCE 7 will be problematic with fiber-cement until suitable coefficient(s) have been developed.

Stainless steel straps, fasteners, and clips are recommended for roofs located within 3,000 feet of an ocean shoreline (including sounds and back bays). For underlayment recommendations, refer to the recommendation at the end of Section 11.5.4.1.

11.5.2.2 Seismic

Fiber-cement products are relatively heavy and, unless they are adequately attached, they can be dislodged during strong seismic events and fall from the roof. At press time, 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 Section 11.5.4.2 is recommended until guidance is developed for cement-fiber products.

11.5.2.3 Hail

It is recommended that fiber-cement shingle systems on buildings in areas vulnerable to hail be specified to pass UL 2218 at a class rating that is commensurate with the hail load. If products with the desired class are not available, another type of product should be considered.

11.5.3 Liquid-Applied Membranes

Liquid-applied membranes are not common on the U.S. mainland but are common in Guam, the U.S. Virgin Islands, Puerto Rico, and American Samoa.

11.5.3.1 High Winds

Investigations following hurricanes and typhoons have revealed that liquid-applied membranes installed over concrete and plywood decks have provided excellent protection from high winds if the deck remains attached to the building. This conclusion is based on performance during Hurricanes Marilyn and Georges. This type of roof covering over these deck types has high-wind-resistance reliability.

Unprotected concrete roof decks can eventually experience problems with corrosion of the slab reinforcement, based on performance observed after Hurricane Marilyn. All concrete roof decks are recommended to be covered with some type of roof covering.

11.5.3.2 Hail

It is recommended that liquid-applied membrane systems on buildings in areas vulnerable to hail be specified to pass UL 2218 or Factory Mutual Global testing with a class rating that is commensurate with the hail load.

11.5.4 Tiles

Clay and extruded concrete tiles are available in a variety of profiles and attachment methods.

11.5.4.1 High Winds

During storm damage investigations, a variety of tile profiles (e.g., S-tile and flat) of both clay and concrete tile roofs have been observed. No significant wind performance differences were attributed to tile profile or material (i.e., clay or concrete).

Figure 11-39 illustrates the type of damage that has often occurred during moderately high winds. Blow-off of hip, ridge, or eave tiles is caused by inadequate attachment. Damage to field tiles is typically caused by wind-borne debris (which is often tile debris from the eaves and hips/ridges). Many tile roofs occur over waterproof (rather than water-shedding) underlayment. Waterproof underlayments have typically been well-attached and therefore have not normally blown off after tile blow-off. Hence, many residences with tile roofs have experienced significant tile damage, but little, if any water infiltration from the roof. Figure 11-40 shows an atypical underlayment blow-off, which resulted in substantial water leakage into the house.

The four methods of attaching tile are wire-tied, mortar-set, mechanical attachment, and foam-adhesive (adhesive-set). Wire-tied systems are not commonly used in high-wind regions of the continental United States. On the roof shown in Figure 11-41, wire-tied tiles were installed over a concrete deck. Nose hooks occurred at the nose. In addition, a bead of adhesive occurred between the tiles at the headlap. Tiles at the first three perimeter rows were also attached with wind clips. The clips prevented the perimeter tiles from lifting. However, at the field of the roof, the tiles were repeatedly lifted and slammed against deck, which caused the tiles to break and blow away.

Damage investigations have revealed that mortar-set systems often provide limited wind resistance (Figure 11-42).[17] As a result of widespread poor performance of mortar-set systems during Hurricane Andrew (1992), adhesive-set systems were developed. Hurricane Charley (2004) offered the first opportunity to evaluate the field performance of this new attachment method during very high winds (see Figures 11-43 and 11-44).

Figure 11-43 shows a house with adhesive-set tile. There were significant installation problems with the foam paddies, including insufficient contact area between the patty and the tile. As can be seen in Figure 11-43, most of the foam failed to make contact with the tile. Some of the foam also debonded from the mineral surface cap sheet underlayment (see Figure 11-44).

Figure 11-45 shows tiles that were mechanically attached with screws. At the blow-off area, some of the screws remained in the deck, while others were pulled out. The ridge tiles were set in mortar.

17 Fact Sheet 7.4, *Tile Roofing for High-Wind Areas,* in FEMA 499 recommends that mechanical or adhesively attached methods be used in lieu of the mortar-set method.

Figure 11-39.
Blow-off of eave and hip tiles and some broken tiles in the field of the roof. Hurricane Ivan (Alabama, 2004)

Figure 11-40.
Large area of blown-off underlayment on a mortar-set tile roof. The atypical loss of waterproofing tile underlayment resulted in substantial water leakage into the house. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Figure 11-41.
Blow-off of wire-tied tiles installed over a concrete deck. Typhoon Paka (Guam, 1997)

Figure 11-42.
Extensive blow-off of mortar-set tiles. Hurricane Charley (Florida, 2004)

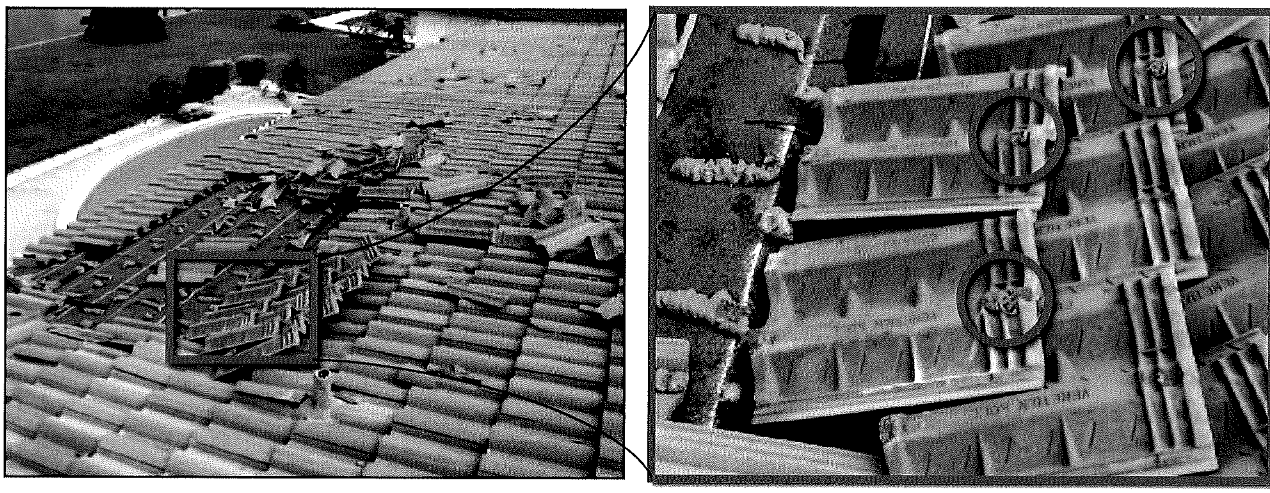


Figure 11-43.
Blown-off adhesive-set tile. Note the very small contact area of the foam at the tile heads (left side of the tiles) and very small contact at the nose (circles). Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Damage investigations have revealed that blow off of hip and ridge failures are common (see Figures 11-39, 11-45, and 11-46). Some of the failed hip/ridge tiles were attached with mortar (see Figure 11-45), while others were mortared and mechanically attached to a ridge board. At the roof shown in Figure 11-46, the hip tiles were set in mortar and attached to a ridge board with a single nail near the head of the hip tile.

Because of the brittle nature of tile, tile is often damaged by wind-borne debris, including tile from nearby buildings or tile from the same building (see Figure 11-47).

At houses on the coast, fasteners and clips that are used to attach tiles are susceptible to corrosion unless they are stainless steel. Figure 11-48 shows a 6-year-old tile roof on a house very close to the ocean that failed because the heads of the screws attaching the tile had corroded off. Stainless steel straps, fasteners, and clips are recommended for roofs within 3,000 feet of an ocean shoreline (including sounds and back bays).

Figure 11-44.
Adhesive that debonded from the cap sheet

Figure 11-45.
Blow-off of mechanically attached tiles. Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)

Figure 11-46.
Blow-off of hip tiles that were nailed to a ridge board and set in mortar. Hurricane Ivan (Florida, 2004)

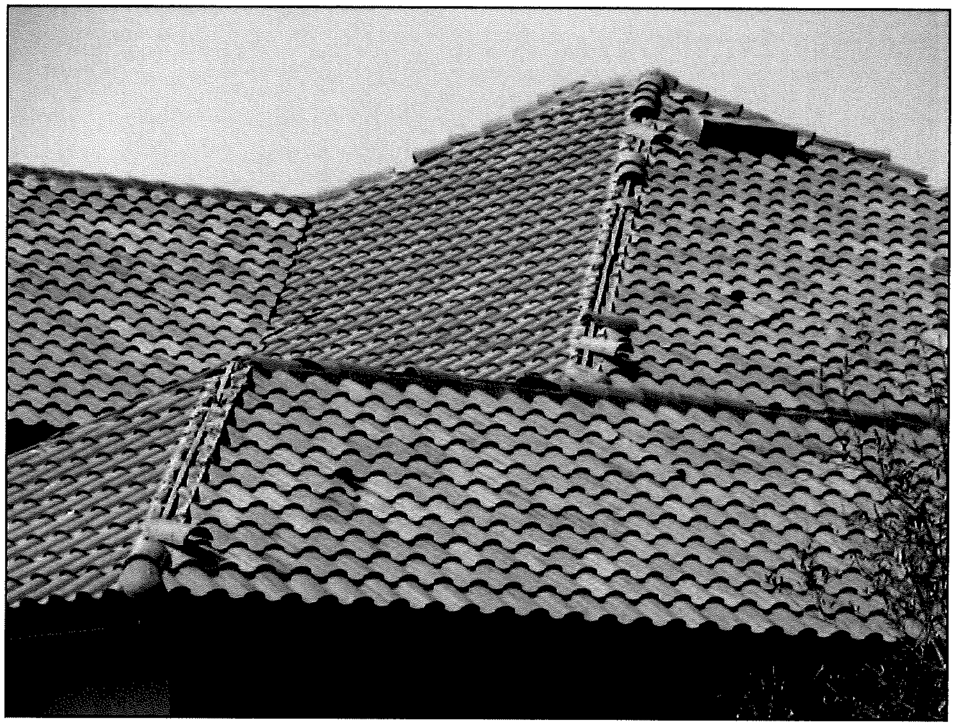
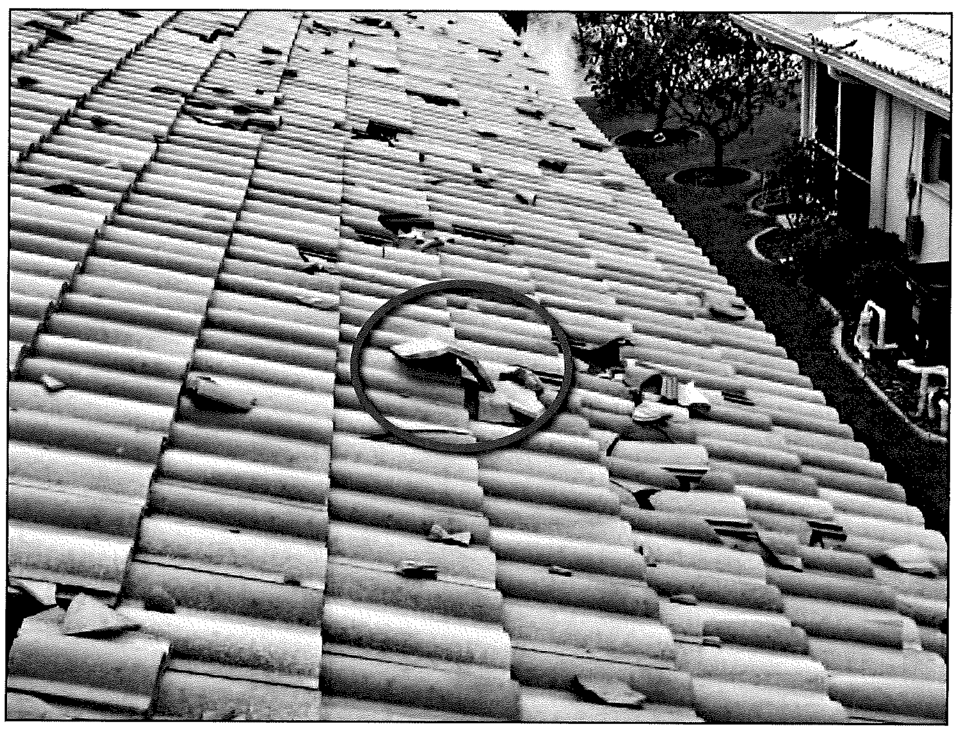


Figure 11-47.
Damage to field tiles caused by tiles from another area of the roof, including a hip tile (circle). Estimated wind speed: 140 to 160 mph. Hurricane Charley (Florida, 2004)



The house in Figure 11-48 had a lightning protection system (LPS), and the LPS conductors were placed under the ridge tile. Conductors are not susceptible to wind damage if they are placed under the tile and the air terminals (lightning rods) are extended through the ridge.

Figure 11-48.
The fastener heads on this mechanically attached tile roof had corroded; air terminals (lightning rods) in a lightning protection system (circle). Hurricane Ivan (Alabama, 2004)

To avoid the type of problems shown in Figures 11-39 through 11-48, see the guidance and recommendations regarding attachment and quality control in Fact Sheet 7.4, *Tile Roofing for High-Wind Areas,* in FEMA P-499. Fact Sheet 7.4 references the Third Edition of the *Concrete and Clay Roof Tile Installation Manual* (FRSA/TRI 2001) but, as of press time, the Fourth Edition is current and therefore recommended (FRSA/TRI 2005). The Manual includes underlayment recommendations.

11.5.4.2 Seismic

Tiles are relatively heavy, and 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, but design guidance for these products in areas prone to large ground-motion accelerations has not been developed. As shown in Figures 11-49, 11-50, and 11-51, tiles can be dislodged if they are not adequately secured.

In seismic areas where short period acceleration, Ss, exceeds 0.5g, the following are recommended:

- If tiles are laid on battens, supplemental mechanical attachment is recommended. When tiles are only loose laid on battens, they can be shaken off, as shown in Figure 11-49 where most of the tiles on the 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.
- 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 OSB sheathing, pullout can occur. Figure 11-50 shows tiles that were nailed to thin wood sheathing. During the earthquake, the nose of the tiles bounced and pulled out the nails. Specifying ring-shank or screw-shank nails or screws is recommended, but even with these types of fasteners, the nose of the tile can bounce, causing enlargement of the nail hole by repeated pounding. To overcome this problem, wind clips near the nose of the tile or a bead of adhesive between the tiles at the headlap should be specified.

Figure 11-49.
Area of the roof where tiles were not nailed to batten strips. Northridge Earthquake (California, 1994)

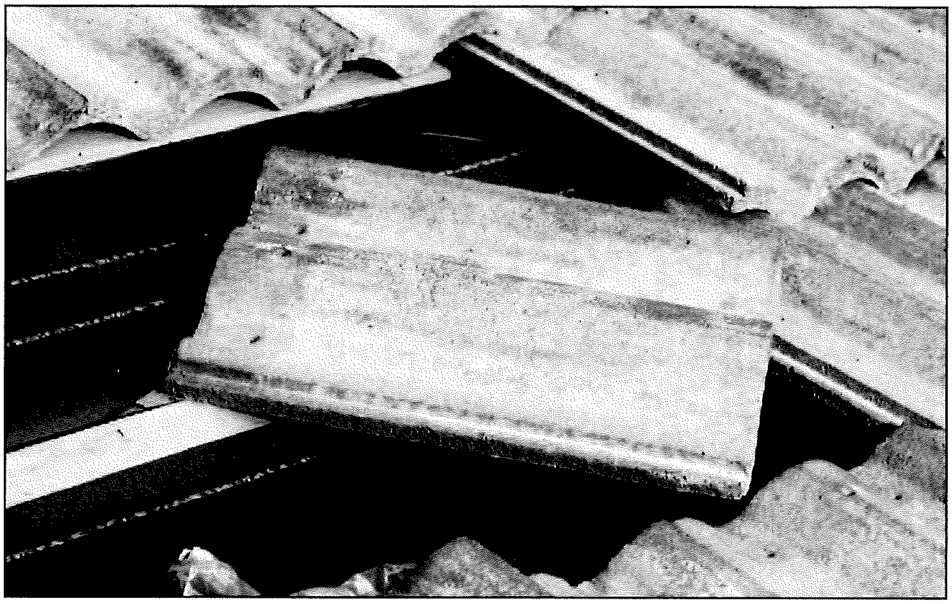


Figure 11-50.
Tiles that were nailed to thin wood sheathing. Northridge Earthquake (California, 1994)

- Tiles that are attached by only one fastener experience eccentric loading. This problem can be overcome by specifying wind clips near the nose of the tile or a bead of adhesive between the tiles at the headlap.
- 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. Figure 11-51 shows a tile that slipped out from under 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. Tiles must have supplemental securing to avoid displacement of these loose tiles.

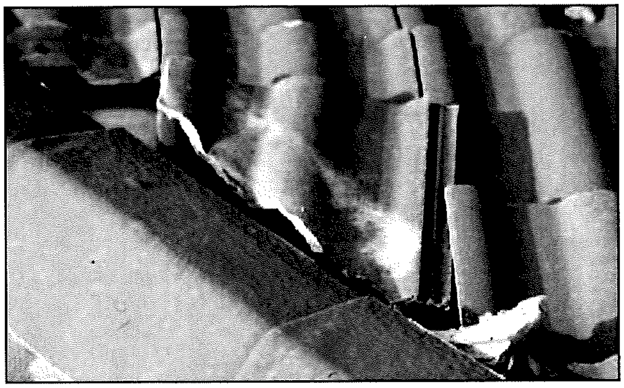


Figure 11-51.
Tile that slipped out from under the hip tiles.
Northridge Earthquake (California, 1994)

- Securing 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.
- Stainless steel straps, fasteners, and clips are recommended for roofs within 3,000 feet of an ocean shoreline (including sounds and back bays).

11.5.4.3 Hail

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, but as of press time, it had not been recognized as a consensus test method.

11.5.5 Metal Panels and Metal Shingles

A variety of metal panel and shingle systems are available. Fact Sheet 7.6, *Metal Roof Systems in High-Wind Regions,* in FEMA P-499 discusses metal roofing options. Some of the products simulate the appearance of tiles or wood shakes.

11.5.5.1 High Winds

Damage investigations have revealed that some metal roofing systems have sufficient strength to resist extremely high winds, while other systems have blown off during winds that were well below the design speeds given in ASCE 7. Design and construction guidance is given in Fact Sheet 7.6 in FEMA P-499.

Figure 11-52 illustrates the importance of load path. The metal roof panels were screwed to wood nailers that were attached to the roof deck. The panels were well attached to the nailers. However, one of the nailers was inadequately attached. This nailer lifted and caused a progressive lifting and peeling of the metal panels. Note the cantilevered condenser platform (arrow), a good practice, and the broken window (circle).

Figure 11-52.
Blow-off of one of the nailers (dashed line on roof) caused panels to progressively fail; cantilevered condenser platform (arrow); broken window (circle). Estimated wind speed: 130 mph. Hurricane Katrina (Louisiana, 2005)

11.5.5.2 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, the dents can be aesthetically objectionable. The appearance of the system is not included in the UL 2218 evaluation criteria.

11.5.6 Slate

Some fiber-cement and tile products are marketed as "slate," but slate is a natural material. Quality slate offers very long life. However, long-life fasteners and underlayment are necessary to achieve roof system longevity.

11.5.6.1 High Winds

Because of limited market share of slate in areas where research has been conducted after high-wind events, few data are available on its wind performance. However, as shown in Figure 11-53, wind damage can occur.

Methods to calculate uplift loads and evaluate load resistance for slate have not been incorporated into the IBC or IRC. Manufacturers have not conducted research to determine a suitable pressure coefficient. Demonstrating slate's compliance with ASCE 7 will be problematic until a coefficient has been developed. A consensus test method for uplift resistance has not been developed for slate.

In extreme high-wind areas, mechanical attachment near the nose 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.

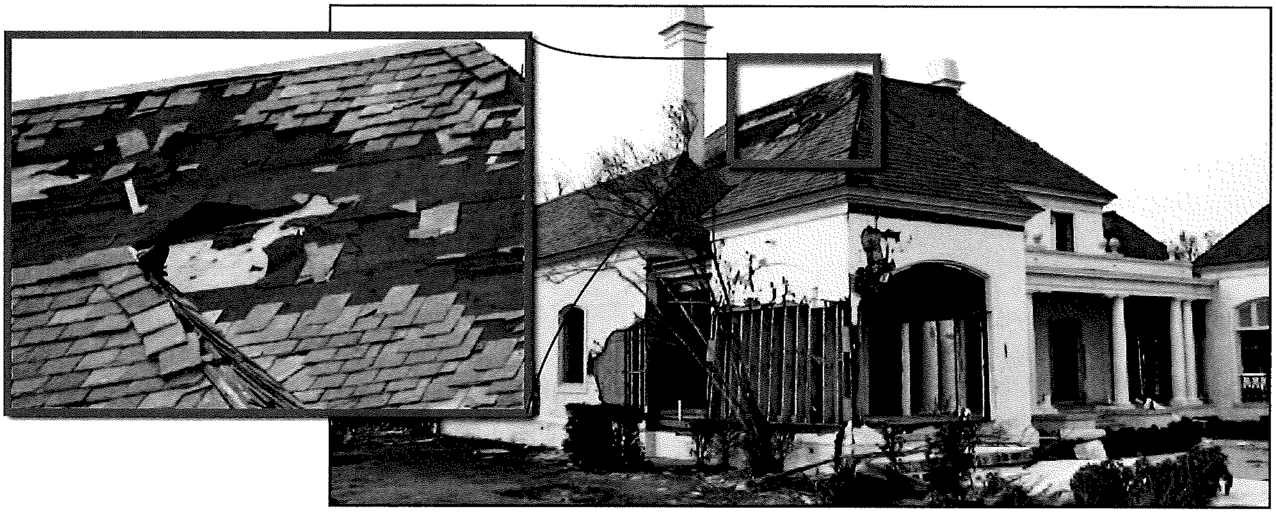


Figure 11-53.
Damaged slate roof with nails that typically pulled out of the deck. Some of the slate broke and small portions remained nailed to the deck. Estimated wind speed: 130 mph. Hurricane Katrina (Mississippi, 2005)

Stainless steel straps, fasteners, and clips are recommended for roofs within 3,000 feet of an ocean shoreline (including sounds and back bays). For underlayment recommendations, refer to the recommendation at the end of Section 11.5.4.1.

11.5.6.2 Seismic

Slate is relatively heavy and 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 11.5.4.2 is recommended until guidance has been developed for slate.

11.5.6.3 Hail

See Section 11.5.4.3.

11.5.7 Wood Shingles and Shakes

11.5.7.1 High Winds

Research conducted after high-wind events has shown that wood shingles and shakes can perform very well during high winds if they are not deteriorated and have been attached in accordance with standard attachment recommendations.

Methods to calculate uplift loads and evaluate load resistance for wood shingles and shakes have not been incorporated into the IBC or IRC. Manufacturers have not conducted research to determine suitable pressure coefficients. Demonstrating compliance with ASCE 7 will be problematic with wood shingles and shakes until such coefficients have been developed. A consensus test method for uplift resistance has not been developed for wood shingles or shakes.

For enhanced durability, preservative-treated wood is recommended for shingle or shake roofs on coastal buildings. Stainless steel fasteners are recommended for roofs within 3,000 feet of an ocean shoreline (including sounds and back bays). See Figure 11-54 for an example of shingle loss due to corrosion of the nails.

Figure 11-54.
Loss of wood shingles due to fastener corrosion. Hurricane Bertha (North Carolina, 1996)

11.5.7.2 Hail

At press time, no wood-shingle assembly had passed UL 2218, but heavy shakes had passed Class 4 (the class with the greatest impact resistance) and medium shakes had passed Class 3.

The hail resistance of wood shingles and shakes depends partly on their condition when affected by hail. Resistance is likely to decline with roof age.

11.5.8 Low-Slope Roof Systems

Roof coverings on low-slope roofs need to be waterproof membranes rather than the water-shedding coverings that are used on steep-slope roofs. Although most of the low-slope membranes can be used on dead-level substrates, it is always preferable (and required by the IBC and IRC) 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 systems. Liquid-applied membranes (see Section 11.5.3), structural metal panels (see Section 11.5.5), and sprayed polyurethane foam may also be used on low-slope roofs. Information on low-slope roof systems is available in *The NRCA Roofing Manual* (NRCA 2011).

Low-slope roofing makes up a very small percentage of the residential roofing market. However, when low-slope systems are used on residences, the principles that apply to commercial roofing also apply to residential work. The natural hazards presenting the greatest challenges to low-sloped roofs in the coastal environment are high winds (see Section 11.5.8.1), earthquakes (see Section 11.5.8.2), and hail (see Section 11.5.8.3).

11.5.8.1 High Winds

Roof membrane blow-off is typically caused by lifting and peeling of metal edge flashings (gravel stops) or copings, which serve to clamp down the membrane at the roof edge. In hurricane-prone regions, roof membranes are also often punctured by wind-borne debris.

Following the criteria prescribed in the IBC will typically result in roof systems that possess adequate wind uplift resistance if properly installed. IBC references ANSI/SPRI ES-1 for edge flashings and copings. ANSI/SPRI ES-1 does not specify a minimum safety factor. Accordingly, a safety factor of 2.0 is recommended for residences.

NOTE

The 2009 edition of the IBC prohibits the use of aggregate roof surfacing in hurricane-prone regions.

A roof system that is compliant with IBC (and the FBC) is susceptible to interior leakage if the roof membrane is punctured by wind-borne debris. If a roof system is desired that will avoid interior leakage if struck by debris, refer to the recommendations in FEMA P-424, *Design Guide for Improving School Safety in Earthquakes, Floods and High Winds* (FEMA 2010a). Section 6.3.3.7 also provides other recommendations for enhancing wind performance.

11.5.8.2 Seismic

If a ballasted roof system is specified, its weight should be considered during seismic load analysis of the structure. 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.

11.5.8.3 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.

If the pavers are installed over a single-ply membrane, it is recommended that a layer of extruded polystyrene intended for protected membrane roof systems be specified over the membrane to provide protection if the pavers break. Alternatively, a stone protection mat intended for use with aggregate-ballasted systems can be specified.

11.6 Attic Vents

High winds can drive large amounts of water through attic ventilation openings, which can lead to collapse of ceilings. Fact Sheet 7.5, *Minimizing Water Intrusion Through Roof Vents in High-Wind Regions,* in FEMA P-499 provides design and application guidance to minimize water intrusion through new and existing attic ventilation systems. Fact Sheet 7.5 also contains a discussion of unventilated attics.

Continuous ridge vent installations, used primarily 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.

Roof sheathing is normally 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 assuming a blocked roof diaphragm, is accomplished with nails. This approach is illustrated by Figure 11-55.

The problem with the continuous ridge vent installation 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, cutting slots or holes in the sheathing is required. If a saw is used to cut off 1 to 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.

NOTE

When cutting a slot in a deck for a ridge vent, it is important to set the depth of the saw blade so that it only slightly projects below the bottom of the sheathing. Otherwise, as shown in Fact Sheet 7.5, the integrity of the trusses can be affected.

The two methods of providing the proper ventilation while maintaining the continuity of the blocked roof diaphragm are as follows:

1. Drill 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, they should be spaced at 6 inches o.c., which will allow about 12 square inches of vent area per linear foot when the holes are placed along either side of the ridge. This concept is illustrated in Figure 11-56.

Figure 11-55.
Method for maintaining a continuous load path at the roof ridge by nailing roof sheathing