
Chapter 12: Designing the Building

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Designing the Building

12.1 The Design Process

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

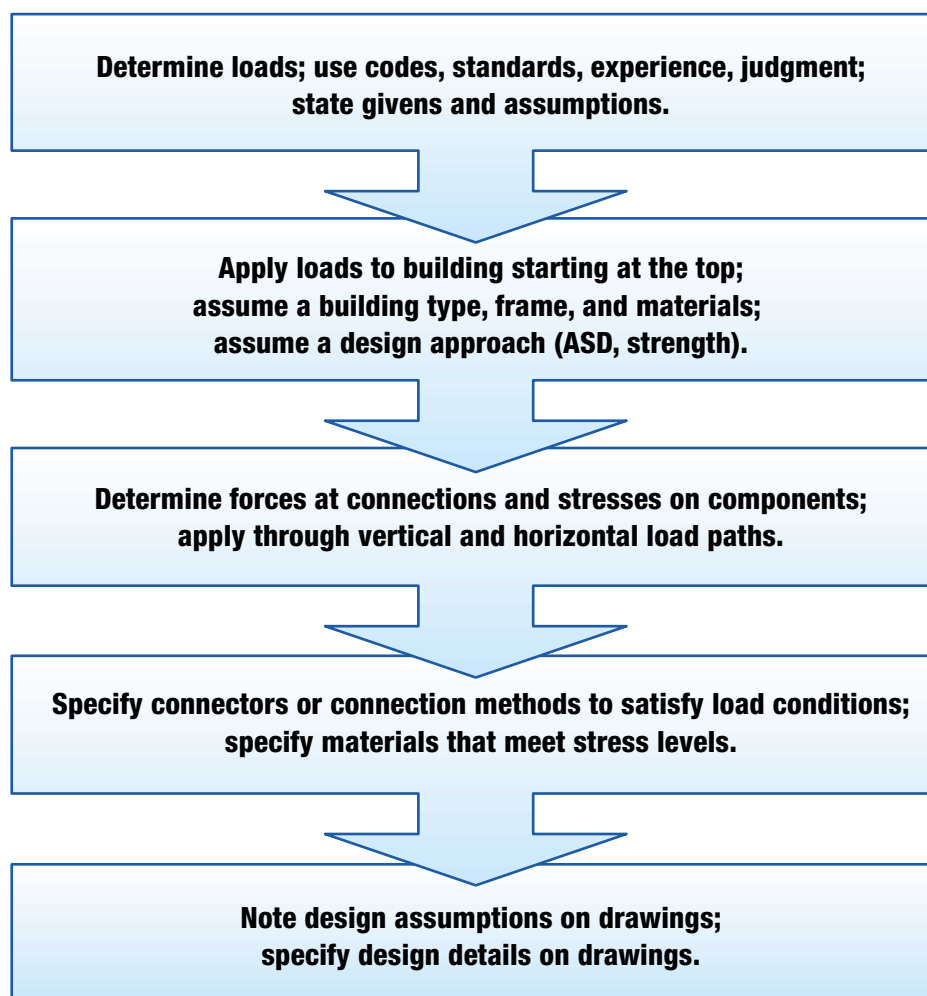


Figure 12-1
Coastal construction design process.



NOTE

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

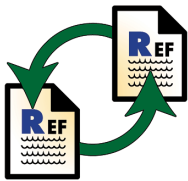
Each step of this process is covered as a separate section of this chapter. The many variations of designs, the location of buildings in different hazard areas, the varieties of building shapes, and other building parameters are discussed where appropriate in each of the design steps. A building located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

The design process involves the following:

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

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

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



CROSS-REFERENCE

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



NOTE

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

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

12.2 Step 1 – Determining Loads

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

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

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

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

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

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

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

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

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

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

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

12.3 Step 2 – Applying Loads to the Building

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

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

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

12.3.1 Failure Modes

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

Primary Failure Modes

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

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

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

Secondary Failure Mode

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

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

12.3.1.1 Uplift

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

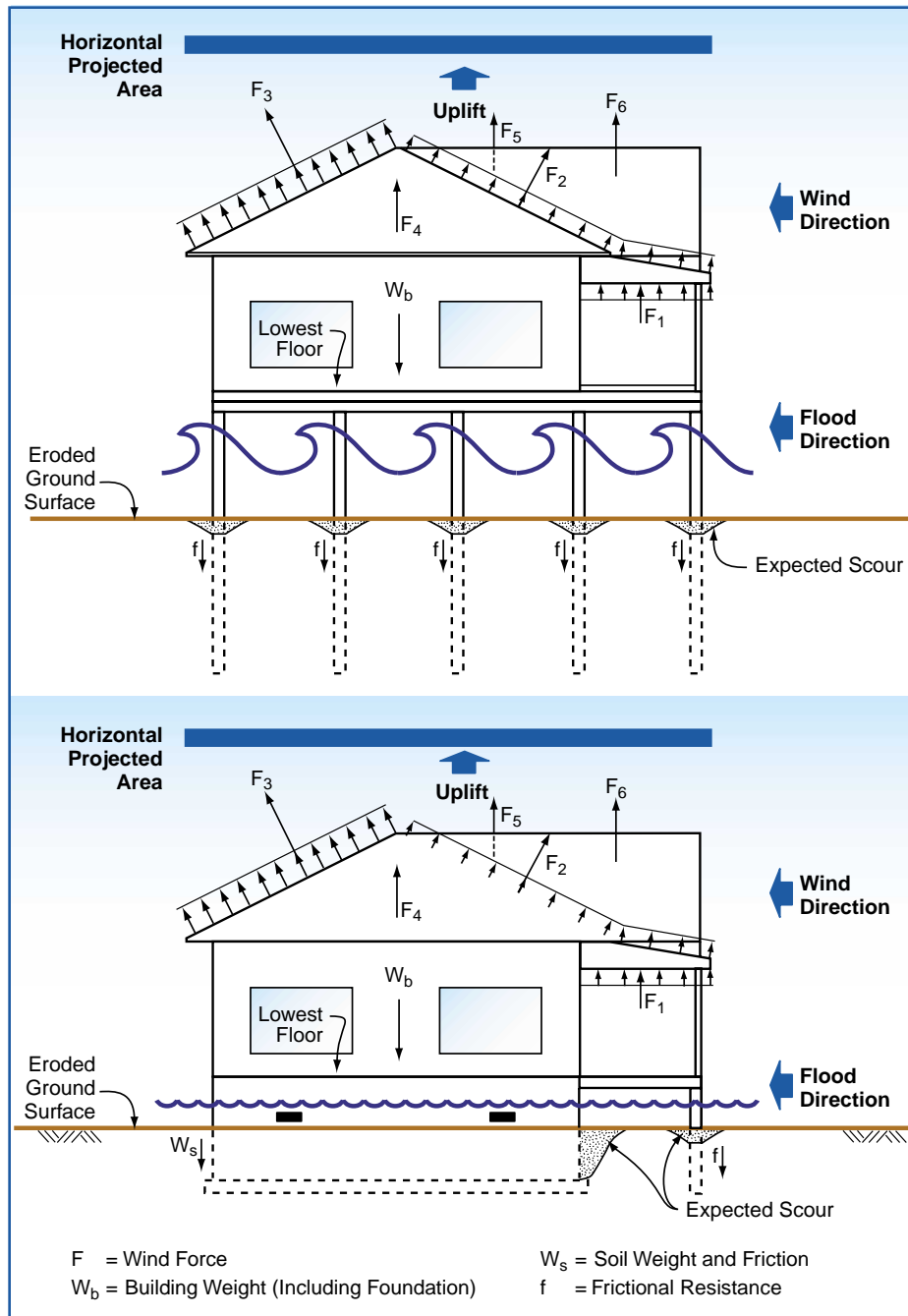


Figure 12-2
Uplift.

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

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

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

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



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

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



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

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

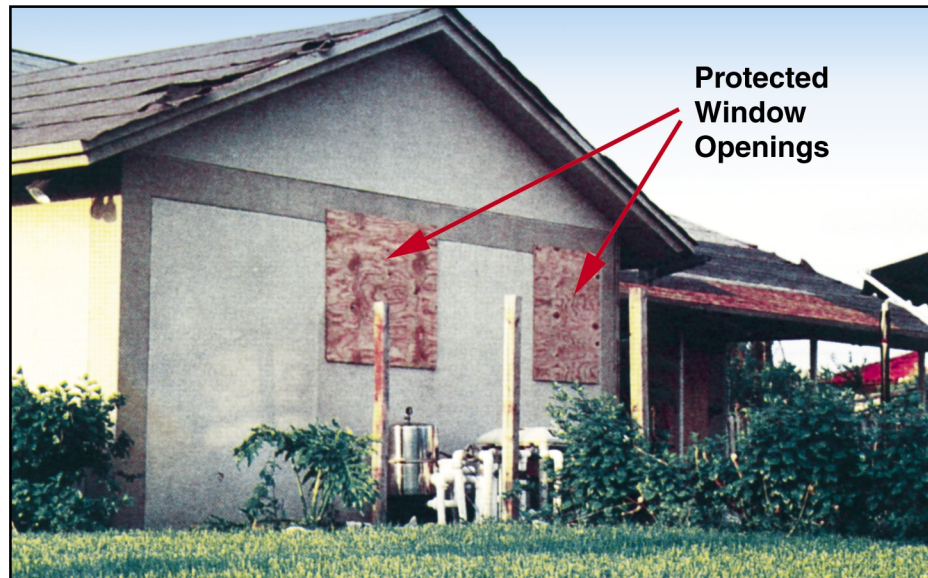


Figure 12-5
Hurricane Diana (1984). Uplift
failure of a porch roof.

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

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

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



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

12.3.1.2 Overturning

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

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

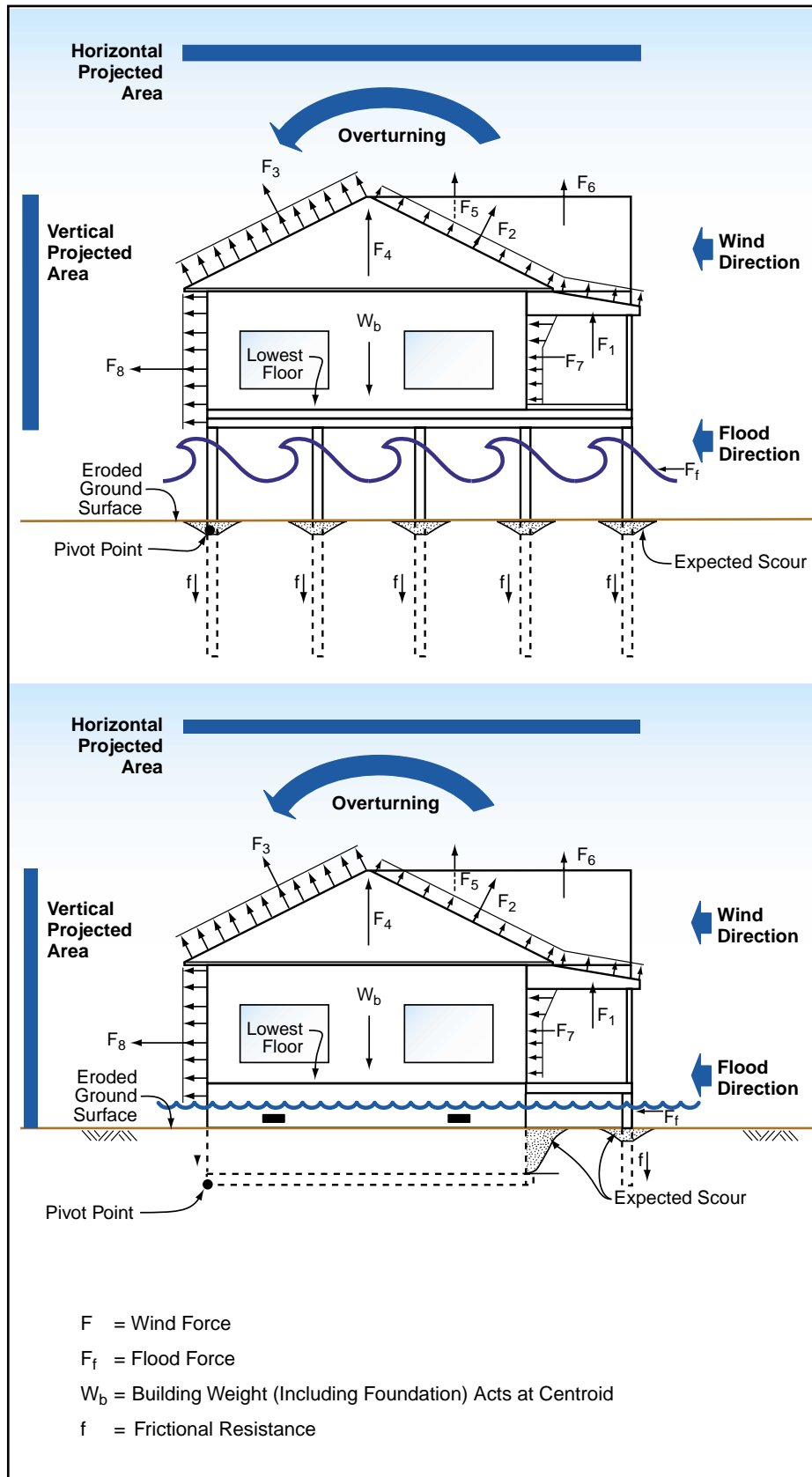


Figure 12-7
Overturning.

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

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

Figure 12-8

Hurricane Fran (1996), North Carolina. House overturned by flood and wind forces.



Figure 12-9

Hurricane Fran (1996), North Carolina. Lightweight building (manufactured home) overturned by wind forces.



12.3.1.3 Sliding or Shearing

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

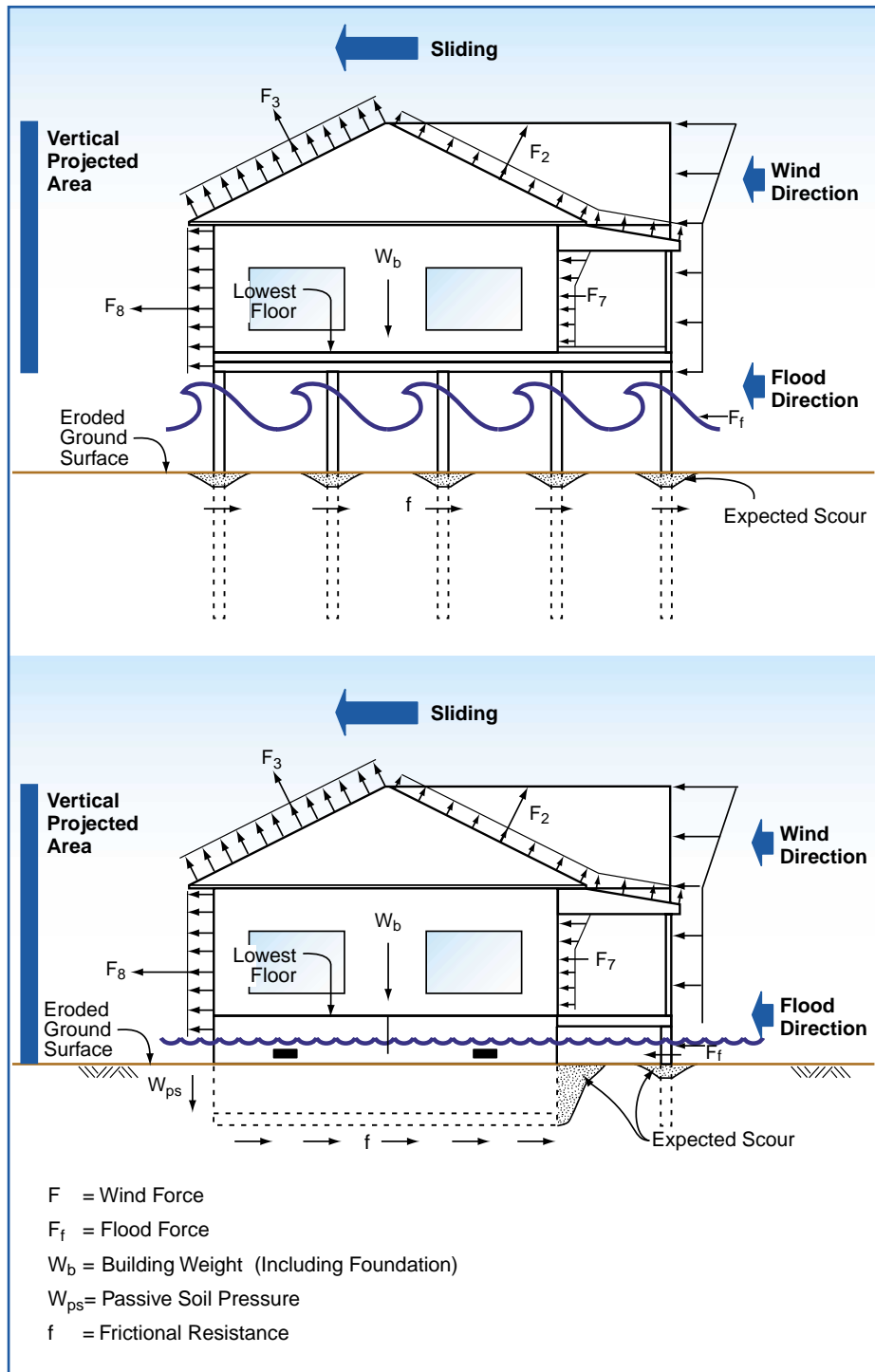


Figure 12-10
Sliding failure.

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

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

Figure 12-11

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



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

Figure 12-12

Hurricane Fran (1996), North Carolina. Building pilings (circled) snapped at top.



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



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

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

Formula 12.1 Frictional Resistance of Soil

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

where: F = sliding resistance
 ϕ = angle of internal friction of soil

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



Frictional Resistance
of Soil

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

12.3.1.4 Collapse

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



NOTE

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

Figure 12-14

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



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

12.3.2 Load Path

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

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

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

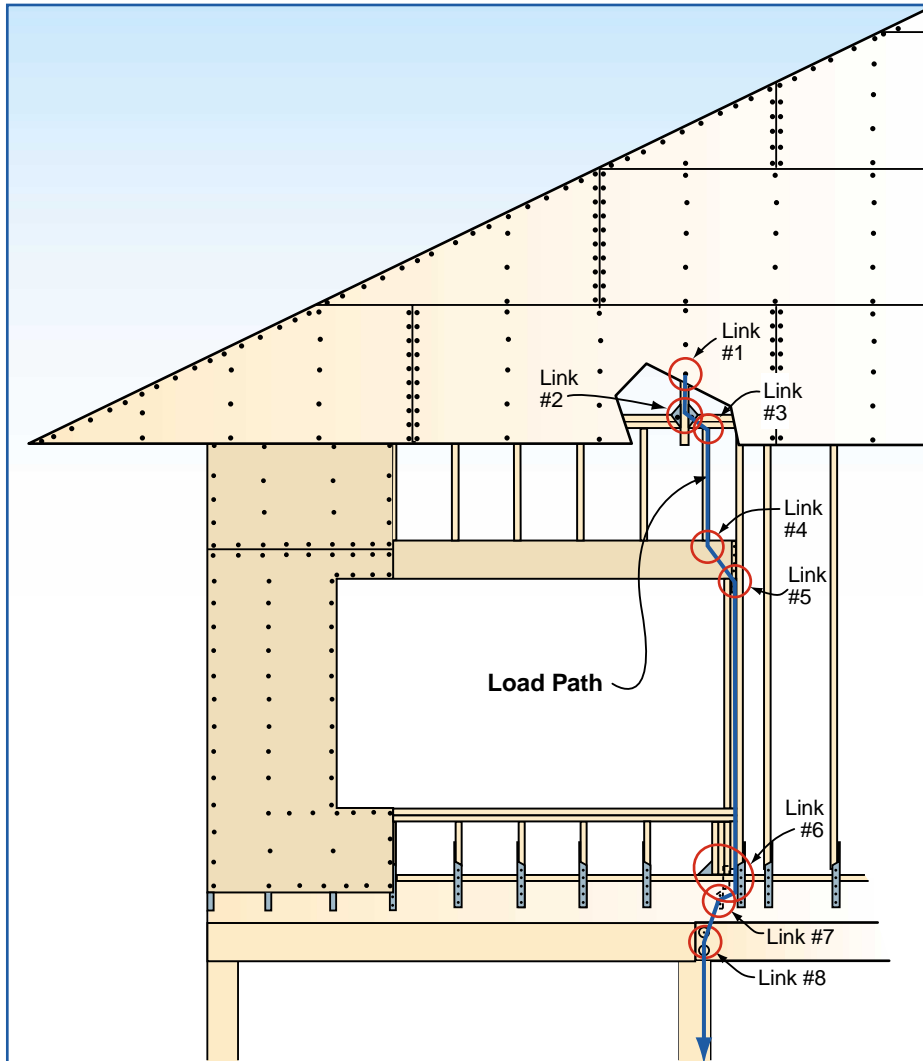


Figure 12-15
Load path at southwest
corner window of case study
building.

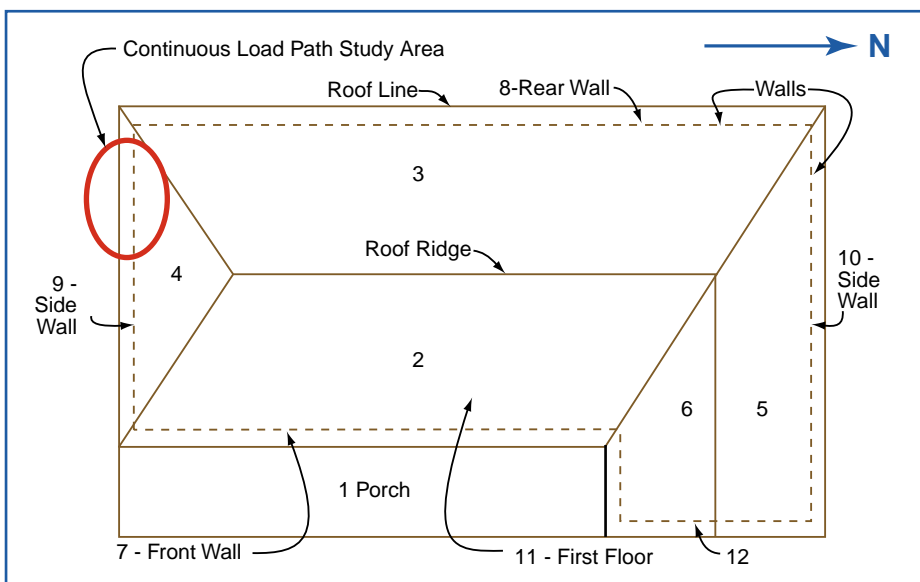


Figure 12-16
Roof plan of case study
building.

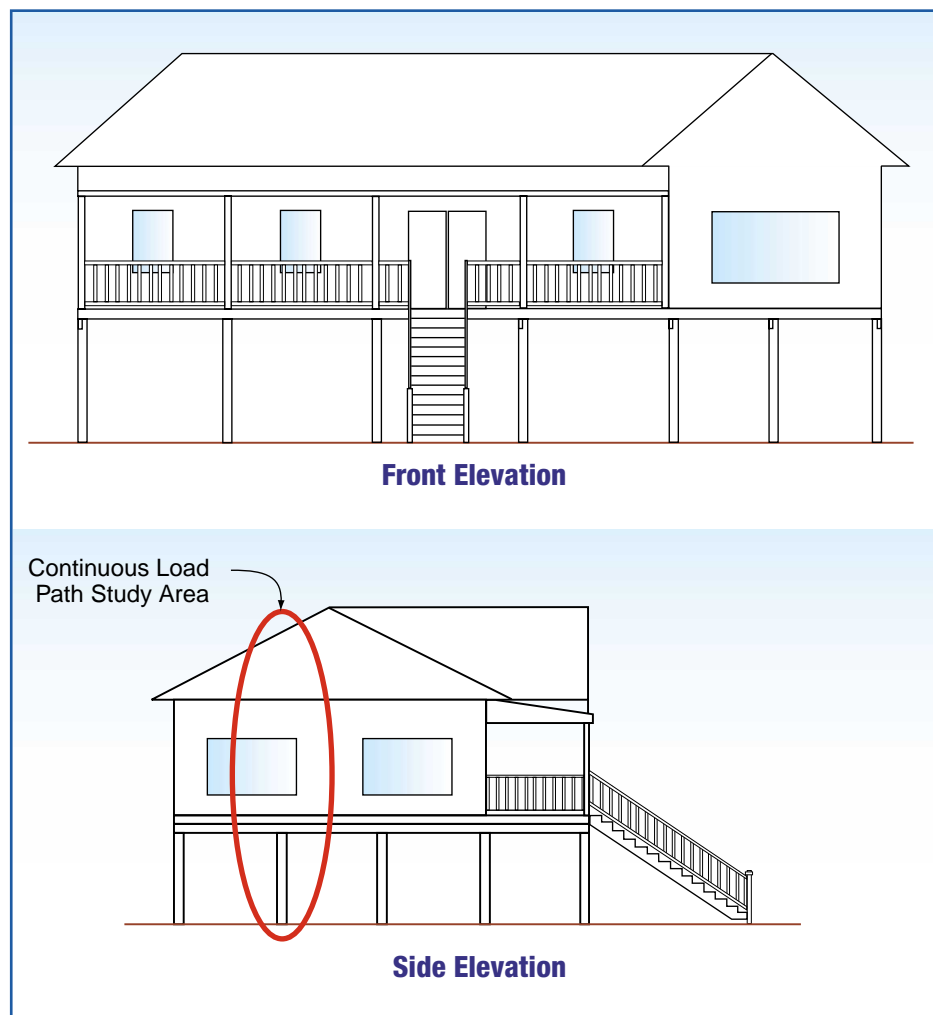
12.3.3 Structural Building Systems

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

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

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

Figure 12-17
Elevations of case study building.



12.3.3.1 Platform Framing

Across the United States, this is by far the most common method of framing a wood-stud or steel-stud residential building. In the platform framing method, a floor assembly consisting of beams, joists, and a subfloor creates a “platform” that supports the exterior and interior walls. The walls are

normally laid out and framed flat on top of the floor, tilted up into place, and attached at the bottom to the floor through the wall bottom plate. The walls are attached at the top to the next-level floor framing or (in a one-story building) to the roof framing. Figure 12-18 is an example of platform framing in a two-story building.

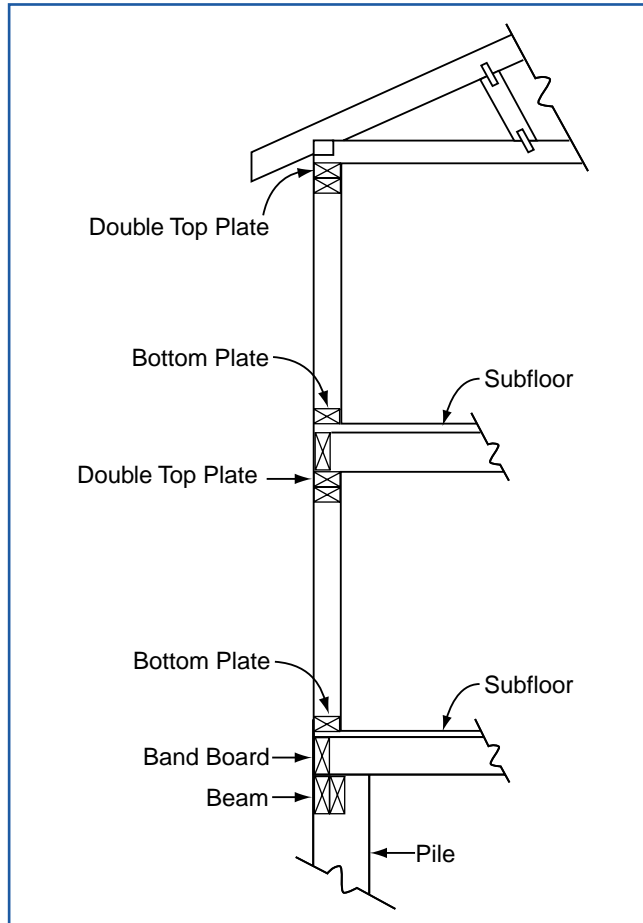


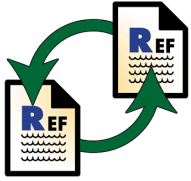
Figure 12-18

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

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

Advantages

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

**CROSS-REFERENCE**

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

Disadvantages

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

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

12.3.3.2 Balloon Framing

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

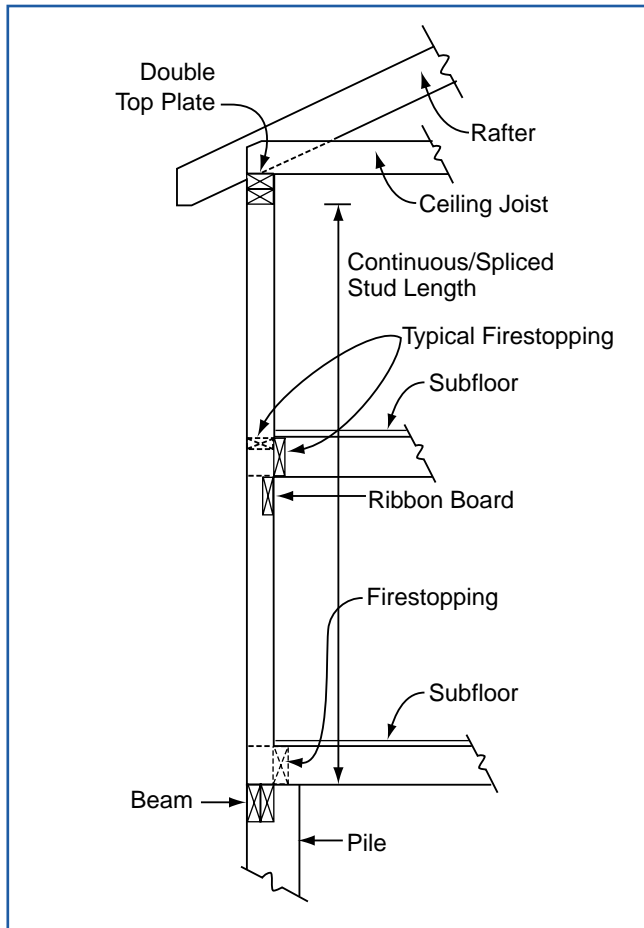
Some of the advantages and disadvantages of this framing method are listed below.

Advantages

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

Disadvantages

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

**Figure 12-19**

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

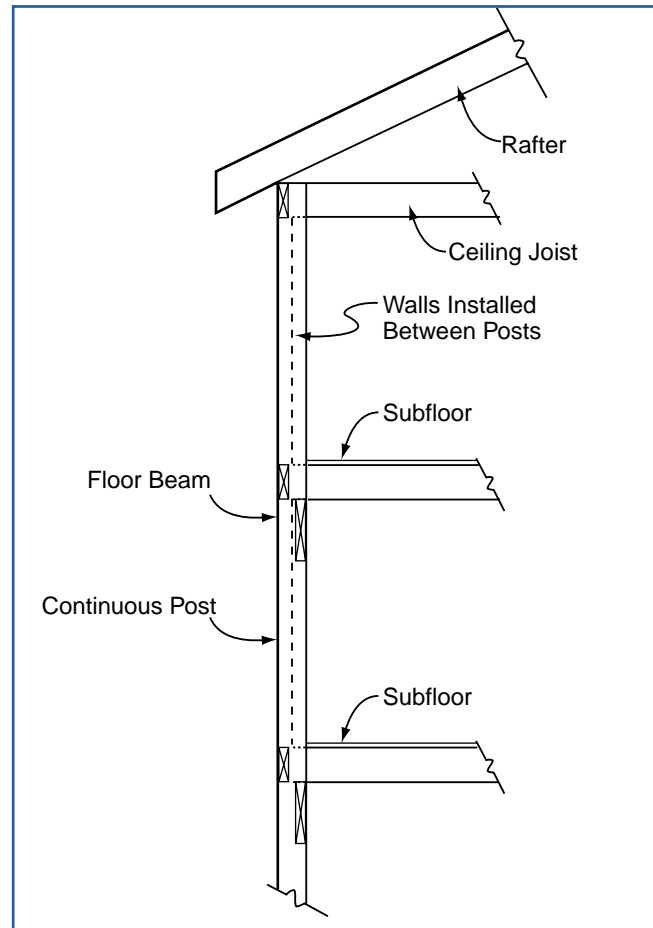
Because it produces continuous exterior walls, balloon framing lacks the failure planes inherent in the platform method and therefore produces a more rigid frame.

12.3.3.3 Post-and-Beam Framing

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

Figure 12-20

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



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

Advantages

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

Disadvantages

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

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

12.3.3.4 Concrete/Masonry

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

Advantages:

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

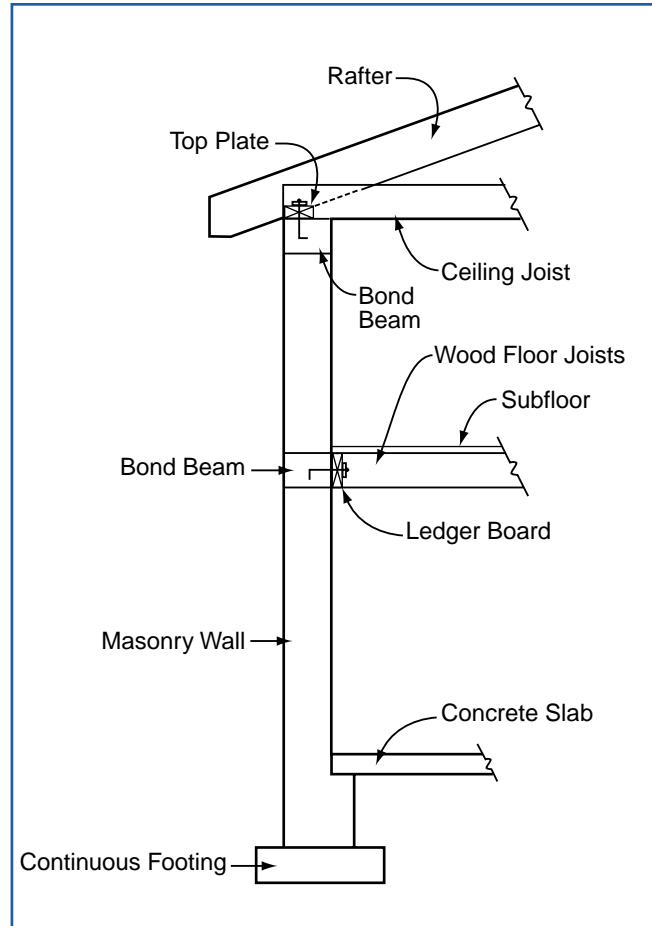
In addition, reinforced masonry buildings have good high-wind damage history.

Disadvantages:

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

Figure 12-21

Two-story masonry wall with wood floor and roof framing.



NOTE

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

12.3.4 Construction Materials

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

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

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

Component	Primary Product	Primary Sizes*
Floor Joists	Dimension lumber	2 x 8 – 2 x 12
	Engineered floor beams or trusses	Varies
Subfloor	Wood structural panel	$\frac{5}{8}$ – $\frac{3}{4}$
Wall Studs	Dimension lumber	2 x 4 – 2 x 6
Wall Sheathing	Wood structural panel	$\frac{1}{2}$ – $\frac{5}{8}$
Roof Framing	Dimension lumber	2 x 8 – 2 x 12
	Trusses	Varies
Roof Sheathing	Wood structural panel	$\frac{1}{2}$ – $\frac{5}{8}$

Table 12.1
Wood Products Used for
Structural Frame Components

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

box nails in length and/or diameter. **Designers and builders are encouraged to specify nailed connections by length and diameter of nail required** instead of the more customary method of “pennyweight” or $\times d$ nail size.

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

12.3.5 Building Layouts and Architectural Shapes

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

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

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



NOTE

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

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

(Source: NAHB Research Center 1994)

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

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

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

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

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

Table 12.3
Building Layout and Impacts
on Natural Hazard Design

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

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

12.4.1 Getting Started

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



NOTE

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

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

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

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

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

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

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

* Numbers refer to Figure 12-16.

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

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

12.4.1.1 Uplift

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

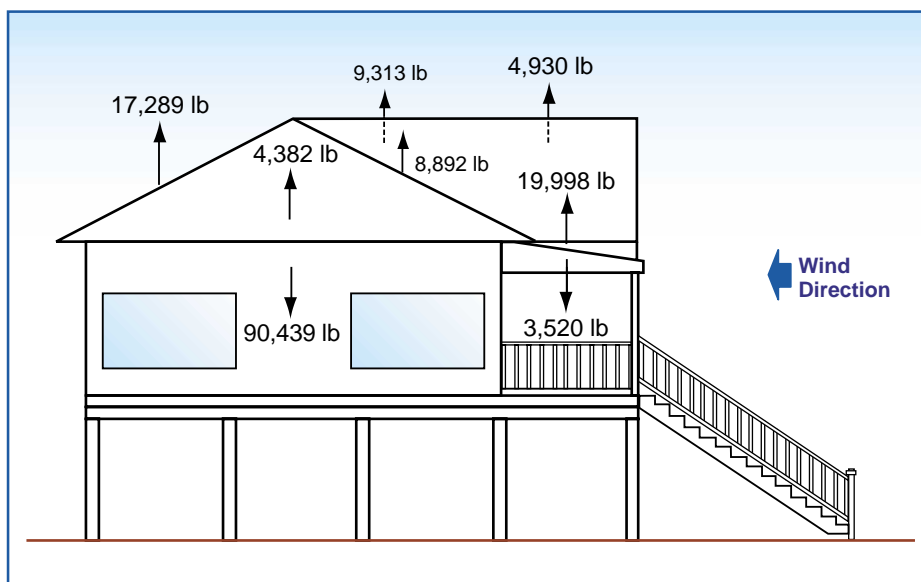


Figure 12-22

Uplift and gravity forces on the case study building.



NOTE

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

**NOTE**

Designers should determine actual dead loads from building components.

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

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

$$\text{Roof} = (10 \text{ lb/ft}^2)(2,248 \text{ ft}^2) = 22,480 \text{ lb}$$

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

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

$$\text{Interior walls} = (8 \text{ lb/ft}^2)(2,000 \text{ ft}^2 - \text{assumed}) = 16,000 \text{ lb}$$

$$\text{Piles} = (409 \text{ lb/each})(31 \text{ piles}) = 12,679 \text{ lb}$$

$$\text{TOTAL WEIGHT} = 90,439 \text{ lb}$$

$$\text{The front porch roof weighs } (8 \text{ lb/ft}^2)(440 \text{ ft}^2) = 3,520 \text{ lb}$$

[12.1]

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

**Formula 12.2 Net Uplift Force**

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

**NOTE**

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

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

$$\text{Net uplift force} = F1 + F2 + F3 + F4 + F5 + F6 - \text{weight of building}(0.6) - \text{weight of porch roof}(0.6)$$

$$\text{Net uplift force (lb)} = 19,998 + 8,892 + 9,313 + 4,930 + 4,382 + 14,243 - (90,439)(0.6) - (3,520)(0.6)$$

$$\text{Uplift} = 64,804 - 56,375 \text{ lb} = 8,429 \text{ lb}$$

[12.2]

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

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

$$\text{Porch roof uplift} = (440 \text{ ft}^2)(45.45 \text{ lb/ft}^2) - (3,520) = 16,478 \text{ lb net uplift}$$

[12.3]

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

12.4.1.2 Over turning

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

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

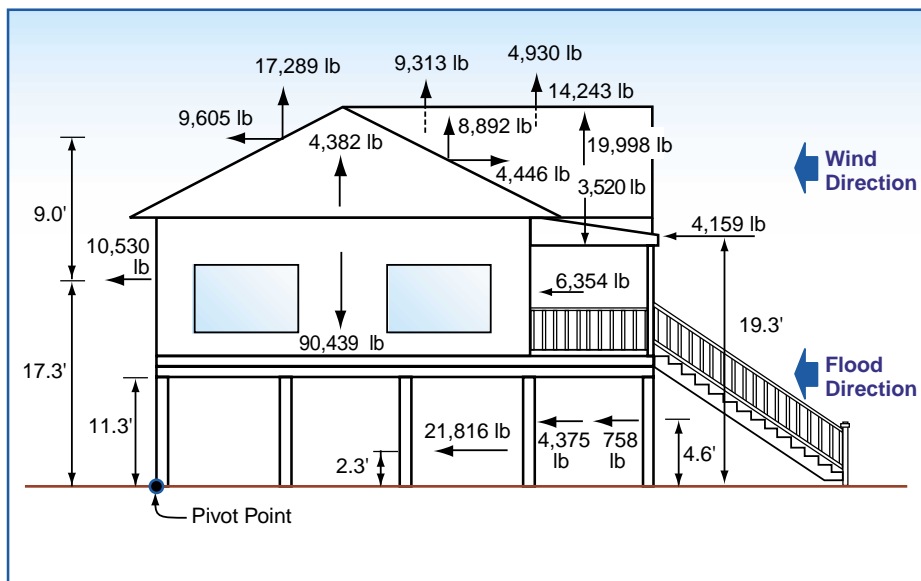


Figure 12-23
Overturning moment on the case study building.

formula Overturning Moment



NOTE

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



NOTE

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

Formula 12.3 Overturning Moment

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

Overturning Moment = $\Sigma[(\text{wind forces}) (\text{distances from line of force to pivot point})] + 1.5 (\text{flood forces}) (\text{distance from pivot point})$

For equilibrium: Righting Moment – Overturning Moment > 0

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

$$\text{Righting moment} = (90,439 \text{ lb})(14 \text{ ft})(0.6) + (3,520 \text{ lb})(33 \text{ ft})(0.6) = 829,384 \text{ ft-lb}$$

$$\begin{aligned} \text{Overturning moment from lateral forces and vertical} \\ \text{projected areas} = & F2(26.3 \text{ ft}) + F3(26.3 \text{ ft}) + \\ & F7(17.3 \text{ ft}) + F8(17.3 \text{ ft}) + F12(19.3 \text{ ft}) + \\ & (F_9)(1.5)(4.6 \text{ ft or } 2.3 \text{ ft}) \end{aligned}$$

[12.4]

$$\begin{aligned} \text{Overturning moment from vertical forces and} \\ \text{horizontal projected areas} = & F1(33 \text{ ft}) + F2(22 \text{ ft}) + \\ & F3(6 \text{ ft}) + F4(14 \text{ ft}) + F5(20.7 \text{ ft}) + F6(28.4 \text{ ft}) \end{aligned}$$

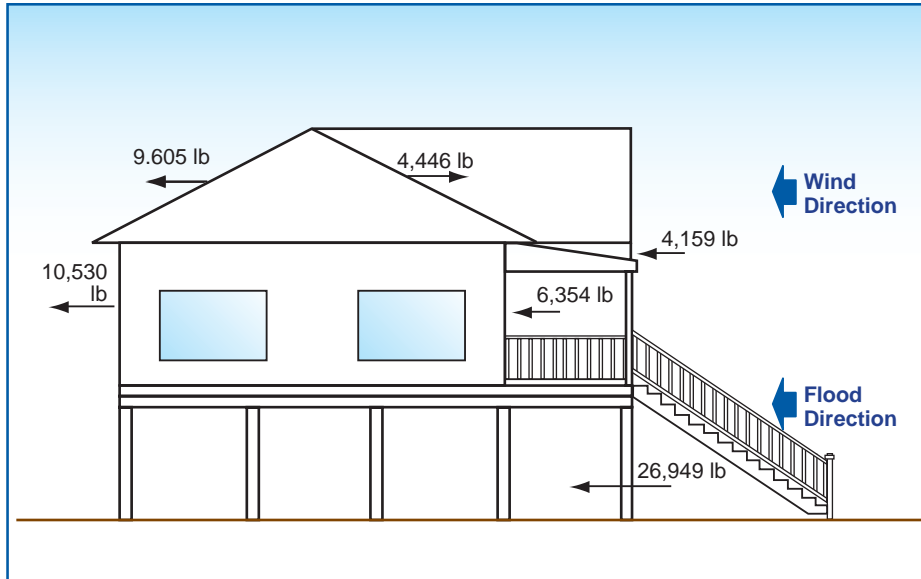
$$\begin{aligned} \text{Righting moment} - \text{Overturning moment} = \\ 829,384 - 2,089,088 = -1,259,704 \text{ ft-lb} \end{aligned}$$

[12.5]

Equilibrium is not satisfied and overturning must be resisted by frictional resistance of the piles.

12.4.1.3 Sliding

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

**Figure 12-24**

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

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

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

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

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

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



NOTE

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

[12.6]

[12.7]

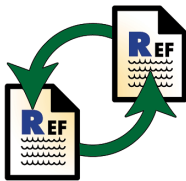


Sliding Resistance of
Foundation Walls and
Footing



NOTE

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



CROSS-REFERENCE

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

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

Formula 12.4

Sliding Resistance of Foundation Walls and Footing

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

where: ϕ = internal angle of soil friction

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

12.4.2 Analyze Load Path Links

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

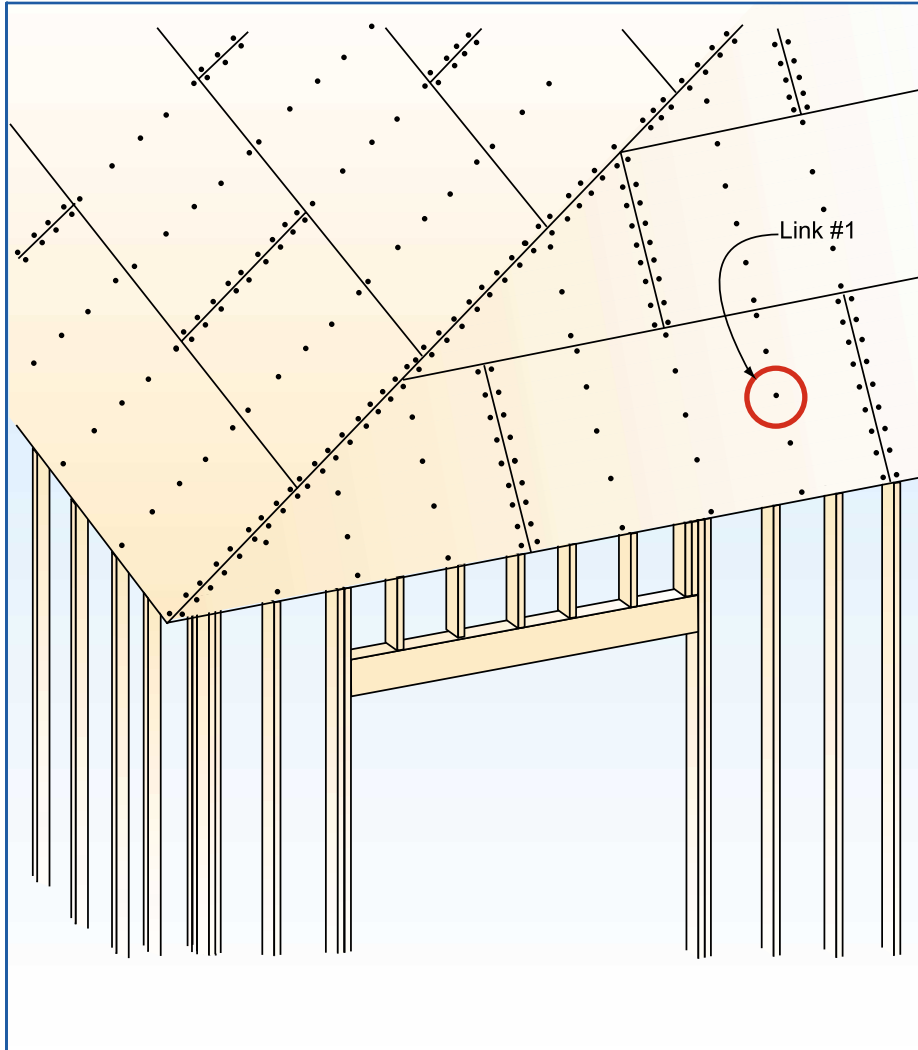
Link #1 – Roof Sheathing to Roof Framing

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

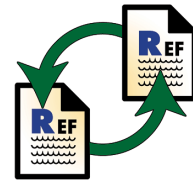
Withdrawal:

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

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

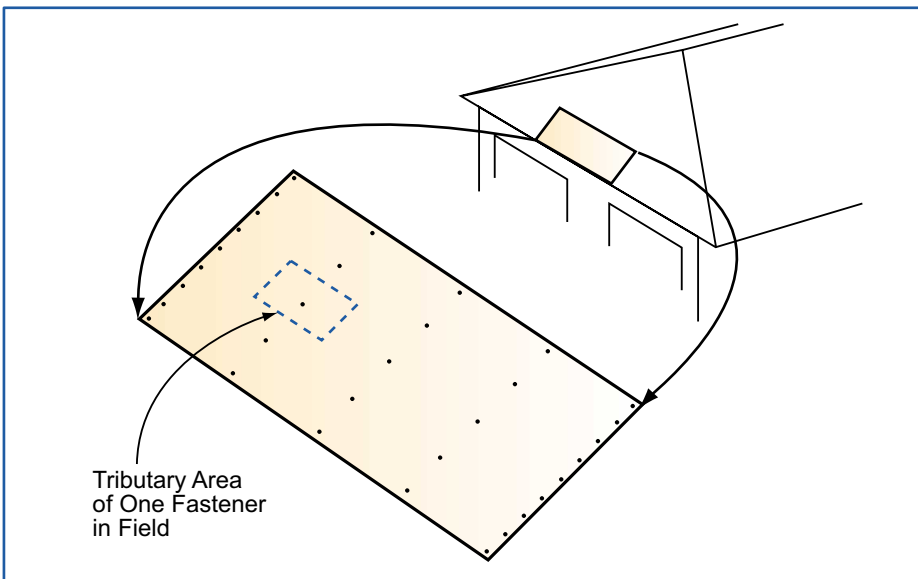
**Figure 12-25**

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



CROSS-REFERENCE

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

**Figure 12-26**

Effective area of a roof sheathing fastener in the field.

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

- nail withdrawal value
- effective area
- uplift pressure

Nail withdrawal values depend on the following:

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

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

[12.8]

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

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

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

Notes:

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

Most prescriptive standards for nail withdrawal values recommend that spacing in the field of the roof not exceed 12 inches on center (o.c.). The American Plywood Association's Report T325, *Roof Sheathing Fastening Schedules for Wind Uplift* (APA 1997) recommends a minimum fastening

schedule of 6 inches o.c. at the panel edges and 12 inches o.c. in the field. To illustrate forces on roof sheathing, a finite element analysis of a 4-foot x 8-foot roof panel included in the APA report showed that there were two critical fasteners in the field when one fastener is either missing or ineffective. This sheathing deflection model and the critical fasteners are shown in Figure 12-27. Uplift resistance of roof sheathing is also a function of support at panel edges (blocked or unblocked). Designers should consider specifying blocked panel edges to increase uplift resistance.

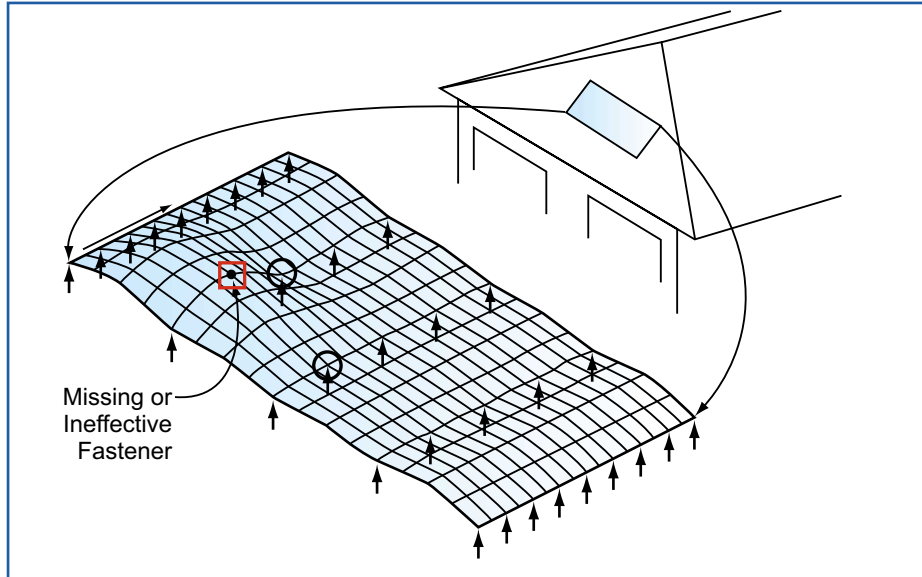


Figure 12-27

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

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

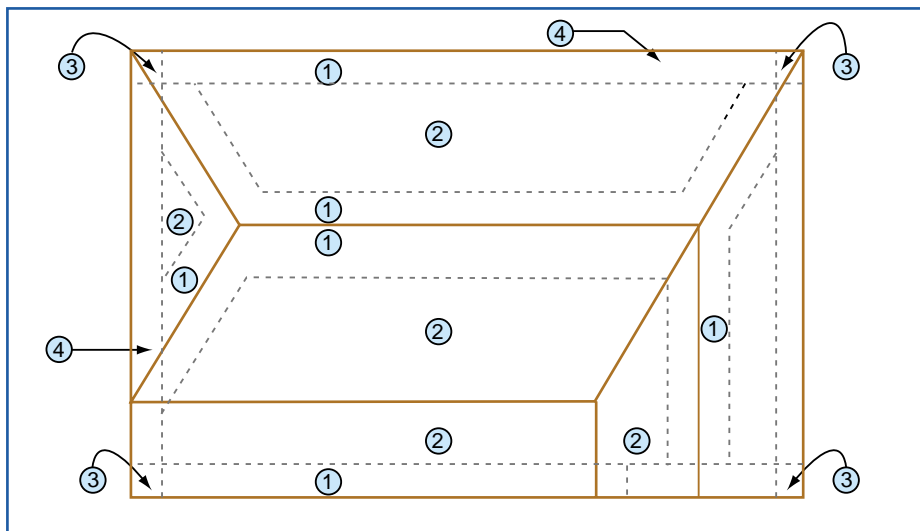


Figure 12-28

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

Table 12.6

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

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

**NOTE**

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

**NOTE**

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

**NOTE**

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

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

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

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



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

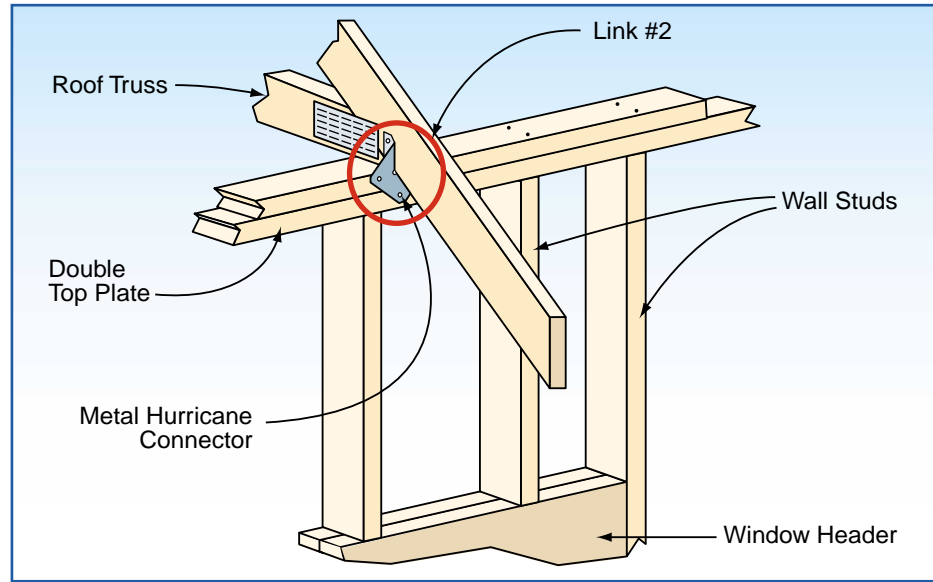


Figure 12-30
Hurricane Marilyn (1995),
U.S. Virgin Islands. Hip roof
that survived high winds with
little to no damage.

Link #2 – Roof Framing to Exterior Walls

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

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



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

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

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

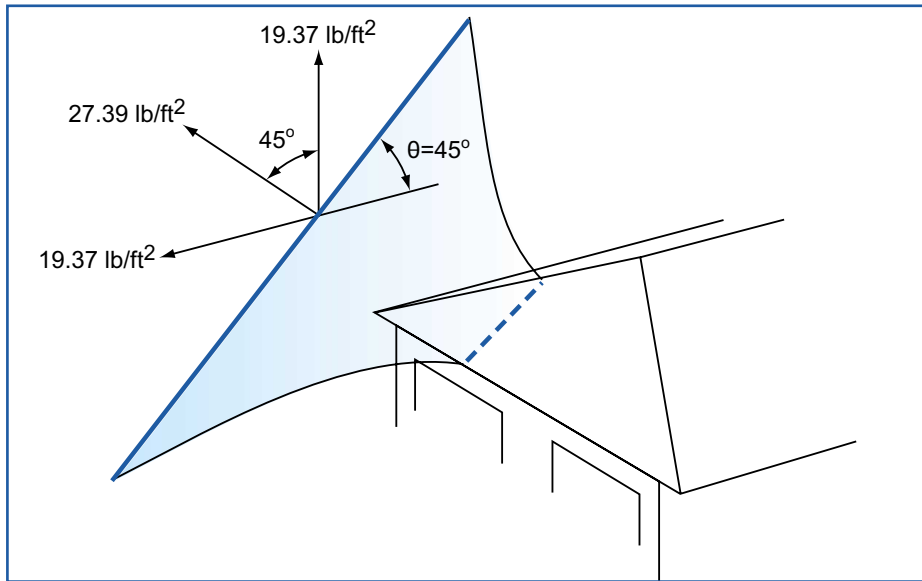


Figure 12-32
Uplift force components
on roof.

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

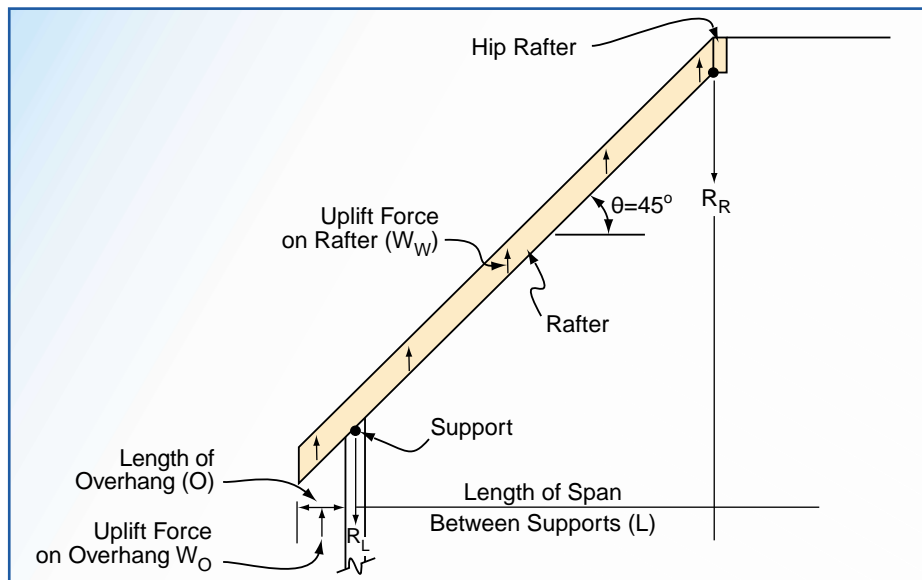


Figure 12-33
Uplift forces on the
connection of the roof
framing to the exterior wall
for this case study example.



Determining Uplift Forces at Each Connection in Hip Roof

Formula 12.5

Uplift Forces at Each Connection in Hip Roof

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

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

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

$$\text{Uplift force} = R_L =$$

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

where:

R_L = reaction left in lb

R_R = reaction right in lb

W_w = uplift force on rafter in lb/ft²

L = length of span between supports in ft

O = length of overhang in ft

W_o = uplift force on overhang in lb/ft²

θ = angle of roof, as measured from horizontal plane

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

Table 12.7

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



NOTE

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

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

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

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

$$(6\text{-ft rafter span})(2\text{-ft rafter spacing})(19.37 \text{ lb/ft}^2) = 232 \text{ lb} \quad [12.9]$$

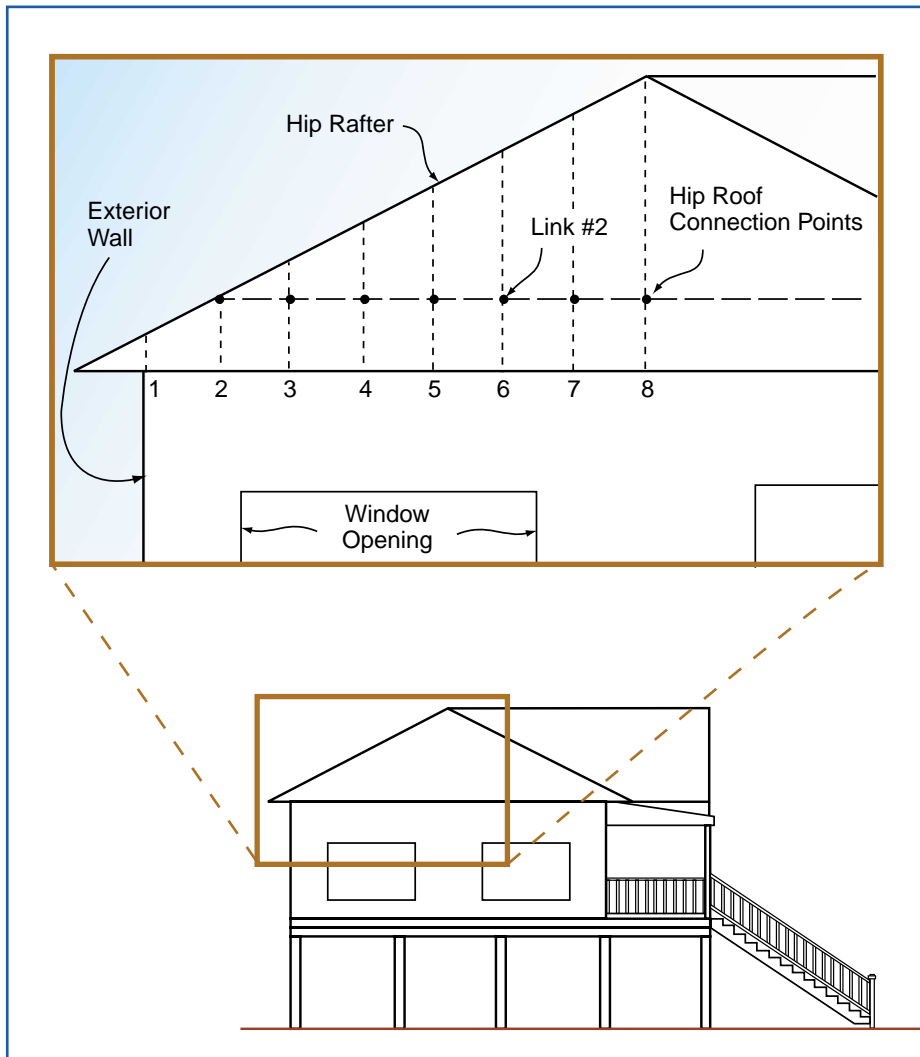


Figure 12-34
Uplift connections on hip roof.

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

So the total normal force = 232 lb + 225 lb = 457 lb

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

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

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

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

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

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



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



**Figure 12-37**

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

Link #3 – Top Wall Plate to Wall Studs

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

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

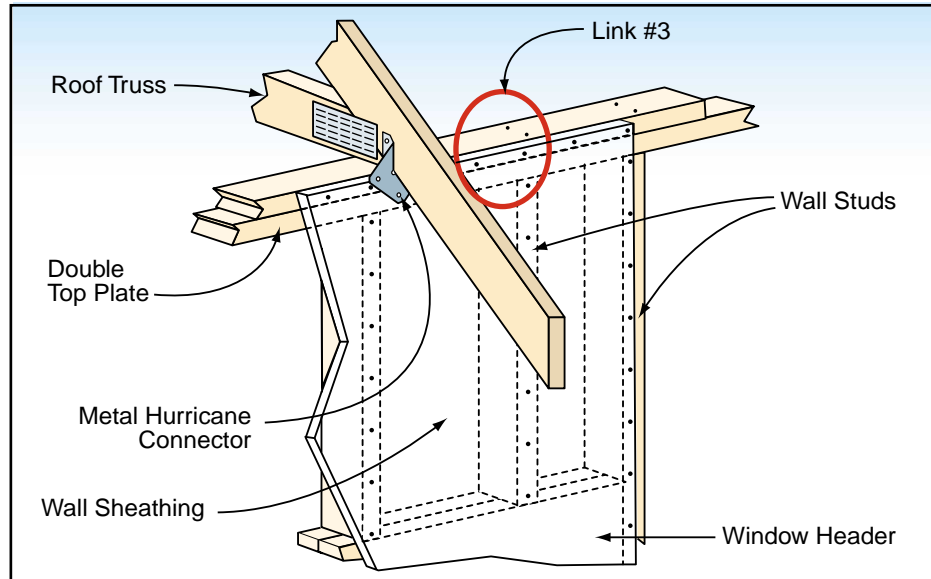
The shear capacity of 8d box nails (used commonly to install sheathing) is 78.4 lb each.



NOTE

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

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



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

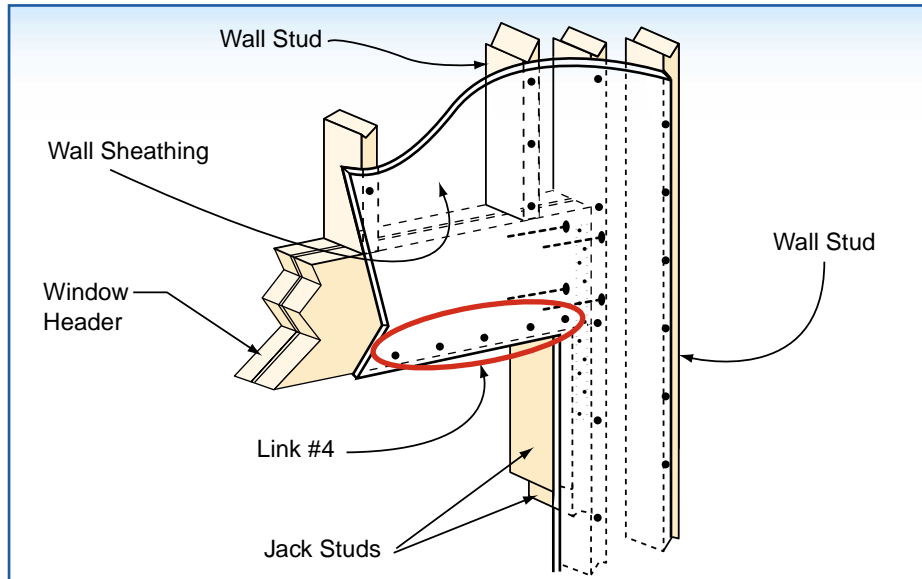
[12.10]

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

Link #4 – Wall Sheathing to Window Header

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

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

**Figure 12-39**

Link #4 – connection of wall sheathing to window header.

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

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

Link #5 – Window Header to Exterior Wall

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

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

The outward force at this connection results from the negative (outward) pressure of wind as it travels around the south side of the building. The wind

produces a suction pressure of 22.47 lb/ft^2 on the MWFRS, but nearly two times that (43.13 lb/ft^2) on the components and cladding (see Wind Load Example Problem on page 11-45). This force acts on a tributary area that includes part of the window and is calculated with pressure coefficients for components and cladding. Figure 12-41 shows the tributary area for this link.

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

Figure 12-40

Link #5 – connection of window header to exterior wall.

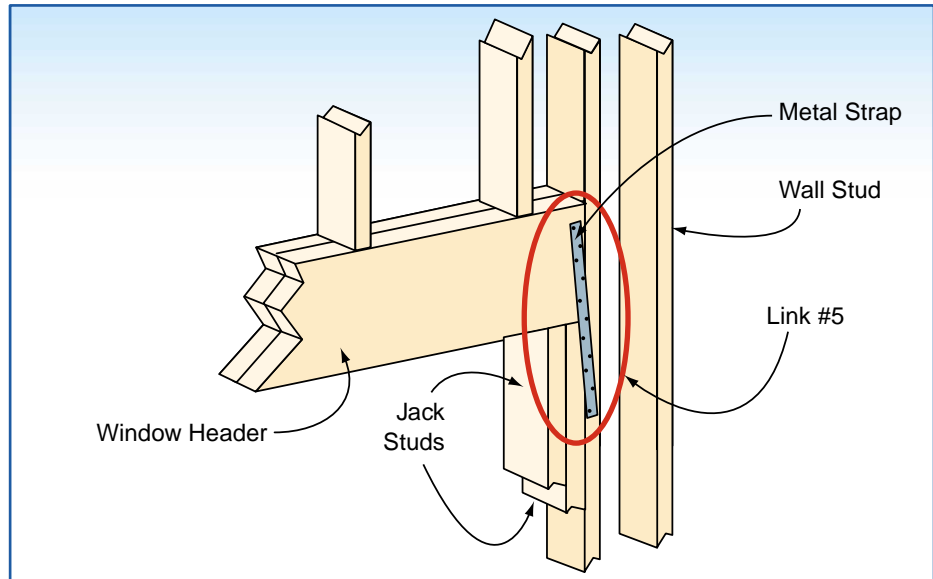
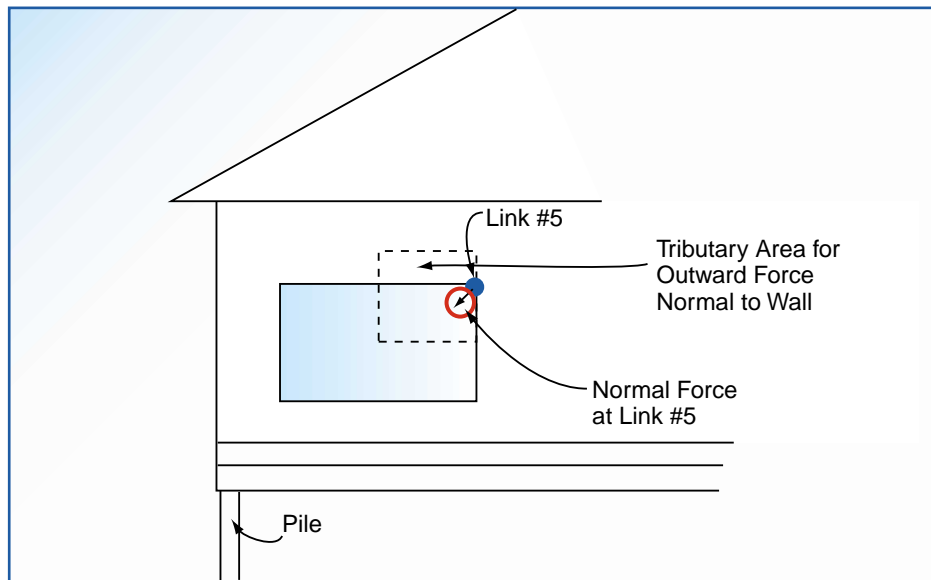


Figure 12-41

Link #5 – tributary area for wind force normal to wall.



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

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

[12.11]

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

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



NOTE

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

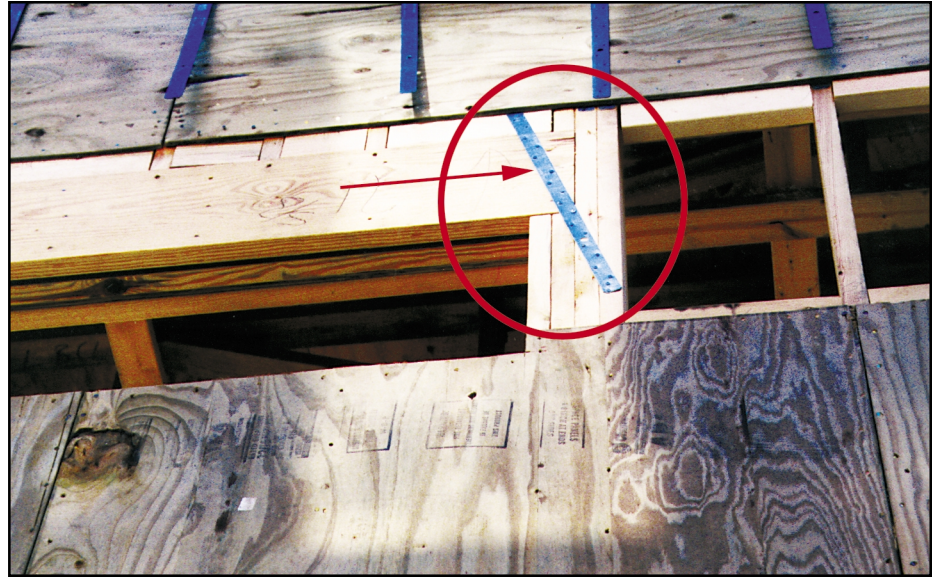
Number of 16d Nails	Shear (lb)
1	142
2	284
3	426
4	568
5	710
6	852
7	994
8	1,136

Table 12.8
Shear Resistance Provided
by 16d Box Nails

Notes:

1. Hem-fir with Specific Gravity = 0.43 is assumed.
2. 1/2" thick side member for withdrawal and 1-1/2" side member for shear is assumed.
3. Wind load factor = 1.6 is included.

Figure 12-42
Metal connector at window opening.

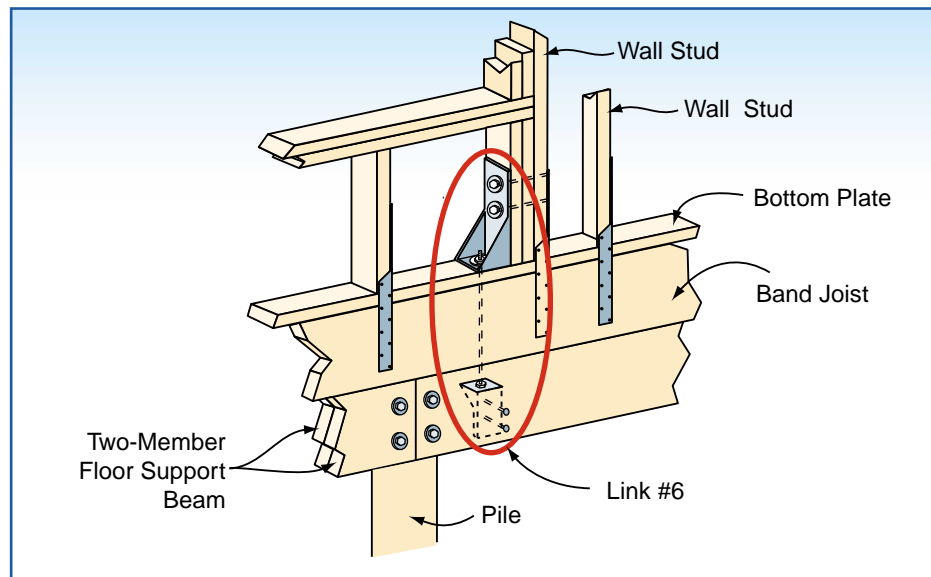


Link #6 – Wall to Floor Framing

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

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

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



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

Shearwalls

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

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

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

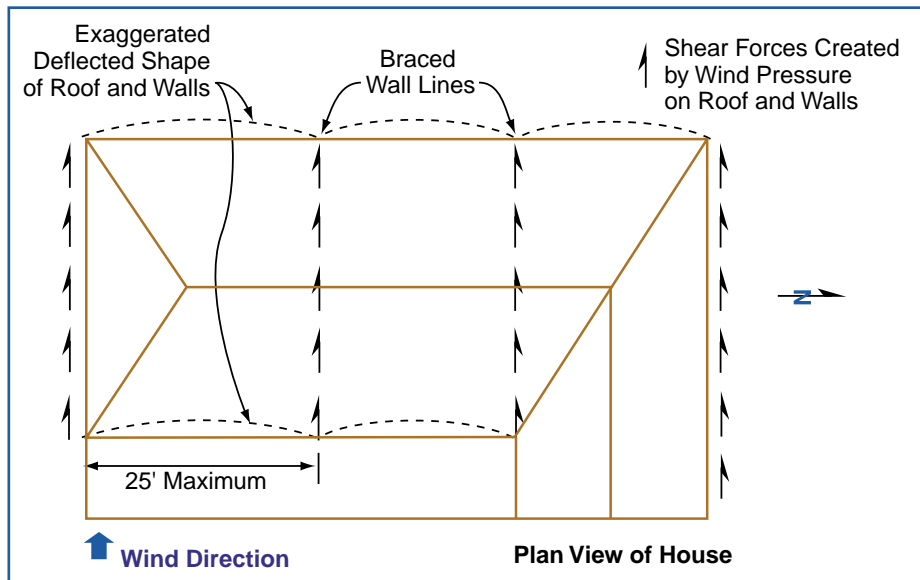


Figure 12-44
Shear force distribution from east wind at roof diaphragm.

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

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

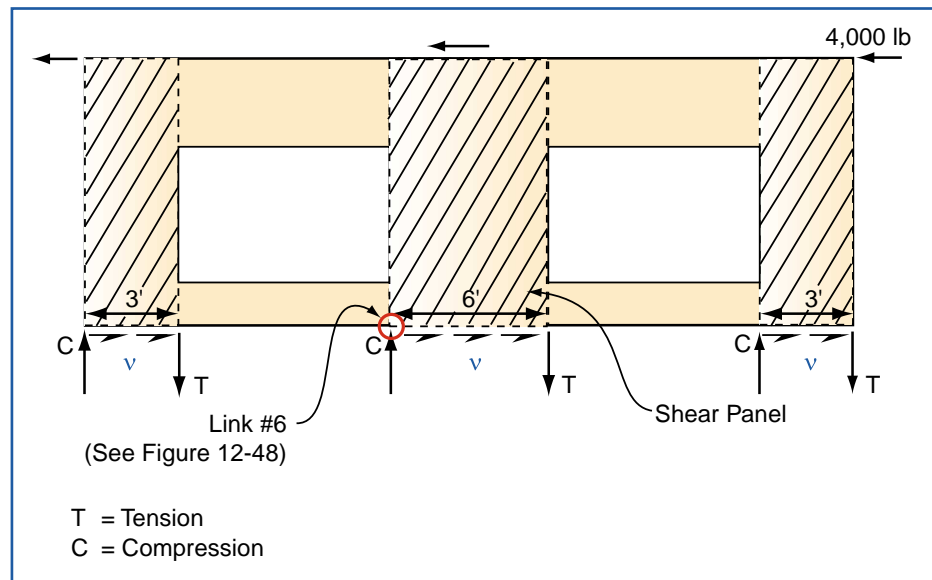
[12.12]

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

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

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

Figure 12-45
Loads on south shearwall.



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

$$4,000 \text{ lb}/(6 \text{ ft} + 3 \text{ ft} + 3 \text{ ft}) = 333.3 \text{ lb/ft}$$

[12.13]

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

$$(\text{unit shear } v)(\text{wall height})$$

[12.14]

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

$$(333.3 \text{ lb/ft})(10 \text{ ft}) = 3,333 \text{ lb} \\ + 731 \text{ lb (direct uplift from Link \#5)} = 4,064 \text{ lb}$$

[12.15]

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



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

Figure 12-47

Shearwall holddown connector (without bracket to post) attached to a concrete beam.



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

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

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

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

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

Alternative methods of increasing shear capacity include the following:

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

The forces at Link #6 are as follows:

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

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

Moment Frames

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

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

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

Figure 12-48
Load summary at Link #6.

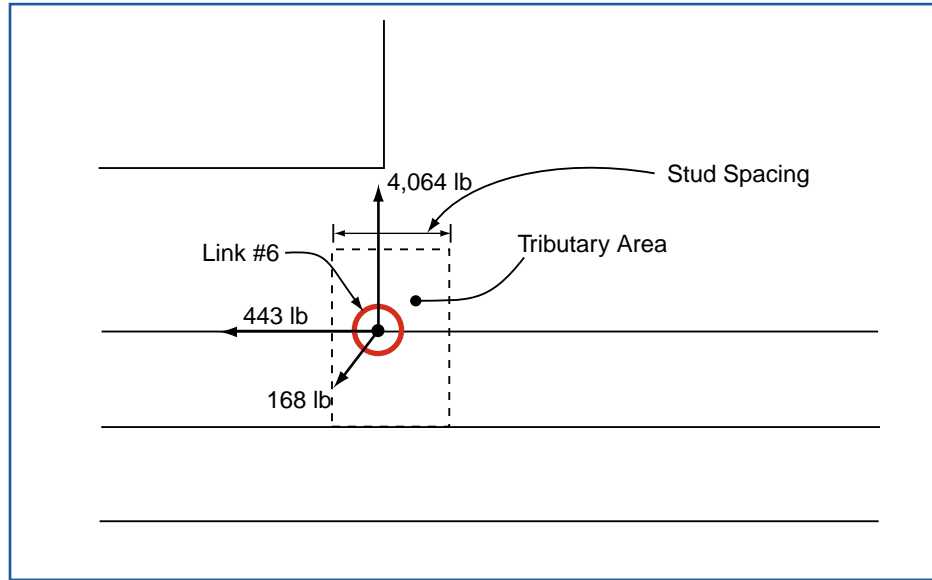


Figure 12-49
Steel moment frame
at large opening.



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

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

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

Link #7 – Floor Framing to Support Beam

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

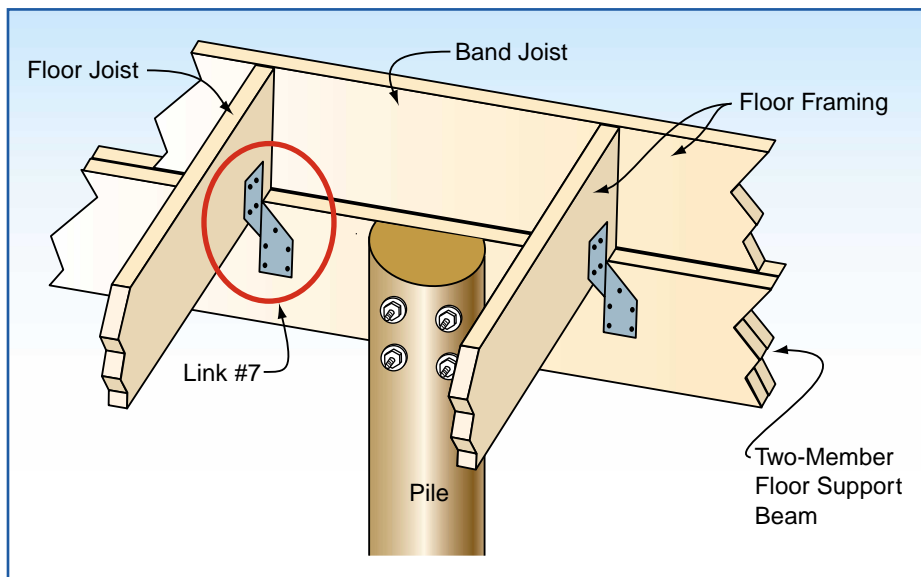
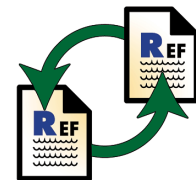


Figure 12-50

Link #7 – connection of floor framing to support beam.



CROSS-REFERENCE

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

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

The connection at Link #7 is normally made with metal connectors (see Figure 12-51); however, most metal connectors cannot be used for both uplift and lateral load transfer. When the connectors shown in Figure 12-50 are used for uplift resistance, additional clips must be used for lateral force transfer. This connection can also be made with a wood block nailed to the beam and

floor joists (see Figure 12-52). Advantages of a wood connection, provided it is sufficient to transfer the uplift forces, are that it eliminates the need for a metal connector, which is easily corroded, and can be used for both uplift and lateral load transfer.

Figure 12-51
Metal joist/beam connector.



Figure 12-52
Wood joist/beam
connector.

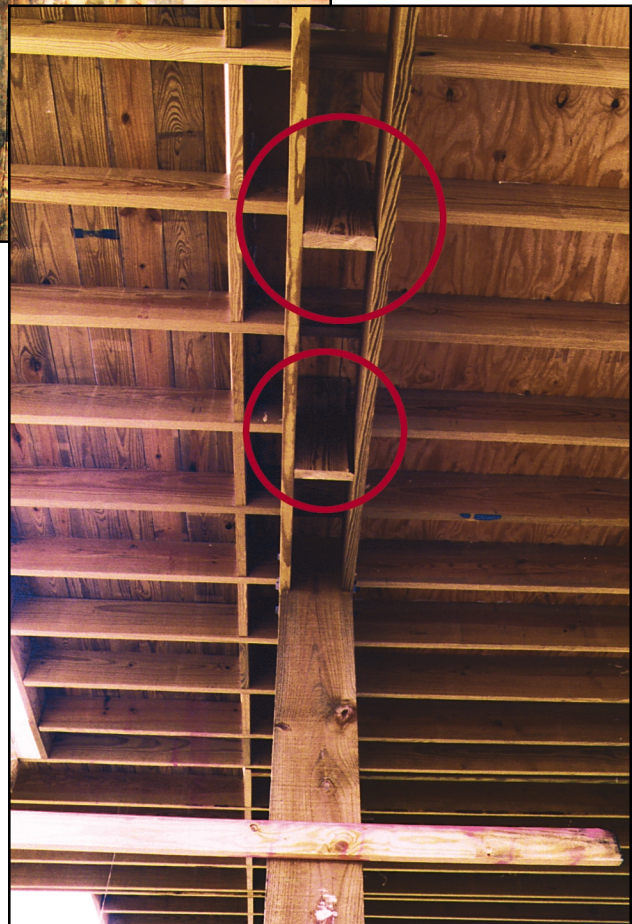


Table 12.8 (on page 12-47) lists the uplift resistance provided by 16d nails acting in single shear. It is common practice to frame wood buildings with 16d nails, so they have been used in this table of values. The wood floor members will normally be 2x10s so there is sufficient width of the floor joist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

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

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

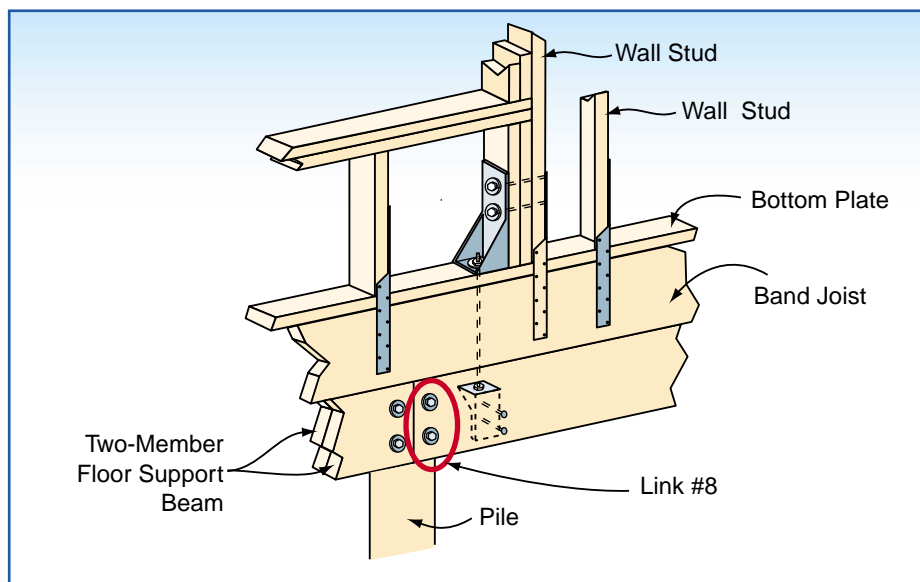
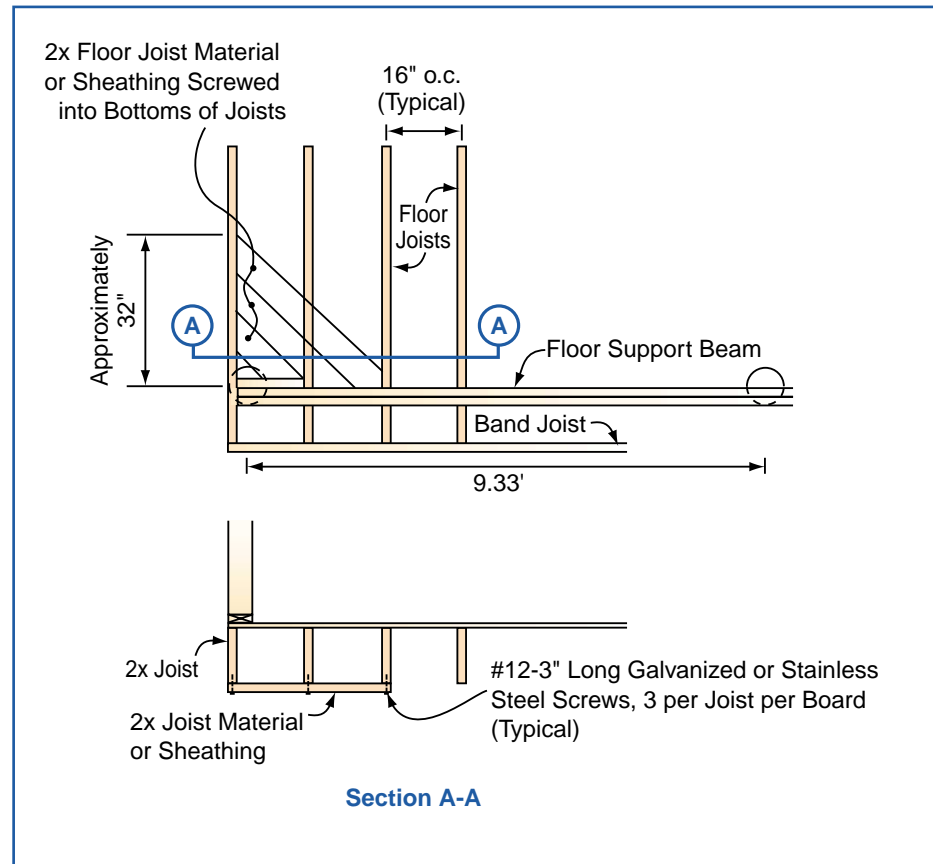


Figure 12-53

Link #8 – connection of floor support beam to foundation.

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

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



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

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

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

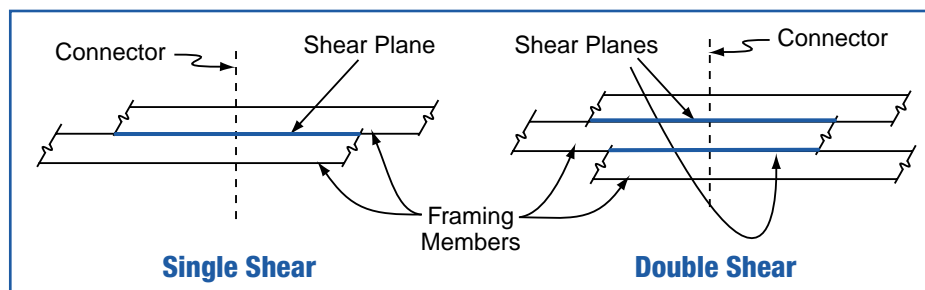


Figure 12-55
Single and double shear connections.

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

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

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

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

12.4.3 Foundation Design

12.4.3.1 Pile Design

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

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

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

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

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



NOTE

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



NOTE

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



NOTE

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

Gravity Loads

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



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

Formula 12.6 Ultimate Compression Capacity of a Single Pile

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

where:

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

P_T = effective vertical stress at pile tip (lb/ft²)

N_q = bearing capacity factor

A_T = area of pile tip (ft²)

k_{HC} = earth pressure coefficient in compression

P_0 = effective vertical stress over depth of embedment, D (lb/ft²)

δ = friction angle between pile and soil

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

D = depth of embedment (ft)

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

From the determination of dead load (Calculation 12.1), plus a live load of 20 lb/ft² on the roof and 40 lb/ft² on the floor of the tributary area, the total dead and live load on the case study pile is 5,974 lb. The compression load from overturning adds another 4,736 lb, so the total compression load is 10,710 lb. The allowable compression load is greater than the required load, so the pile is adequate for dead load, live load, and compression and overturning loading.

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

Uplift Loads

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

Formula 12.7 Ultimate Tension Capacity of a Single Pile

$$T_{ult} = \Sigma(k_{HT})(P_0)(\tan \delta)(s)(D)$$

where:

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

k_{HT} = earth pressure coefficient in tension

P_0 = effective vertical stress over depth of embedment, D (lb/ft²)

δ = friction angle between pile and soil

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

D = depth of embedment (ft)

Formula
Ultimate Tension
Capacity of a Single
Pile (Developed by
Frictional Resistance)

The formula predicts a tension capacity (T_{ult}) (with 1.8 feet of scour) of 14,965 lb. With the recommended FS of 3.0, the allowable tension capacity of the 15-foot pile is $T_{allow} = 14,965/3 = 4,988$ lb.

The frictional resistance required of the row of piles under the front porch if only those piles resisted the net overturning moment shown in Calculation 12.5 is 1,259,704 ft-lb/38 feet/7 piles or 4,736 lb/pile. Therefore, a 15-foot embedment satisfies overturning tension as well as compression.

Direct tension must also be checked. Figure 12-56 shows the tributary area that creates this tension. Depending on shearwall connection location, it is also possible that the tension or compression load from the shearwall connector will be concentrated on one pile (as it is in this case study).

The total tension load on the pile highlighted in Figure 12-56 is 8,586 lb, which exceeds the allowable tension load for the 15-foot embedment. This pile needs to be embedded 25 feet to resist the total tension load in this case study.

Table 12.9 summarizes the allowable compression and tension (uplift) capacities of wood piles placed 15 feet below the ground surface.

Figure 12-56
Uplift tributary area from hip roof to a single pile.

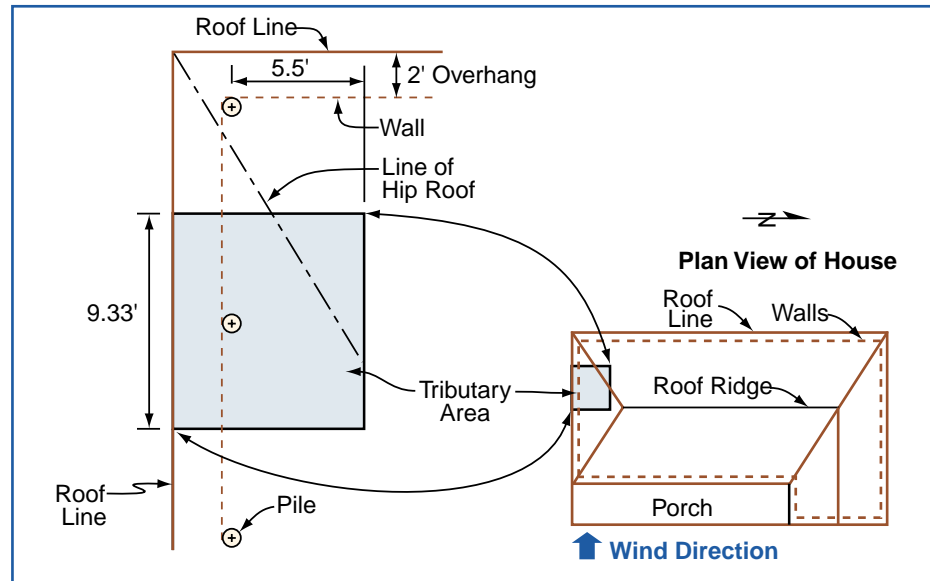


Table 12.9
Allowable Capacities of
Wood Piles (lb) for the Case
Study Example Only, Based
on End Bearing and
Frictional Resistance

Pile Type	Compression		Tension	
	No Scour	1.8 ft Scour	No Scour	1.8 ft Scour
Driven	11,338	10,594	5,477	4,988
Augered	4,764	4,466	1,905	1735
Jetted	6,099	5,822	2,143	1,952

Combined Bending and Compressive Loads

Structural members that carry both compressive and bending loads are more susceptible to buckling failure than those carrying only one or the other type of load. The effect of the combination of these loads is checked by an interaction formula shown as Formula 12.8.

Formula
Combined Axial and
Bending Stresses

Formula 12.8 Combined Axial and Bending Stresses

$$(f_c/F_c')^2 + f_b/[F_b(1-f_c/F_{cE})] \leq 1.0$$

where:

f_c = actual compressive stress (lb/in²)

F_c' = allowable compressive stress (lb/in²)

f_b = actual bending stress (from Formula 12.9)

F_b = allowable bending stress (lb/in²)

F_{cE} = Euler-based buckling stress (lb/in²)

Actual compressive stress f_c is 10,710 lb (from page 12-60)/pile area of 63.62 in² = 168 lb/in². The allowable compressive stress F_c' from the NTPC (1995) is:

$$F_c' = F_c C_d C_{ld} C_t C_{pt} C_{sp} C_p C_{cs}$$

$$F_c' = (1,278)(1.0)(1.6)(1.0)(0.9)(0.8)(0.15)(1.054) = 233 \text{ lb/in}^2$$

[12.16]

From the NTPC (1995), the allowable bending stress F_b' equals:

$$F_b' = F_b C_d C_{ld} C_t C_{pt} C_f C_{sp} C_{cs}$$

$$F_b' = (2,612)(1.0)(1.6)(1.0)(0.9)(1.0)(0.77)(1.054) = 3,052 \text{ lb/in}^2$$

[12.17]

The adjustment factors are listed in the NTPC. $C_d C_t C_{pt}$ are functions of the pile material and loading; $C_f C_{sp} C_{cs}$ are functions of the pile dimensions and length.

From Formula 12.9, $f_b = (989 \text{ lb})(13.5 \text{ ft})(12 \text{ in/ft})/71.57 \text{ in}^3 = 2,239 \text{ lb/in}^2$

Formula 12.9

Determination of Actual Bending Stress

$$f_b = (PxL)(12 \text{ in/ft})/S$$

where:

f_b = actual bending stress (lb/in²)

P = applied lateral load (lb)

L = allowable unbraced pile length (feet) (from Formula 12.10)

S = section modulus (in³)



NOTE

C_p in Calculation 12.16 is a column stability factor. The value of this factor depends on the effective column length from Formula 12.10. Section 3.7 of the NDS provides a formula for C_p .

The adjustment factors used in Calculations 12.16 and 12.17 are described in detail in the *Technical Guidelines for Construction With Treated Round Timber Piling*, by the National Timber Piling Council (NTPC 1995).



Determination of
Actual Bending
Stress

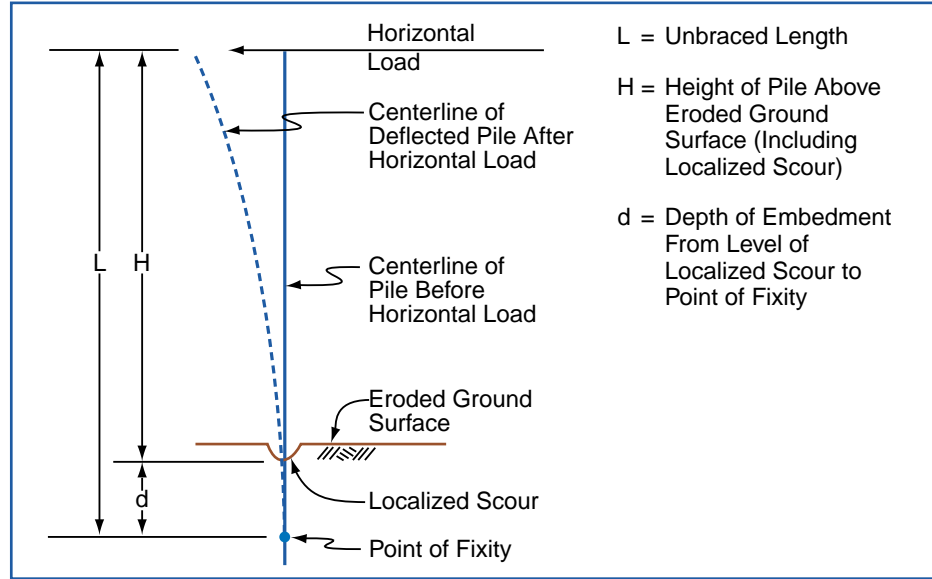
Horizontal Loads

The horizontal load on an unbraced pile causes the pile to deflect as illustrated in Figure 12-57. ASCE 24-98, Section C5.2.4.7 (ASCE 1998a), includes a formula (also included in this manual as Formula 12.10) for determining the allowable length of a pile above the point of fixity. The point of fixity is the point at which the pile is assumed to be fixed for translation and rotation, and it occurs at some distance (d) below the ground surface.

In Figure 12-57, the distance H includes the height above original ground plus the additional height resulting from any expected erosion and scour. In this case study, $H = 9.5 \text{ feet} + 1.8 \text{ feet} = 11.3 \text{ feet}$. The total effective length L is determined by Formula 12.10.

Figure 12-57

Deflected pile shape for an unbraced pile. The effective length of this pile is equal to $L \times (K=2.1)$.



Formula
Effective Length of Unbraced Pile

Formula 12.10 Effective Length of Unbraced Pile

$$L = K(H + d/12)$$

where:

L = length of pile (feet)

d = depth of embedment from level of localized scour to point of fixity = $(1.8)(EI/n_h)^{1/5}$ (inches)

E = Modulus of Elasticity of pile material (lb/in²)

I = moment of inertia of pile material (in⁴)

n_h = modulus of subgrade reaction (lb/in²)

K = effective length factor based on end conditions

H = length of pile between horizontal load and eroded ground surface (see Figure 12-57) (feet)

Table 12.10 lists recommended values for n_h , the modulus of subgrade reaction, for a variety of soils (Bowles 1996), which is used in Formula 12.10.

Table 12.10

Values of n_h , Modulus of Subgrade Reaction

Soil	n_h Modulus of Subgrade Reaction (lb/in ³)
Dense sandy gravel	800-1,400
Medium dense coarse sand	600-1,200
Medium sand	400-1,000
Fine or silty fine sand	290-700
Medium clay (wet)	150-500
Soft clay	6-150

The pile is assumed to have the following characteristics:

- material: southern pine with a modulus of elasticity (E) of 1,500,000 lb/in²
- tabulated fiber bending stress: $F_b = 2,612$ lb/in² 1995 National Timber Piling Council (NTPC)
- section: round timber with a nominal 9-inch diameter (8 inches at base; 10 inches at top); moment of inertia (I) = 322 in⁴; section modulus (S) = 71.57 in³ (at 9-inch section)
- end restraints: free at the top and fixed at the base; the fixed/free end conditions of a cantilever pile produce an effective length factor (K) of 2.1 (from Appendix G of the NDS).

Calculating the pile length L for a round timber pile using Formula 12.10, assuming $n_h = 700$ lb/in³:

$$d = (1.8)[(1.5 \times 10^6)(322)/700]^{1/5} = 26.5 \text{ in}$$

$$L = (11.3 \text{ ft} + 26.5/12) = (11.3 + 2.2) = 13.5 \text{ ft}$$

[12.18]

This length represents the unbraced length of the pile to its point of fixity. This length is used to determine the actual bending stress f_b (Formula 12.9) and the C_p factor used in Calculation 12.16

Now using Formula 12.8, the effect of combined bending and axial loads is determined. Using $f_b = 2,239$ lb/in², $F_b = 3,052$ lb/in², $f_c = 168$ lb/in², and $F_c = 233$ lb/in², the interaction formula = 3.15, which is much greater than 1.0. Therefore, this pile design does not work. The alternatives are as follows:

- larger piles
- reduced spacing
- shorter effective length
- piles with greater strength than wood

In order for a wood pile to be acceptable, the effective length shown in the last column of Table 12.11 must be greater than approximately 13.5. This length is determined by working backwards through Formula 12.8 and the factor C_p to determine what the maximum effective length could be when Formula 12.8 yields a value of 1.0. As can be seen from Table 12.11, the pile must be at least 12 inches in diameter for the load and height criteria to be met.

Another way to make this pile design acceptable is to shorten the effective length with bracing. This method is discussed in Section 12.4.5

Table 12.11 Maximum Unbraced Lengths of Various Pile Materials for the Case Study Loads^a

Pile	Material ^b	E (lb/in ²)	I (in ⁴)	S (in ³)	F _B (lb/in ²)	Length (ft)
Wood – 9" diameter	P.T. S.Y.P.	1,500,000	322	71.57	3,052	7.1
Wood – 10" x 10" square	P.T. S.Y.P.	1,500,000	678.8	142.9	3,052	12.8
Wood – 12" diameter	P.T. S.Y.P.	1,500,000	1,018	169.65	3,052	15.1

^a Buckling and the interaction formula not considered.

^b P.T. = Pressure Treated; S.Y.P. = Southern Yellow Pine.

Allowable Shear Stress

From the NTPC, the allowable shear stress F_v' equals:

$$F_v' = F_v C_{ld} C_t C_{pt}$$

$$F_v' = (110)(1.6)(1.0)(0.9) = 158 \text{ lb/in}^2$$

The actual average shear stress is the horizontal load at the pile-ground intersection from Calculation 12.7 divided by the area of the pile, or $2,293 \text{ lb}/63.62 \text{ in}^2 = 36 \text{ lb/in}^2$. The maximum shear stress is $4/3$ the average shear stress, or 48 lb/in^2 . Thus shear is not a failure mode for the pile under the case study conditions.

Deflection

From beam theory, the deflection at the top of the case study 13.5-foot-long (H+d only) cantilevered pile horizontally deflected by a 989-lb load is 2.9 inches. This is an excessive amount of deflection that most building occupants would find unacceptable. This deflection is being caused by very high winds, however, in what is expected to be an unoccupied house. This deflection can be reduced by bracing to reduce the unbraced length. See Section 12.4.5 for additional information.

Other Pile Types

Other types of piles and methods of installation are available, including the following:

- precast concrete
- cast-in-place concrete
- steel H-sections
- concrete filled steel pipe piles

Some advantages of, and special considerations for, of each of these are listed in Table 12.12.

Pile Material Type	Advantages	Special Considerations
Wood (ASTM-D25)	<ul style="list-style-type: none"> • Comparatively low initial cost • Permanently submerged piles are resistant to decay • Relatively easy to drive in soft soil • Easy to connect to wood framing • Suitable for friction and end bearing pile 	<ul style="list-style-type: none"> • Difficult to splice • Subject to eventual decay when in soil or intermittently submerged in water • Vulnerable to damage from driving — easy to split • Comparatively low compressive load • Relatively low allowable bending stress
Concrete (ACI 318 for concrete, ASTM-A15 for reinforcing steel)	<ul style="list-style-type: none"> • High compressive load capacity for precast/prestressed piles • Corrosion resistant • Can be driven through some hard material • Suitable for friction and end bearing pile • Reinforced piles have high bending resistance 	<ul style="list-style-type: none"> • High initial cost • Difficult to splice • Vulnerable to breakage • Must be reinforced, no bending allowed in plain concrete • More difficult to attach to wood framing • Not usable in high seismic areas
Steel (ASTM-A36)	<ul style="list-style-type: none"> • High resistance to bending • Easy to splice • Available in many lengths and sizes • Able to drive through hard subsurface material • Suitable for end bearing • High compressive capacity 	<ul style="list-style-type: none"> • Vulnerable to corrosion • May be permanently deformed if impacted by heavy object • High initial cost • Some difficulty attaching wood framing

Table 12.12
Pile Material Selection

There are several methods for installing piles, including driving, augering, and jetting.

Driving a pile is hitting the top of the pile with a pile driver hammer until the pile reaches the desired depth. In some areas, piles are driven with vibratory hammers, which generate vertical oscillating movements that reduce the soil stress against the pile and make the piles easier to drive. Ultimate load resistance is achieved by a combination of end bearing of the pile and frictional resistance between the pile and the soil. A record of the

blow counts from the pile driver can be used with a number of empirical formulas to determine capacity.

Augering is drilling a full or partial pile-diameter hole in the soil to some predetermined depth, installing the pile, and then driving the pile into the hole until it reaches refusal.

Jetting is removing soil with a jet of water (or air) as the pile is driven, and then driving the pile down until it reaches refusal. Both augering and jetting remove natural, undisturbed soil along the side of the pile. Load resistance for both of these methods is achieved by a combination of end bearing and frictional resistance, although the frictional resistance is much less than that provided by driven piles.

Figure 12-58 illustrates the three pile installation methods. Table 12.13 lists advantages and special considerations for each method.

It is important to note from Table 12.13 that the 40-percent reduction in the k factors for compression and tension loads on augered and jetted piles reduces the allowable loads by the same percentage. Therefore, for the case study building, if the piles were augered or jetted into place, the allowable load in compression would be reduced to 2,710 lb, which is then less than the required load of 10,710 lb. The allowable load in tension is reduced to 1,220 lb, which is less than the net uplift load of 8,429 lb. Consequently, if the piles were installed by augering or jetting, they would have to be embedded further to resist both the compression and tension loads .

12.4.3.2 Masonry or Concrete Columns

While local floodplain regulations require the use of open foundations in V zones, foundation materials are not restricted to wood as long as the foundation is anchored to resist flotation, collapse, and lateral movement. Solid foundation walls may be constructed in A zones, but must comply with the NFIP and have openings to equalize hydrostatic pressure. Columns constructed of concrete or reinforced masonry can be used, but must be constructed on a solid foundation such as a spread footing or a pile cap to distribute loads onto undisturbed soils below the anticipated erosion and scour depth. This type of foundation is normally assumed to be fixed at its base.

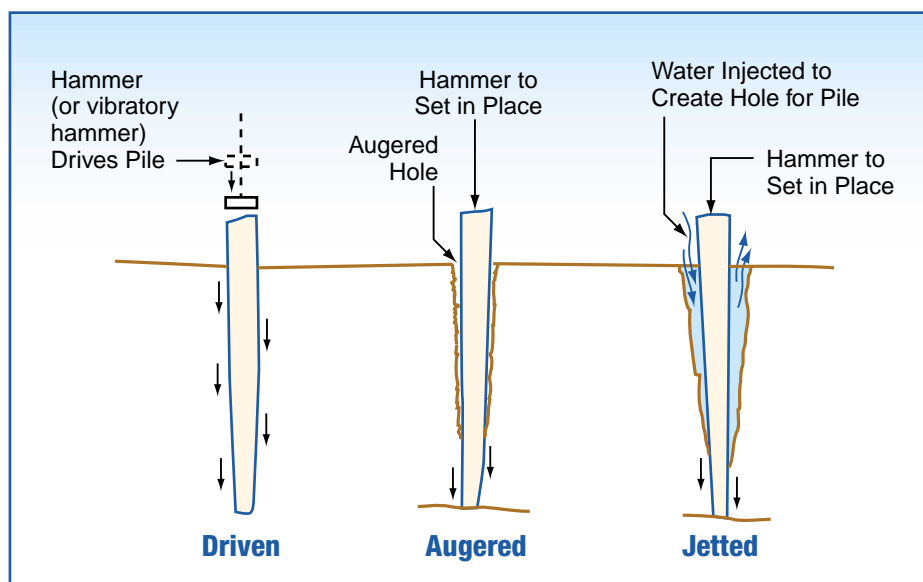


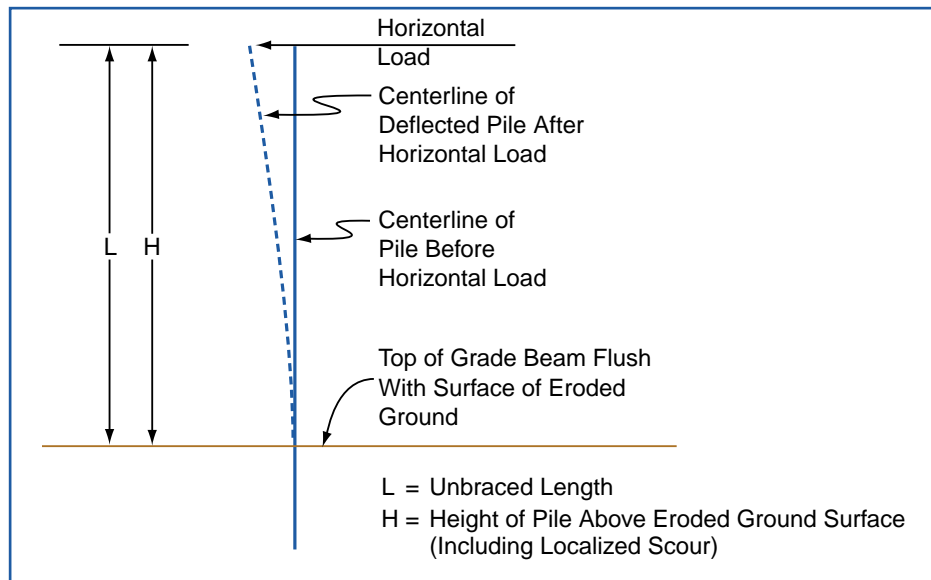
Figure 12-58
Pile installation methods.

Table 12.13 Selection of Pile Installation Method

Pile Installation Method	Advantages	Special Considerations
Driven	<ul style="list-style-type: none"> • Well suited for friction pile • Common construction practice • Pile capacity can be determined empirically 	<ul style="list-style-type: none"> • Difficult at times to reach terminating soil strata, which is not necessary for friction piles • Difficult to maintain plumb during driving and thus maintain column lines
Augered	<ul style="list-style-type: none"> • 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 	<ul style="list-style-type: none"> • Requires subsurface investigation • Not suitable for highly compressible material • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients k_{HC} and k_{HT} to 40 percent of that for driven piles • Capacity must be determined by engineering judgment or load test
Jetted	<ul style="list-style-type: none"> • Minimal driving vibration to adjacent structures • Well suited for end bearing • Easier to maintain column lines 	<ul style="list-style-type: none"> • Requires subsurface investigation • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients k_{HC} and k_{HT} to 40 percent of that for driven piles • Capacity must be determined by engineering judgment or load test

A way to reduce this unbraced length is to “fix” the column in both translation and rotation at some point along its length. A timber pile can be fixed in only translation by the addition of horizontal grade beams. These grade beams may be constructed of concrete or wood; they must be installed so that they are self-supporting and the effects of scour have been considered. The effectiveness of grade beams in “fixing” the piles is subject to engineering judgment. See Figure 12-59 for an illustration of this design concept.

Figure 12-59
Design method for reducing effective column length of a cantilever pile.



Guidance on the design of concrete or masonry piers on spread footings follows. It will be obvious why the use of these foundation methods in V zones and coastal A zones is often not appropriate. It should be noted, however, that foundations of this type can be appropriate in V zones and coastal A zones where bedrock, coral rock, or some other stratum that terminates erosion and scour is present.

CALCULATION OF UNBRACED PILE LENGTH (L)

Distance from top of pile to DFE = 2 feet

Distance from DFE to eroded ground surface = 9.5 feet

Scour = 1.8 feet

**L = 2 feet + 9.5 feet + 1.8 feet
= 13.3 feet**

First for V zones, the design example that follows will use the loads developed in previous sections. From previous examples and the Flood Load Example Problem presented on page 11-30 in Chapter 11, $L = 13.3$ feet (see calculation at left). It is assumed that the allowable soil bearing pressure is $2,000 \text{ lb/ft}^2$. Figure 12-60 illustrates a column on a spread footing with the axial (P_a) and lateral (P_l) forces determined previously.

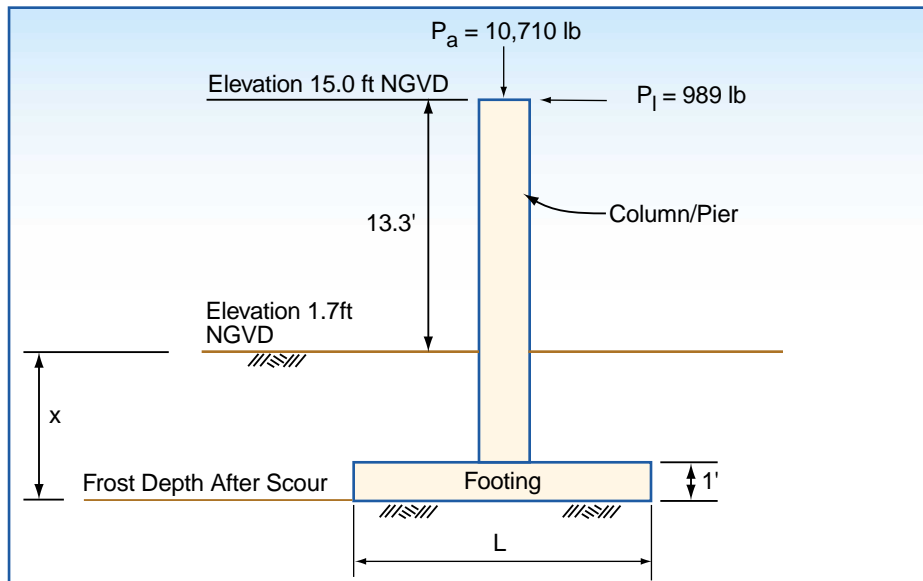


Figure 12-60
Column/pier on a spread footing.

The following foundation elements and design parameters must be determined:

- soil bearing pressure assuming a footing size or a footing size assuming 2,000-lb/ft² soil bearing pressure
- bending stresses in the column, to determine the amount of reinforcing steel to place in the column
- compressive stresses in the column, to determine whether these stresses exceed the allowable for the material selected for the column
- bending stresses in the footing, to determine the amount of reinforcing steel to place in the footing

This foundation type must be designed for the same conditions as the pile – gravity loads, uplift loads, horizontal loads, and the resulting bending that occurs in the column.

Gravity Loads

The footing must be large enough to carry the following loads:

- weight of the foundation
- axial load P_a

\sqrt{f} formula Determination of Square Footing Size for Gravity Loads for the Case Study Building

The footing size can be determined by Formula 12.11.

Formula 12.11 Determination of Square Footing Size for Gravity Loads

$$L = \{P_a + [\text{col. } h + (x-1)](\text{col. } w)(\text{col. } t)(w_c)\} / [q - (\text{foot. } t)w_c]^{0.5}$$

where:

L = square footing dimension

P_a = axial load

h = column height above scour elevation

x = depth of bottom of footing from scour elevation

w = column width

t = column thickness

w_c = unit weight of column and footing material

q = soil bearing pressure

foot. t = footing thickness

For a concrete column and footing with $h = 13.3$ feet, $x = 2$ feet, column dimensions are 16 inches square, $\text{foot. } t = 1$ foot, $q = 2,000 \text{ lb/ft}^2$, $P_a = 10,710 \text{ lb}$, and the unit weight of concrete is 150 lb/ft^3 , the required footing size to support only the gravity load is 2.8 feet square.

Uplift Loads

Uplift resistance is provided by the weight of the foundation and column and the weight of soil on top of the footing. In this case, where the height of the column is 13.3 feet and concrete weighs 150 lbs/ft^3 , the footing and column weight of 4,920 lb does not exceed the uplift load of 8,429 lb. The uplift load can be resisted by a footing that is 5.6 feet square.

Horizontal Loads

Overturning of the foundation must be resisted by the allowable soil bearing pressure and the weight of soil above the “heel” of the footing. This footing design concept is very similar to that of a retaining wall. The centroid of resistance of the soil to the overturning forces acts at a point R located as shown in Figure 12-61. It is assumed for this analysis that q is always in the middle one-third of the footing, so overturning stability is satisfied.

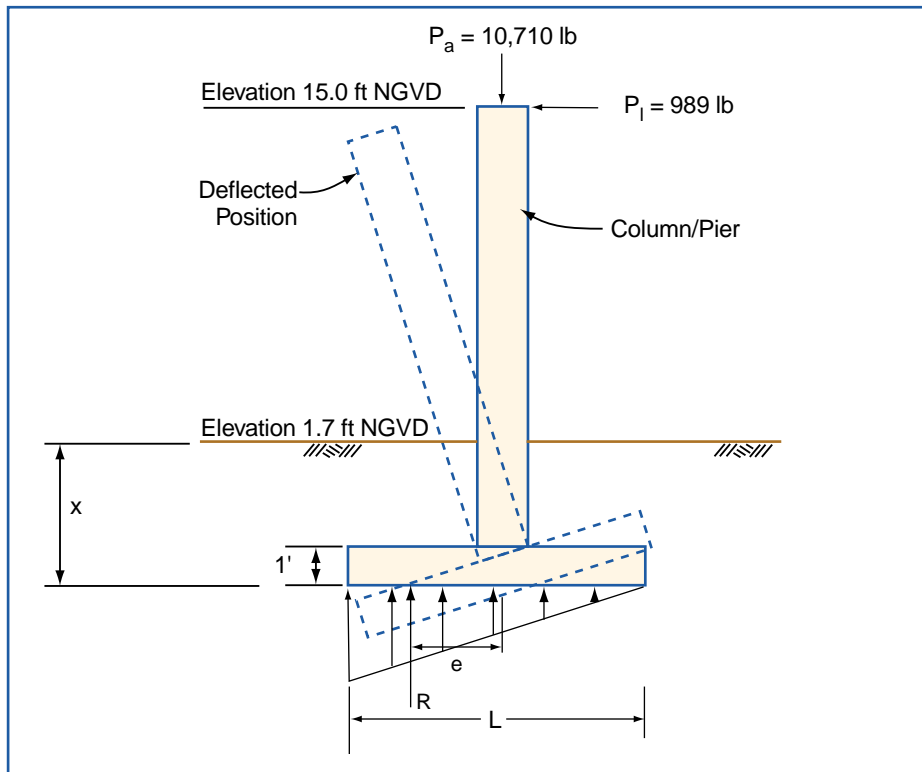


Figure 12-61
Horizontal load diagram for
columns/piers on spread
footings.

As illustrated, if the soil bearing capacity is not adequate to support the overturning forces, the column and footing rotate. Sufficient rotation will result in either failure of the foundation due to excessive settlement or failure in the connection at the top of the column. See *Foundation Analysis and Design* (Bowles 1996) for additional information on foundation design.

Determining the length of footing L required to resist the loads given in this case study without exceeding the 2,000-lb/ft² allowable soil bearing pressure is an iterative process. Using Formula 12.12, the square footing length L must be such that q_{\max} is 2,000 lb/ft² and q_{\min} is 0 lb/ft². The footing size for this case study and this unique set of parameters is 5 feet.

**Formula 12.12 Determination of Soil Pressure
for the Case Study Building**

$$q = P_t/L^2 \pm 6M/L^3$$

where:

q = soil bearing pressure

P_t = total vertical load, including P_a , weight of column and weight of footing

L = square footing dimension

M = moment = $P_l(13.3 + x)$, where x = the depth of the bottom of the footing from the scour elevation

e = eccentricity (see Figure 12-61) = M/P , which cannot exceed $L/6$

formula

Determination of Soil
Pressure for the Case
Study Building—
From *Foundation
Analysis and Design*
(Bowles 1996)

In order for the pier and footing to act as one unit, the connection between the footing and the base of the pier must be fixed. This fixed condition can only be developed with sufficient steel reinforcement that is adequately anchored into both the footing and the pier.

When the uplift load of 8,429 lb is considered with the lateral loads (which would be the case for the windward- or seaward-most row of piles), and q_{\max} is 2,000 lb/ft² and q_{\min} is 0 lb/ft², the required footing size is 9.7 feet square. Thus the footing size is governed by the combination of uplift and lateral forces.

A 9.7-foot-square footing is not practical when the pile spacing is 9.33 feet o.c. in one direction and 11 feet o.c. in the other direction. In addition, excavation in sand for a 9.7-foot-square footing 4 feet below grade results in, for all practical purposes, a continuous 1-foot-thick concrete mat under the house, approximately 4 feet below grade. This type of foundation, known as a “mat” or “raft” foundation, is used in some locations along the coast of the United States.

Allowable Bending Stress and Moment

Design for bending in concrete or masonry involves designing steel reinforcement to resist tensile stresses (because neither concrete nor masonry has much tensile strength) and designing the section to resist failure in compression or crushing. Design guidance can be found in many of the references listed at the end of this chapter. The determination of these stresses involves effort beyond the scope of this manual and is left to the designer.

If the case study building were placed in an A zone where the breaking wave height is just less than 3 feet (see Chapter 6 for a description of A zones) and all other loads on the building were the same as the example above, the following design parameters would apply:

- The lateral force is minimally reduced.
- The height of the building above the ground (after scour) is only 1 foot lower than the V zone building (see Figure 11-13 for BFEs for the case study) so “moment arms” are practically the same as before.
- With the moments on the concrete or masonry piers practically the same as before and forces only slightly reduced, the footing length is practically the same as the footing length required in the V zone.

As a result, and for other reasons noted in previous chapters, this manual recommends that the foundation standards applied in V zones also be used for coastal A zone construction.

Prescriptive designs, including column sizes, reinforcing requirements, and allowable heights, are available for both masonry and concrete materials. Designers should refer to the *Masonry Designers' Guide* (ACI 1993) and *Building Code Requirements for Masonry Structures*, ACI 530, (ACI 1999a) for information on how to design masonry columns, and to *Building Code Requirements for Structural Concrete and Commentary*, ACI 318, (ACI 1999b) for information on how to design concrete columns.

12.4.4 Other Important Load Paths

Several additional design considerations must be investigated for the case study building in order for the design to be complete. The details of these investigations are left to the designer, but they are mentioned here to more thoroughly cover the subject of continuous load paths and to point out that many possible paths require investigation.

From Figure 12-16, the following vertical load paths should also be investigated:

- uplift of the front porch roof
- uplift of the main roof section that spans the 28-foot width of the building

The following lateral load paths also need to be investigated:

- lateral load on the front gable wall (see “Shearwalls” on page 12-49)
- interior shearwalls noted on Figure 12-44
- possible twisting of the L-shaped front during either a wind or seismic event (see Section 12.4.4.2)

12.4.4.1 Gable Wall Support

There are many cases of failures of gable-end walls during high-wind events. The primary failure modes in gable-end walls are as follows:

- A gable wall that is not braced into the structure collapses, and the roof framing falls over (see Figure 12-62).
- An unsupported gable-end eave ladder used for overhangs is lifted off by the wind and takes the roof sheathing with it.
- The bottom chord of the truss is pulled outward, twisting the truss and causing an inward collapse.

Figure 12-62
Hurricane Andrew (1992),
Dade County, Florida. Gable-
end failure.



It is common in non-coastal areas to expect the roof sheathing to provide the required lateral bracing of the roof system. However, the wind pressures are too great in coastal areas, and certainly during high-wind events, to expect that the sheathing is sufficient to provide the required lateral support.

Recommendations for additional gable wall bracing are provided in the *Guide to Wood Construction in High Wind Areas*, by the Wood Products Promotion Council (WPPC 1996), and in retrofitting and rebuilding guidance for the U.S. Virgin Islands. The bracing recommendation shown in Figure 12-63 is based on an illustration in the WPPC publication. In addition, truss manufacturers and the Truss Plate Institute recommend that permanent lateral bracing be installed on all roof truss systems.

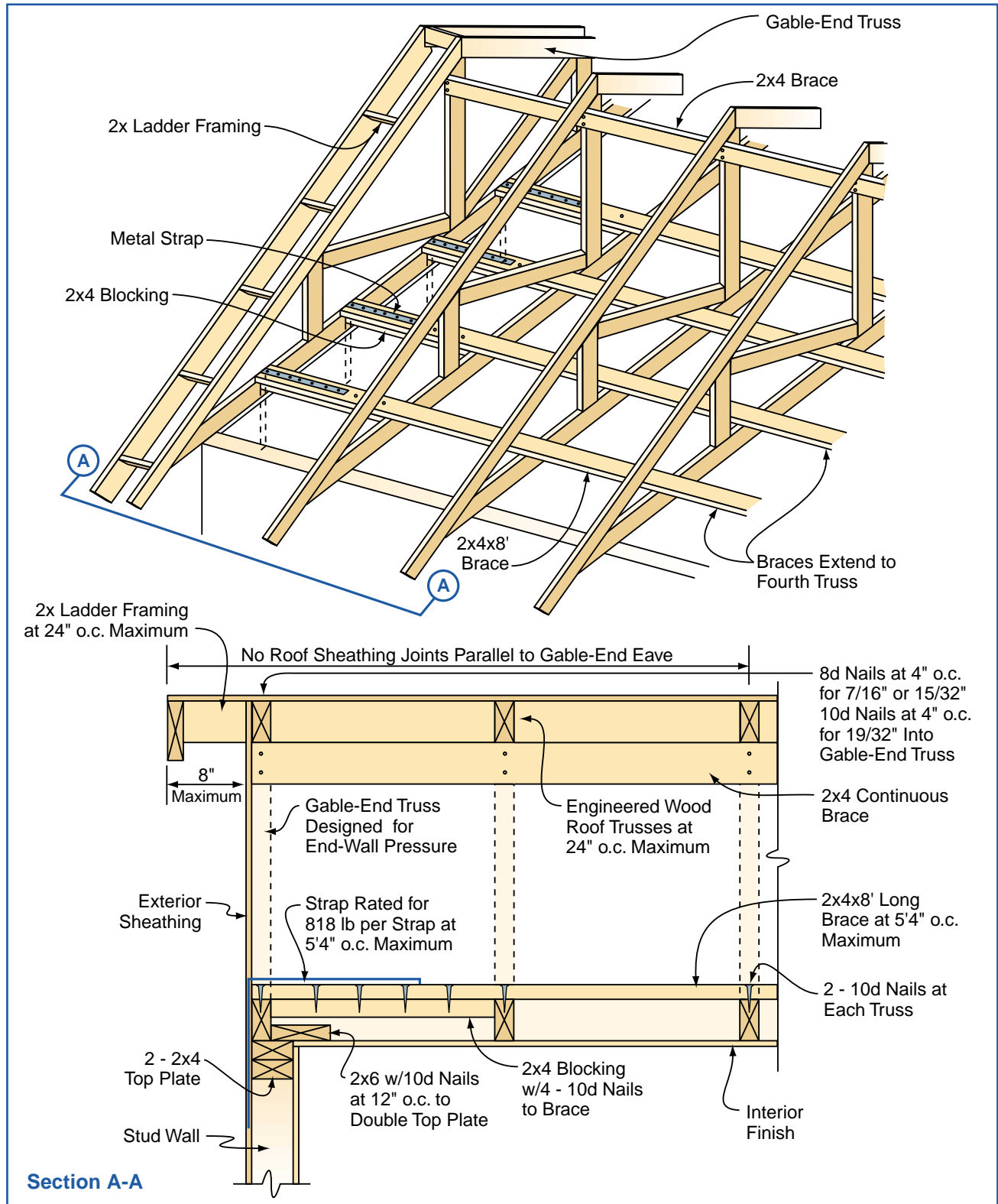
12.4.4.2 Building Eccentricities

In the simple one-story, L-shaped case study building, the L-shape contributes an eccentricity to the distribution of forces. High winds can cause the L to rotate about the inside corner where the extension meets the main portion of the building. A seismic event can also create rotation about the inside corner. The differences in the two “masses” located at the two centers of gravity will create stress at the inside corner and on the pile-supported foundation.

12.4.5 Bracing

In a pile foundation, piles can be braced in three ways:

- grade beams
- diagonal bracing across the full pile heights
- knee bracing

Figure 12-63 Gable-end bracing recommendations.

Source: Wood Products Promotion Council (1996)

Grade beams provide support in the horizontal plane parallel to the floors. **Diagonal braces** are normally attached to the pile near the top and secured to the adjacent pile either near the ground surface or at the height that reduces the unbraced length to the required height. **Knee braces** are normally installed at 45° angles between the floor framing and the pile and are usually placed within 4 feet of the top of the pile. Figures 12-64, 12-65, and 12-66 show examples of these bracing methods.

Figure 12-64
Grade beams.



NOTE

This manual recommends that if full-height diagonal bracing is used, it be placed parallel to the direction of current and waves.



Figure 12-65
Diagonal bracing.



NOTE

ASCE 24-98 (ASCE 1998a) recommends that designers strive to establish a stable design free of bracing and use bracing only to add rigidity to the design for the comfort of the occupants. As noted in ASCE 24-98, past experience has shown that cross-bracing often fails during a storm event and does not provide the expected degree of support.





Figure 12-66
Knee bracing.

Bracing is used to reduce the effective unbraced pile length, which will reduce the lateral movement of a cantilever foundation system when a lateral load is applied and reduce the stresses in the pile.

12.4.5.1 Grade Beams

Because grade beams are usually placed at or below grade, they are normally constructed with either wood or concrete. The maximum allowable unbraced pile length in this case study is 7.1 feet. In the case study, erosion and scour expose 11.3 feet of pile, thus grade beams cannot be used in this case to reduce the effective pile length.

Grade beams have advantages and disadvantages.

Advantages:

- When grade beams are placed near the ground surface, they do not obstruct the area between piles, making the space more usable.
- Grade beams reduce the potential for catching debris during a storm because they do not block the area between the piles, the floor, and the ground surface.

Disadvantages:

- Grade beams become an obstruction around which high-velocity flow must be redirected. Scour potential around and under grade beams can be significant.



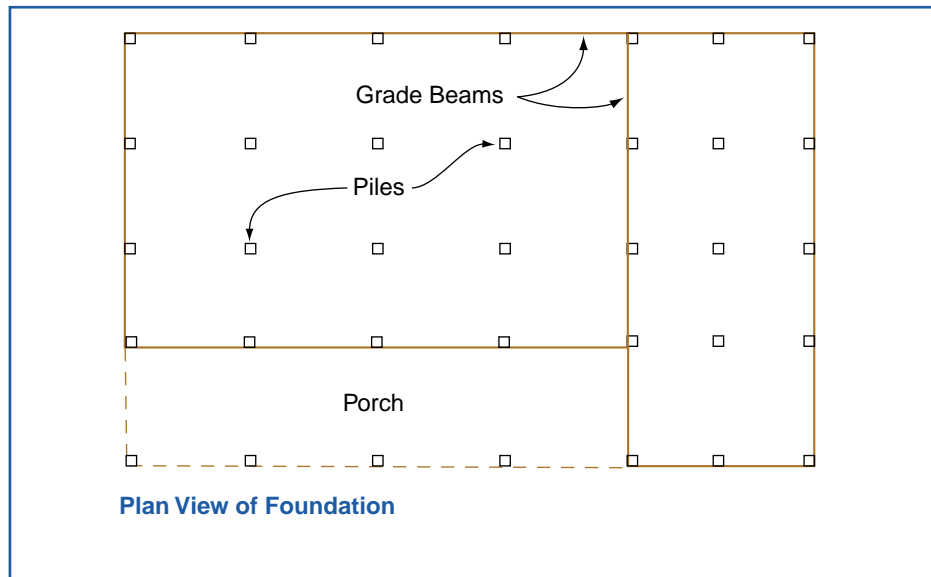
COST CONSIDERATION

See Tables 9.5 and 9.6, in Chapter 9, regarding the financial implications of having the lowest floor below the BFE.

- Grade beams shall not be used as structural support for a concrete slab below an elevated building in a V zone. Although such a slab may serve as the floor of a ground level enclosure-usable only for parking, storage, and building access-the slab must be independent of the building foundation. If a grade beam is used to support the slab, the slab becomes lowest floor of the building, the beam becomes the lowest horizontal structural member supporting the lowest floor, and the bottom of the beam becomes the reference elevation for flood insurance purposes. The NFIP, IBC and IRC, require buildings in V zones to have the bottom of the lowest horizontal structural member supporting the lowest floor elevated to or above the BFE. Therefore, to avoid the slab being considered the lowest floor, slabs must be separate from grade beams. This requires that the slab not be monolithic with the grade beam, and that the slab and grade beam not be tied together through such means as reinforcing steel. (Chapter 9 describes how the elevation of the bottom of the lowest horizontal structural member can affect flood insurance premiums.)

Lateral resistance to bending and excessive deflection is required for all piles that support lateral loads, so installing lateral bracing along the perimeter of the building and at any intermediate shearwall placed in either direction is one way to provide the lateral resistance. Grade beams that provide lateral bracing must be continuous in order to transfer the horizontal loads to adjacent piles. Figure 12-67 illustrates a grade beam layout.

Figure 12-67
Grade beam layout.



12.4.5.2 Diagonal Bracing

Diagonal bracing is normally constructed with wood framing members or steel rods or angles. This bracing is attached at the top of one pile and near grade level at adjacent piles. Diagonal bracing usually acts in tension only, so the bracing attachment to the pile must be capable of resisting that tension force.

Diagonal bracing has advantages and disadvantages.

Advantages:

- Bending stresses in piles are greatly reduced, so piles can be taller, and fewer piles may be required
- Piles can be braced to practically any required unbraced length as long as the lumber or steel used for the bracing is manufactured in lengths that will reach from one pile to the next.
- There are no obstructions placed at grade; therefore, the risk of scour at and around the piles is reduced.

Disadvantages:

- Diagonal bracing creates an obstruction between the piles, thereby increasing the risk that debris will become trapped. When trapped debris obstructs the flow of flood water, loads on the foundation are increased.
- Bracing only in the direction parallel to flood flow (which would be acceptable if properly done) is difficult with diagonal bracing.
- Braces provide unwanted support to breakaway walls (see Section 12.4.6 and FEMA NFIP Technical Bulletin 9 in Appendix H) and therefore can prevent breakaway walls from failing as intended. When this occurs, loads on the foundation are increased.

Figure 12-68 shows the forces that occur at the top of the pile when diagonal bracing is used to reduce unbraced length.

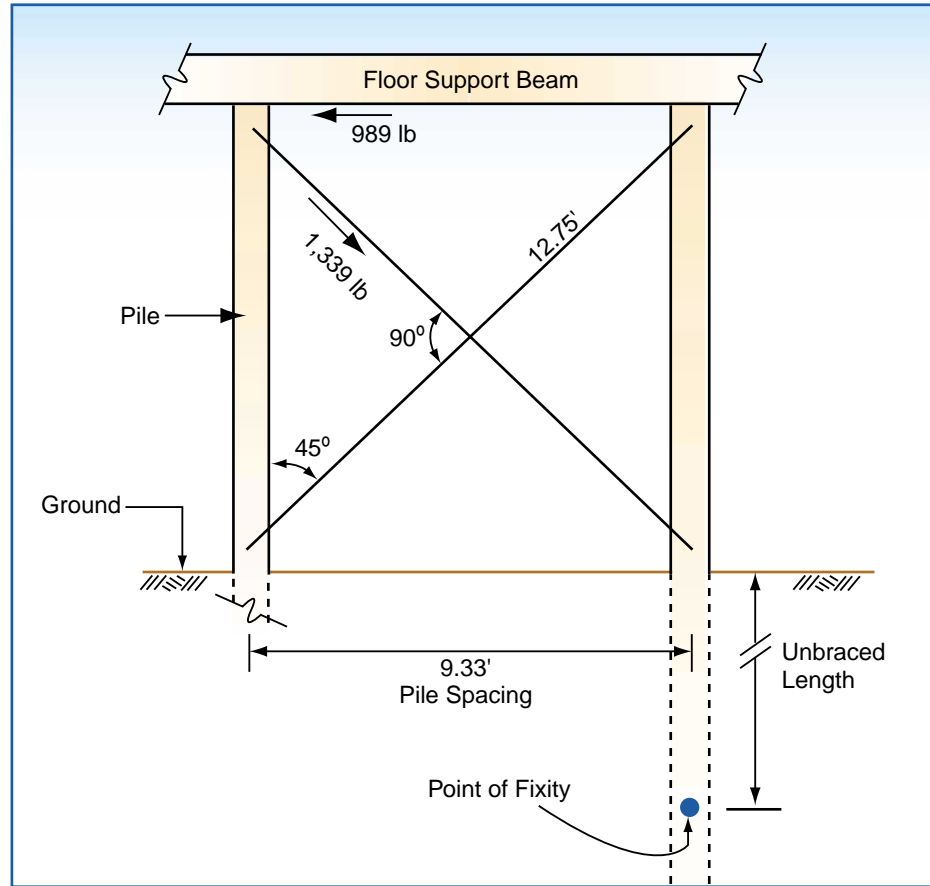
The tension force in the diagonal is determined as follows:

$T_{\text{diagonal}} = (989/\cos 45^\circ) = 1,399 \text{ lb}$. The interaction of the soil with the pile is expected to fully resist the tension load on the pile from the diagonal.

[12.20]

The restraint provided by the brace reduces the effective length factor K for the 9-inch-diameter wood pile to about 1.5, and the bending stress is reduced because the restraint at the top of the pile causes a double curvature in the

Figure 12-68
Force diagram for diagonal
bracing.



NOTE

Steel rods are subject to corrosion in salt air environments. For buildings in such environments, designers should specify rods with a greater cross-sectional area, or stainless steel or hot-dip galvanized rods.



WARNING

Because of considerations regarding serviceability, grade of lumber, and out-of-plane bending from flood impacts, the minimum size wood brace recommended is 2 inches x 6 inches.

pile. These effects reduce the result of the interaction formula (Formula 12.8) to less than 1.0, thus this method of reducing stress and effective length has provided a means by which the 9-inch-diameter wood pile can be used throughout for the case study building.

Assuming southern pine, the allowable tension stress in the brace is:

$$F_t = (825 \text{ lb/in}^2)(C_d) = (825)(1.6) = 1,320 \text{ lb/in}^2.$$

[12.21]

The required size of the southern pine bracing is:

$$A_t = 1,399 \text{ lb} / 1,320 \text{ lb/in}^2 = 1.06 \text{ in}^2. \text{ The minimum size brace is 1 in x 3 in with an area of 1.875 in}^2.$$

[12.22]

If steel rods are used, the minimum size of the bracing is $1,399 \text{ lb} / (60,000 \text{ lb/in}^2 \text{ yield strength})(0.67 \text{ allowable stress factor}) = 0.015 \text{ in}^2$ or a bar with a diameter of at least $3/16$ inch that has an area of 0.03 in^2 . Steel rods and most 1x and 2x wood braces are used only in tension.

The connection between the diagonal brace and the pile must be sufficient to transfer loads from the pile to the brace. Through-bolts are the preferred

method. Lag bolts, while they may have sufficient capacity, can work their way out of the wood as the wood dries and as the brace is loaded and unloaded.

Assuming 1x bracing and an 8-inch-diameter pile, the 1,399-lb load requires two 5/8-inch-diameter bolts (1997 NDS, Table 8.2A). Section 8.5 of the 1997 NDS is used to determine the bolt placement. Figure 12-69 illustrates one method of placing the two bolts.

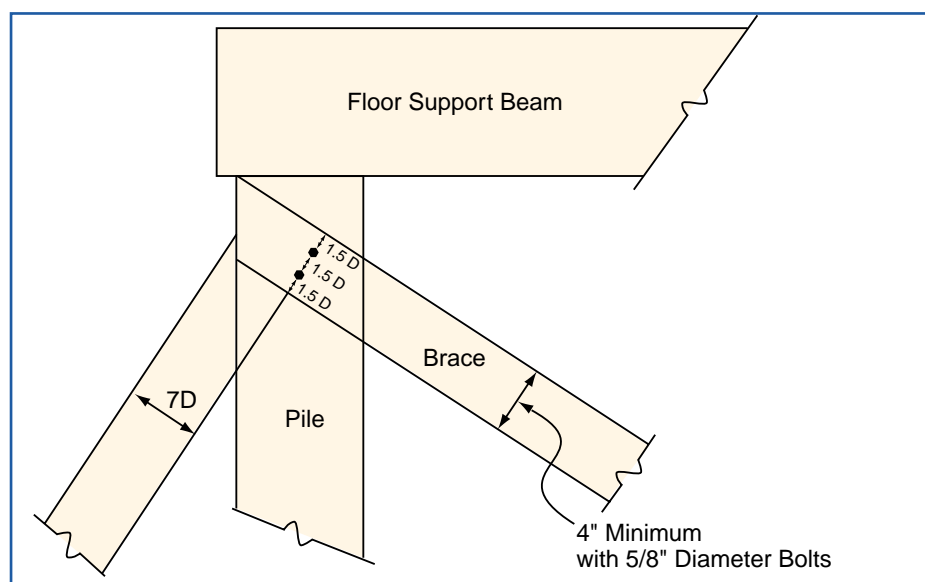


Figure 12-69
Bolt layout for diagonal brace. A space equal to 1.5 bolt diameters (1.5D) should be provided between bolts and between the bolts and the edge of the brace. This spacing dictates a minimum brace width of 2.8 inch, which means the minimum brace size is 1 inch x 4 inch for this case study example.

12.4.5.3 Knee Bracing

Like grade beams and diagonal bracing, knee bracing is installed to reduce the unbraced pile length. The unbraced length starts at the point where the knee bracing is connected to the pile (see Figure 12-70).

Knee bracing has advantages and disadvantages.

Advantages:

- Piles can be braced with shorter members. Using shorter bracing reduces the obstructions between piles, and therefore the potential for catching debris, and increases the usability of the area between the piles.
- No obstructions are placed at grade, which reduces the risk of scour around the piles.

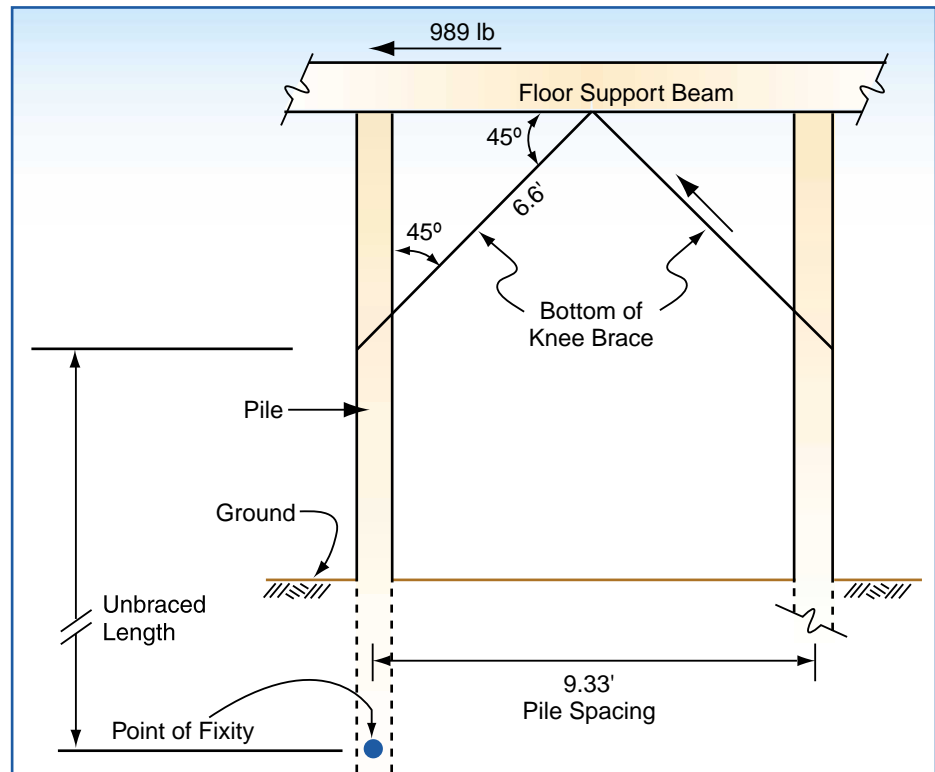
Disadvantages:

- Knee bracing is not as effective as full-height diagonal bracing in reducing the bending stress in piles subjected to high lateral loads.
- It is difficult to brace only in the direction parallel to flood flow (which would be acceptable if properly done).

- Braces provide unwanted support to breakaway walls (see Section 12.4.6 and FEMA NFIP Technical Bulletin 9 in Appendix H) and therefore can prevent breakaway walls from failing as intended. When this occurs, loads on the foundation are increased.
- Shear at top of pile is increased.

The bottoms of the knee braces are placed to reduce the unbraced length. Figure 12-70 illustrates the placement of knee braces on the pile.

Figure 12-70
Knee brace placement and forces.



WARNING

Because of considerations regarding serviceability, grade of lumber, and out-of-plane bending from flood impacts, the minimum size wood brace recommended is 2 inches x 6 inches.

The tension force in the knee brace is larger than that in the diagonal brace. In compression, the 1,098-lb load requires a minimum member size of 1 inch x 12 inches or 2 inches x 3 inches to prevent buckling failure. The knee brace has reduced the effective length factor K to 1.2 for the 9-inch-diameter pile and has reduced the bending stress because the restraint at the top causes double curvature in the pile. These effects have reduced the result of the interaction formula to less than 1.0, which indicates that this bracing method also has provided a means by which the 9-inch-diameter pile can be used.

12.4.6 Breakaway Wall Enclosures

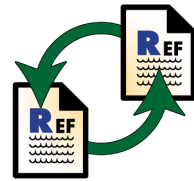
In V and coastal A zones, breaking waves are almost certain to occur simultaneously with peak flood conditions. As breaking waves pass an open piling or column foundation, the foundation experiences cyclic fluid impact and drag forces. The flow peaks at the wave crest, just as the wave breaks. Although the flow creates drag on the foundation, most of the flow under the building is undisturbed. This makes open foundations somewhat resistant to wave actions and makes pile and column foundations a manageable design.

When a breaking wave hits a solid wall, the effect is quite different. When the crest of a breaking wave impacts a vertical surface, a pocket of air is trapped and compressed by the wave. As the air pocket compresses, it exerts a high-pressure burst on the vertical surface, centered at the stillwater level. The pressures can be extreme. For example, a 5-foot wave height can produce a peak force of 4,500 lb/ft², roughly 100 times the force caused by a 130-mph wind. These extremely high loads make designing solid foundation walls for small buildings impractical in areas subject to the effects of breaking waves. Prudent design dictates elevating buildings on an open foundation above potential breaking waves. In fact, the IBC 2000 (ICC 2000a) and the IRC 2000 (ICC 2000b) require that new, substantially damaged, and substantially improved buildings in V zones be elevated above the BFE on an open (e.g., pile, post, column, or pier) foundation.

The IBC and IRC prohibit obstructions below elevated buildings but allow enclosures below the BFE as long as they are constructed with insect screening, lattice, or walls designed and constructed to fail under the loads imposed by floodwaters. Because such enclosures will fail under flood forces, they will not transfer additional significant loads to the foundation. Regulatory requirements and design criteria concerning enclosures below elevated buildings in V zones are discussed in FEMA NFIP Technical Bulletin 9 (see Appendix H). As explained in the bulletin, breakaway walls may be of wood- or metal-frame or masonry construction.

Figure 12-71 shows how a failure begins in a wood-frame breakaway wall—with the failure of the connection between the bottom plate of the wall and the floor of the enclosed area. Figure 12-72 shows a situation in which utility components placed on and through a breakaway wall prevented it from breaking away completely.

If screening is used it may be either metal or synthetic. Lattice is available in pre-manufactured 4-foot x 8-foot sheets. Either wood or plastic lattice is acceptable, provided the material used to fabricate it is no thicker than 1/2 inch and the finished sheet has an opening ratio of at least 40 percent. Figure 12-73 shows lattice used to enclose an area below an elevated building.



CROSS-REFERENCE

NFIP compliance provisions, as described in the IBC 2000 and the IRC 2000, are discussed in Chapter 6 of this manual.

Figure 12-71

Hurricane Hugo (1989), South Carolina. Typical failure mode of breakaway wall beneath an elevated building—failure of the connection between the bottom plate of the wall and the floor of the enclosed area.

**Figure 12-72**

Hurricane Opal (1995), Florida. Utility penetrations prevented this breakaway wall panel from breaking away cleanly.



12.5 Step 4 – Develop Connections at Each Link

12.5.1 Connection Choices

Alternatives for joining building components include the following:

- mechanical connectors such as those available from a variety of manufacturers
- fasteners such as nails, screws, bolts, pegs, and reinforcing steel
- connectors such as wood blocks
- alternative materials such as adhesives and strapping



Figure 12-73
Lattice installed beneath an elevated house in a V zone.

Most commercially available mechanical connectors that have been approved by the model building code product evaluation services are fabricated metal devices formed into shapes designed to fit snugly around components such as studs, rafters, and wall plates. To provide their rated load, these devices must be properly nailed or bolted as specified by the manufacturer. Mechanical connectors are typically galvanized with 1 to 2 oz of zinc per ft². **Thicker coatings are recommended because they provide greater protection against corrosion.** Welded steel products are generally hot-dip galvanized or painted for corrosion protection. Stainless steel (A304 and A316 SS) connectors are also available. Because exposed metal fasteners (even when galvanized) can corrode within a few years of installation in coastal areas, **stainless steel is recommended where rapid corrosion is expected.**

Connector manufacturers provide specifications and ratings for their products. Often this information is not reviewed sufficiently and improper connectors are selected. Therefore, the designer should review the specifications carefully. Particular attention should be given to the following:

- corrosion protection provided
- wood species or lumber type used in framing (e.g., ordinary framing lumber, wood I-beams, LVL products)
- ultimate capacity of connector for all modes of failure (e.g., shear, uplift, gravity loading)
- corrosion protection provided for the nails
- nail size and type required to achieve rated loads
- use in new or retrofit applications



NOTE

Table 1 in FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas* (see Appendix H), lists recommendations for corrosion-resistant connectors.

In the following sections, each link in the studied load path is investigated and connection alternatives are suggested.

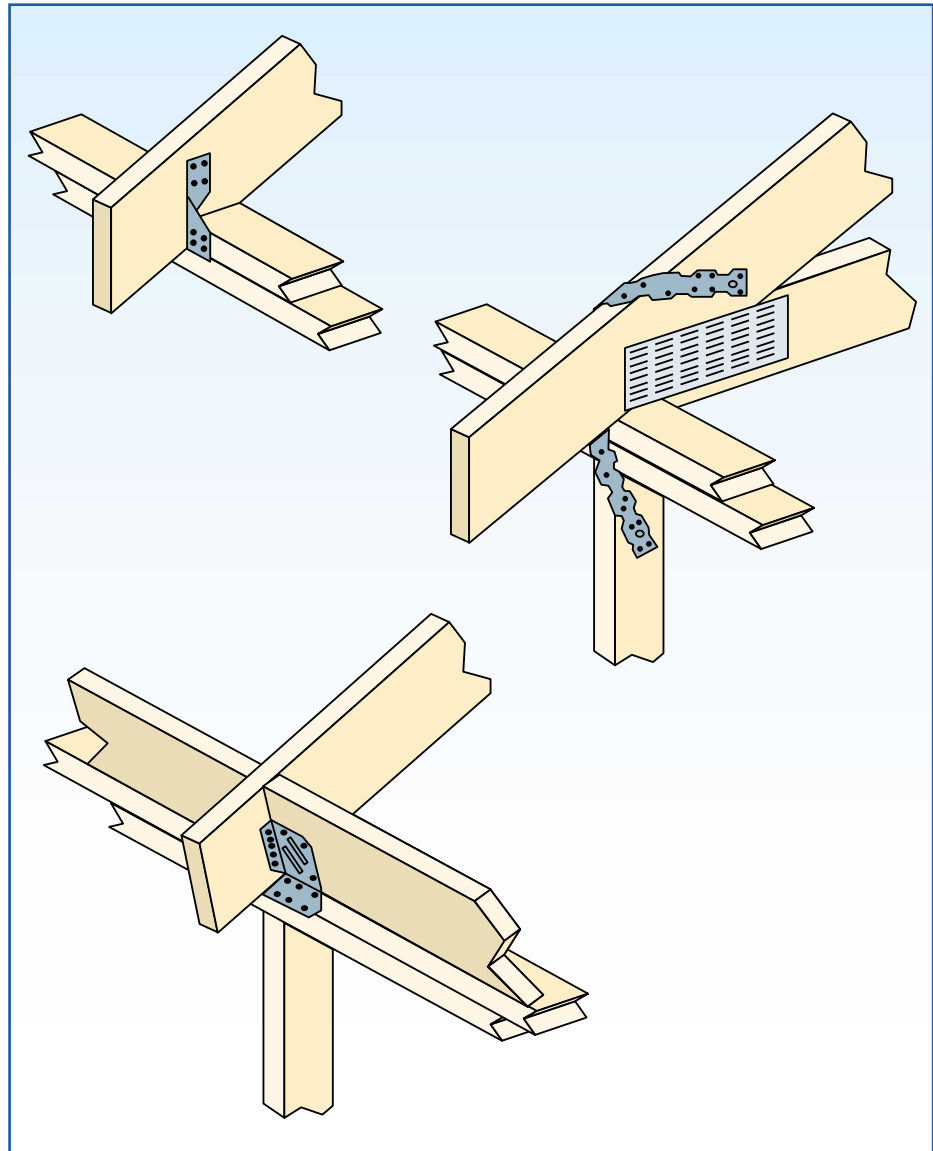
12.5.2 Link #1 – Roof Sheathing to Roof Framing

Link #1 connects roof sheathing to roof framing. This connection can be made with nails or screws.

12.5.3 Link #2 – Roof Framing to Exterior Walls

Link #2 is the connection between the roof rafter or truss and the top plate of the exterior wall. This connection is usually made with a metal strap designed to resist uplift and lateral loads (see Figure 12-74). The connector must be sized so that loads in all three planes are adequately resisted.

Figure 12-74
Metal connectors.



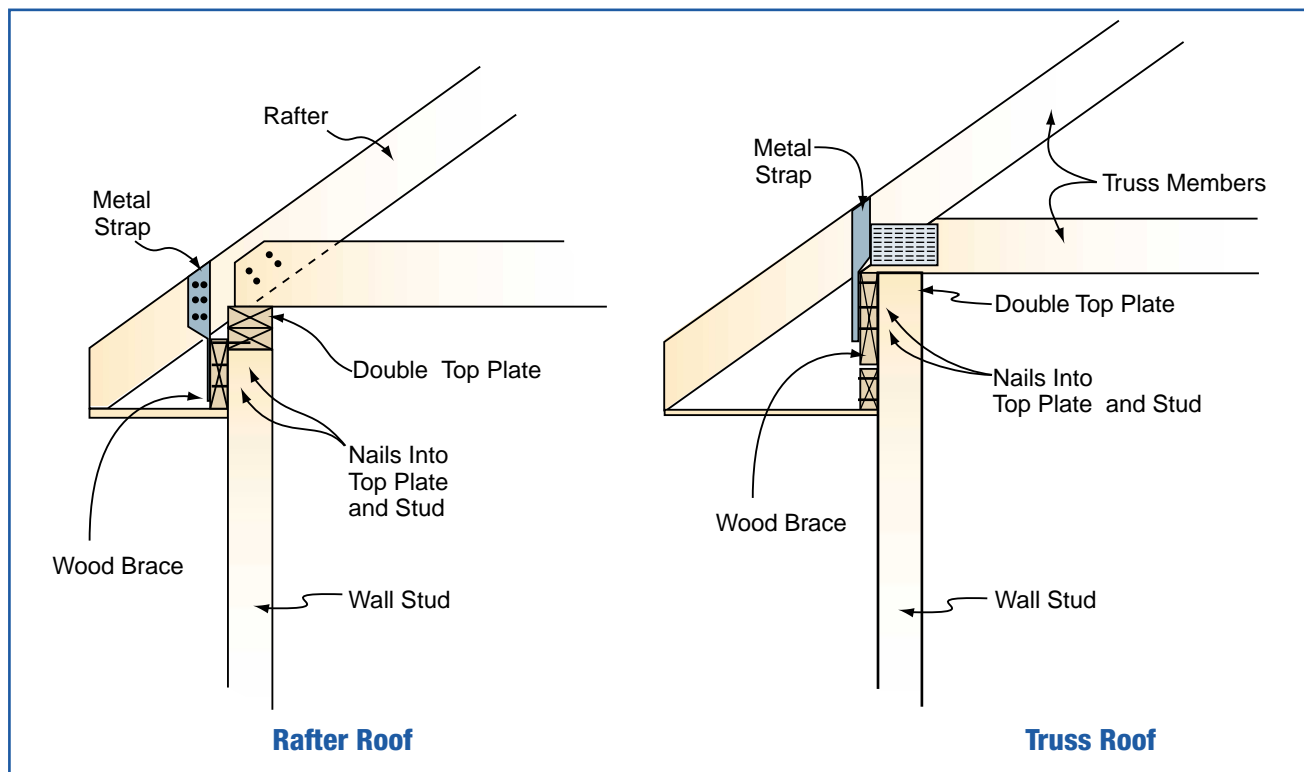
The loads at Link #2 (from Section 12.4.2) are as follows:

- uplift = 383 lb (from Table 12.7)
- lateral force in plane of the wall = 286 lb (from page 12-41)
- normal force = 457 lb (from page 12-41)

The designer must specify connectors and their locations and capacities in the plans. A number of manufacturers supply mechanical connectors for use in residential construction.

In a retrofit situation, it is possible to nail a connecting board to both the roof member and the top plate of the wall. Figure 12-75 illustrates this connection with wood. The installation must be done carefully so that no wood splits during the installation of the nails. Wood splitting will significantly reduce the capacity of the member in any of the primary directions. Each of the primary loads can be resisted with 16d nails that each resist 224 lb in single shear [from 1997 NDS where shear value is $(122)(1.6)(1.15)$] and 85 lb in withdrawal. For this connection, the uplift can be resisted by two 16d nails and by two to four 16d nails in lateral and normal directions.

Figure 12-75 Board connected to both the roof member and the top plate of the wall.



12.5.4 Link #3 – Top Wall Plate to Wall Studs

This link is the connection between the top wall plate and the wall studs. Because roof framing members are normally spaced 24 inches o.c. and wall studs are spaced 16 inches o.c., the same connector usually cannot be used to link the roof framing to the wall stud. Where the spacings are the same, a connector that accomplishes a link at both locations may be preferable. This link can normally be accomplished with wall sheathing.

Figure 12-76 shows a typical metal connector that can be used when such a connector is required. Calculation 12.10 indicated the connection with wall sheathing could be completed with five 8d box nails over a 24-inch truss spacing.

The loads at Link #3 (same as Link #2) are:

- uplift = 383 lb
- lateral force in plane of the wall = 176 lb
- normal force = 232 lb

Figure 12-76
Top-plate-to-wall-stud metal connector.

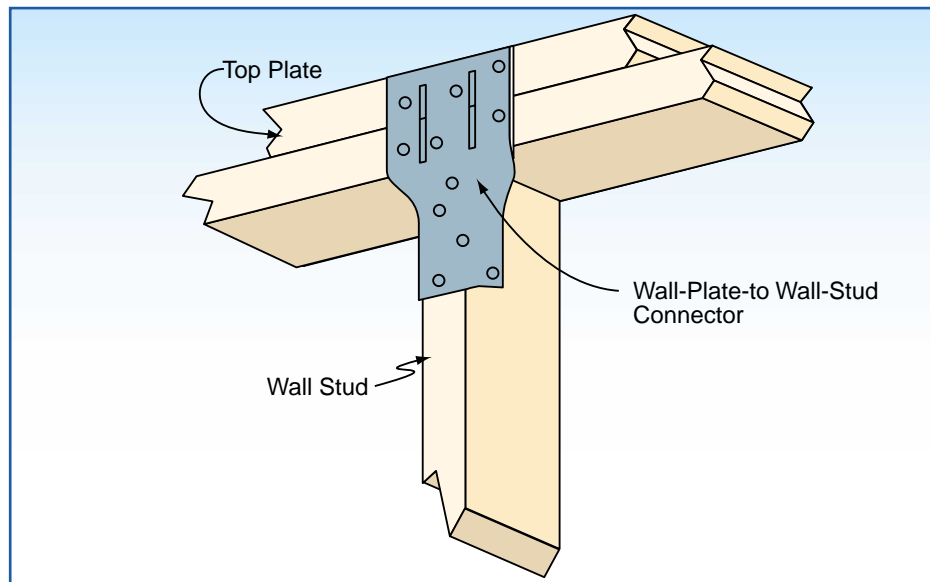


Table 12.14 lists the capacities for 8d box nails for Link #3.

Table 12.14
Nail Capacities for Link # 3

Product	Uplift (lb)	Lateral Force (lb)	Normal Force (lb)
Required load	383	176	232
8d box nails	392	342	305

12.5.5 Link #4 – Wall Stud to Window Header

The connection method and the loads required at this connection are the same as those used at Link #3.

12.5.6 Link #5 – Window Header to Exterior Wall

This link is the connection between the end of the window header and the wall studs. This connector will normally have to be placed flat against the framing, so it will be in the shape of a strap. The connector must be sized so that loads in all three planes are adequately resisted.

The loads at Link # 5 (from Section 12.4.2) are:

- uplift = 731 lb (from page 12-45)
- lateral force in plane of the wall is distributed into shearwall
- normal force = 449 lb (from Calculation 12.11) (Use end nails through stud into header.)

12.5.7 Link #6 – Wall to Floor Framing

This link is the connection between the bottom of the wall framing and first floor. The connector used here is also the shearwall holddown connector. It is bolted to a stud (or more than one stud) and through the floor framing into the beam (as in the case study house) or, in a multi-story house, to the wall below. This connector must be sized to resist tension. Figure 12-77 shows a shearwall holddown connector. The uplift load at this holddown is 4,064 lb (from Calculation 12.15).

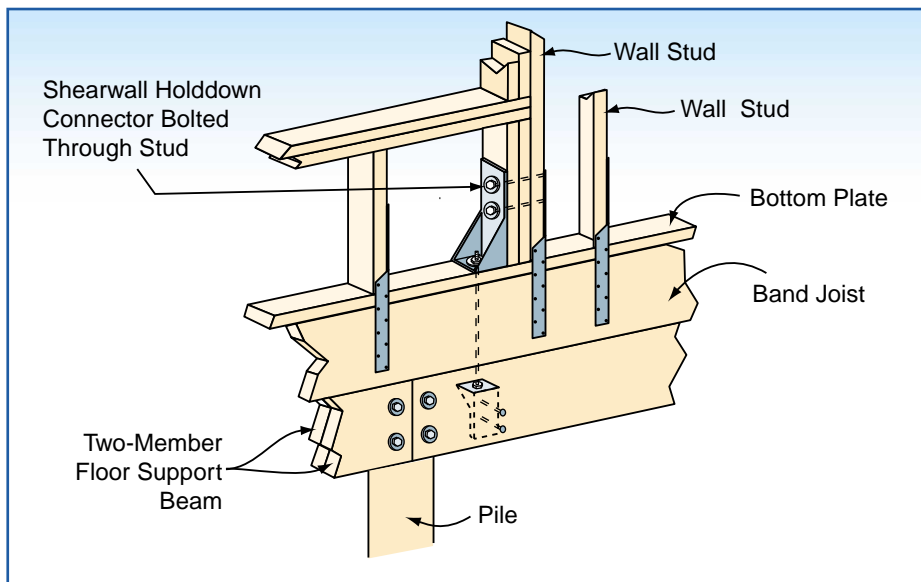


Figure 12-77
Shearwall holddown
connector.

12.5.8 Link #7 – Floor Framing to Support Beam

This link is the connection between the floor framing and the floor support beam and does not include the shearwall overturning uplift. This connection can be made with either a metal connector (see Figure 12-51) or a block of wood installed between the floor support beam and nailed to the floor joists (see Figure 12-52). This connector must be sized so that loads in all three planes are adequately resisted.

The loads at Link #7 (from Section 12.4.2) are:

- uplift – 4,064 lb
- lateral force in plane of the wall – 443 lb (from page 12-53)
- normal force – 168 lb (from page 12-53)



WARNING

Overnotching of pilings during construction is a common problem. Designers should specify maximum pile notching depths.

12.5.9 Link #8 – Floor Support Beam to Pile Foundation

The connection between the beam and the top of the pile has already been discussed. This connection is normally completed with hex head bolts (bolts installed through the beam and top of the pile). When this connection is made by notching the pile to provide a “seat” for the beam, the pile is often overnotched. The top of the pile at the cut is a failure plane in shear.

For the case study example problem, the lateral load at the pile top is 989 lb. When the top of the pile is held rigidly by bracing, the load could shear off the top of the pile. Using guidelines from the NTPC (1995), the allowable shear stress is

$$(F_v)(C_d)(C_{pt}) = (110 \text{ lb/in}^2)(1.6)(0.90)$$

= 158.4 lb/in² (and as noted on page 12-66, the maximum shear stress is higher than this average shear stress)

[12.23]

The minimum area of the pile required to resist shear failure is 989 lb(1.5 factor for rectangular shape)/(158.4) lb/in². The minimum area = 9.4 in².

The area of a 9-inch-diameter pile is 63.62 in², so the minimum area required is **15 percent** of the pile area. Overnotching to provide a beam “seat” does increase the risk of shear failure in the pile. Figure 12-78 illustrates failure of a pile that has been overnotched. Reinforcing the pile at the overnotched section with steel plates on both sides of the pile is an acceptable method of increasing the shear capacity at this critical point. This detail must, however, be carefully designed.

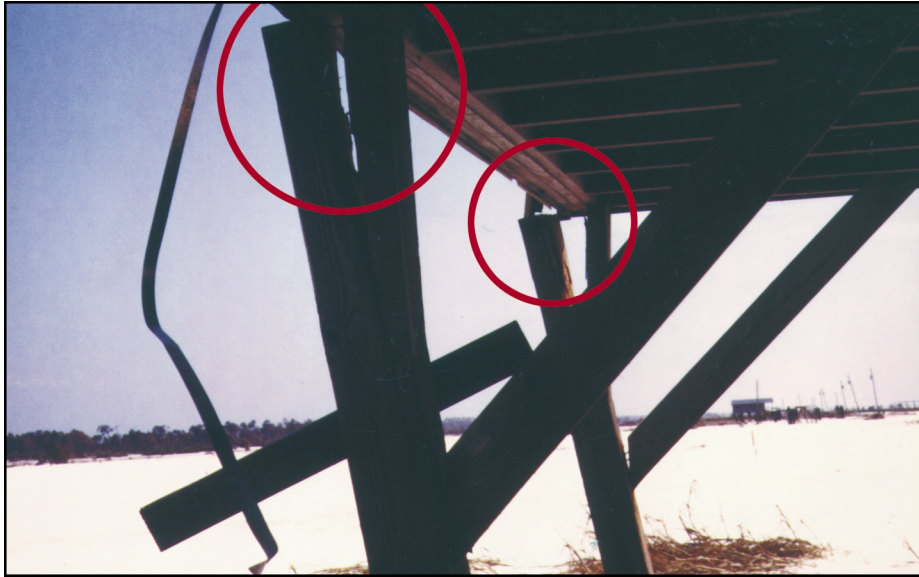


Figure 12-78
Failure of overnotched pile.

12.6 Step 5 – Select Building Materials

12.6.1 Introduction

The designer and builder of coastal structures must choose the materials to use for all parts of the building, including the foundation, structural frame, exterior envelope, and interior finishes. The foundation and structural frame are usually wood, concrete, steel, or masonry. The exterior envelope and interior finishes can be of these materials or of a wide selection of other materials.

The choice of materials will be influenced by many considerations, including whether they will be used above or below the DFE. Below the DFE, the risk of inundation by seawater must be anticipated. Significant forces due to wave action, water velocity, and waterborne debris impact must also be considered. Materials intermittently wetted by flood water below the BFE are subject to corrosion and decay.

Above the DFE, building materials still face significant environmental effects. The average wind velocity increases with height above ground. Wind-driven salt water spray can cause corrosion and moisture intrusion. The evaporation of salt water leaves crystalline salt that retains water and is corrosive.

Each of the commonly used materials (wood, concrete, steel, and masonry) has characteristics that can be advantageous or that can require special consideration when the materials are used in the coastal environment (see Table 12.15). Usually, a coastal residential structure will have a combination of these materials, with each used to advantage in a specific application.

Table 12.15
General Guidance for
Selection of Materials –
Advantages and Special
Considerations

Material	Advantages	Special Considerations
Wood	<ul style="list-style-type: none"> • Generally available and commonly used • With proper design, can generally be used in most structural applications • Variety of products available • Can be treated to resist decay 	<ul style="list-style-type: none"> • Easily over-cut, over-notched, and over-nailed • Many designs done by "rule-of-thumb" • Requires special treatment and continued maintenance to resist decay and damage from termites and marine borers • Requires protection to resist weathering • Subject to warping and deterioration
Steel	<ul style="list-style-type: none"> • Used for larger forces than wood can resist • Can span long distances • Can be coated to resist corrosion 	<ul style="list-style-type: none"> • Not corrosion-resistant • Heavy and not easily handled and fabricated by carpenters • May require special connections such as welding
Reinforced Concrete	<ul style="list-style-type: none"> • Resistant to corrosion if reinforcing is properly protected • Good material for compressive loads • Can be formed into a variety of shapes • Pre-stressed members have high load capacity 	<ul style="list-style-type: none"> • Salt water infiltration into concrete cracks will cause reinforcing steel corrosion • Pre-stressed members require special handling • Water intrusion and freeze-thaw cause deterioration and spalling
Masonry	<ul style="list-style-type: none"> • Resistant to corrosion if reinforcing is properly protected • Good material for compressive loads • Material commonly used in residential construction 	<ul style="list-style-type: none"> • Not good for beams and girders • Water infiltration into cracks will cause reinforcing steel corrosion • Requires reinforcement to resist loads in coastal areas

12.6.2 Selection of Materials for Foundations and Enclosures Below the DFE

The use of flood-resistant materials below the BFE is covered in FEMA NFIP Technical Bulletin 2 (see Appendix H). The introduction in this bulletin indicates that “All construction below the lowest floor is susceptible to flooding and must consist of flood-resistant materials. Uses of enclosed areas below the lowest floor in a residential building are limited to parking, access, and limited storage—areas that can withstand inundation by floodwater without sustaining significant structural damage.” The IBC 2000 and IRC 2000 require that all new construction and substantial improvements in the Special Flood Hazard Area be constructed with materials resistant to flood damage.

Compliance with these requirements in coastal areas means that the only building elements that will be below the BFE include the following:

- foundations – either treated wood, concrete or steel piles, concrete or masonry piers, or concrete, masonry, or treated wood walls
- breakaway walls
- enclosures used for parking, building access, or storage below elevated buildings
- garages in enclosures under elevated buildings or attached to buildings

Material choices for these elements are limited to the following:

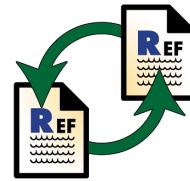
- pressure-treated lumber, 0.40 CCA or equivalent (see Appendix J and references listed at the end of this chapter for additional information on wood preservatives)
- naturally decay-resistant lumber
- concrete – 5,000 lb/in² minimum compressive strength recommended in coastal environments, with a 0.40 water-cement ratio
- masonry – reinforced and fully grouted in coastal environments
- steel – must resist corrosion
- closed-cell foam insulation
- other flood-resistant materials approved by local building officials

Each of these materials has characteristics that can be advantageous or that can require special consideration when the materials are used for various building elements. Some of these are presented in Table 12.16. Additional information about material selection for various locations in the building and for various uses is included in Appendix J.



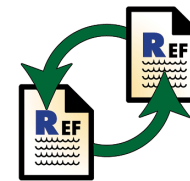
NOTE

Although the NFIP regulations, IBC 2000 (ICC 2000a), and IRC 2000 (ICC 2000b) specify that flood-resistant materials be used below the BFE, this manual recommends that flood-resistant materials be used below the DFE.



CROSS-REFERENCE

NFIP compliance provisions, as described in the IBC 2000 and the IRC 2000, are discussed in Chapter 6 of this manual.



CROSS-REFERENCE

Table 9.6 in Chapter 9 lists sample flood insurance premiums for buildings in which the lowest floor is below the BFE and in which there is an enclosure below the BFE.

Table 12.16

Selection of Materials for Use
in Foundation Elements
Below the DFE – Advantages
and Special Considerations

	Material/Use	Advantages	Special Considerations
Deep Foundations	Tapered Round Wood Piles	<ul style="list-style-type: none"> • Normally longer than square piles • Shape is advantageous for pressure treatment to resist decay • Cross-sectional area and stiffness greater than square piles 	<ul style="list-style-type: none"> • May not be straight, thus driving accuracy is affected • May be more costly than square wood piles
	Square Wood Piles	<ul style="list-style-type: none"> • May be more economical than round piles in some areas • May be more available than round piles in some areas • Can be pressure-treated to resist decay • May be easier to frame to flat surfaces 	<ul style="list-style-type: none"> • Knots can be located at pile edge and weaken pile (see Figure 12-79) • Pressure-treated square wood piles can twist • Smaller cross-section make these piles weaker and less stiff than round piles
	Pre-Stressed Concrete Piles	<ul style="list-style-type: none"> • Constructed straight and true to any specified length and strength • Can support large vertical loads • Material is decay-resistant 	<ul style="list-style-type: none"> • Reinforcing steel must be protected to prevent corrosion • More expensive than wood piles • Special handling equipment may be required because of weight
	Steel Piles	<ul style="list-style-type: none"> • Delivered straight and true to any specified length • Can support large vertical and lateral loads • Material is easily cleaned after flooding 	<ul style="list-style-type: none"> • Special handling equipment may be required because of weight • Must be coated and maintained to be corrosion-resistant • More expensive than wood piles
Shallow Foundations	Reinforced Concrete Piers or Walls on Footings		<ul style="list-style-type: none"> • Reinforcing steel must be protected to prevent corrosion • Requires extensive excavation in order to be used as deep foundation
	Masonry Piers or Walls on Footings	<ul style="list-style-type: none"> • Easy to construct 	<ul style="list-style-type: none"> • Reinforcing steel must be protected to prevent corrosion • Requires extensive excavation in order to be used as deep foundation • Performance very sensitive to quality of construction



Figure 12-79
Hurricane Fran (1996), North
Carolina. Square wood pile
failure at edge knot.

12.6.3 Selection of Materials for Use Above the DFE

The selection of materials for use above the DFE is usually a function of one or more of the following:

- exposure – outside or inside the building envelope
- use – structural support or finish material
- structural requirement – span, deflection, load
- availability – cost and delivery

Long-term durability as well as architectural and structural considerations will normally be the most important determiners of material selections. Material in the coastal environment is very susceptible to weathering, corrosion, termite damage, and decay from water infiltration, in addition to the stresses induced by loads from natural hazard events. These influences must be considered in the selection of the appropriate materials. Appendix J contains additional information about a variety of wood products and considerations important in their selection and use.

12.6.4 Material Combinations

Materials are frequently combined in the construction of a single residence. The most common combinations are as follows:

- masonry or concrete lower structure with wood on upper level
- wood piles supporting concrete pile caps and columns that support a wood superstructure
- steel framing with wood sheathing

For the designer of coastal buildings, the important design considerations when combining materials include:

1. There must be **material compatibility** so that corrosion is not caused by the contact of dissimilar metals in the presence of salt and moisture. Appendix J addresses a possible problem with galvanized fasteners and hardware in contact with certain types of treated wood.
2. **Connecting the materials together** is crucial. The connector must be properly embedded (if into concrete or masonry) and placed so that alignment and vertical or horizontal load path continuity are maintained. Altering a connector location after it has been cast into concrete or grout is a difficult and expensive task.
3. Material combinations used for the same building add the **complexity of additional skills** being required to construct the project. This is normally of more concern to the builder than to the designer, but may impact decisions regarding which materials will be acceptable. Figure 12-80 shows a coastal house being constructed with ordinary wood piles that support a welded steel frame that will be used as the floor frame support beams.
4. Material properties such as stiffness of one material relative to another will affect movement or deflection of one material relative to the other. This **difference in material behavior** can affect the resultant damage to the building. For example, movement that occurs in a wood-frame building during high wind can fail masonry piers because masonry is less flexible than wood and will fail with small deflection.

Figure 12-80
House being constructed
with a steel frame on
wooden piles.



12.6.5 Fire Safety Considerations

Construction of multi-family coastal dwellings that must withstand natural hazards and meet the building code requirements for adequate fire separation presents some challenges. In multi-family buildings, the IBC 2000 and IRC 2000 require that the common walls between living units be constructed of materials that provide a minimum of 2-hour fire resistance. The code requires that the units be constructed such that if a fire were to occur in one unit, the structural frame of that unit would collapse within itself and not affect either the structure or the fire resistance of adjacent units.

For townhouse-like units, the common framing method is to use the front and rear walls for the exterior load-bearing walls so that firewalls can be placed between the units. If these walls are also used as the support for the floor framing, the floor support beams must be supported by the side walls (or side wall support structure such as piles). This beam orientation may make the structure more vulnerable to flood damage. Figure 12-81 illustrates the framing system for a series of townhouses and the potential difficulties in framing these units to minimize flood and wind damage.

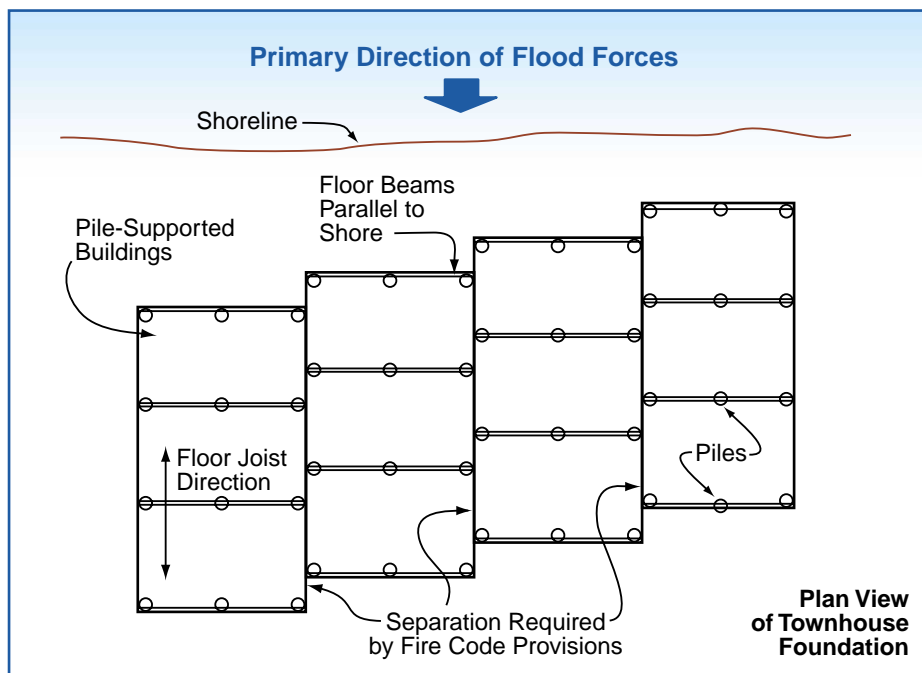


Figure 12-81
Townhouse framing system.

**NOTE**

Consult the local building code or building official to determine how to resolve the conflict between fire-resistant construction and flood-resistant construction.

The figure suggests several design difficulties that must be overcome. These difficulties include, but are not limited to the following:

1. The floor support beams are parallel to the shore and perpendicular to the expected flow, and may therefore create an obstruction during a greater than design flood event.
2. The fire separation between the units makes structural rigidity difficult to achieve because the interior walls between townhouse units cannot be attached to the framing system and used for support. Therefore, the transfer of lateral loads to the foundation, particularly for multi-directional loads from wind or earthquake forces, is difficult to achieve.
3. The exposed undersides of buildings elevated on an open foundation (e.g., pile, pier, post, or column) must be protected from fire with a 2-hour rated material. Where sheathing is desired on the underside of the floor framing—for example, in a below-building enclosure used for parking, storage, or building access—the use of fire-resistant gypsum board will comply with the requirement for fire protection; however, gypsum board is not a flood-resistant material. A better approach would be to use cement-fiber board, which has a greater resistance to damage from flood waters. If the building design requires fire separation and the fire-protection material is not flood-resistant (e.g., gypsum board), larger wood framing members or other techniques may be required in order to meet the competing demands of flood- and fire-resistance.
4. The requirement for separation of the foundation elements between townhouse units makes structural rigidity in the direction parallel to the shore difficult to achieve. If the houses in Figure 12-81 were located in a seismic hazard area, the designer might want to place a shearwall or diagonal bracing in the direction parallel to the shore (i.e., perpendicular to the primary flood flow direction). Shearwall segments or diagonal bracing will provide rigidity, but when they are constructed below the first-floor framing (and thus below the DFE) and perpendicular to the expected flood flow, they will create an obstruction below the DFE. The designer should consult FEMA NFIP Technical Bulletin 5, *Free of Obstruction Requirements for Buildings Located in Coastal High Hazard Areas* (see Appendix H) for information about the types of construction that constitute an obstruction.

Designers should consult local building officials for guidance about how to create a design compatible with both fire and natural hazard design requirements. Understanding local requirements will influence how the designer resolves potential conflicts between various code requirements.

12.6.6 Corrosion

Modern construction techniques often rely heavily on metal fasteners and connectors to resist the forces of various coastal hazards. To be successful, these products must have lifetimes comparable to those of the other materials used for construction. The metal materials in common use have proved to have an adequate lifetime in most inland applications. However, for some uses near saltwater coastlines, corrosion has been found to drastically shorten the lifetime of standard fasteners and connectors. Corrosion is one of the most underestimated hazards affecting the overall strength and lifetime of coastal buildings. To be successful, hazard-resistant buildings must match the corrosion exposure of each component with the proper corrosion-resistant material. Although standard materials may not have sufficient corrosion resistance for some uses in coastal buildings, a variety of corrosion-resistant materials and techniques are readily available, often at a small increase in cost.

FEMA NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas*, represents the current state of knowledge and research concerning this subject. This bulletin is included in Appendix H.

12.6.7 Additional Environmental Considerations

In addition to water intrusion and possible resulting decay, several other environmental factors must be considered in the selection of materials to be used in coastal buildings. The coastal environment is extremely harsh, and materials should be selected that will not only provide protection from the harsh elements, but also require minimal maintenance. The following environmental factors will be discussed:

- sun (heat and ultraviolet [UV] radiation)
- wind-driven rain

12.6.7.1 Sun

Buildings at or near the coast are typically exposed to extremes of sun, which produces high heat and UV radiation. This exposure creates the following effects:

- The sun will bleach out many colors.
- The heat will build up in enclosed spaces like attics so the design must consider ways to reduce the heat buildup.
- The heat and UV will shorten the life of many organic materials such as asphalt roof shingles.
- The heat and UV affect the life of sealants, protective materials placed on siding, and exterior wood used for decks, walkways, and other external components.



NOTE

See FEMA NFIP Technical Bulletin 8 (Appendix H) for additional information about corrosion of metal connectors in coastal construction.

- The heat dries out oils and lubricants such as those contained in door and window operating mechanisms.

To overcome these problems:

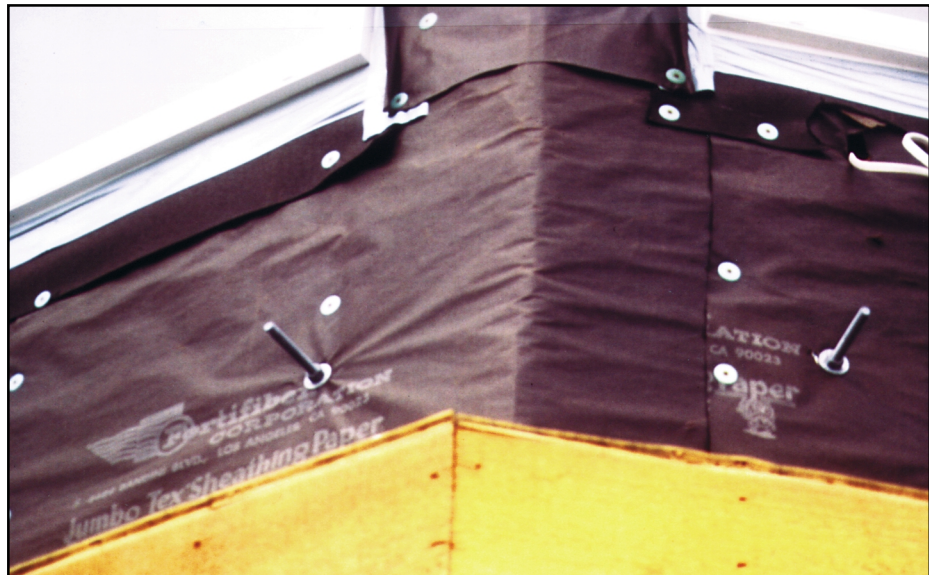
- use materials that are heat/UV-tolerant,
- shield heat/UV susceptible materials with other materials, and
- perform periodic maintenance and repair.

12.6.7.2 Wind-Driven Rain

Wind-driven rain is primarily a problem for elements of the building envelope, particularly elements that operate such as windows and doors. High winds can carry water droplets into the smallest of openings, even up, into, and behind flashings, vents, and drip edges. When houses are built to provide what is considered to be complete protection from the effects of natural hazards, any small “hole” in the building envelope becomes an area of weakness into which sufficiently high wind can drive a large amount of rain.

The designer must address any area of potential weakness in the building envelope with the proper technique—flashings, sealants, product design of windows and doors, attachment of elements that penetrate the building envelope (e.g., decks, porches, window boxes)—as further discussed in Section 12.7. Figure 12-82 illustrates a design detail for the attachment of a deck that minimizes penetrations of bolts through the building envelope.

Figure 12-82
Deck attachment scheme
that minimizes penetration of
the building envelope.



12.7 Design of the Building Envelope

The building envelope comprises the roof covering, exterior wall covering, and exterior doors and windows. For buildings elevated on open foundations, the floor is also considered a part of the envelope .

To avoid injury and minimize damage to a building and its contents, good structural system performance is critical; however, it does not ensure occupant or building protection. Good performance of the building envelope and exterior-mounted mechanical and electrical equipment is also necessary. Satisfactory building envelope performance is critical for buildings exposed to high-wind and wildfire hazards.

Satisfactory performance of the envelope depends on suitable design, materials, application, maintenance, and repair. Of these, design is the key element. Although design inadequacies frequently cannot be compensated for by the other four elements, good design (to some extent) can compensate for other inadequacies.

Breaching of the building envelope and subsequent water infiltration has historically been the predominant cause of damage to buildings and their contents during high-wind events. Breaching includes catastrophic failure (e.g., loss of the roof covering or windows) as well as water infiltration through small openings at walls, windows, doors, or the roof. Water infiltration also occurs during high winds because wind-driven rain will penetrate into even the smallest opening. Preventing this type of damage requires the application of a high-quality sealant around components such as windows and doors. Other openings that must be properly sealed include mechanically attached roof or wall penetrations such as exhaust fans and vents.

Building envelope components have also been the predominant source of windborne missiles generated from damaged buildings. Close design attention should be given to buildings in special wind regions (see Figure 11-18, in Chapter 11) or in areas where the basic wind speed is greater than 90 mph (3-sec peak gust).

Building integrity in earthquakes also is dependent on the performance of the building envelope, particularly the integrity of roof-to-wall connections and diaphragm-to-wall connections and detailing. Residential buildings have historically performed very well when the structural integrity of these components of the building envelope has been maintained.

In addition, poor resistance of building envelopes to wildfires has historically resulted in building losses as illustrated in Figure 12-83.



NOTE

Special attention to the design of the building envelope must be given to buildings located in areas subject to wind speeds greater than 90 mph (3-sec peak gust).

Basic design issues and general good practices that are applicable to all residential design are not addressed in this section. Rather, this section builds on the basics by addressing the special design considerations related to buildings susceptible to natural hazards. It provides recommendations regarding doors, floors, roofs, walls, and windows. Flooding influence on the building envelope is not addressed, because it is assumed that the envelope will not be inundated by water; however, envelope resistance to wind-driven rain is addressed. The recommended measures taken to protect against wind-driven rain should also be adequate to protect against wave spray.

Figure 12-83

The fire-resistant walls and roof of this house helped it survive a wildfire while surrounding houses were destroyed. Photograph courtesy of Decra Roofing Systems.



NOTE

For A-zone buildings elevated on solid walls with openings, a number of manufacturers produce vents that reduce heat loss in cold-weather environments but still allow the entry and exit of flood waters.

12.7.1 Floors for Elevated Buildings

12.7.1.1 Corrosion and Wave Spray

For buildings near the ocean, sheathing the underside of the bottom-floor joists or trusses helps minimize corrosion of framing connectors and fasteners. The sheathing also protects insulation installed between the joists/trusses from wave spray. (If fiberglass insulation is installed, the paper or foil face should be installed adjacent to the underside of the floor decking, or the insulation should be unfaced so that downward water vapor migration is not impeded.) For long-term durability, exterior grade sheathing is recommended for the exposed sheathing and it should be fastened with stainless steel or hot-dip galvanized nails or screws.

12.7.1.2 High Winds

For buildings in high-wind areas, if sheathing is applied to the underside of joists or trusses, its attachment should be specified in order to avoid blowoff.

12.7.1.3 Wildfires

For buildings in areas prone to wildfires, sheathing the underside of joists or trusses with a fire-resistant material such as cement-fiber panels is

recommended. Cement-fiber panels should be attached with stainless steel or hot-dip galvanized screws.

12.7.2 Exterior Walls and Soffits

High winds and wildfires are the natural hazards that can cause the greatest damage to exterior wall systems. Seismic events can also damage heavy wall systems or coverings. Although hail can damage walls, significant damage is not common.

A variety of systems can be used for wall construction. The following wall coverings are commonly used over wood-frame construction: aluminum siding, cement-fiber panels or siding, exterior insulating finishing system (EIFS), stucco, vinyl siding, sawn wood siding boards, and wood panel siding. Concrete or masonry wall construction may also be used. These systems are discussed in the following sections.

12.7.2.1 High Winds

The discussion of air-permeable roof coverings in Section 12.7.5 is also applicable to air-permeable wall coverings such as siding. Research on special pressure coefficients has not been conducted for air-permeable wall claddings. Therefore, according to ASCE 7, these claddings have to be designed for the full wind load. Wind-load resistance of non-load-bearing walls, wall coverings, and soffits should be based on testing in accordance with ASTM E 1233. Walls and soffits must be designed to resist positive and negative wind pressure.

Siding, panels (e.g., textured plywood), and stucco over masonry and concrete typically perform well during high winds. More blowoff problems have been experienced with vinyl siding than with other siding or panel materials. Aluminum and cement-fiber siding problems have also occurred. The key to the successful performance of siding and panel systems is attachment with a sufficient number of proper corrosion-resistant fasteners (based on design loads and tested resistance) that are correctly located. Blowoff of stucco applied directly to concrete walls (i.e., wire mesh was not applied over the concrete) has occurred. This problem can be avoided by leaving the concrete exposed or painted.

A secondary line of protection against wind-driven water infiltration (e.g., an air-barrier film) is recommended underneath wall coverings. Designers should specify that horizontal laps be installed so that water is allowed to drain from the wall (i.e., the top sheet should lap over the bottom sheet so that water running down the sheets remains on their outer face). The bottom of the secondary protection needs to be detailed to allow drainage.

EIFS can be applied over wood-frame, concrete, or masonry construction. The EIFS assembly is composed of several types of materials. Some of the layers are



NOTE

Throughout this manual, references to ASTM standards are based on the *Annual Book of ASTM Standards* (ASTM 1998).



WARNING

There have been documented problems with deterioration of wall sheathing in EIFS systems that are non-drainable. The use of these systems has been restricted in some jurisdictions. The designer should consult with local authorities to determine whether such restrictions exist, and what their effect may be on design and construction.

adhered to one another, and one or more of the layers is typically mechanically attached to the wall. If mechanical fasteners are used, they need to be correctly located, of the proper type, and of sufficient number (based on design loads and tested resistance). Failures have been observed where the rated wind load capacity of the fasteners was not sufficient for a design event. Proper application of the components that are adhered together is also necessary to avoid blowoff. It is strongly recommended that if EIFS is used, it be designed with a drainage system that allows for dissipation of water leaks.

Punctures of EIFS by windborne debris are also common in high-wind events. EIFS must be installed on a solid substrate such as plywood in order to provide enhanced resistance to missile penetration and thus occupant protection. A minimum plywood thickness of 15/32-inch is recommended.

Concrete and masonry walls (or veneers) typically provide excellent windborne missile resistance provided they are adequately designed and constructed to resist the wind load.

Durability: To avoid corrosion problems, stainless steel or hot-dip galvanized fasteners (preferable heavy-duty hot dip galvanized) are recommended for buildings located within 3,000 feet of an ocean shoreline. If air can freely circulate in a cavity (e.g., above a soffit), access panels should be provided so components within the cavity can be periodically observed for corrosion.

In areas with severe termite problems, if wood is specified, it should be pressure-treated. See Appendix J for additional information.

12.7.2.2 Wildfires

For buildings in areas prone to wildfires, concrete, masonry, stucco, or cement-fiber panels or siding offer the greatest protection. If one of these wall surfaces is specified, a fire-resistive system should also be specified for soffits (e.g., stucco or cement-fiber).

Gable and soffit vents should have openings covered with wire mesh that has openings no greater than 1/4 inch, in order to inhibit the entry of burning brands. For added protection, noncombustible hinged shutters that can quickly be placed in the closed position could be designed and installed.

12.7.2.3 Seismic

Where required by code, concrete and masonry walls (or veneers) need to be designed for the seismic load. When a heavy covering such as stucco, cement-fiber panels or siding, or brick veneer, is specified, the seismic design should account for the added weight of the material, and its connection to the base material in the case of veneer. Inadequate connection of veneer material to the base substrate has been a problem in past earthquakes and can result in a life safety hazard.

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

12.7.2.4 Flashings

Poor performance of flashing and water intrusion protection is a common problem in many coastal homes. In areas that frequently experience strong winds, enhanced flashing details are recommended. Enhancements include use of flashings that have extra-long flanges, and use of sealant and tapes. General guidance is offered below, but it is recommended that designers also attempt to determine what type of flashing details have successfully been used in the area where the residence will be constructed.

Flashing design should recognize that wind-driven water can be pushed vertically. The height to which water can be pushed increases with wind speed. Water can also migrate vertically and horizontally by capillary action between layers of materials (e.g., between a flashing flange and housewrap).

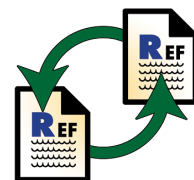
Conceptually, the exterior siding should not be thought of as the only barrier to water intrusion. The housewrap (if used), flashings, and underlayment must be used to shed and direct water away from openings in the building envelope. The overriding principle of successful water diversion is to install the layers of building materials correctly so that water can not get behind any one layer and into an opening.

- **Roof/wall flashing:** Where enhanced protection is desired, use step flashing that has a 2- to 4-inch-longer vertical leg than normal. Alternatively (or for a more conservative design, in addition to the long leg), tape the top of the vertical flashing to the wall sheathing with 4-inch-wide self-adhering modified bitumen roof tape (apply about 1 inch of tape on the metal flashing, 3 inches on the sheathing). Extend the housewrap over the flashing in the normal fashion. Do not seal the housewrap to the flashing—if water reaches the housewrap further up the wall, it needs to be able to drain out at the bottom of the wall. Figure 12-84 illustrates a good roof/wall flashing detail. This detail has been used successfully by a builder on the Delaware coast.
- **Window flashings:** For windows with nailing flanges, apply a generous bead of butyl sealant to the wall sheathing before setting the window. Place the sealant inward of the fasteners. At sheathing joints, place sealant over the joint, from the window opening out past the flange. Place the housewrap over the head trim flashing, and tape the flange to the housewrap with duct tape or modified bitumen roof tape as illustrated in Figure 12-85.



NOTE

Housewrap is a material used on the exterior skin of the house prior to siding installation that is primarily intended to reduce air infiltration into the building.



CROSS-REFERENCE

The designer should consult *Nail-On-Windows, Installation & Flashing Procedures for Windows & Sliding Glass Doors* (Bateman 1995) for additional information.

Figure 12-84 Roof/wall flashing detail. Courtesy of Journal of Light Construction.

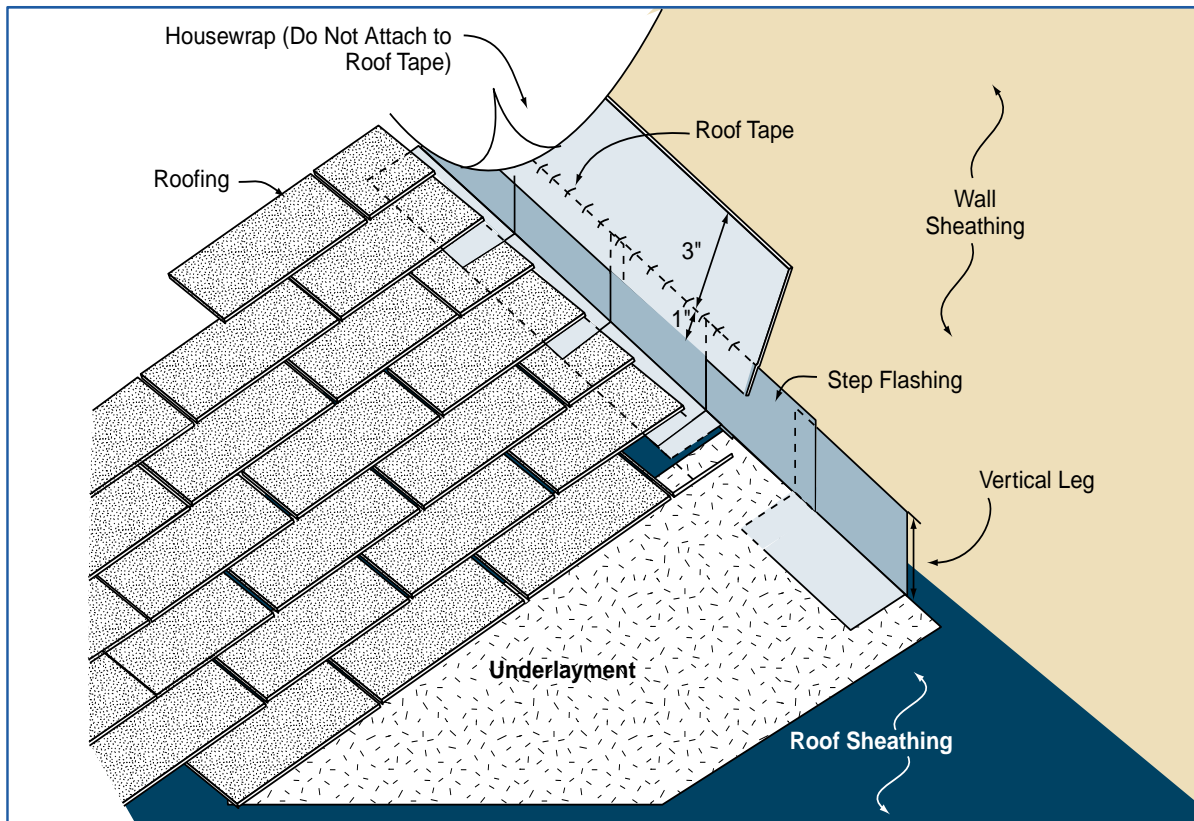
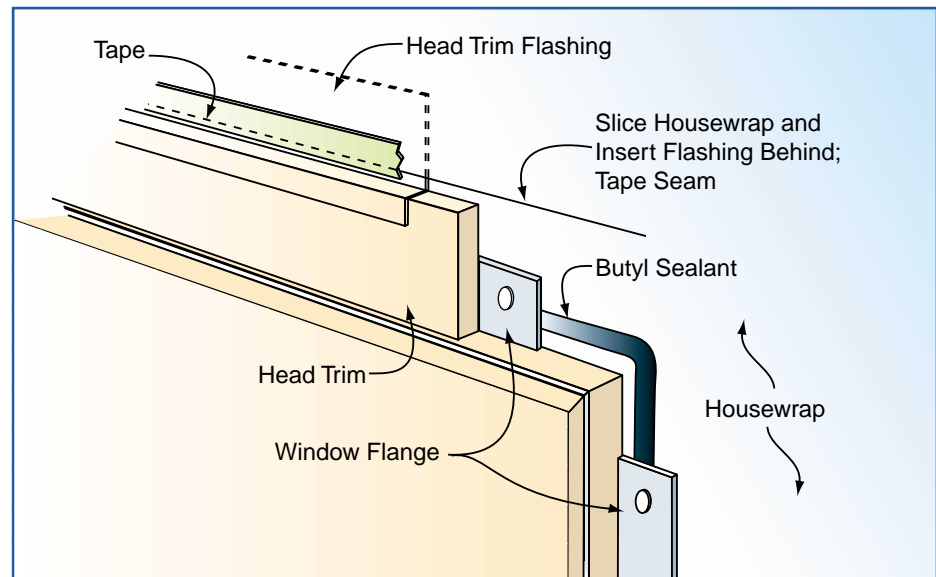


Figure 12-85
Window flashing detail.
Courtesy of Journal of Light
Construction.



12.7.3 Doors and Door Assemblies

High winds and wildfires are the natural hazards that present the greatest threat to exterior doors.

12.7.3.1 High Winds

Loads and resistance: The door assembly (i.e., door, hardware, frame, and frame attachment to the wall) should be of sufficient strength to resist positive and negative design wind pressures (see Section 11.8 and the Wind Load Example Problem on page 11-45). The assembly should be specified to comply with wind load testing in accordance with ASTM E 1233. (Note: ASTM E 330 is the most commonly used method to evaluate door assemblies; however, because E 1233 is a cyclic test method, whereas E 330 is a static test, E 1233 is the preferred test method, particularly in hurricane-prone areas.) It is important to specify frame attachment to the wall, either by performance or prescriptive criteria, as inadequate frame attachment due to lack of design guidance is a common problem.

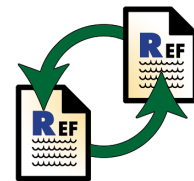
Except for glazing, doors typically do not need to be resistant to loads from hurricane-generated windborne missiles. If a door is hit with a missile, the missile may penetrate the door, but, in most cases, the missile opening will not be large enough to result in significant water infiltration problems or in a substantial increase in internal pressure. For information on glazing in doors, see Section 12.7.4.2. If enhanced missile resistance of solid doors is desired for occupant protection, specify door assemblies that have demonstrated compliance with testing discussed in Section 12.7.4.2.

Durability: To avoid corrosion problems with metal doors or frames, aluminum or galvanized steel units are recommended for buildings located within 3,000 feet of an ocean shoreline. Stainless steel frame anchors and hardware are also recommended. Fiberglass doors may also be used with wood frames. Galvanized steel doors and frames should be painted for additional protection.

In areas with severe termite problems, metal door assemblies are recommended. If concrete, masonry, or metal wall construction is used to eliminate termite problems, it is recommended that wood not be specified for blocking or nailers. If wood is specified, see Appendix J, for information on wood treatment methods.

Water infiltration: Hurricanes and coastal storms present enormous wind-driven water infiltration problems. Leakage can occur between the door and frame or between the frame and wall. Because of the extremely high design wind pressures and numerous opportunities for development of leakage paths, some leakage should be anticipated when design wind-speed conditions are approached. Examples of design responses to possible water infiltration include the following:

- Vestibule – designing a vestibule is one method of accounting for the infiltration problem. With this approach, both the inner and outer doors



CROSS-REFERENCE

Section 12.7.4.2 provides information about windborne missiles and glazing in doors.

can be equipped with weatherstripping, and the vestibule itself can be designed to tolerate water. For example, water-resistant finishes (e.g., concrete or tile) can be specified, and the floor can be equipped with a drain.

- Door swing – if weatherstripping is specified, out-swinging doors offer an advantage over in-swinging doors. With out-swinging doors, the weatherstripping is located on the interior side of the door, where it is less susceptible to degradation. Also, some interlocking weatherstripping assemblies are available for out-swinging conditions. However, there is a security disadvantage to out-swinging doors; without door stops, an out-swinging door is much easier to break into.
- Pan flashing – adding flashing under the door threshold will help prevent penetration of water into the subflooring. This is a common place for water entry and subsequent wood dry rot in coastal homes.

A variety of pre-manufactured weatherstripping components are available, including drips, door shoes and bottoms, thresholds, and jamb/head weatherstripping. A few examples of weatherstripping options are presented below:

- Drips are intended to divert water away from the opening between the frame and door head, and the opening between the door bottom and the threshold as shown in Figures 12-86 and 12-87. Alternatively, a door sweep can be specified (Figure 12-88); however, for high-traffic doors, periodic replacement of the neoprene will be necessary.
- Door shoes and bottoms are intended to minimize the gap between the door and the threshold. Figure 12-87 illustrates a door shoe that incorporates a drip. Figure 12-89 illustrates an automatic door bottom. Door bottoms can be surface-mounted or mortised. For high-traffic doors, periodic replacement of the neoprene/vinyl will be necessary.
- Thresholds are available in a variety of configurations. Thresholds with high vertical offsets offer enhanced resistance to water; however, where handicap access thresholds are required, the offset is limited. Thresholds that can be interlocked with the door, or thresholds with a stop and seal, are recommended. Examples are shown in Figure 12-90. Designers should specify setting the threshold in sealant to avoid water infiltration between the threshold and floor. Butyl sealant is recommended. If the threshold has a drain pan (Figure 12-90), designers should specify that the weep holes not be obstructed during installation.
- Adjustable jamb/head weatherstripping is recommended. These units offer good door contact because they have wide sponge neoprene and they can be adjusted to fit the door (Figure 12-91).

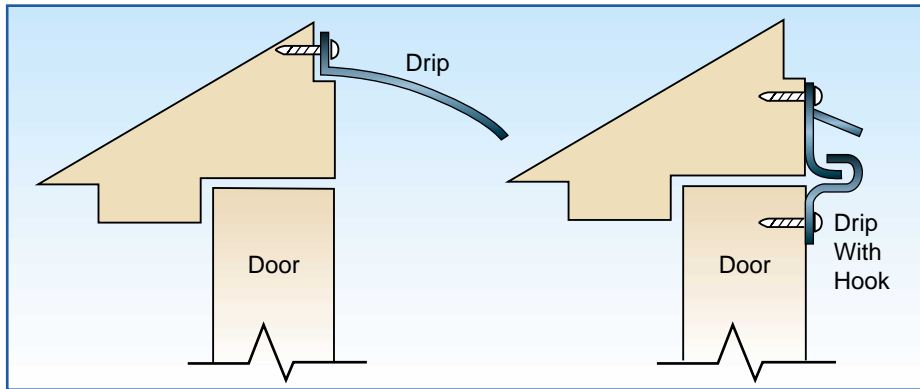


Figure 12-86
Drip at door head and drip with hook at head.

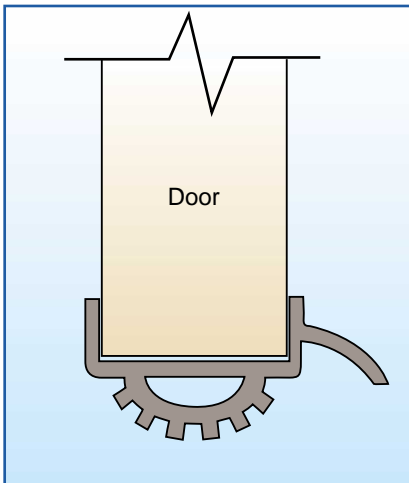


Figure 12-87
Door shoe with drip and vinyl seal.

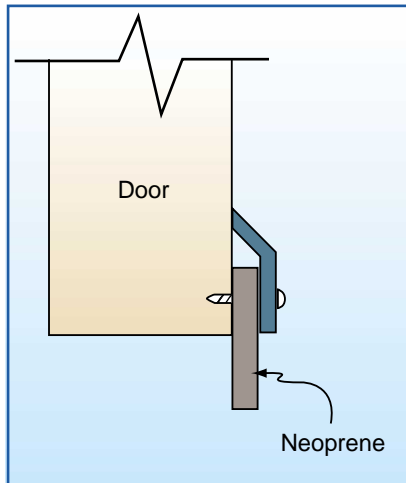


Figure 12-88
Neoprene door bottom sweep.

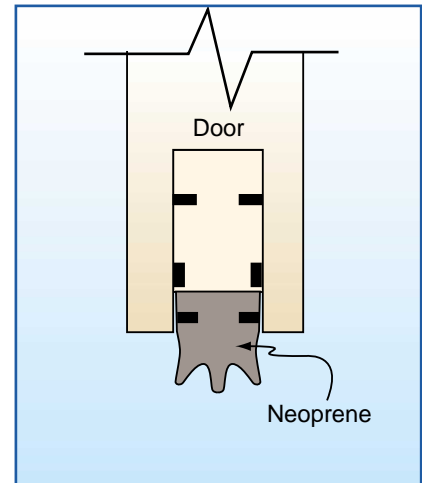


Figure 12-89
Automatic door bottom.

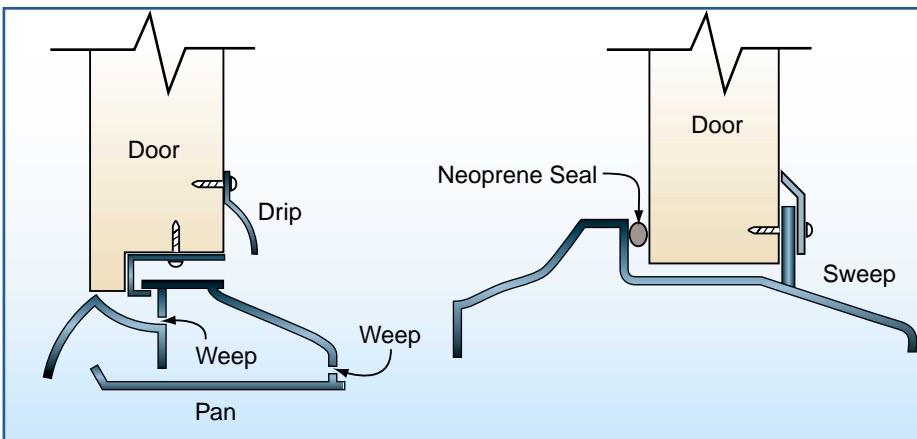
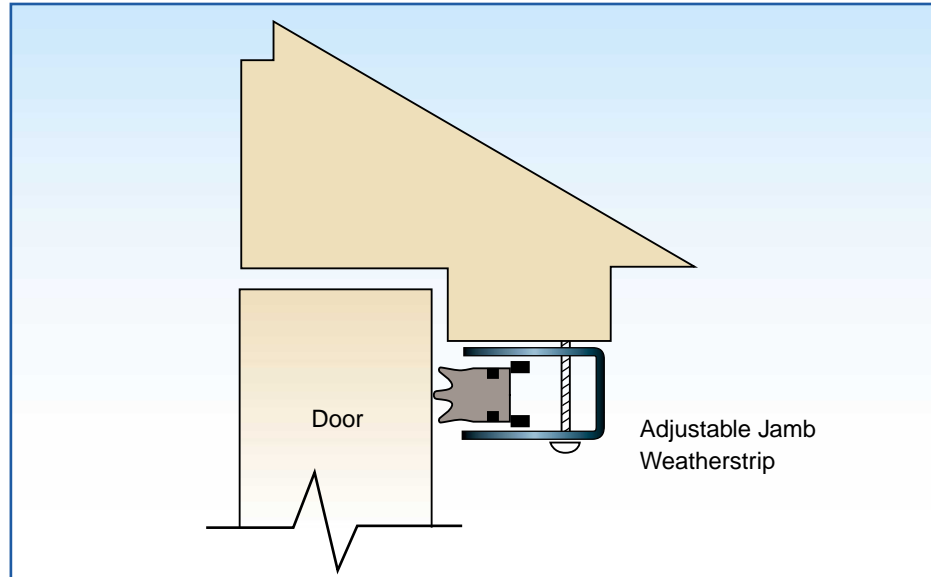


Figure 12-90
Interlocking threshold for doors with exit hardware.

Figure 12-91
Adjustable jamb/head
weatherstripping.



12.7.3.2 Wildfires

For buildings in areas prone to wildfires, fire-rated metal doors and frames are recommended. Door/frame assemblies are available in 3/4-, 1-1/2-, and 3-hour ratings.

Although construction of the door and frame are usually the same for these three ratings, the amount of allowable glass opening is decreased as the hourly rating is increased. Maximum protection is provided by a 3-hour rated assembly with no glazing. When glazed openings are desired, maximum protection is provided by fire-rated glass ceramic (this type of glass is discussed in Section 12.7.4.7).

For glass sliding doors, protection of the opening with fire-rated glass ceramic or a metal shutter is recommended; however, because a sliding glass door/shutter assembly has not been subjected to fire testing, its reliability is unknown. See Section 12.7.4 for shutter information.

12.7.4 Windows, Shutters, and Skylights

High winds and wildfires are the natural hazards that present the greatest threat to windows and skylights. Strong seismic events can also damage windows, but, in residential construction, this is not common. Hail can be very damaging to skylights and occasionally causes window breakage. Although this section focuses on windows, shutters, and skylights, it also addresses door glazing.

12.7.4.1 High Winds

Window and skylight assemblies (i.e., glazing, hardware for operable units, frame, and frame attachment to the wall/curb) must be of sufficient strength to resist positive and negative design wind pressures. The assembly should be specified to comply with wind load testing in accordance with ASTM E 1233. (Note: ASTM E 330 is the most commonly used method for evaluating window assemblies; however, because E 1233 is a cyclic test method and E 330 is a static test, E 1233 is the preferred test method, particularly in hurricane-prone areas). Designers must specify the frame attachment to the wall, either by performance or prescriptive criteria, as inadequate frame attachment due to lack of design direction is a common problem as illustrated by Figure 12-92.



Figure 12-92
Hurricane Georges (1998),
Puerto Rico. Inadequate
window frame attachment
caused the window frame to
be pulled out of the wall.

12.7.4.2 Windborne Missiles

A special consideration of glazing is its susceptibility to breakage by windborne missiles (debris). When a missile penetrates most materials, only a small opening is made; when a missile penetrates most glazing materials, however, it can result in a very large opening. The opening is often sufficiently large to increase the building's internal pressure, which may overstress other building envelope components or the structure itself. Increased internal pressure can also damage interior partitions and ceilings. A substantial amount of wind-driven water may also enter through the breached openings. In windstorms other than hurricanes, the probability of a window or skylight being struck by a missile is extremely low; however, in hurricane-prone regions, the missile load issue is of concern. Figure 12-93 shows a large window broken by windborne debris.

**NOTE**

Because of the extensive interior damage that can be caused by wind and rainwater if glazing is breached, and because of the large internal pressure created by a breach of the building envelope, this manual recommends the use of shutters or impact-resistant glass rather than the design of partially enclosed buildings.

Missile impact criteria are included in the IBC 2000 and IRC2000. ASCE 7-98, the IBC, and the IRC require that one of the following conditions be met for glazing (including skylights) in windborne debris regions:

1. The glazing must either be designed to resist missiles or be protected by shutters, or
2. The building in which the glazing is used must be designed as a partially enclosed building.

According to ASCE 7-98, the IBC, and the IRC, windborne debris regions are areas within hurricane-prone regions located:

- within 1 mile of the coast where the basic wind speed is equal to or greater than 110 mph (3-sec peak gust) and in Hawaii, or
- in all areas where the basic wind speed is equal to or greater than 120 mph (3-sec peak gust), including Guam, Puerto Rico, the U.S. Virgin Islands, or American Samoa.

Figure 12-93

Hurricane Georges (1998), Puerto Rico. Window broken by windborne debris.



It is recommended that in addition to being tested for air pressure (i.e., ASTM E 1233 and E 330), glazed assemblies in windborne debris regions have sufficient strength to resist missile loads specified in the IBC and IRC and be tested in accordance with ASTM E 1886-97 and E 1996-99. (Note: A special impact-resistant glazing assembly or shutters will be necessary.) Glazing protected with shutters should also be designed to resist the positive and negative design wind pressure.

The typical missile test criteria include the following provisions:

- Small missiles weighing 2 grams shall impact the surface at a speed of 80 ft/sec (54 mph).
- Each test specimen shall receive 30 small missile impacts; these 30 impacts are to be distributed over the surface of the window/door unit.
- Large missiles shall be wood 2x4s weighing 9 lb and impacting the surface at a speed of 50 ft/sec (34 mph).
- Each test specimen shall receive two large-missile impacts, one in the center of the assembly, and one near the corner.

In addition to the missile impact standards, each of the window/door assemblies must pass a cyclic pressure test. The assembly must first pass the missile impact test, then that same assembly is used in the cyclic pressure test. A pressure loading sequence for both inward- and outward-acting pressure, a range of pressure, the number of cycles, and the duration of each cycle is given. Tests should also be conducted for shutters, skylights, and glazing protective film.

12.7.4.3 Durability

To avoid corrosion problems, aluminum, wood or vinyl frames are recommended for buildings located within 3,000 feet of an ocean shoreline. Stainless steel frame anchors and hardware are also recommended in these areas. In areas with severe termite problems, wood frames should either be treated or not used.

12.7.4.4 Water Infiltration

Hurricanes and other coastal storms present enormous wind-driven rainwater infiltration problems. Leakage can occur at the glazing/frame interface, at the frame itself, or between the frame and wall. Because of the high design wind pressures and numerous opportunities for leakage path development, some leakage should be anticipated when design wind speed conditions are approached. A design approach to deal with this problem is to not run carpet all of the way to walls that have a large amount of glazing. Instead, a strip of water-resistant material such as tile could be specified along the wall. During a storm, towels could be placed along the strip to absorb water infiltration. These actions can help protect carpets from water damage.

It is recommended that window units be tested in accordance with ASTM D 1233 for water infiltration. The challenge with prefabricated window units is meeting the need for successful integration between the window units and the walls. To the extent possible, detailing of the interface between the wall and window units should rely on sealants as the secondary line of defense against water intrusion, rather than making them the primary protection.

The design of joints between walls and window units should consider the shape of the sealant joint (i.e., a square joint is typically preferred) and type of sealant to be specified. The sealant joint should be detailed so that the sealant is able to bond on only two opposing surfaces (i.e., a backer rod or bond-breaker tape should be specified). Butyl is recommended for concealed sealants, and polyurethane is recommended for exposed sealants. During installation, cleanliness of the sealant substrate is important (particularly if polyurethane or silicone sealants are specified), as well as tooling of the sealant.

A removable stop, as illustrated in Figure 12-94, protects the sealant from direct exposure to the weather, and the protection can reduce the wind-driven rain demand on the sealant.

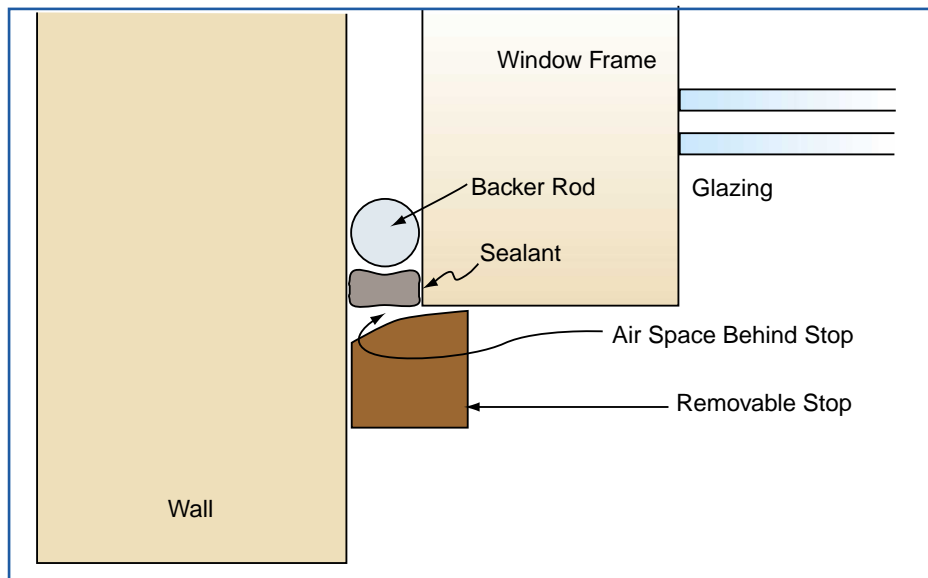
On-site water infiltration testing in accordance with ASTM E 1105 can be specified in instances where water infiltration is particularly important; however, this is an expensive test.

Figure 12-94
Protection of sealant
with a stop.



NOTE

Missile impact standards (ASTM E 1886 and E 1996) cover criteria for small and large missiles. Shutters can be costly, so the designer or owner may want to evaluate debris and impact potential vs. the cost of shutters for the design wind event. The most important envelope protection goal is to prevent an opening from allowing internal pressure increases in the building and to prevent windblown water from causing interior damage.



12.7.4.5 Shutters

If shutters are used to provide missile protection, they should be tested as discussed above for glazing (see Section 12.7.4.2). Miami-Dade County, Florida, has established a product approval mechanism for shutters to ensure that these important protective devices and the method used for securing them to the building are rated for particular wind and missile loads. A variety of shutter designs and materials are available. For windows that are difficult to reach, motor-driven rollup shutters are available as illustrated in Figure 12-95. Shutter designs that use permanently installed tracks facilitate rapid attachment of shutter panels as illustrated in Figure 12-96.

Plywood panels can also provide protection, provided they are thick enough (a minimum thickness 15/32 inch is recommended). Attaching the panels with closely spaced screws will help prevent them from blowing off. Figure 12-97 illustrates an attachment scheme for plywood panels on either wood-frame or masonry walls.

Shutters should be attached to the wall rather than the window frame, because the attachment between the window frame to the wall may be weak, as illustrated in Figure 12-92. Shutters also need to be stiff enough, or set far enough away from the glazing, that under design wind loads they do not deflect and break the glass. Shutters constructed of 2x4 boards can also provide protection from missile impacts as shown in Figure 12-98.

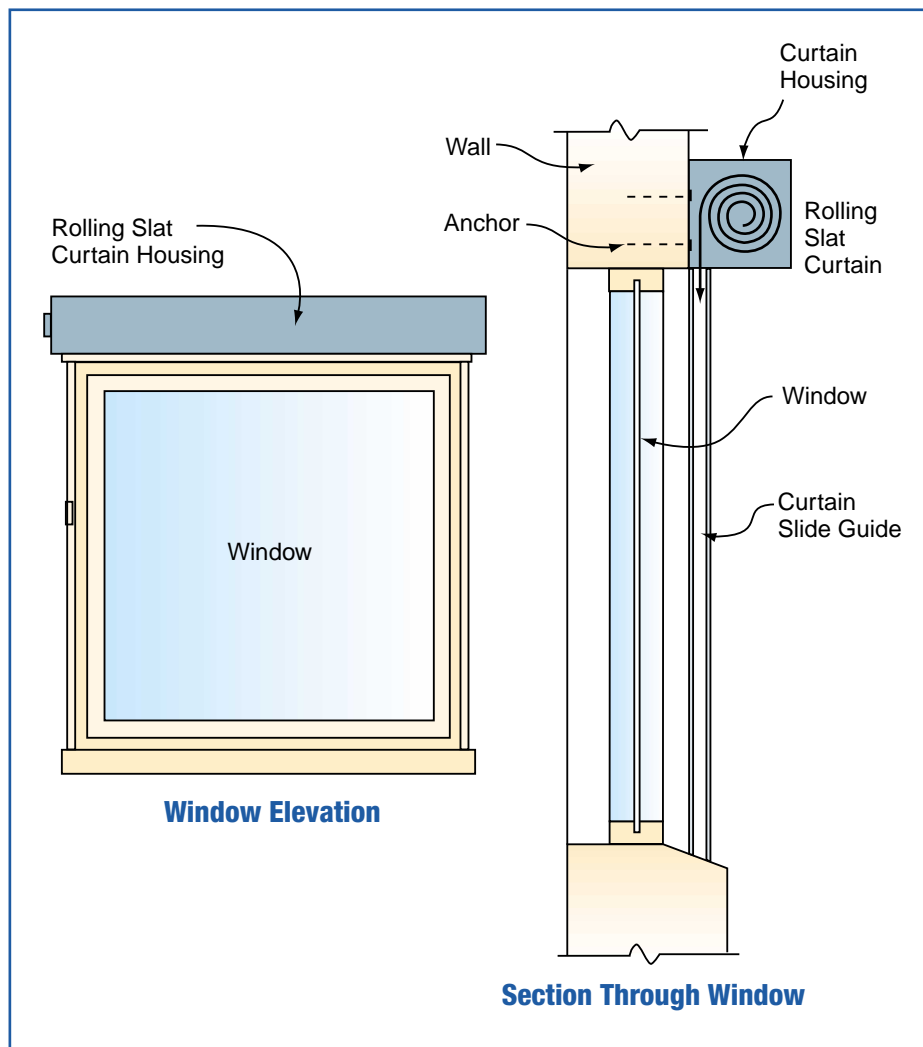


Figure 12-95
Motor-driven rollup shutters.

Figure 12-96

Hurricane Georges (1998),
Puerto Rico. Permanently
installed shutter tracks.

**Figure 12-97**

Plywood “shutter”
attachment on either wood-
frame or masonry walls.

**NOTE**

The Engineered Wood Association (formerly the American Plywood Association) has prepared five brochures that present hurricane shutter designs for wood-frame and masonry buildings. The brochures are available on-line at the Engineered Wood Association website at <http://www.apawood.org/>

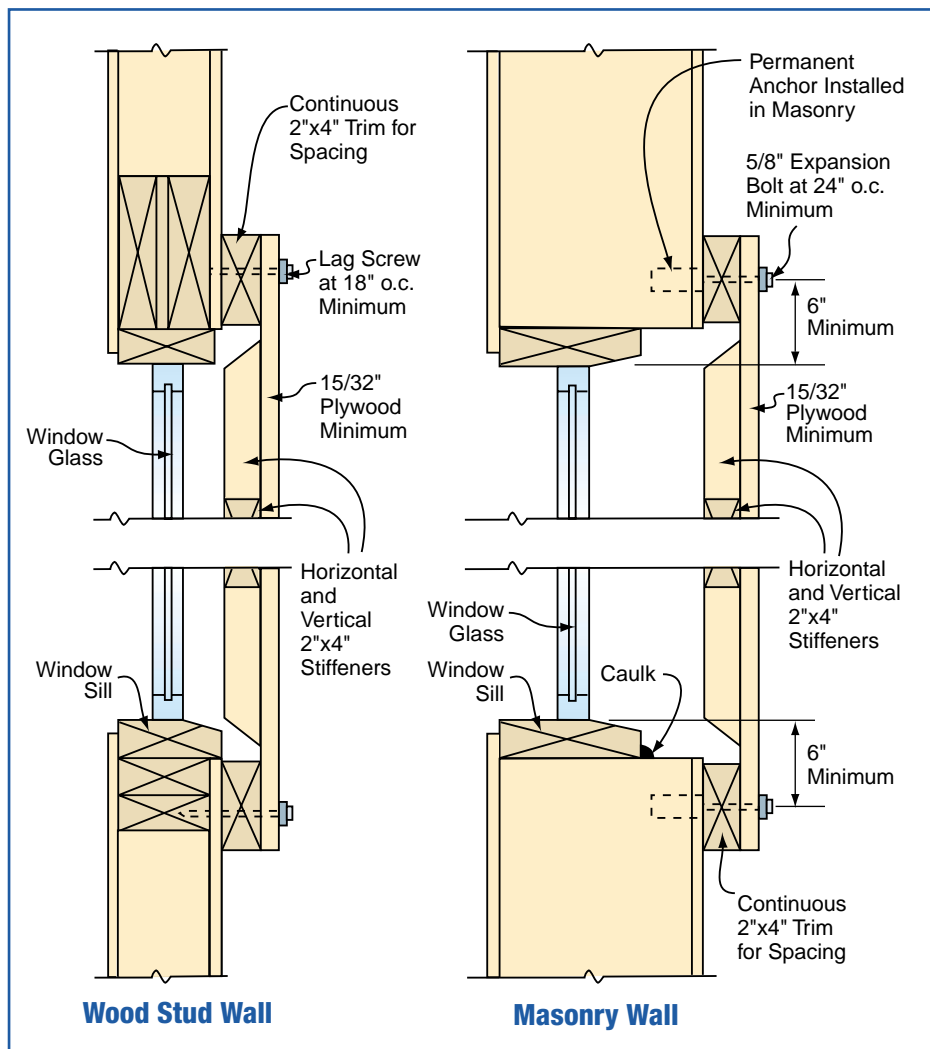




Figure 12-98
Hurricane Marilyn (1995),
U.S. Virgin Islands.
Shutters constructed of 2x4
lumber.

12.7.4.6 Seismic

Glass breakage due to in-plane wall deflection is unlikely; however, special consideration should be given to walls that have a high percentage of windows and limited shear capacity. In these cases, it is important to analyze the in-plane wall deflection to verify that it will not exceed the limits prescribed in the building code.

12.7.4.7 Wildfires

Limited data and guidelines are available related to performance of glazing subjected to wildfires. Reports from California (McMullen 1994) indicate that insulated (i.e., double-pane) windows were found to be an effective insulator against radiant heat exposure. Fire-rated glass ceramic is also a very heat-resistant material, but it is relatively expensive. Insulated windows could be composed of a fire-rated glass ceramic for the outer lite and annealed glass for the inner lite. With these approaches, metal frames should provide greater reliability than wood or vinyl frames.

Another approach is to protect the window openings with metal shutters similar to those shown in Figures 12-95 and 12-96; however, because these types of shutter assemblies have not been subjected to fire testing, their reliability is unknown.

12.7.4.8 Hail

A test method has not been developed specifically for testing skylights for hail-resistance. However, ASTM E 822, for testing hail-resistance of solar collectors, could be used for assessing the hail-resistance of skylights.

**NOTE**

The IBC and IRC include requirements for roof coverings in high-wind areas.

**NOTE**

Roofing failures in high-wind areas are one of the most common causes of substantial building damage and dollar loss.

**NOTE**

When retrofitting in high-wind areas, it is recommended that the existing roof covering be removed so that the roof deck can be checked for deterioration and to verify that it is adequately attached.

12.7.5 Roof Coverings

High-winds, seismic events, wildfires, and hailstorms are the natural hazards that can cause the greatest damage to roof covering systems. In addition to damage that can be inflicted on the roof covering itself, there are secondary issues that are of greater importance. When high winds damage the roof covering, water infiltration commonly causes significant damage to the interior of the building and its contents. Water infiltration may also occur after very large hail impact. During seismic events, heavy roof coverings such as tiles or slate may be dislodged and fall from the roof and present a hazard if the building is occupied during the event. A roof covering system that is not highly resistant to fire exposure can result in the destruction of the building during a wildfire.

Residential buildings typically have steep-slope roofs (i.e., a slope greater than 3:12). Some residential buildings have low-slope roofs, which can be roofed with a variety of systems. Low-slope roof systems are discussed in Section 12.7.5.8.

A variety of products can be used for steep-slope coverings. The following commonly used products are discussed below: asphalt composition shingles, cement-fiber shingles, liquid-applied membranes, tiles, metal panels, metal shingles, slate, and wood shingles and shakes. All of these steep-slope roofing coverings are air-permeable, except for the liquid-applied membrane and metal panel systems, which are air-impermeable. The wind loads derived in Chapter 11 represent the pressure differential between the exterior and interior surfaces of the roof system. For air-impermeable coverings, the roof covering should typically be designed to resist the full wind load; however, because of partial air-pressure equalization provided by air-permeable coverings, the wind loads derived in Chapter 11 can overestimate the load on air-permeable coverings. ASCE 7 allows the designer to use the full design load or calculate the load by using a pressure coefficient that was specifically developed for the air-permeable element being considered. Research on special pressure coefficients have been developed only for asphalt shingles and tiles. Therefore, according to ASCE 7, the other air-permeable roofing products have to be designed for the full wind load, which is problematic for several of the coverings as discussed below.

12.7.5.1 Asphalt Shingles

The discussion of asphalt shingles relates only to shingles with self-seal tabs. Mechanically interlocked shingles are not addressed because of their limited use.

High Winds

A method for calculating uplift loads on asphalt shingles has been developed (Peterka et al. 1997). A pressure coefficient for use in the calculation has also been developed; however, that coefficient may not be applicable to all asphalt shingles. Additional research needs to be undertaken to determine coefficients on all asphalt shingles. Until work on the uplift coefficients is complete, it is not possible to analyze loads on asphalt shingles. Therefore, a prescriptive solution based on results of field investigations and judgement is necessary. However, with this approach, the uplift resistance provided by the system is unknown.

The key elements to successful wind performance include the bond-strength of the self-seal strip, the mechanical properties of the shingle, and correct application of the shingle fasteners. If the tab lifts, the number of fasteners used to attach the shingle may influence whether or not shingles are blown off. Bond strength can be assessed by ASTM D 6381.

Underlayment

In the event of shingle blowoff, subsequent water infiltration damage can be avoided if the underlayment remains attached and is adequately sealed at penetrations. However, to achieve reliable secondary protection, an enhanced underlayment design is needed. The design enhancements include increased blowoff resistance of the underlayment itself, increased resistance to water infiltration (primarily at penetrations), and increased resistance to extended weather exposure.

If shingles are blown off, in some cases the underlayment is exposed for only a week or two before a new roof covering is installed. But many roofs damaged by hurricanes go unrepaired for several weeks. If a hurricane strikes a heavily populated area, roof covering damage is typically extensive. Therefore, because of the great work load, large numbers of roofs go unrepaired for several months. It is not uncommon for some roofs to go unrepaired for nearly a year.

The longer an underlayment will be exposed to weather, the more durable it will need to be to provide adequate water infiltration protection for the residence. The three options presented on page 12-122 are listed in order of decreasing resistance to long-term weather exposure. Option 1 provides the greatest reliability for long-term exposure—it is advocated in areas that are heavily populated and where the design wind speed is equal to or greater than 120 mph. Option 3 provides limited protection and is advocated only in areas that have a modest population density and the design wind speed is less than or equal to 110 mph (3-sec peak gust).



WARNING

ASTM D 3161 and Underwriters Laboratories (UL) 997 are virtually identical test methods for determining uplift resistance of asphalt shingles. However, complying with these methods does not ensure successful performance of shingles during high-wind events.



WARNING

If the roof has a ridge vent that is blown off, the underlayment recommendations do not provide water infiltration protection at ridge vent slots. To avoid leakage at ridge vent slots, gable end vents and/or well-anchored stack vents in lieu of ridge vents should be considered.

Option 1. Specify taping the joints of the plywood sheathing with roof tape (i.e., self-adhering modified bitumen). Specify that the tape be a minimum of 4 inches wide, applied to a broom-clean deck, and rolled with a roller. Seal around deck penetrations with roof tape, sealant, or asphalt roof cement. Apply a single layer of ASTM D 226 Type II (#30) underlayment felt, attached with low-profile capped-head nails or thin metal disks (“tin caps”) attached with roofing nails. Fasten at approximately 6 inches on center (o.c.) along the laps and at approximately 12 inches o.c. along two rows in the field of the sheet between the side laps. Apply a single layer of self-adhering modified bitumen complying with ASTM D 1970. Seal the modified bitumen sheet to the deck penetrations with roof tape or asphalt roof cement.

Note: (1) Because of its enhanced resistance to weather exposure, plywood is recommended in lieu of OSB, in the event the underlayment is blown off. (2) The purpose of the tape over the plywood joints is to provide secondary protection if the underlayment is blown off. (3) As an alternate to the modified bitumen sheet, install two plies of felt complying with ASTM D 2178 Type IV. Set each sheet in a continuous mopping of hot asphalt complying with ASTM D 312 Type IV.

Option 2. Specify taping the sheathing joints and sealing around penetrations as described in Option 1. Specify two plies of underlayment felt with offset side laps, complying with ASTM D 226 Type I (#15). Attach the underlayment with low-profile capped-head nails or thin metal disks (“tin caps”) attached with roofing nails. Fasten at approximately 6 inches o.c. along the laps and at approximately 12 inches o.c. along a row in the field of the sheet between the side laps.

Note: If the building is within 3,000 feet of salt water, stainless steel or hot-dip galvanized fasteners are recommended for the underlayment attachment.

Option 3. Specify taping the sheathing joints and sealing around penetrations as described in Option 1. Apply a single layer of ASTM D 226 Type I (#15) underlayment felt in the normal fashion.

Note: (1) If the roof slope is less than 4:12, tape and seal the sheathing and follow the recommendations given in *The NRCA Roofing and Waterproofing Manual* (NRCA 1996). (2) With this option, the underlayment has limited blowoff resistance. Water infiltration resistance is provided by the taped and sealed sheathing panels. This option is intended where temporary or permanent repairs are likely to be made within several days after the roof covering is blown off.

Shingle Products

If fiberglass-reinforced shingles are desired, specify shingles that comply with ASTM D 3462. If organic-reinforced shingles are desired, specify shingles that comply with ASTM D 225. SBS modified bitumen shingles are another option to consider. Because of the flexibility imparted by the SBS polymers, if a tab on a modified bitumen shingle lifts, it is less likely to break and blowoff. Compared to fiberglass-reinforced shingles, organic-reinforced shingles typically have substantially lower resistance to pulling over fastener heads. ASTM D 3462 specifies a minimum fastener pull-through resistance of 20 lb/ft at 70° F; however, fiberglass-reinforced shingles are available with a fastener pull-through resistance in excess of 30 lb/ft. In high-wind (less than or equal to 90 mph 3-sec peak gust) areas, it is recommended that a minimum pull-through resistance of 25 lb/ft be specified. In areas with extremely high design wind speeds (greater than 120 mph 3-sec peak gust), it is recommended that a minimum value of 30 lb/ft be specified.

At the time this manual was produced, data on bond-strength of the self-seal adhesive were typically unavailable. Bond strengths range from approximately 3 to 20 lb/ft at 70° F. It is recommended that bond-strength data be sought from manufacturers and that products be specified with a minimum of 12 lb/ft in high-wind areas. In areas with extremely high design wind speeds, it is recommended that a minimum value of 17 lb/ft be specified.

Attaching and Sealing

Specify attachment with six nails (rather than staples) per shingle. Locate the nails as indicated in *The NRCA Steep Roofing Manual*. For roofs within 3,000 feet of the ocean, hot-dip galvanized (rather than electroplated [EG]) or stainless steel nails are recommended.

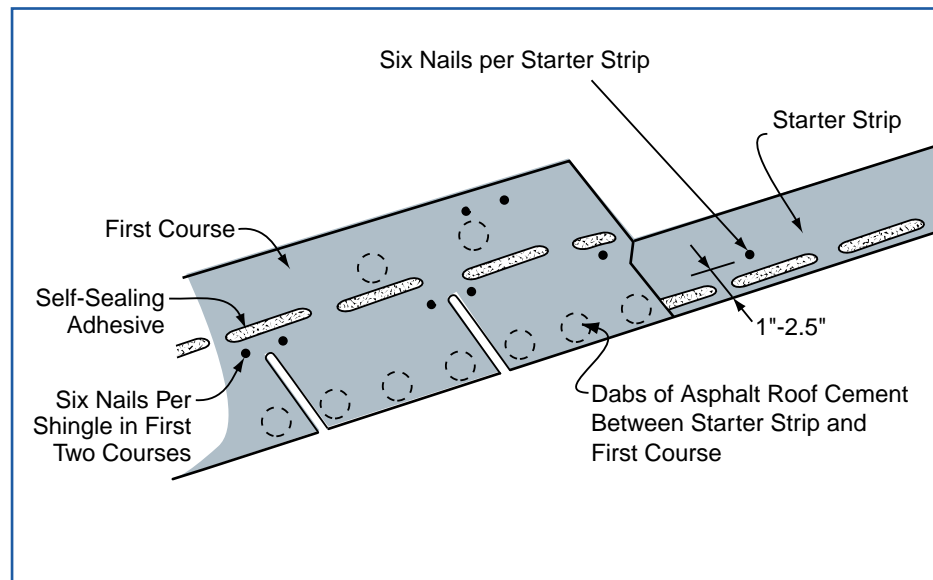
It is recommended that the designer specify that the starter strip be nailed approximately 1–2-1/2 inches from the eave edge of the starter strip. One inch is preferred, but framing conditions may require that the nails be placed further away. The fasteners should be placed in sheathing or framing lumber, rather than in trim boards. Six nails per starter strip are recommended. Specify that the starter strip and first course overhang the eave metal by approximately 1/4 inch. The 1/4-inch overhang will provide a drip edge without exposing much shingle beyond the eave for wind to lift. It is recommended that the designer specify sealing the first course of shingles by placing three (approximately 1-inch-diameter) dabs of asphalt roof cement over the starter strip so that the overlying tab of the first course will be adhered. The eave detail is illustrated in Figure 12-99.

It is recommended that the designer specify hand tabbing rakes, ridges, and hips as illustrated in Figures 12-100 and 12-101. At rakes, two dabs of asphalt roof cement are placed on the shingle about 1 inch from the rake edge, and two dabs of cement are placed on the metal drip edge. The next shingle is then set in place and fasteners, except for the one at the rake, are installed. The rake end of the shingle is then pressed to set the shingle in the dabs of cement. Finally, the fastener at the rake is installed. (Note: If a bleeder strip is used at the rake, omit the dabs of cement on the drip edge.) Figure 12-100 shows a rake detail. Specify that the rake shingles overhang the metal drip edge by approximately 1/4 inch.

At hips and ridges, two dabs of cement are placed on each side of the hip/ridge line on a hip/ridge shingle that has been installed already, as shown in Figure 12-101. Two additional dabs are placed on the field shingles to adhere the next hip/ridge shingle. For the starter hip/ridge shingle, four dabs of tab cement are placed on the field shingles.

After the next hip/ridge shingle is set in place, it is pressed to set the shingle in the dabs of cement and one nail is applied on each side of the hip/ridge line.

Figure 12-99
Eave detail for asphalt shingles.



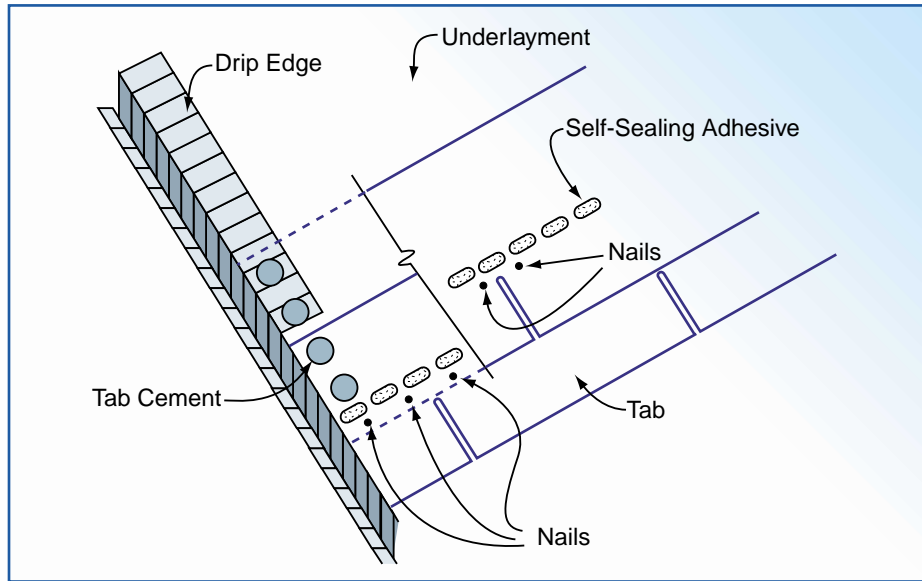


Figure 12-100
Rake detail for asphalt shingles.

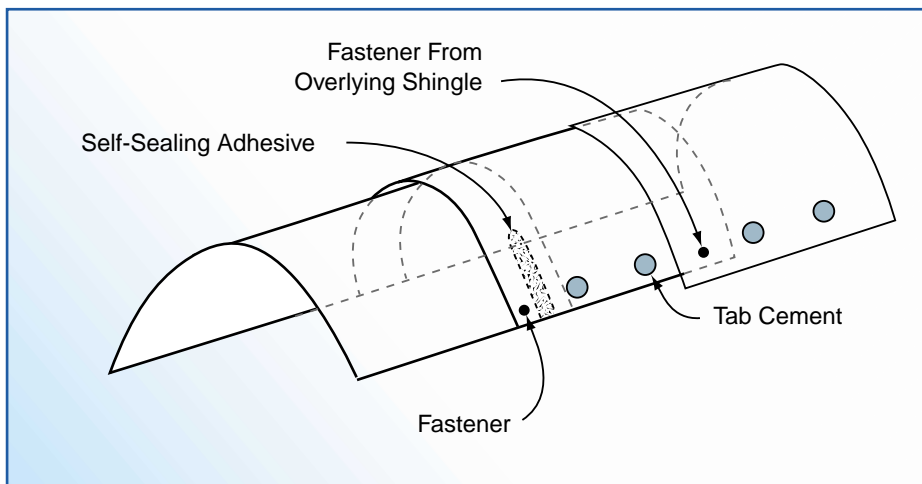


Figure 12-101
Hip and ridge detail for asphalt and composition shingles.

Hip and ridge shingles are normally cut from a strip shingle. Premanufactured hip/ridge shingles are also available and may or may not have self-sealing adhesive. If they do not, specify application of cement to replicate the adhesive on the strip shingles.

Because of extra thickness of materials at hips and ridges, longer nails than those used in the roof field are needed. All nails should penetrate the underside of the sheathing, or penetrate at least 3/4 inch into wood plank decks.

Some building codes require that hips, ridges, rakes, eaves, and valleys be set in wide continuous bands of asphalt roof cement; however, this practice may result in shingle blistering, which could shorten shingle life and be

aesthetically objectionable to the homeowner. If the code requires wide continuous bands of cement, it is recommended that the designer request the building official to allow the use of dabs of cement as recommended above.

Ridge Vents

Ridge vents should be evaluated for susceptibility to penetration by wind-driven rain. Test methods or specific design guidance have not been developed. See Section 12.7.6 for additional information.

Wildfires

Organic-reinforced shingle systems achieve only a Class C fire-resistance rating, which indicates very little fire resistance. Fiberglass-reinforced shingle systems achieve a Class A fire-resistance rating, which is the highest rating achieved through ASTM E 108 fire testing. Under typical fire exposure conditions, fiberglass-reinforced shingle systems normally provide sufficient fire resistance; however, they typically do not provide sufficient resistance to the extreme conditions induced by wildfires. Where enhanced resistance to wildfires is desired, another type of roof covering is recommended.

Hail

UL 2218 is a test method for assessing simulated hail resistance of roofing systems. This test yields four ratings (Classes 1–4). Systems rated Class 4 have the greatest impact resistance. Asphalt shingles are available in all four classes. It is recommended that asphalt shingle systems on buildings located in areas vulnerable to hail be specified to pass UL 2218, at a Class level commensurate with the hail load. Hail resistance of asphalt shingles depends partly on the condition of the shingles at the time they are impacted and is likely to decline with roof age.

12.7.5.2 Cement-Fiber Shingles

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

High Winds

Because of limited market share of cement-fiber shingles in areas where research has been conducted after high-wind events, few data are available on their wind performance. At the time this manual was produced, manufacturers had not conducted research to determine a suitable pressure coefficient or coefficients; therefore, demonstrating compliance with ASCE 7 is problematic with these products until such a coefficient(s) is

developed. A consensus test method for uplift resistance has not been developed for these products.

In high-wind areas, mechanical attachment should be specified in corner and perimeter roof zones. With some of the lighter-weight products, mechanical attachment in the field of the roof may also be prudent. In extremely high-wind areas, mechanical attachment in the field of the roof should be specified. Additional mechanical attachment near the tail of the shingle should also be specified in perimeter and corner zones, and perhaps in the field of the roof. Because these prescriptive attachment suggestions are based on limited information, the uplift resistance that they provide is unknown.

To minimize water damage in the event of blowoff of the cement-fiber covering, the underlayment recommendations for asphalt shingles are suggested. For cement-fiber systems, however, a Type II (i.e., #30) felt is recommended in order to provide more puncture resistance during application. Stainless steel or hot-dip galvanized fasteners are recommended for roofs within 3,000 feet of the ocean. Note that cement-fiber systems are vulnerable to breakage from missile impact. See the discussion in Section 12.7.5.4.

Seismic

Cement-fiber products are relatively heavy and, unless they are adequately attached, they can be dislodged during strong seismic events and fall from the roof. At time this manual was produced, manufacturers had not conducted research or developed design guidance for use of these products in areas prone to large ground motion accelerations. The guidance provided in the Section 12.7.5.4 is suggested until specific guidance is developed for cement-fiber products.

Wildfires

Most cement-fiber systems can achieve a Class A fire-resistance rating; however, some only achieve a Class B. The Class A systems should provide relatively good fire resistance during wildfires; however, because cement-fiber products typically have less thermal mass than tiles, they probably do not offer as much protection as tiles. Cement-fiber products may not be as fire-resistant as metal-panel or metal-shingle systems.

Hail

At the time this manual was being produced, one cement-fiber assembly had passed UL 2218 with a Class 2 rating. It is recommended that cement-fiber shingle systems on buildings in areas vulnerable to hail be specified to pass UL 2218, at a class level commensurate with the hail load. If products with the desired class are not available, another type of product should be considered.

12.7.5.3 Liquid-Applied Membranes

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

High Winds

Investigations following strong hurricanes and typhoons have revealed that liquid-applied membranes installed over concrete and plywood decks have offered excellent performance (provided the deck remains attached to the building). This type of roof covering has extremely high wind-resistance reliability.

It has been found that unprotected concrete roof decks could eventually experience problems with corrosion of the slab reinforcement. It is recommended that all concrete roof decks be covered with some type of roof covering.

Wildfires

Liquid-applied membranes over concrete decks should provide excellent wildfire-resistance. A Class A rating can be achieved for liquid-applied membranes over a plywood deck; however, where enhanced resistance to the extreme conditions induced by wildfires is desired, a roof covering other than a liquid-applied membrane over plywood is recommended.

Hail

At the time this manual was being produced, no liquid-applied membranes over concrete or plywood decks had been evaluated by UL 2218.

12.7.5.4 Tiles

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



NOTE

For additional information, see SBCCI standard SSTD 11-97, *Test Standard for Determining Wind Resistance of Concrete or Clay Roof Tiles* (SBCCI 1997b) for additional information.

High Winds

Loads and Resistance

Methods to calculate uplift loads and load resistance on tiles have been developed and incorporated into the *IBC* for loose-laid and mechanically attached tiles. Coefficients have not been determined for mortar-set or foam-set systems.

Storm damage investigations have revealed poor performance of wire-tied systems as illustrated in Figure 12-102.

**Figure 12-102**

These wire-tied tiles (installed over a concrete deck and attached with stainless steel clips at the perimeter rows) failed during Typhoon Paka (1997) in Guam.

Use of mortar to adhere tiles is problematic. For a variety of reasons, performance of mortar-set tile systems during Hurricane Andrew in South Florida was poor. Foam-set securement of tiles was developed after Hurricane Andrew. At the time this manual was being produced, foam-set systems had not been exposed to high-wind conditions; therefore it is unknown how they will perform.

Missile Loads

Tile roof coverings are very vulnerable to breakage from windborne missiles. Even when their attachment is well-designed and the tiles are properly installed, because of their brittle nature, they can easily be broken by relatively low-energy missiles. If a tile is broken, debris from a single damaged tile can impact other tiles on the roof, which can lead to a progressive cascading failure. In addition, a substantial number of high-energy missiles can be injected into the wind field.

In windstorms other than hurricanes, the probability of a roof being struck by a missile is extremely low; however, in hurricane-prone regions where the basic wind speed is equal to or greater than 110 mph (peak gust), the missile impact issue is of concern.

To minimize water damage in the event of damage to the tile covering, the underlayment recommendations for asphalt shingles are suggested; however, for tiles, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application. If a mortar-set or foam-set system is specified, a mineral surface cap sheet adhered to a nailed base sheet is suggested.



WARNING

In high-wind areas, there is a significant potential for tile blowoff. The resulting wind-borne debris can injure people and damage adjacent buildings and vehicles.

For roofs within 3,000 feet of the ocean, stainless steel fasteners are recommended.

Seismic

Tiles are relatively heavy. Unless they are adequately attached, they can be dislodged during strong seismic events and fall away from the roof. Manufacturers have conducted laboratory research on seismic resistance of tiles; however, specific design guidance for use of these products in areas prone to large ground motion accelerations has not been developed. As shown in Figures 12-103, 12-104, and 12-105, investigations after seismic events have revealed that tiles can be dislodged if they are not adequately secured.

Figure 12-103

Most of the tiles on this roof were nailed to batten strips. However, in one area, several tiles were not nailed. Because of the lack of nails, the tiles were shaken off the battens. Northridge Earthquake, California.



Figure 12-104

These tiles were nailed to thin wood sheathing. During the earthquake, the tail of the tiles bounced and pulled out the nails. Northridge Earthquake, California.





Figure 12-105
Northridge Earthquake, California. The tile in the center of this photograph slipped out from underneath the hip tiles. The tile that slipped was trimmed to fit at the hip. The trimming eliminated the nail holes and no other attachment was provided. The friction fit was inadequate to resist the seismic forces.

The following attachment guidance is recommended in seismic areas where short period acceleration exceeds 0.5g.

- When tiles are only loose-laid on battens, they can be shaken off as shown in Figure 12-103. If tiles are laid on battens, supplemental mechanical attachment is recommended.
- Tiles nailed only at the head may or may not perform well. If they are attached with a smooth-shank nail into a thin plywood or oriented strand board (OSB) deck, pullout can occur (see Figure 12-104). Specifying ring-shank or screw-shank nails or screws is recommended; however, even with these types of fasteners, the tail of the tile can bounce, causing enlargement of the nail hole by repeated pounding. To overcome this problem, specify wind clips near the tail of the tile.
- Tiles that are attached by only one fastener experience eccentric loading. This problem can be overcome by specifying wind clips near the tail of the tile.
- Two-piece barrel (i.e., mission) tiles attached with straw nails can slide downslope a few inches because of deformation of the long straw nail. This problem can be overcome by specifying a wire-tied system or proprietary fasteners that are not susceptible to downslope deformation.
- When tiles are cut to fit near hips and valleys, the portion of the tile with the nail hole(s) is often cut away (see Figure 12-105). Supplemental securement is necessary to avoid displacement of these loose tiles.
- Securement of rake, hip, and ridge tiles with mortar is ineffective. If mortar is specified, it should be augmented with mechanical attachment.

- Rake trim tiles fastened just near the head of the tile often slip over the fastener head because the nail hole is enlarged by repeated pounding. Additional restraint is needed for the trim pieces. Also, the design of some rake trim pieces makes them more inherently resistant to displacement than other rake trim designs.

For roofs within 3,000 feet of the ocean, stainless steel fasteners are recommended.

Wildfires

Tiles are noncombustible and have a relatively large thermal mass. Therefore, tile roof systems should provide excellent wildfire-resistance. Lightweight tile products are available, but it is recommended that normal-weight tiles be specified in lieu of lightweight tiles when enhanced wildfire resistance is desired.

Hail

At the time this manual was being produced, no tile assembly had passed UL 2218. Tile manufacturers assert that UL 2218 is not a good test method to assess non-ductile products such as tiles. A proprietary alternative test method is available to assess non-ductile products; however, at the time this manual was being produced, it had not been recognized as a consensus test method.

12.7.5.5 Metal Panels and Metal Shingles

A variety of metal panel and shingle systems are available. Some of these products simulate the appearance of tiles or wood shakes.

High Winds

Because of the great differences in system designs, the wind performance of metal systems varies widely. For metal-panel systems, it is recommended that uplift resistance be based on the ASTM E 1592 test method; for metal-shingle systems, it is recommended that uplift resistance be based on ASTM E 330. Both of these systems should be considered as air-impermeable for the purpose of calculating uplift loads.

For panel systems, it is recommended that two rows of fasteners be placed along the eaves, hip, and ridges (see Figure 12-106). The first row should be near the edge/end of the panel (i.e., within 2–3 inches), and the next row should be approximately 3–4 inches from the first row. Hip and ridge flashings should be attached with a double row of fasteners. For exposed fastener systems, the hip and ridge flashing fasteners can also serve as the panel fasteners. For concealed clip systems, the hip and ridge fasteners are in addition to the two clips along either side of the hip and ridge. Screws rather than nails should be used to attach clips, panels, and flashings.

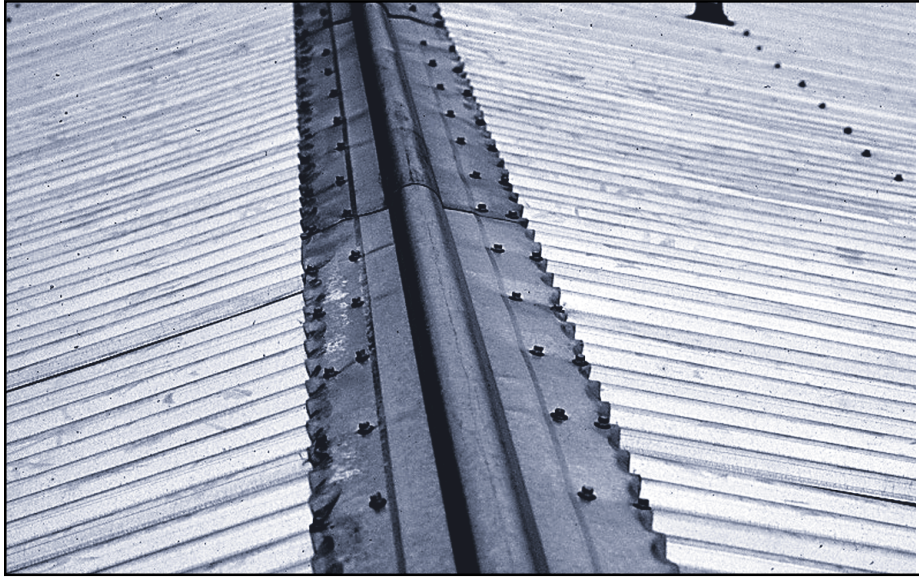


Figure 12-106
Hurricane Marilyn (1995),
U.S. Virgin Islands. The ridge
flashing on this corrugated
metal panel roof was well
attached, with a double row
of closely spaced fasteners
on each side of the ridge.

Stainless steel clips and fasteners are recommended. For enhanced corrosion protection of steel panels/shingles, an aluminum zinc alloy (Galvalume[®]) coating is recommended in lieu of galvanizing. (Additional design information regarding corrugated metal roofing is provided in Appendix K.)

To minimize water damage in the event of damage to the metal covering, the underlayment recommendations for asphalt shingles are suggested; however, for metal coverings, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application.

Wildfires

Metal panel/shingle systems are noncombustible; however, they can readily transmit heat to the substrate below. If metal roofing is installed over a combustible substrate such as wood, a thermal barrier is needed between the substrate and metal to achieve enhanced resistance to wildfires. The design should be based on system testing.

Hail

Several metal panel and shingle systems have passed UL 2218. Although metal systems have passed Class 4 (the Class with the greatest impact resistance), they often are severely dented by the testing. Although they may still be effective in inhibiting water entry, they can be aesthetically objectionable. (Note: The appearance of the system is not included in the UL 2218 evaluation criteria.)

12.7.5.6 Slate

Some cement-fiber and tile products are marketed as “slate”; however, slate is a natural material. Cement-fiber and tile products that simulate slate are addressed in Section 12.7.5.2.

High Winds

Because of limited usage of slate in areas where research has been conducted after high-wind events, few data are available on its wind performance. Manufacturers have not conducted research to determine a suitable pressure coefficient. Demonstrating slate’s compliance with ASCE 7 is problematic until such time as the coefficient is developed. A consensus test method for uplift resistance has not been developed for slate.

In extremely high-wind areas, mechanical attachment near the tail of the slate should be specified in perimeter and corner zones, and perhaps in the field. Because this prescriptive attachment suggestion is based on limited information, the uplift resistance that it provides is unknown.

To minimize water damage in the event of slate blowoff, the underlayment recommendations for asphalt shingles are recommended for slate as well; however, for slate systems, a Type II (i.e., #30) felt is recommended for greater puncture resistance during application.

Seismic

Slate is relatively heavy. Unless adequately attached, it can be dislodged during strong seismic events and fall away from the roof. Manufacturers have not conducted research or developed design guidance for use of slate in areas prone to large ground motion accelerations. The guidance provided for tiles, in Section 12.7.5.4, is suggested until specific guidance is developed for slate.

Wildfires

Slate is noncombustible and has a relatively large thermal mass. Therefore, slate roof systems should provide excellent wildfire-resistance.

Hail

At the time this manual was being produced, no slate assembly had passed UL 2218.

12.7.5.7 Wood Shingles and Shakes

High Winds

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

At the time this manual was produced, manufacturers had not conducted research to determine a suitable pressure coefficient. Demonstrating compliance with ASCE 7 is problematic with wood shingles/shakes until such a coefficient is developed. At this time, a consensus test method for uplift resistance has not been developed for wood shingles/shakes.

For enhanced durability, preservative-treated wood is recommended for shingle/shake roofs on coastal buildings. For those roofs located within 3,000 feet of the ocean, stainless steel nails (normally Grade 316) are recommended. See Figure 12-107 for an example of shingle loss due to corrosion of the nail fasteners.



Figure 12-107

Loss of wood shingles on this North Carolina house during Hurricane Bertha (1996) was due to corrosion of fasteners.

Wildfires

Wood shingle and shake roofs (including those with fire-resistive treatment) are not recommended in areas prone to wildfires.

Hail

At the time this manual was being produced, no wood-shingle assembly had passed UL 2218; however, heavy shakes have passed Class 4 (the class with the greatest impact resistance) and medium shakes have passed Class 3.

The hail resistance of wood shingles/shakes depends partly on the condition of the shingles/shakes at the time they are impacted and is likely to decline with roof age.

12.7.5.8 Low-Slope Roof Systems

Roof coverings used on low-slope roofs need to be waterproof membranes, rather than water-shedding coverings as used on steep-slopes. Although most of the low-slope membranes can be used on dead-level substrates, it is always preferable (and typically required by building codes) to install them on substrates that have some slope (e.g., 1/4 inch in 12 inches [2 percent]). The most commonly used coverings on low-slope roofs are built-up, modified bitumen, and single-ply. Liquid-applied membranes (Section 12.7.5.3), structural metal panels (Section 12.7.5.5), and sprayed polyurethane foam may also be used on low-sloped roofs. Information on low-slope roof system can be found in *The NRCA Roofing and Waterproofing Manual* (NRCA 1996).

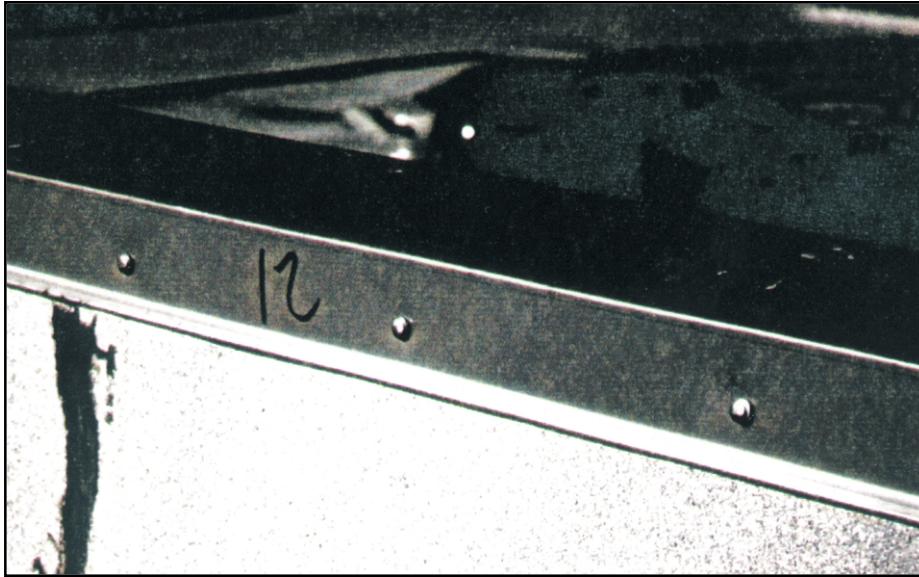
Low-slope roofing makes up a very small percentage of the residential roofing market. However, when low-slope systems are used on residences, the same principles that apply to commercial roofing also apply to residential work. The natural hazards presenting the greatest challenges to low-sloped roofs are high-winds, earthquakes, wildfires, and hail.

High Winds

Edge Flashings/Copings

Roof membrane blowoff is almost always a result of lifting and peeling of metal edge flashings (gravel stops) or copings, which serve to clamp down the membrane at the roof edge. For many years, continuous cleats have been recommended to restrain the vertical leg of edge flashings and the outer leg of copings. However, because light-gauge metals are typically used, the cleat and flashing/coping legs deflect outward during high winds. As a result, the flashing/coping leg often disengages from the cleat. The disengaged leg is then free to roll up and allow the wind to lift and peel the membrane.

An effective and reliable deterrent to leg deformation is to face-fasten vertical legs of flashings/copings. Figure 12-108 shows this attachment with concrete spikes. For attachment to other building materials, #14 stainless steel screws with stainless steel washers are recommended. To determine screw spacing and thickness of the flashing/coping refer to ANSI/SPRI ES-1 *Wind Design Standard for Edge Systems Used with Low Slope Roofing Systems*. This standard can also be used in the design of edge flashings/copings that are fastened with continuous cleats; however, cleat-attached systems are not as reliable as face-attached systems. The standard can also be used in selecting pre-manufactured edge flashings and copings.

**Figure 12-108**

Typhoon Paka (1997), Guam.
24-gauge coping attached
with stainless steel concrete
spikes at 12 inches o.c.

Built-Up Roofs (BUR)

BURs can offer exceptionally good high-wind performance, provided the edge flashing/coping does not lift. These membranes are also resistant to penetration by low- and moderate-energy missiles. After the uplift load is determined (see Chapter 11), a system should be selected that has demonstrated adequate capacity to meet the uplift load through testing in accordance with Factory Mutual (FM) 4470, UL 580, or UL 1897. A factor of safety (FS) should be applied to the test load results (an FS of 2 is commonly used).

To avoid the potential hazard of aggregate blowoff, it is recommended that a cap sheet or field-applied coating be specified in lieu of aggregate surfacing.

Modified Bitumen

These systems are related and are similar to BURs. The BUR information presented above is applicable to modified bitumen systems.

Single-Ply

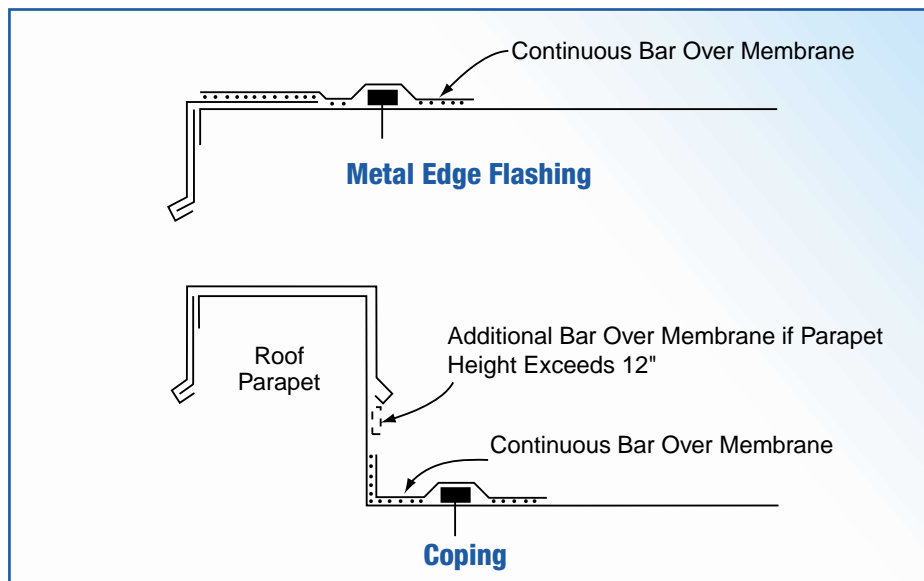
There are four primary types of single-ply membranes: CSPE (Hypalon), EPDM, PVC, and TPO. They are normally attached by ballasting, fully adhering to the substrate, or by rows of mechanical fasteners. Because single-ply membranes do not possess the stiffness or thickness of BURs or modified bitumen membranes, single-ply membranes are more vulnerable to low-energy windborne missiles and they are more susceptible to lifting and peeling from the substrate.

To avoid the potential hazard of aggregate blowoff, it is recommended that heavyweight concrete pavers (i.e., 17 lb/ft² minimum) be specified in lieu of aggregate if a ballasted system is desired.

Because of the complex load distribution that occurs in most mechanically attached single-ply systems, it is recommended that a fully adhered membrane be specified in lieu of one that is mechanically attached. After the uplift load is determined (see Chapter 11), a fully adhered system should be selected that has demonstrated adequate capacity to meet the uplift load through testing in accordance with FM 4470, UL 580, or UL 1897. An FS should be applied to the test load results (again, an FS of 2 is commonly used).

It is recommended that a “bar” be placed over the membrane near the edge flashing/coping as illustrated in Figure 12-109. The purpose of the bar is to provide secondary protection against the membrane lifting and peeling in the event that the edge flashing/coping fails. A strong bar specifically made for “bar-over” mechanically attached systems is recommended. The bar needs to be very well attached to the parapet or deck. Depending upon wind conditions, a spacing between 6 inches o.c. and 12 inches o.c. is recommended. A gap of a few inches should be left between each bar to allow for water flow across the membrane. After the bar is attached, it is stripped over with a piece of membrane.

Figure 12-109
Continuous bar near the
edge of flashing/coping.



Sprayed Polyurethane Foam (SPF)

SPF systems typically provide excellent wind pressure performance, provided the substrate to which they are applied does not lift. SPF roofs are typically surfaced with a coating; however, they can be surfaced with aggregate. To avoid the potential hazard of aggregate blowoff, it is recommended that a coating be specified. Note that these systems are vulnerable to puncture by large windborne missiles.

Seismic

If a ballasted system is specified, its weight should be considered during seismic load analysis of the structure (see Chapter 11). Also, a parapet should extend above the top of the ballast to restrain the ballast from falling over the roof edge during a seismic event.

Wildfires

Many low-slope systems are available with a Class A fire-resistance rating. However, enhanced protection can be provided by a heavyweight concrete paver ballasted roof system. If pavers are to be placed over a BUR or modified bitumen membrane, it is recommended that a layer of extruded polystyrene intended for protected membrane systems be specified over the membrane. Additionally, for smooth-surfaced BUR and modified bitumen membranes, it is recommended that a sheet of polyethylene (4-mil minimum) be specified between the membrane and polystyrene to keep the polystyrene from bonding to the membrane. To protect the base flashings, it is recommended that a mortar-faced extruded polystyrene board be installed over the base flashings as shown in Figure 12-110.

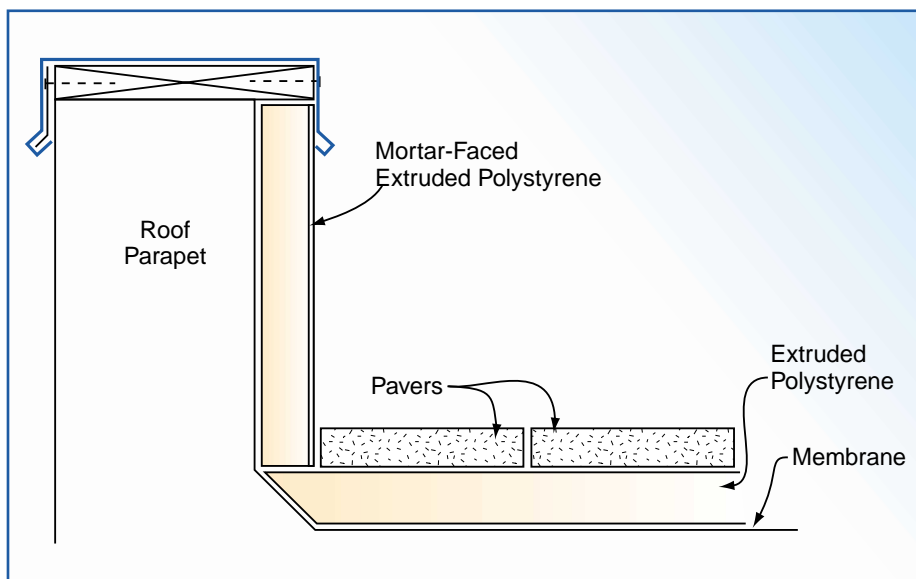


Figure 12-110
Mortar-faced extruded polystyrene over base flashing for wildfire protection.

Hail

It is recommended that a system that has passed the Factory Mutual Research Corporation's severe hail test be specified. Enhanced hail protection can be provided by a heavyweight concrete-paver-ballasted roof system, as discussed in the section above.

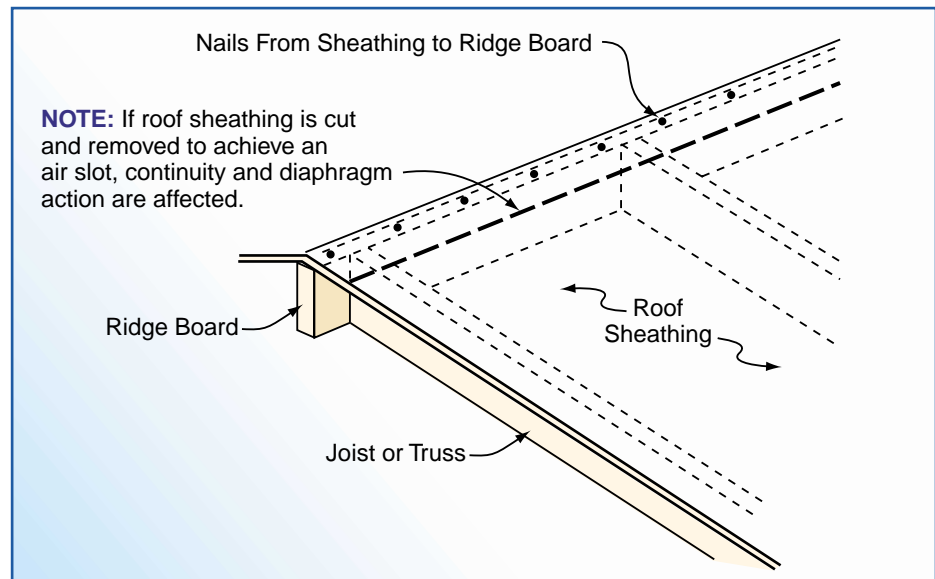
If the pavers are installed over a single-ply membrane, it is recommended that a layer of extruded polystyrene intended for protected membrane systems be specified over the membrane to provide protection in the event the pavers break. Alternatively, a stone protection mat intended for use with aggregate-ballasted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

12.7.6 Roof Ridge Vents

Continuous ridge vent installations, primarily used on roofs with asphalt shingles, have typically not addressed the issue of maintaining structural integrity of the roof sheathing. When the roof sheathing is used as a structural diaphragm as it is in high-wind and seismic hazard areas, the structural integrity of the roof can be compromised by the continuous vent.

The roof sheathing, usually plywood or oriented strand board (OSB), is intended to act as a diaphragm. The purpose of the diaphragm is to resist lateral forces. To properly function, the diaphragm must have the capability of transferring the load at its boundaries from one side of the roof to the other; it normally does this through the ridge board. The continuity, or load transfer is accomplished with nails. This approach is illustrated by Figure 12-111.

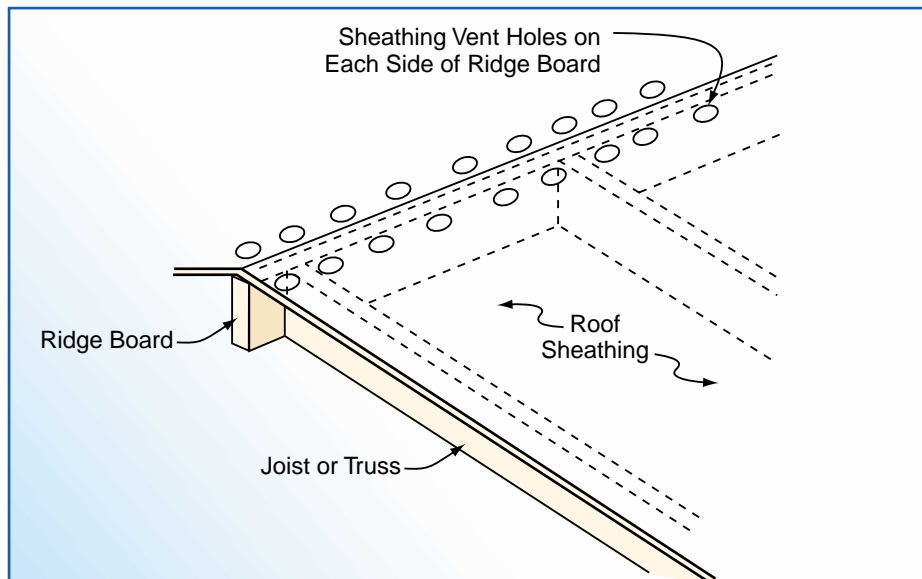
Figure 12-111
Method for maintaining a continuous load path at the roof ridge by nailing roof sheathing.



The problem with the continuous ridge vent application is the need to develop openings through the diaphragm to allow air to flow from the attic space up to and through the ridge vent. For existing buildings not equipped with ridge vents, this requires cutting slots or holes in the sheathing. If a saw is used to cut off 1–2 inches along either side of the ridge, the integrity of the diaphragm is affected. This method of providing roof ventilation should not be used without taking steps to ensure proper load transfer.

There are methods, however, of providing the proper ventilation while maintaining the continuity of the roof diaphragm. They include the following:

1. Drilling 2- to 3-inch-diameter holes in the sheathing between each truss or rafter approximately 1-1/2 inches down from the ridge. The holes should be equally spaced and should remove no more than one-half of the total amount of sheathing area between the rafters. For example, if the rafters are spaced 24 inches o.c. and 2-inch-diameter holes are drilled, space them at 6 inches o.c., which will allow about 12 in² of vent area per linear foot when holes are placed along either side of the ridge. This concept is illustrated in Figure 12-112.
2. Install two ridge boards separated by an air space of at least 3 inches, with solid blocking between the ridge boards at each rafter or truss. Stop the sheathing at the ridge board and fully nail the sheathing as required. The ridge vent must be wide enough to cover the 3-inch gap between the ridge boards. The ridge board and blocking must be nailed to resist the calculated shear force.



NOTE

Refer to Chapter 12 of the IBC 2000 or Chapter 8 of the IRC 2000 for additional information about the required vent area for roof and attic spaces. Ventilation may be provided by any of the following methods:

1. mechanical
2. turbine vents
3. gable end vents
4. soffit vents
5. ridge vents

Figure 12-112

Holes drilled in roof sheathing for ventilation—roof diaphragm action is maintained. (For clarity, sheathing nails are not shown.)



WARNING

Designers and builders are cautioned not to inadvertently weaken the lateral load path by cutting back the sheathing for a ridge vent without providing for an alternative method of transferring the loads.

For new construction, the designer should detail the ridge vent installation with the proper consideration for the load transfer requirement. Where high-diaphragm loads may occur, a design professional should be consulted regarding the amount of sheathing that can be removed or other methods of providing ventilation while still transferring lateral loads. The need to meet these requirements may become a significant problem in large or complex residential buildings where numerous ventilation openings are required. In these instances, ridge vents may need to be augmented with other ventilating devices (e.g., static or gable-end vents).

Many ridge vent products are not very wide. When these products are used, it may be difficult to provide sufficiently large openings through the sheathing and still maintain diaphragm integrity if holes are drilled through the sheathing. Manufacturers' literature often illustrates large openings at the ridge with little or no consideration for the transfer of lateral loads.



NOTE

For additional information, see *Protecting Building Utilities From Flood Damage – Principles and Practices for the Design and Construction of Flood-Resistant Building Utility Systems*, FEMA 348 (FEMA 1999).

12.8 Utilities/Mechanical Equipment

12.8.1 Elevators

Elevators are being installed with increasing frequency in elevated, single-family homes in coastal areas. These elevators are used for primarily small numbers of people, but are large enough to provide handicap accessibility and accommodate small pieces of household equipment.

Small personal-size elevators are almost always designed with a shaft that is installed away from an outside wall. The elevator shaft must have a landing, usually at the very bottom level (usually ground level), and a cab platform near the top. The bottoms of elevators that have a landing at the lower level are almost always below the DFE.



DEFINITION

DFE = Design Flood Elevation

See Chapter 11, Section 11.6.2, for additional information.

FEMA's NFIP Technical Bulletin 4 (see Appendix H) discusses the installation of elevator equipment in the floodplain. As explained in the bulletin, elevator accessory equipment should be installed above the BFE to prevent damage. It is important to note that one of the safety features of elevators causes them to descend to the lowest level during power outages so that occupants will not be trapped in the cab. During flooding, this feature may expose the cab to flood waters. For elevator installation in coastal buildings, the designer must ensure the elevator stops at a level above the DFE when the power is lost. One method is to install a system of interlocking controls and float switches, as described in NFIP Technical Bulletin 4.

12.8.2 Design of Exterior-Mounted Mechanical Equipment

High winds, flooding, and seismic events are the natural hazards that can cause the greatest damage to exterior-mounted mechanical and electrical equipment.

12.8.2.1 High Winds

Blowoff of exhaust fans, fan cowlings, and vent hoods commonly occurs during high winds. The resulting windblown debris can cause damage to other buildings, and water can enter the building that lost the equipment. Tearing away of the equipment typically occurs because of inadequate anchorage of the equipment to the roof, inadequate strength of the equipment itself (i.e., loss of fan cowlings), and corrosion.

Considering the small size of most exhaust fans, vent hoods, and air-conditioning units used on residential buildings, the following prescriptive attachment recommendations should be sufficient for most residences:

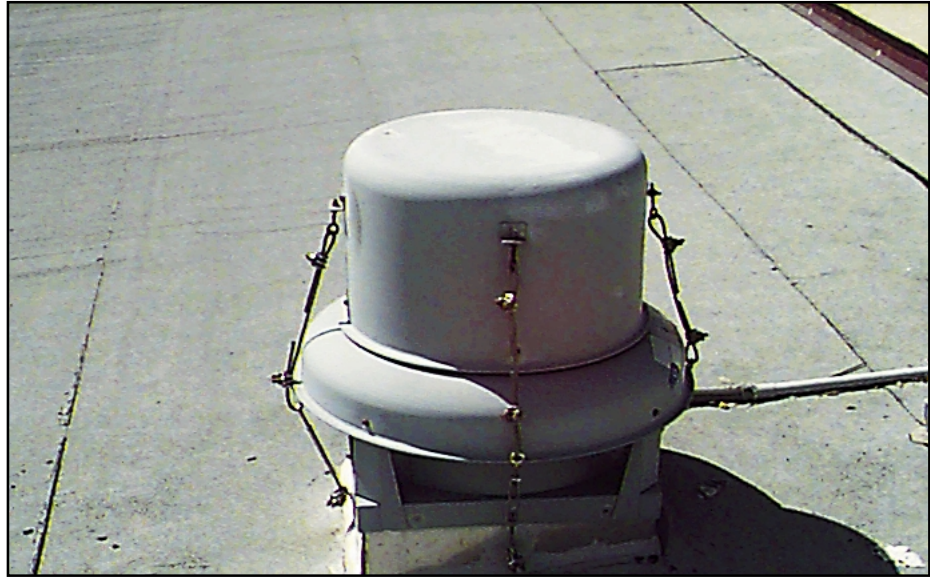
- For curb-mounted units, specify #14 screws with gasketed washers.
- For curbs with sides less than 12 inches, specify one screw at each side of the curb.
- For curbs between 12 and 24 inches, specify two screws per side.
- For curbs between 24 and 36 inches, specify three screws per side.
- For buildings within 3,000 feet of the ocean, stainless steel screws are recommended.
- For units that have flanges attached directly to the roof, attachment with #14 pan-head screws is recommended. A minimum of two screws per side, with a maximum spacing of 12 inches o.c., is recommended.

If the equipment is more than 30 inches above the curb, the attachment design should be based on calculated wind loads. ASCE 7-98 does not provide adequate guidance for determining equipment loads. Until such criteria are provided, the following approach is recommended:

- Assume a negative (i.e., uplift) load on the top of the equipment, a negative (i.e., suction) load on one side of the equipment, and a positive load on the opposite side of the equipment.
- Apply the loads to the longest side of the equipment.
- Consider the equipment as partially enclosed.
- Use component and cladding coefficients (consider the top of the equipment as the roof, and the sides as walls).
- Select coefficients for the field of the roof and field of the wall (i.e., do not use perimeter or corner coefficients).

Until equipment manufacturers produce more wind-resistant equipment, job-site strengthening of fan cowlings and vent hoods is recommended. One approach is to use 1/8-inch-diameter stainless steel cables, as shown in Figure 12-113. Two or four cables are recommended, depending on design wind conditions. Alternatively, additional, heavy straps could be screwed to the cowling and curb.

Figure 12-113
Typhoon Paka (1997), Guam.
Stainless steel cables for
strengthening fan cowlings
and vent hoods.



To avoid corrosion problems, nonferrous metal, stainless steel, or steel with minimum G-90 hot-dip galvanized coating is recommended for the equipment itself, equipment stands, and equipment anchors when the equipment is on buildings located within 3,000 feet of the ocean. Stainless steel fasteners are also recommended.

12.8.2.2 Flooding

Flood damage to mechanical equipment is typically caused by failure to sufficiently elevate equipment as shown in Figure 12-114. Figure 12-115 shows proper elevation of an air-conditioning condenser in a floodprone area.



Figure 12-114
Hurricane Georges (1998),
U.S. Gulf Coast. Mechanical
equipment damaged as a
result of insufficient
elevation.



Figure 12-115
Proper elevation of
mechanical equipment in a
floodprone area.

Outdoor or exposed mechanical equipment for one-to-four family residential buildings will normally be limited to the following:

- air-conditioning condensers
- ductwork (air supply and return)
- exhaust fans
- well pumps

Flood waters can force mechanical equipment from its supports and sever its connection to mechanical or electrical systems. Mechanical equipment can also be damaged or destroyed by inundation in flood waters, especially salt

water. A very short period of inundation may not destroy some types of mechanical equipment, but any inundation of electrical equipment will, at a minimum, cause significant damage to wiring and other components.

Minimizing flood damage to mechanical equipment requires elevating it above the DFE. Because of the uncertainty of wave heights and the probability of waves splashing, the designer should consider additional elevation for this equipment to help minimize damage.

In V zones, equipment must be installed either on a cantilevered platform (see Section 13.6.2) supported by the first floor framing system or on an open foundation like that used for the primary building. It is strongly recommended that any open foundation used to support mechanical equipment be of the same size, depth, and structural integrity as the main building foundation.

In A zones, mechanical equipment must be elevated to the DFE on either an open or solid foundation or otherwise protected from flood waters entering or accumulating in the system components. For houses constructed over crawlspaces, some HVAC systems are installed in such a way that the ductwork is routed through the crawlspace. This ductwork must be installed above the DFE or be made watertight in order to minimize flood damage. Many ductwork systems today are constructed with insulated board and thus would not withstand flood inundation without being destroyed.

12.8.2.3 Seismic

Residential mechanical equipment units are normally not heavy. Therefore, with some care in attachment design for resistance to shear and overturning forces, these units should perform well during seismic events. Air-conditioning units that are mounted on tall elevated platforms will experience higher accelerations than ground-mounted units; therefore, extra attention should be given to attachment in areas prone to large ground motion accelerations.

12.8.3 Design of Interior Mechanical Equipment

High winds will normally not affect the operation of indoor equipment; however, flood waters can cause substantial damage to furnaces, boilers, water heaters, and distribution ductwork. Flood waters can extinguish a flame, short-circuit the equipment's electrical system, and inundate equipment and ductwork with sediment.

There are only two primary methods of reducing flood damage to interior equipment:

1. Elevate the equipment and the ductwork above the DFE. This elevation may be accomplished by hanging the equipment from the existing first floor or placing the equipment in the attic or some other location above the DFE.
2. Build a waterproof structure around the equipment, allowing access for maintenance and replacement of some of the equipment parts.



NOTE

Although the IBC and IRC specify that flood-resistant materials be used below the BFE, this manual recommends that flood-resistant materials be used below the DFE.

12.8.4 Electrical, Telephone, and Cable TV Systems

Electric utilities serving residential buildings in coastal areas are frequently in a harsh and corrosive environment where increased maintenance and a shorter life can be expected. Common electrical components on residential buildings that might be exposed to severe wind or flood events consist of electric meters, electric panels, electric feeds from the utility company, receptacles, lights, security system connections, and telephone connections.

The design of an electric utility system (and any other utility service) must consider that the success of the system depends on numerous components operating successfully—all of which are at some risk of failure. For instance, if there are 10 components in the system that must operate for the system to be successful and the reliability of each component is 95 percent, the reliability of the entire system is $(0.95)^{10} = 60$ percent. A system of this type has a high probability of failing during a natural hazard event.

The primary method of protection is to elevate all of these components above the DFE; however, it frequently is not possible to accomplish this—in fact, there are conflicts between the concepts embodied in floodplain management requirements and other building code regulations. Depending on the DFE, the two most likely conflicts that are difficult to resolve involve the placement of the electric meter and the location of a light switch at the base of entry/exit stairs required for safe egress by the building code.

Meters are normally required to be no higher than eye level for easy reading by utility company employees; however, this height is often below the expected flood level for coastal homes. Figure 12-116 shows an electric meter that is easily accessible by the utility company, but that is below the DFE. Figure 12-117 shows a bank of meters and electric feeds that failed during Hurricane Opal.

Figure 12-116
Elevated electric meter.

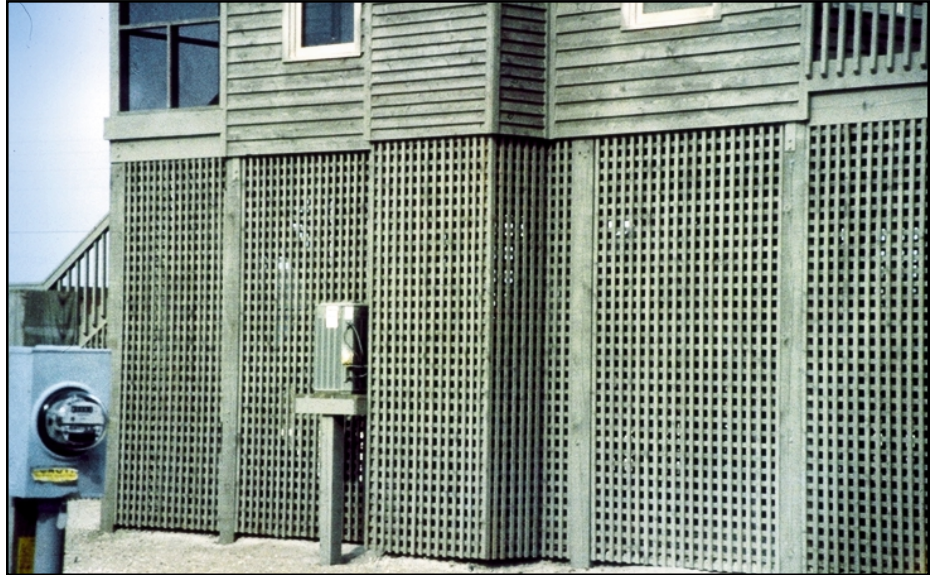


Figure 12-117
Electric meters and feed
lines that failed during
Hurricane Opal (1995).



Traditionally, the utility company is responsible for the service to a house to the point that is metered. The service to the house is provided either underground or overhead. Underground service must be designed for flood conditions and must be buried deep enough for protection from scour and erosion. The length of the service connection from grade to the meter is usually a few feet. This short connection should be designed so that the conduit or electric cable will not be contaminated by flood water and so that debris will not strike the conduit or cable and disable the power.

Wind damage to exterior-mounted electrical equipment is infrequent, in part because of the small size of most equipment (e.g., disconnect switches and conduit). Exceptions are satellite dishes and electrical service penetrations through the roof, as shown in Figure 12-118. This problem can be avoided with underground service. It can also be avoided by terminating the overhead lines at a service mast mounted on a freestanding concrete pylon, and providing underground lines between the pylon and residence. A concrete pylon is shown in Figure 12-119.



Figure 12-118
Hurricane Marilyn (1995),
U.S. Virgin Islands. Damage
caused by dropped overhead
service.



Figure 12-119
Hurricane Georges (1998),
Puerto Rico. Concrete
pylon used for overhead
service mast.

Other recommendations for protecting electrical system components include the following:

- Install panel feeds and conduits on the landward side of piles or other types of foundation elements to protect them from floating debris and frontal flood forces.
- Install the minimum number of switches below the DFE that will provide compliance with the electrical code. The feeds to the switch receptacles should come from above the DFE to minimize flood inundation.
- Install lights that are energized by a motion detector on the level below the DFE instead of lights operated by a switch.
- Install the feed from the utility company to the house below the ground so that damage to this connection is minimized.
- Install the electric panel above the DFE in a location that is easily accessible from the interior of the house.

Do not install electrical components behind or through breakaway walls. Components installed in this way are likely to be damaged when the breakaway wall fails. In addition, they may prevent complete failure of the wall and may transfer loads into the supporting structure of the house.

Satellite dish failures are typically caused by the designer's failure to perform wind load calculations and provide for adequate anchorage.



NOTE

See Section VI-D of *Engineering Principles and Practices for Retrofitting Flood Prone Residential Buildings*, FEMA 259 (FEMA 1995), for guidance on determining the proper size of an emergency generator.

Because a severe wind event will certainly interrupt electric service, designers and homeowners need to make decisions about the need for temporary power. If a decision is made to install a generator in a new or existing building, this manual recommends the following:

- Locate the generator above the DFE.
- Locate the generator so that engine exhaust fumes are vented to the outside.
- Locate the fuel source above the DFE, and store an amount of fuel adequate for the expected use and length of time the generator will operate.
- Install the generator where its noise and vibration will cause the least disruption.
- Size the generator for the expected load, including the starting and running load. Determine what the expected load will be (e.g., does the load include heat, refrigeration, lights, sump pumps, sewer ejector pumps). Heating and cooling require very large generators and should probably be provided only for safety and freeze protection.

- Provide an “emergency load” subpanel to supply critical circuits. Do not rely on extension cords. Supply the emergency panel from the “load side” of a manual or automatic transfer switch.
- Determine whether operation of the generator will be manual or automatic. (Manual operation is simpler and less expensive.)
- Decide whether to (1) install a quick connection at the building’s electric panel so that the generator can feed the building’s electric system or (2) run extension cords from the generator to the selected equipment. Protection must be provided at the panel so that generator power will not “back feed” into the supply line and endanger electric workers when they are reconnecting power to the building.

**WARNING**

Do not “backfeed” emergency power through the service panel. Utility workers can be killed!

12.8.5 Water and Wastewater Systems

These systems include the following:

- wells
- septic systems
- sanitary systems
- municipal water connections
- fire sprinkler systems

12.8.5.1 Wells

Protection of well systems from a severe event (primarily a flood) must consider the following, at a minimum:

- Flood waters that enter aquifers or that saturate the soil can contaminate the water supply.
- Non-submersible well pumps must be above the DFE.
- If water is to be available following disasters, a alternative power source must be provided.
- The water supply line riser must be protected from water and debris flow damage; this means the supply line must be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.
- Backflow valves must be installed to prevent flood waters from flowing into the water supply when water pressure in the supply system is lost.

**WARNING**

In some areas, high groundwater levels may preclude the installation of septic tanks below the level of expected erosion and scour.

12.8.5.2 Septic Systems

Protection of septic systems must consider the following, at a minimum:

- If the septic tank is dislodged from its position in the ground, the piping will come disconnected, releasing sewage into floodwaters, and the tank could damage the nearest structure. Therefore, the system must be buried below the expected depth of erosion and scour, and the tank must be anchored to prevent a buoyancy failure
- The sewage riser lines and septic tank risers must be protected from water and debris flow damage; this means risers should be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.

12.8.5.3 Sanitary Systems

Protection of sanitary systems must consider the following, at a minimum:

- Sanitary riser lines must be protected from water and debris flow damage; this means risers should be behind a pile or other structural member or inside an enclosure designed to also withstand the forces from the event.
- When the line breaks at the connection of the building line and main sewer line, a check valve in the line may help prevent raw sewage from flowing back out of the line and contaminating the soil near the building.

12.8.5.4 Municipal Water Connections

Protection of municipal water connections is accomplished primarily by protecting the water riser into the building from damage by debris. If water risers are severed during a coastal event, damage to the water supply system can include the following:

- waste matter from flooded sewer or septic systems intruding into the water system
- sediment filling some portion of the pipes
- breaks in the pipes at multiple locations

12.8.5.5 Fire Sprinkler Systems

Protection of this system is similar to the others - the primary issue is to locate the sprinkler riser such that the location provides shielding from damage. In addition, there must be consideration to the location of shutoff valves, etc., so that, if there is damage to an unprotected portion of the fire water supply line, this damage is not unnecessarily added to the damage caused by the natural hazard event.

12.9 Appurtenant Structures

12.9.1 Decks, Gazebos, and Covered Porches Attached to Buildings

Many decks and other exterior attached structures have failed during hurricanes. For decks and other structures without roofs, the primary cause of failure has been inadequate support — the pilings have either not been embedded deep enough to prevent failure or have been too small to carry the large forces from natural hazards.

The following are recommendations for the design of decks and other exterior attached structures:

- Where possible, the deck should be supported with the same type of foundation and structural system as the primary building. If this is not possible, care should be taken to ensure that the main building and attachment have similar stiffness.
- Either decks and other structures should be structurally independent of the main structure, or the additional forces they will generate should be expressly considered in the design of the main structure and the attachment to the main structure. If the attachment relies on the lateral-force-resisting system of the main structure, its anchorage must not be allowed to fail. Unless the deck is structurally independent, any attachment method that is based on the “breakaway deck” concept will create debris. Note that any construction seaward of mean high tide must be carried out in such a way that damage to the insured structure is minimized.
- If the deck surface is constructed at floor level, the deck surface/floor level joint provides a point of entry for wind-driven rain. Eliminate this problem by lowering the deck surface below the floor level.
- Cantilevering a deck from a building eliminates the need for piles altogether and should be considered when the deck dimensions can be accommodated with this structural technique. Caution must be exercised with this method to keep water out of the main house framing. Chapter 13 discusses construction techniques for flashing cantilever decks that will minimize water penetration into the house.
- Exposure to the coastal environment is severe for decks and other exterior appurtenant structures. Wood must be preservative-treated or naturally decay resistant, and fasteners must be corrosion resistant.

12.9.1.1 Handrails

To minimize the effects of wind pressure, flood forces, and wave impacts, handrails for decks should be open, with slender vertical or horizontal



WARNING

The NFIP regulations define “appurtenant structure” as “a structure which is on the same parcel of property as the principal structure to be insured and the use of which is incidental to the use of the principal structure.” In this manual, appurtenant structure means any other building or constructed element on the same property as the primary building.



WARNING

Decks should not cantilever over bulkheads or retaining walls where waves can run up the vertical wall and under the deck.



NOTE

See FEMA NFIP Technical Bulletin 5, *Free of Obstruction Requirements* (Appendix H), for additional information about the types of construction allowed in Coastal High Hazard Areas.

members spaced in accordance with the locally adopted building code. Many deck designs include solid panels (some made of plexiglass) between the top of the deck handrail and the deck. These solid panels must resist the design wind and flood loads (below the DFE); otherwise, they will become debris.

12.9.1.2 Stairways

Many coastal homes have stairways leading to ground level. During flooding, flood forces often move the stairs and frequently separate them from the point of attachment. When this occurs, the stairs become debris and can cause damage to nearby houses and other buildings.

Recommendations for stairs that descend below the BFE include the following:

- To the extent permitted by code, use open-riser stairs to let flood water through the stair stringers, and anchor the stringers to a permanent foundation such as piles driven to a depth sufficient to prevent failure from scour.
- Extend the bottom of the stair carriages several feet below grade to account for possible scour. Stairs constructed in this fashion are more likely to remain in place during a coastal hazard event and therefore more likely to be usable for access after the event. In addition, by decreasing the likelihood of damage, this approach will reduce the likelihood of the stairs becoming debris.
- Construct the stairs so they are retractable. This requires a winch, cable, and hinged stair so that when a storm is expected, the winch can be used to raise the stairs out of harm's way (check local building codes for construction requirements of these systems).



NOTE

See Appendix I for guidance concerning the design and construction of dune walkovers.

12.9.2 Walkways, Sidewalks, and Other Ground-Level Structures

The walkway and sidewalk least susceptible to damage is the one that does not exist, so eliminating ground-level structures is one way a designer can reduce potential damage. In most instances, these ground-level accessories become debris during a flood event and can cause damage to neighboring buildings.

The designer should first consult state or local officials concerning regulations that govern ground-level structures. Some states have specific guidelines that must be followed for these types of structures. Ground-level structures include the following:

- wood walkways
- wood dune walkovers
- wood ground-level or near-ground-level decks

- planters
- concrete walkways
- concrete driveways

For wood structures, one of the following is recommended:

- Secure the structure with a foundation that will resist both the expected scour and the flood force that will come up from under the structure and lift it out of the soil. Figure 12-120 illustrates this concept.
- Construct the structure in such a way that it can be removed and stored prior to the storm event.

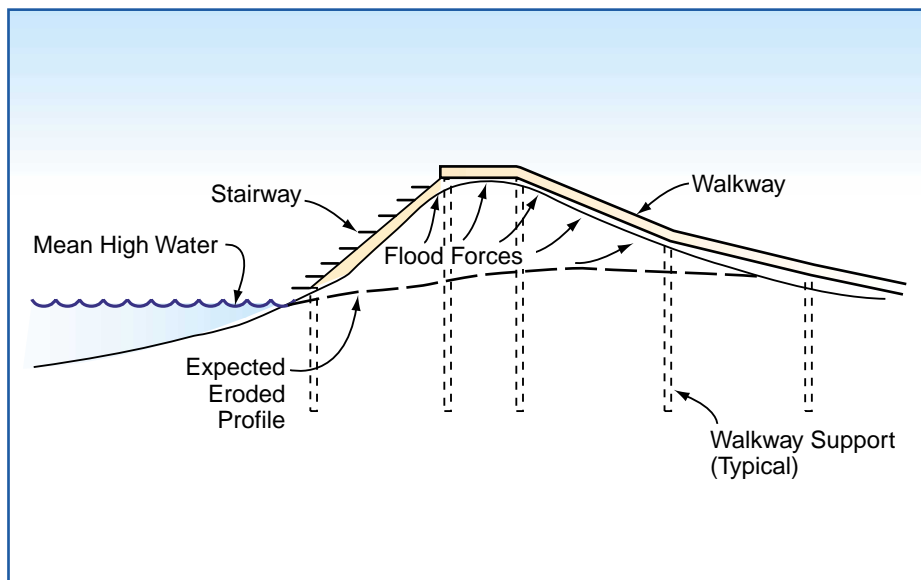


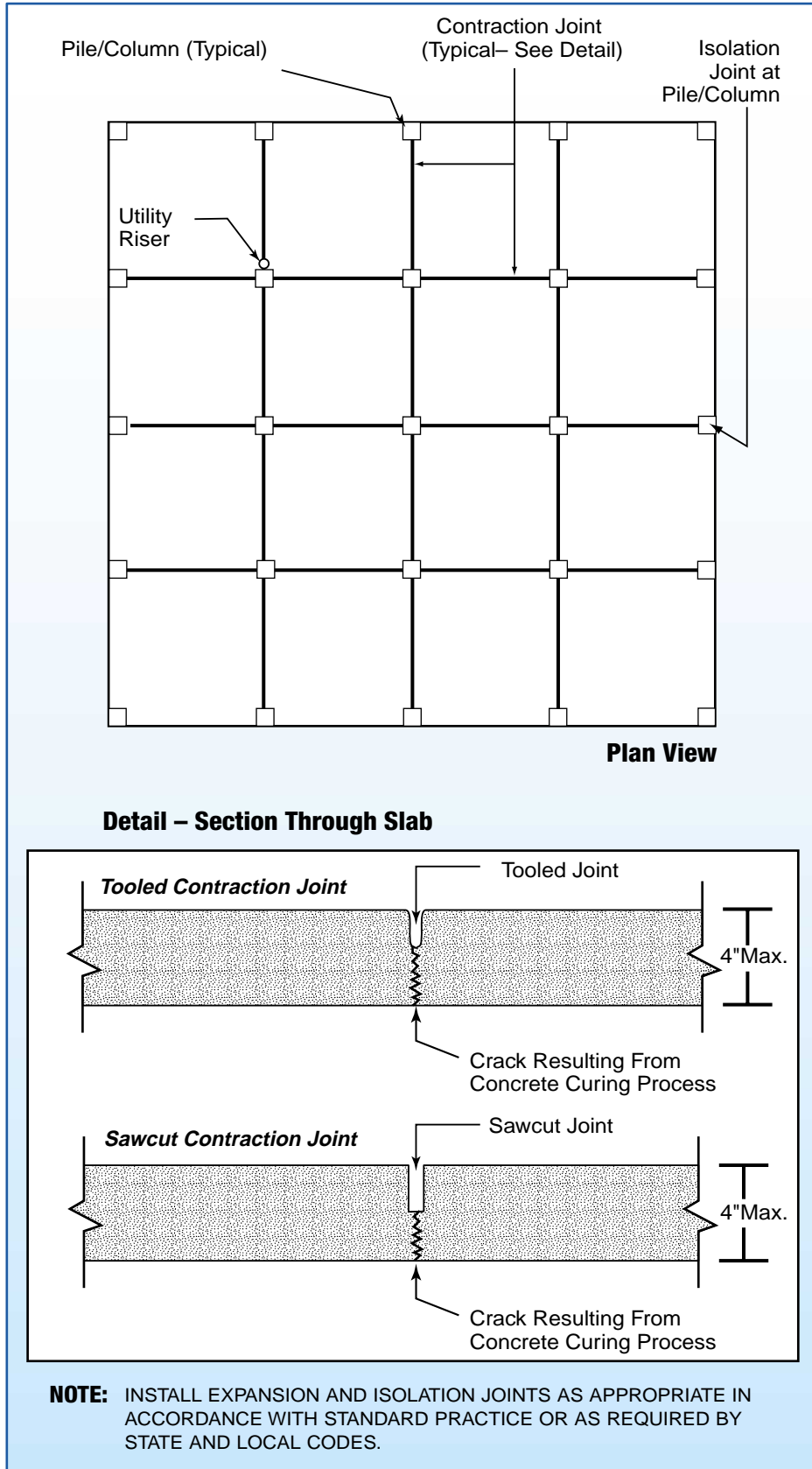
Figure 12-120
Wood walkover construction.

For concrete structures, the following is recommended:

- Do not install any reinforcement in walkways or driveways located below the building
- Do not secure the concrete to any structural element of the building such as a pile or column
- Install the concrete in small segments (approximately 4 feet x 4 feet) that will easily break up during a severe storm event
- Install the concrete with a maximum thickness of 3–4 inches

Figure 12-121 (FEMA 1997) illustrates these recommendations.

Figure 12-121
Recommended contraction
joint layout for frangible
slab-on-grade below
elevated building.



12.9.3 Access to Elevated Buildings

The first floor of buildings in the Special Flood Hazard Area will be elevated a few feet to many feet above the exterior grade in order to protect the building and its contents from flood damage. Buildings in A zones may be only a few feet above grade; buildings in V zones may be 8 feet to more than 12 feet above grade. Access to these elevated buildings must be provided by one or more of the following:

- stairs
- ramps
- elevator

Stairs must be constructed in accordance with the local building code so that the run and rise of the stairs conform to the requirements. Chapter 10 of the IBC and IRC require a minimum run of 11 inches per stair tread and a maximum rise of 7 inches per tread. An 8-foot elevation difference requires 11 treads or almost 12 feet of horizontal space for the stairs. Local codes will also have requirements concerning other stair characteristics, such as stair width and handrail height.

Ramps that comply with regulations for access by persons with disabilities must have a maximum slope of 1:12 with a maximum rise of 30 inches and a maximum run of 30 feet without a level landing. The landing length must be a minimum of 60 inches. As a result, access ramps generally will not be practical for buildings elevated more than a few feet above grade, and then only when adequate space is available.

Elevators are being installed in many 1- to 4-family residential structures and provide an easy way to gain access to elevated floors of a building (including the first floor). There must be an elevator entrance on the lowest floor; therefore, in flood hazard areas, some of the elevator equipment will be below the BFE. FEMA's NFIP Technical Bulletin 4 provides guidance on how to install elevators so that damage to elevator components is minimized during a flood.

12.9.4 Pools and Hot Tubs

Many homes at or near the coast have a swimming pool or hot tub as an accessory. Some of these pools are fiberglass and are installed on a pile-supported structural frame. Others are in-ground concrete pools. The designer should consider the following when a pool is to be installed at a coastal home:

- Only an in-ground pool may be constructed beneath an elevated V-zone building. In addition, the top of the pool and the accompanying deck or walkway must be flush with the existing grade, and the area below the lowest floor must remain unenclosed.



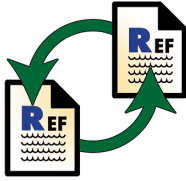
NOTE

For more information about elevator installation in buildings located in SFHAs, see FEMA NFIP Technical Bulletin 4, in Appendix H.



NOTE

Check with local floodplain management officials for information about regulations governing the disturbance of primary frontal dunes. Such regulations can affect various types of coastal construction, including the installation of appurtenant structures such as swimming pools.



CROSS-REFERENCE

Refer to Appendix L for design guidance regarding frangible pools.



NOTE

The construction of pools below or adjacent to buildings in coastal high hazard areas must meet the requirements presented in FEMA's NFIP Technical Bulletin 5, *Free of Obstruction Requirements for Buildings Located in Coastal High Hazard Areas* (see Appendix H). In general, pools must either be (1) elevated above the BFE on an open foundation or (2) constructed in the ground in such a way as to minimize the effects of scour and the potential for the creation of debris.

- Enclosures around pools beneath elevated buildings constitute recreational use and are therefore not allowed, even if constructed to breakaway standards. Lattice and insect screening are allowed, because they do not create an enclosure under a community's NFIP-compliant floodplain management ordinance or law.
- A pool adjacent to an elevated V-zone building may be either constructed at grade or elevated. Elevated pools must be constructed on an open foundation and the bottom of the lowest horizontal structural member must be at or above the DFE so that the pool will not act as an obstruction.
- The design professional must assure community officials that a pool beneath or adjacent to an elevated V-zone building will not be subject to breaking up or floating out of the ground during a coastal flood and will therefore not increase the potential for damage to the foundations and elevated portions of any nearby buildings. If an in-ground pool is constructed in an area that can be inundated by flood waters, the elevation of the pool must account for the potential buoyancy of the pool. If a buoyancy check is necessary, it should be made with the pool empty. In addition, the design professional must design and site the pool so that any increased wave or debris impact forces will not affect any nearby buildings.
- Pools and hot tubs have water pumps, piping, heaters, filters, and other equipment that is expensive and that can be damaged by flood waters and sediment. All such equipment should be placed above the DFE where practical.
- Equipment required for fueling the heater, such as electric meters or gas tanks, should be placed above the DFE. It may also be necessary to anchor the gas tank to prevent a buoyancy failure. If buried, the tank must not be susceptible to erosion and scour and thus failure of the anchoring system.

For concrete pools, buoyancy failure is also possible when flood waters cover the pool. In addition, flood flows can scour the soil surrounding a buried pool and tear the pool from its anchors. When this happens, the pieces of the pool become large waterborne debris.

12.9.5 Boat Houses

In some coastal communities that have access to waterways where boats can be used, covered boat storage areas are sometimes constructed adjacent to the house. When appurtenant structures are constructed in V zones seaward of mean high tide, these structures must be **detached from the main structure** and built to minimize damage to that insured structure.

The important design considerations for an appurtenant structure such as a boat house include the following:

- The roof of the appurtenant structure must be sufficiently anchored to a adequate foundation so that it will not lift off in a high-wind event.
- The appurtenant structure must be detached from the main building so that damage to the appurtenant structure does not result in damage to the main building.
- The structure must be adequate to carry the load of a boat being lifted from the water and stored during a high-wind event.

12.9.6 Storage Buildings

In order for storage buildings to survive severe events, they must be:

- elevated above the expected flood level and anchored to resist flotation, collapse, and lateral movement, or
- anchored to resist lateral movement and wet floodproofed.

In wet floodproofing, flood waters are allowed to enter a building. When this method is used, the portion of the building below the expected flood level must be constructed of flood-resistant materials so that inundation does not cause significant damage (i.e., damage requiring more than cleanup and low-cost repair, such as painting).

Regardless of the method used, the building and/or its foundation must be protected from erosion and scour.

Without sufficient anchorage, storage buildings exposed to flood forces will become large debris that can cause significant damage to nearby structures. In V zones and coastal A zones, anchoring these lightweight buildings adequately is virtually impossible; therefore, it is recommended that storage buildings not be placed in V zones and coastal A zones. When a storage building is anchored to a foundation that resists uplift, overturning, and sliding, it is still possible for portions of the building to come apart in high winds that cause pressures greater than the failed parts of the building were designed to carry. This suggests that storage buildings will fail just like houses unless an adequate load path is provided. A minimal anchoring technique for storage buildings is illustrated in Figure 12-122.

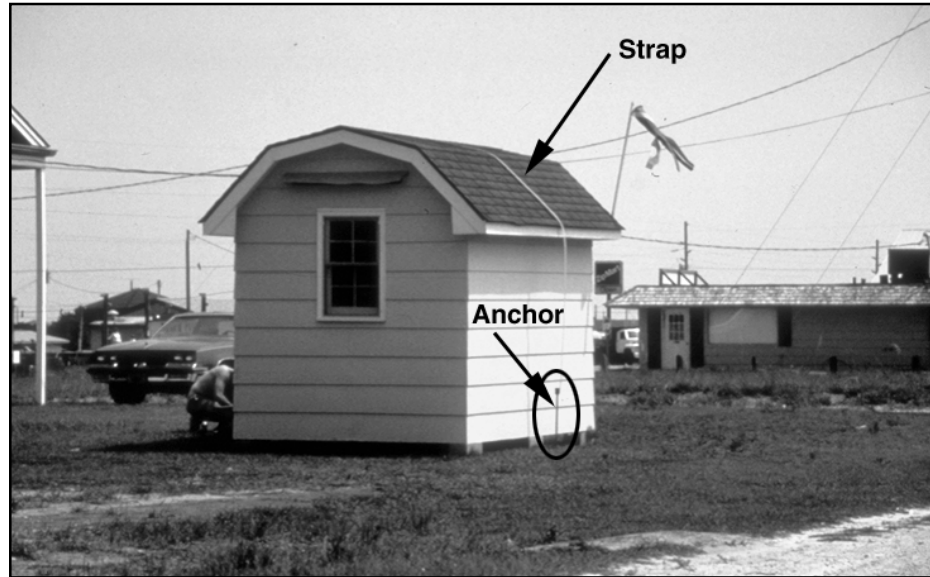


NOTE

Check with local officials to determine whether wet floodproofing is allowed by local floodplain management ordinances or laws.

Figure 12-122

Storage building anchored with strap and ground anchor. This anchoring system will resist only low-level flooding and moderate winds.



12.10 References

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