Worksheet 1. Flood Load Computation Non-Tsunami Coastal A Zones (Solid Foundation)

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

OWNER'S NAME: ______________________ PREPARED BY: ______________________

ADDRESS: ______________________ DATE: ______________________

PROPERTY LOCATION: ______________________

Constants

γ_w = specific weight of water = 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater

ρ = mass density of fluid) = 1.94 slugs/ft^3 for fresh water and 1.99 slugs/ft^3 for saltwater

g = gravitational constant = 32.2 ft/sec^2

Variables

d_s = design stillwater flood depth (ft) =

Vol = volume of floodwater displaced (ft^3) =

V = velocity (fps) =

C_{db} = breaking wave drag coefficient =

H_b = breaking wave height (ft) =

C_p = dynamic pressure coefficient =

C_s = slam coefficient =

C_d = drag coefficient =

w = width of element hit by water (ft) =

h = vertical distance (ft) wave crest extends above bottom of member =

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

W = weight of object (lb) =

C_D = depth coefficient =

C_B = blockage coefficient =

C_{Str} = building structure coefficient =

a = diameter of round foundation element =

L = horizontal length alongside building exposed to waves (ft)

Summary of Loads

F_{sta} =

F_{buoy} =

F_{brkw} =

F_s =

F_{dyn} =

F_i =

S_{max} =

S_{TOT} =

Worksheet 1. Flood Load Computation Non-Tsunami Coastal A Zones (Solid Foundation) (concluded)

Equation 8.3 Lateral Hydrostatic Load (Flood load on one side only)

$$F_{sta} = \frac{1}{2}\gamma_w d_s^2 w =$$

Equation 8.4 Vertical (Buoyancy) Hydrostatic Load

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

Equation 8.6 Breaking Wave Load on Vertical Walls

$F_{brkw} = \left(1.1C_p \gamma_w d_s^2 + 2.4\gamma_w d_s^2\right)w$ (if dry behind wall) =

or $F_{brkw} = \left(1.1C_p \gamma_w d_s^2 + 1.9\gamma_w d_s^2\right)w$ (if stillwater elevation is the same on both sides of wall) =

Equation 8.7 Wave Slam

$$F_S = \frac{1}{2}\gamma_w C_S d_s h w =$$

Equation 8.8 Hydrodynamic Load

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

Equation 8.9 Debris Load

$$F_i = W V C_D C_B C_{Str} =$$

Equation 8.10 Localized Scour Around Single Vertical Pile

$S_{max} = 2a =$

Equation 8.11 Total Localized Scour Around Vertical Piles

$S_{TOT} = 6a + 2$ ft (if grade beam and/or slab-on-grade present) =

$S_{TOT} = 6a$ (if no grade beam or slab-on-grade present) =

Equation 8.12 Total Scour Depth Around Vertical Walls and Enclosures

$S_{MAX} = 0.15L =$

Worksheet 2. Flood Load Computation Non-Tsunamic Zone V and Coastal A Zone (Open Foundation)

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

OWNER'S NAME: ______________________ PREPARED BY: ______________________

ADDRESS: ______________________ DATE: ______________________

PROPERTY LOCATION: ______________________

Constants

γ_w = specific weight of water = 62.4 lb/ft³ for fresh water and 64.0 lb/ft³ for saltwater

ρ = mass density of fluid = 1.94 slugs/ft³ for fresh water and 1.99 slugs/ft³ for saltwater

g = gravitational constant = 32.2 ft/sec²

Variables

d_s = design stillwater flood depth (ft) =

V = velocity (fps) =

C_{db} = breaking wave drag coefficient =

a, D = pile diameter (ft) =

H_b = breaking wave height (ft) =

C_p = dynamic pressure coefficient =

C_s = slam coefficient =

C_d = drag coefficient for piles =

w = width of element hit by water (ft) =

h = vertical distance (ft) wave crest extends above bottom of member =

W = debris object weight (lb) =

C_D = depth coefficient =

C_B = blockage coefficient =

C_{Str} = building structure coefficient =

L = horizontal length alongside building exposed to waves (ft) =

Summary of Loads

F_{brkp} =

F_s =

F_{dyn} =

F_i =

S_{max} =

S_{TOT} =

Equation 8.5 Breaking Wave Load on Vertical Piles

$$F_{brkp} = \frac{1}{2} C_{db} \gamma_w D H_b^2 =$$

Worksheet 2. Flood Load Computation Non-Tsunamic Zone V and Coastal A Zone (Open Foundation) (concluded)

Equation 8.7 Wave Slam
$F_S = \frac{1}{2}\gamma_w C_S d_s h w =$
Equation 8.8 Hydrodynamic Load
$F_{dyn} = \frac{1}{2}C_{dr}V^2 A =$
Equation 8.9 Debris Load
$F_i = W V C_D C_B C_{Str} =$
Equation 8.10 Localized Scour around Single Vertical Pile
$S_{max} = 2a =$
Equation 8.11 Total Localized Scour Around Vertical Piles
$S_{TOT} = 6a + 2$ ft (if grade beam and/or slab-on-grade present) =
$S_{TOT} = 6a$ (if no grade beam or slab-on-grade present) =

8.6 Tsunami Loads

In general, tsunami loads on residential buildings may be calculated in the same fashion as other flood loads; the physical processes are the same, but the scale of the flood loads is substantially different in that the wavelengths and runup elevations of tsunamis are much greater than those of waves caused by tropical and extratropical cyclones (see Section 3.2). If the tsunami acts as a rapidly rising tide, most of the damage is the result of buoyant and hydrostatic forces (see *Tsunami Engineering* [Camfield 1980]). When the tsunami forms a bore-like wave, the effect is a surge of water to the shore and the expected flood velocities are substantially higher than in non-tsunami conditions.

The tsunami velocities are very high and if realized at the greater water depths, would cause substantial damage to buildings in the path of the tsunami. Additional guidance on designing for tsunami forces including flow velocity, buoyant forces, hydrostatic forces, debris impact, and impulsive forces is provided in FEMA P646, *Guidelines for Design of Structures for Vertical Evacuation from Tsunami* (FEMA 2008b). For debris impact loads under tsunami conditions, see Section 6.5.6 of FEMA P646, which recommends an alternative to Equation 8.6 in this Manual for calculating tsunami debris impact loads.

8.7 Wind Loads

ASCE 7-10 is the state-of-the-art wind load design standard. It contains a discussion of the effects of wind pressure on a variety of building types and building elements. Design for wind loads is essentially the same whether the winds are due to hurricanes, thunderstorms, or tornadoes.

Important factors that affect wind load design pressures include:

- Location of the building site on wind speed maps

- Topographic effects (hills and escarpments), which create a wind speedup effect
- Building risk category (one- and two-family dwellings are assigned to Risk Category II; accessory structures may be assigned to Risk Category I) (see Section 6.2.1.1)
- Building height and shape
- Building enclosure category: enclosed, partially enclosed or open
- Terrain conditions, which determine building exposure category

The effects of wind on buildings can be summarized as follows:

- Windward walls and windward surfaces of steep-sloped roofs are acted on by inward-acting, or positive pressures. See Figure 8-17.
- Leeward walls and leeward surfaces of steep-sloped roofs and both windward and leeward surfaces of low-sloped roofs are acted on by outward-acting, or negative pressures. See Figure 8-17.
- Air flow separates at sharp edges and at locations where the building geometry changes.
- Localized suction, or negative, pressures at eaves, ridges, and the corners of roofs and walls are caused by turbulence and flow separation. These pressures affect loads on components and cladding (C&C) and elements of the main wind force resisting system (MWFRS).

NOTE

Basic mapped wind speeds in ASCE 7-10 for Category II structures (residential buildings) are higher than those in ASCE 7-05 because they represent ultimate wind speeds or strength-based design wind speeds. Load factors for wind in ASCE 7-10 are also different from those in ASCE 7-05. In ASCE 7-10, the wind load factor in the load combinations for LRFD strength design (LRFD) is 1.0 (but ASCE 7-05 provides a load factor of 1.6), and the ASD wind load factor in the load combinations for allowable stress design (ASD) for wind is 0.6 (but ASCE 7-05 provides a load factor of 1.0).

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

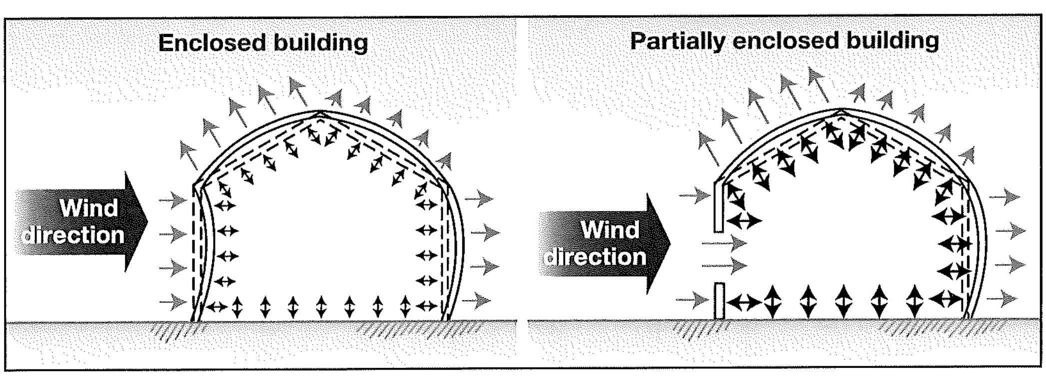


Figure 8-17.
Effect of wind on an enclosed building and a building with an opening

pressure. The magnitude of internal pressures depends on whether the building is enclosed, partially enclosed, or open, as defined in ASCE 7-10. Figure 8-17 shows the effect of wind on an enclosed and partially enclosed building.

In wind-borne debris regions (as defined in ASCE 7-10), in order for a building to be considered enclosed for design purposes, glazing must either be impact-resistant or protected with shutters or other devices that are impact-resistant. This requirement also applies to glazing in doors.

Methods of protecting glazed openings are described in ASCE 7-10 and in Chapter 11 of this Manual.

TERMINOLOGY: HURRICANE-PRONE REGIONS

In the United States and its territories, hurricane-prone areas are defined by ASCE 7-10 as (1) the U.S. Atlantic Ocean and Gulf of Mexico Coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph and (2) Hawaii, Puerto Rico, Guam, the Virgin Islands, and American Samoa.

8.7.1 Determining Wind Loads

In this Manual, design wind pressures for MWFRS are based on the results of the envelope procedure for low-rise buildings. A low-rise building is defined in ASCE 7-10. The envelope procedure in ASCE 7-10 is only one of several for determining MWFRS pressures in ASCE 7-10, but it is the procedure most commonly used for designing low-rise residential buildings. The envelope procedure for low-rise buildings is applicable for enclosed and partially enclosed buildings with a mean roof height (*h*) of less than or equal to 60 feet and where mean roof height (*h*) does not exceed the smallest horizontal building dimension.

Figure 8-18 depicts the distribution of external wall and roof pressures and internal pressures from wind. The figure also shows the mean roof height, which is defined in ASCE 7-10

FORMULA

The following formula converts ASCE 7-05 wind speeds to ASCE 7-10 Risk Category II wind speeds.

$$\text{ASCE 7-10} = (\text{ASCE 7-05})\left(\sqrt{1.6}\right)$$

For conversion from ASCE 7-10 to ASCE 7-05, use:

$$\text{ASCE 7-05} = \frac{\text{ASCE 7-10}}{\sqrt{1.6}}$$

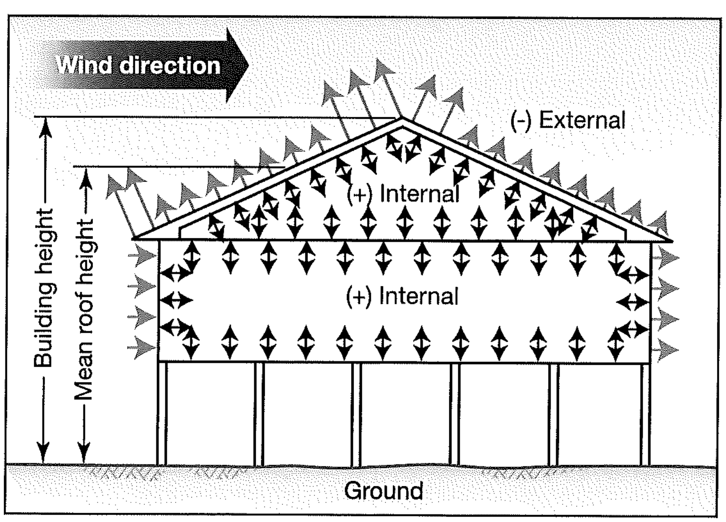


Figure 8-18.
Distribution of roof, wall, and internal pressures on one-story, pile-supported building

as "the average of the roof eave height and the height to the highest point on the roof surface ..." Mean roof height is not the same as building height, which is the distance from the ground to the highest point.

For calculating both MWFRS and C&C pressures, velocity pressures (q) should be calculated in accordance with Equation 8.13. Velocity pressure varies depending on many factors including mapped wind speed at the site, height of the structure, local topographic effects, and surrounding terrain that affects the exposure coefficient.

NOTE

ASCE 7-10 *Commentary* states that where a single component, such as a roof truss, comprises an assemblage of structural elements, the elements of that component should be analyzed for loads based on C&C coefficients, and the single component should be analyzed for loads as part of the MWFRS.

EQUATION 8.13. VELOCITY PRESSURE

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(Eq. 8.13)}$$

where:

q_z = velocity pressure evaluated at height z (psf)

K_z = velocity pressure exposure coefficient evaluated at height z

K_{zt} = topographic factor

K_d = wind directionality factor

V = basic wind speed (mph) (3-sec gust speed at 33 ft above ground in Exposure Category C)

The design wind pressure is calculated from the combination of external and internal pressures acting on a building element. This combination of pressures for both MWFRS and C&C loads in accordance with provisions of ASCE 7-10 is represented by Equation 8.14.

EQUATION 8.14. DESIGN WIND PRESSURE FOR LOW-RISE BUILDINGS

$$p = q_h \left[GC_{pf} - GC_{pi} \right] \qquad \text{(Eq. 8.14)}$$

where:

p = design wind pressure

q_h = velocity pressure evaluated at mean roof height (h) (see Figure 8-18 for an illustration of mean roof height)

GC_{pf} = external pressure coefficient for C&C loads or MWFRS loads per the low-rise building provisions, as applicable

GC_{pi} = internal pressure coefficients based on exposure classification as applicable; GC_{pi} for enclosed buildings is +/- 0.18

Figure 8-19 depicts how net suction pressures can vary across different portions of the building. Central portions of the walls represent the location of the least suction, while wall corners, the roof ridge, and the roof perimeter areas have potential for suction pressures that are 1.3, 1.4, and 2 times the central wall areas, respectively. Wall areas and roof areas that experience the largest suction pressures are shown as edge zones in Figure 8-19. The variation of pressures for different portions of the building is based on an enclosed structure (e.g., GC_{pi} = +/- 0.18) and use of external pressure coefficients of the low-rise building provisions.

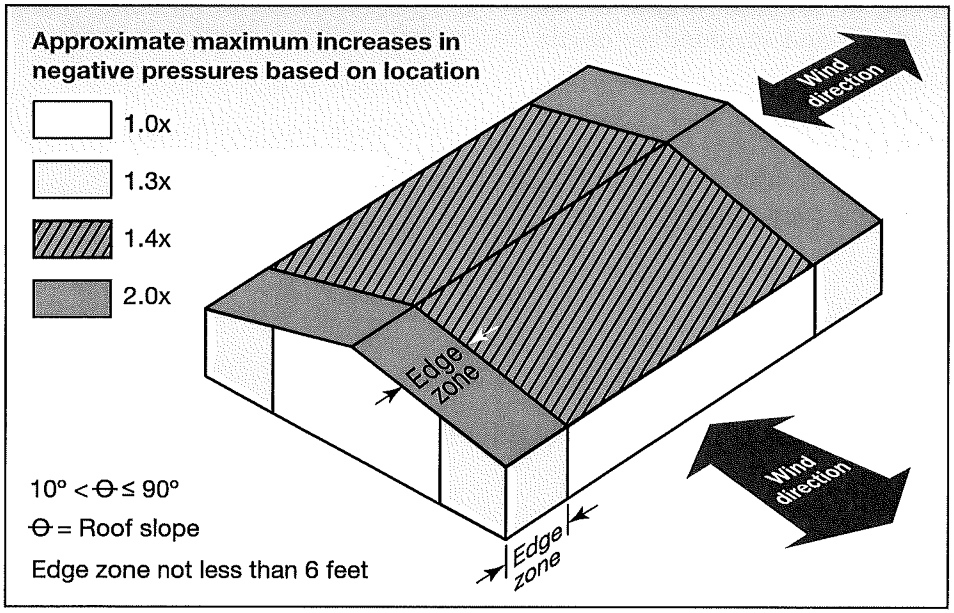


Figure 8-19.
Variation of maximum negative MWFRS pressures based on envelope procedures for low-rise buildings

To simplify design for wind, as well as establish consistency in the application of the wind design provisions of ASCE 7-10, several consensus standards with prescriptive designs tabulate maximum wind loads for the design of specific building elements based on wind pressures (both MWFRS and C&C are often referred to as "prescriptive" standards because they prescribe or tabulate load requirements for pressures) in accordance with ASCE 7-10. These standards, which are referenced in the 2012 IRC, are specific building applications based on factors such as wind speed, exposure, and height above grade. Examples of prescriptive standards for wind design that are referenced in the 2012 IRC are:

- ICC 600-2008, *Standard for Residential Construction in High-Wind Regions* (ICC 2008)
- ANSI/AF&PA, *Wood Frame Construction Manual* (WFCM) (AF&PA 2012)
- ANSI/AISI-S230, *Standard for Cold-Formed Steel Framing-Prescriptive Method for One and Two Family Dwellings* (AISI 2007)

Tabulated wind load requirements in these standards often use conservative assumptions for sizing members and connections. Therefore, load requirements are often more conservative than those developed by direct application of ASCE 7-10 pressures when design loads can be calculated for each element's unique characteristics.

8.7.2 Main Wind Force Resisting System

The MWFRS consists of the foundation; floor supports (e.g., joists, beams); columns; roof rafters or trusses; and bracing, walls, and diaphragms that assist in transferring loads. ASCE 7-10 defines the MWFRS as "... an assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind load from more than one surface." Individual MWFRS elements of shear walls and roof diaphragms (studs and cords) may also act as components and should also be analyzed under the loading requirements of C&C.

For a typical building configuration with a gable roof, the wind direction is perpendicular to the roof ridge for two cases and parallel to the ridge in the other two cases. A complete analysis of the MWFRS includes determining windward and leeward wall pressures, side wall pressures, and windward and leeward roof pressures for wind coming from each of four principal directions. Figure 8-18 depicts pressures acting on the building structure for wind in one direction only. The effect of the combination of pressures on the resulting member and connection forces is of primary interest to the designer. As a result, for each direction of wind loading, structural calculations are required to determine the maximum design forces for members and connections of the building structure.

Prescriptive standards can be used to simplify the calculation of MWFRS design loads. Examples of prescriptive MWFRS design load tables derived from the application of ASCE 7-10 wind load provisions are included in this Manual for the purpose of illustration, as follows:

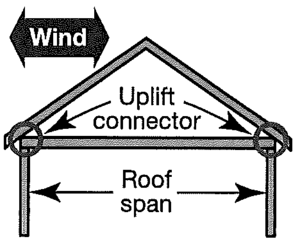
- **Roof uplift connector loads** (see Table 8-6). The application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.5. Equation 8.13 for velocity pressure and Equation 8.14 for determining design wind pressure are used to arrive at the design uplift connector load. The roof uplift connection size is based on moment balance of forces acting on both the windward and leeward side of the roof. The uplift load is used to size individual connectors and also provides the distributed wind uplift load acting at the buildings perimeter walls. Note that while wind speeds are based on 700-year Mean Recurrence Interval, the resulting uplift loads are based on ASD design.

- **Diaphragm loads due to wind acting perpendicular to the ridge** (see Table 8-7). Application of the ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.6. The diaphragm load is based on wind pressures simultaneously acting on both the windward and leeward side of the building. The diaphragm load is used to size the diaphragm for resistance to wind and is also used for estimating total lateral forces for a given wind direction based on combining diaphragm loads for the roof and wall(s) as applicable. Total lateral forces from wind for a given direction can be used for preliminary sizing of the foundation and for determining shear wall capacity requirements.

The example loads in Table 8-6 and Table 8-7, which are based on ASCE 7-10 envelope procedures for low-rise buildings, are used in Examples 8.7 and 8.8 to illustrate their application in the wind design of select load path elements. Tables 8-6 and 8-7 and Examples 8.5 and 8.6 are derived from wind load procedures in the WFCM (AF&PA 2012). Tables 8-6 and 8-7 are not intended to replace requirements of the building code or applicable reference standards for the actual design of a building to resist wind.

Table 8-6. Roof Uplift Connector Loads (Based on ASD Design) at Building Edge Zones, plf (33-ft mean roof height, Exposure C)

Roof Span (ft)	Wind Speed[a] (mph)								
	110	115	120	130	140	150	160	170	180
	Roof uplift connector load[b][c][d] (plf)								
24	189	215	241	298	358	424	494	568	647
32	237	269	303	374	451	534	622	716	816
40	285	324	364	450	544	643	750	864	985
48	333	379	426	527	636	753	879	1,012	1,154



(a) 700-year wind speed, 3-sec gust.

(b) Uplift connector loads are based on 33-ft mean roof height, Exposure C, roof dead load of 10 psf, and roof overhang length of 2 ft (see Example 8.5).

(c) Uplift connector loads are tabulated in pounds per linear ft of wall. Individual connector loads can be calculated for various spacing of connectors (e.g., for spacing of connectors at 2 ft o.c., the individual connector load would be 2 ft times the tabulated value).

(d) Tabulated uplift connector loads are conservatively based on a 20-degree roof slope. Reduced uplift forces may be calculated for greater roof slopes.

Table 8-7. Lateral Diaphragm Load from Wind Perpendicular to Ridge, plf (33-ft mean roof height, Exposure C)

Roof Span (ft)	Wind Speed[a] (mph)								
	110	115	120	130	140	150	160	170	180
	Roof diaphragm load[b][c] for 7:12 roof slope (plf)								
24	138	151	164	192	223	256	291	329	369
32	161	176	191	224	260	299	340	384	430
40	186	203	221	259	301	345	393	443	497
48	210	230	250	294	341	391	445	503	563
	Floor diaphragm load (plf)								
Any	154	168	183	214	249	286	325	367	411

(a) 700-year wind speed, 3-sec gust.

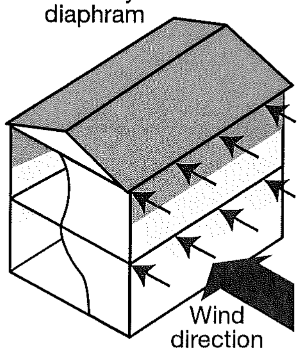
(b) Lateral diaphragm loads are based on 33-ft mean roof height, Exposure C, and wall height of 8 ft (see Example 8.6). Tabulated roof diaphragm loads are for a 7:12 roof slope. Larger loads can be calculated for steeper roof slopes and smaller loads can be calculated for shallower roof slopes.

(c) Total shear load equals the tabulated unit lateral load by the building length perpendicular to the wind direction.

Legend

Tributary area for roof diaphram

Tributary area for floor diaphram



Same figure as Example 8.6, Illustration A

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS

Given:

- Roof span of 24 ft with 2-ft overhangs
- Roof/ceiling dead load of 10 psf
- Wind load based on 150 mph, Exposure C at 33-ft mean roof height
- Building is enclosed
- K_z = 1.0 (velocity pressure exposure coefficient evaluated at height of 33 ft)
- K_{zt} = 1.0 (topographic factor)
- K_d = 0.85 (wind directionality)

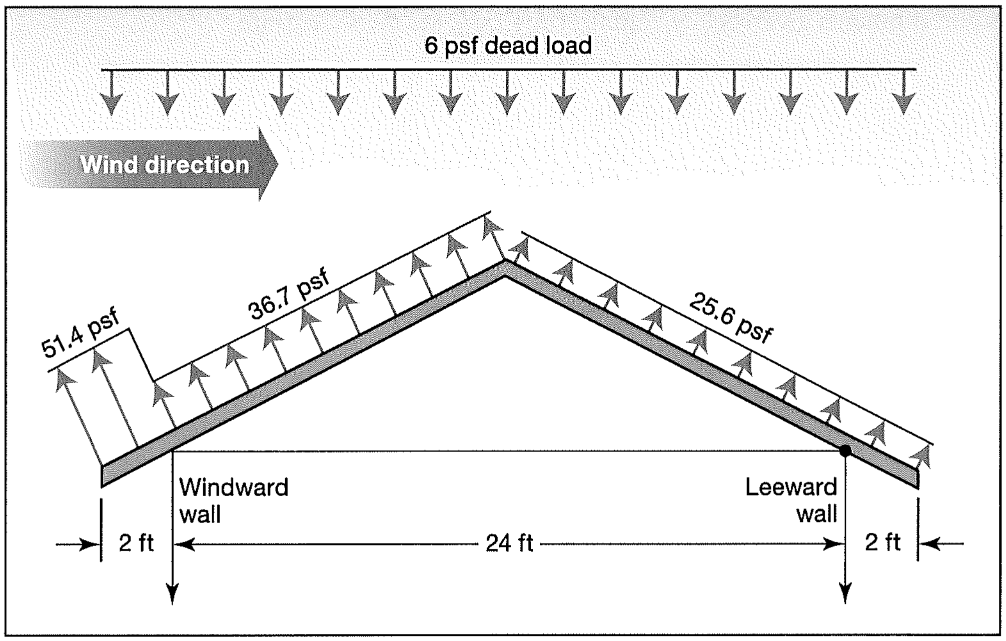


Illustration A. Roof-to-wall uplift connection loads from wind forces

Find: The roof-to-wall uplift connection load using the envelope procedure for low-rise buildings (see Figure 28.4-1 in ASCE 7-10).

Solution: The roof-to-wall uplift connection load can be found using the envelope procedure for low-rise buildings as follows:

- The velocity pressure (q) for the site conditions is determined from Equation 8.13 as follows:

$$q_h = 0.00256 K_z K_{zt} K_d V^2$$

$$q_h = 0.00256(1.0)(1.0)(0.85)(150 \text{ mph})^2$$

$$q_h = 48.96 \text{ psf}$$

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (continued)

- For ASD, multiply by the ASD wind load factor of 0.6, which comes from Load Combination 7 (See Section 8.10) 0.6D + 0.6W:

  $q_h = 48.96 \text{ psf}(0.6) = 29.38 \text{ psf}$

  The largest uplift forces occur for a roof slope of 20 degrees where wind is perpendicular to the ridge. The addition of an overhang also increases the roof-to-wall uplift connection load. For the windward overhang, a pressure coefficient of 0.68 is used based on the gust factor of 0.85 and pressure coefficient of 0.80 from ASCE 7-10. Otherwise, pressure coefficients for other elements of the roof are based on GC_{pi} = 0.18 and GC_{pf} from the edge zone coefficients shown in Figure 28.4-1 of ASCE 7-10.

Pressures and moments given below contain subscripts for their location:

- W = windward
- L = leeward
- O = overhang
- R = roof

The design wind pressure is determined from Equation 8.14 as follows:

$$p = q_h(GC_{pf} - GC_{pi})$$

$$p_{WO} = 29.38 \text{ psf}(-1.07 - 0.68) = -51.4 \text{ psf}$$

$$p_{WR} = 29.38 \text{ psf}(-1.07 - 0.18) = -36.7 \text{ psf}$$

$$p_{LR} = 29.38 \text{ psf}(-0.69 - 0.18) = -25.6 \text{ psf}$$

$$p_{LO} = 29.38 \text{ psf}(-0.69 - 0.18) = -25.6 \text{ psf}$$

- The roof/ceiling dead load is adjusted for the load case where dead load is used to resist uplift forces as follows:

  $\text{Dead load} = 10 \text{ psf}(0.6) = 6 \text{ psf}$ where 0.6 is the ASD load factor for dead load in the applicable load combination

- Wind loads on the roof have both a horizontal and vertical component. The uplift connector force, located at the windward wall, can be determined by summing moments about the leeward roof-to-wall connection and solving for the connector force that will maintain moment equilibrium. Clockwise moments are considered positive.

Moment (M) created by windward overhang pressures is solved as follows:

Vertical component, windward overhang (VWO):

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (continued)

$$M_{VWO} = [(51.4 \text{ psf} \cos(20))]\left(\frac{2 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(2 \text{ ft})](1 \text{ ft} + 24 \text{ ft}) = 2{,}270 \text{ ft-lb}$$

Horizontal component, windward overhang (*HWO*):

$$M_{HWO} = [-51.4 \text{ psf} \sin(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right)\left(-\frac{2\tan(20)}{2}\right) = 13.6 \text{ ft-lb}$$

Moment (*M*) created by windward roof pressures is solved as follows:

Vertical component, windward roof (*VWR*):

$$M_{VWR} = [(36.7 \text{ psf} \cos(20))]\left(\frac{12 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(12 \text{ ft})](18 \text{ ft}) = 6{,}631.2 \text{ ft-lb}$$

Horizontal component, windward roof (*HWR*):

$$M_{HWR} = [-36.7 \text{ psf} \sin(20)]\left(\frac{12}{\cos(20)}\right)\left(\frac{12\tan(20)}{2}\right) = -349.7 \text{ ft-lb}$$

Moment (*M*) created by leeward roof pressures is solved as follows:

Vertical component, leeward roof (*VLR*):

$$M_{VLR} = [(25.6 \text{ psf} \cos(20))]\left(\frac{12}{\cos(20)}\right) + (-6 \text{ psf})(12 \text{ ft})](6 \text{ ft}) = 1{,}411.2 \text{ ft-lb}$$

Horizontal component, leeward roof (*HLR*):

$$M_{HLR} = [25.6 \text{ psf} \sin(20)]\left(\frac{12}{\cos(20)}\right)\left(\frac{12\tan(20)}{2}\right) = 243.9 \text{ ft-lb}$$

Moment(*M*) created by leeward overhang pressures is solved as follows:

Vertical component, leeward overhang (*VLO*):

$$M_{VLO} = [(25.6 \text{ psf} \cos(20))]\left(\frac{2 \text{ ft}}{\cos(20)}\right) + (-6 \text{ psf})(2 \text{ ft})](-1 \text{ ft}) = -39.2 \text{ ft-lb}$$

Horizontal component, leeward overhang (*HLO*):

$$M_{HLO} = [25.6 \text{ psf} \sin(20)]\left(\frac{2 \text{ ft}}{\cos(20)}\right)\left(\frac{-2\tan(20)}{2}\right) = -6.8 \text{ ft-lb}$$

The total overturning moment per ft of roof width = 10,174.3 ft-lb

EXAMPLE 8.5. ROOF UPLIFT CONNECTOR LOADS (concluded)

Solving for uplift load: F_w = 10174.3 ft-lb/roof span ft = 10,174.3 ft-lb/24 ft = 424 lb

Assuming the uplift forces are calculated for a 1-ft-wide section of the roof, the unit uplift connector force can be expressed as f_w = **424 plf.**

Note: *This solution matches the information in Table 8-6.*

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE

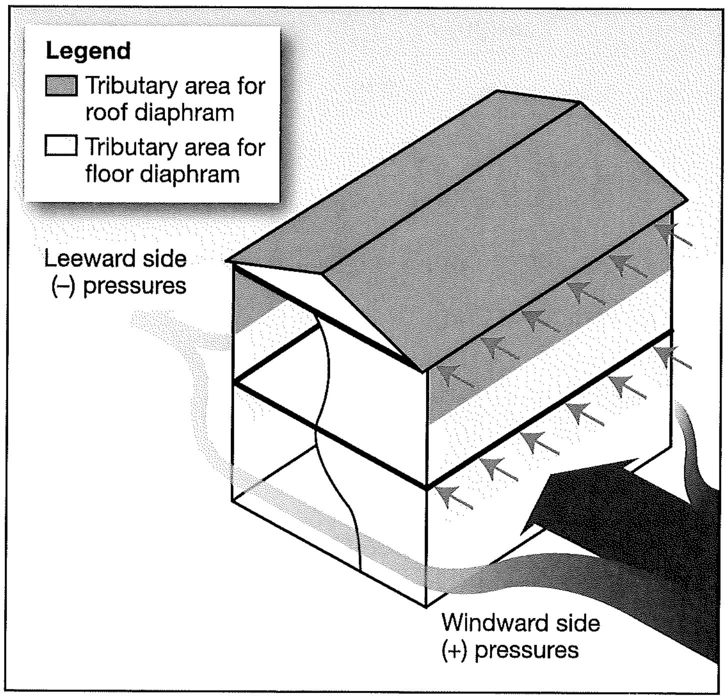


Illustration A. Lateral diaphragm loads from wind perpendicular to building ridge

Given:

- Roof span of 24 ft
- 7:12 roof pitch
- The wind load is based on 150 mph, Exposure C at 33-ft mean roof height
- The building is enclosed
- From Example 8.5, for the same site condition, the ASD velocity pressure q = 29.38 psf

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (continued)

Find: The roof diaphragm load using the envelope procedure for low-rise buildings (see Figure 28.4-1 in ASCE 7-10).

Solution: The roof diaphragm load using the envelope procedure for low-rise buildings can be found as follows:

- Lateral loads (see Illustration A) into the roof diaphragm are a function of roof slope and wall loads tributary to the roof diaphragm.
- Pressure coefficients for elements of the roof GC_{pi} and GC_{pf} are given in Table A.

Table A. Pressure Coefficients for Roof and Wall Zones

Diaphragm Zone			GC_{pi}	GC_{pf}
Roof diaphragm	Wall interior zone	Windward	0.18	0.56
		Leeward	0.18	–0.37
	Wall end zone	Windward	0.18	0.69
		Leeward	0.18	–0.48
	Roof interior zone	Windward	0.18	0.21
		Leeward	0.18	–0.43
	Roof end zone	Windward	0.18	–0.53
		Leeward	0.18	0.27
Floor diaphragm	Wall interior zone	Windward	0.18	0.53
		Leeward	0.18	–0.43
	Wall end zone	Windward	0.18	0.80
		Leeward	0.18	–0.64

- GC_{pi} is determined using the Enclosure Classification (enclosed building in this example) and Table 26.11-1 from ASCE 7-10
- GC_{pf} is determined using Figure 28.4-1 in ASCE 7-10
- Both interior zone and end zone coefficients are used to establish an average pressure on the wall and roof.

The design wind pressure is determined from Equation 8.14 ($q = q_h$ in this case) as follows:

$$p = q \mid GC_{pf} - GC_{pi} \mid$$

Step 1: Roof Diaphragm

- L = leeward
- W = windward

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (continued)

- w = wall

Wall interior zone

$p_{Ww} = 29.38 \text{ psf}(0.56 - 0.18) = 11.16 \text{ psf}$

$p_{Lw} = 29.38 \text{ psf}(-0.37 - 0.18) = -16.16 \text{ psf}$

$Sum = 11.16 \text{ psf} + |-16.16\text{psf}\ | = 27.3 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Wall end zone

$p_{Ww} = 29.38 \text{ psf}(0.69 - 0.18) = 14.98 \text{ psf}$

$p_{Lw} = 29.38 \text{ psf}(-0.48 - 0.18) = -19.39 \text{ psf}$

$Sum = 14.98 \text{ psf} + |-19.39 \text{ psf}\ | = 34.4 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Under the procedures and notes shown in Figure 28.4-1 of ASCE 7-10, end zones extend a minimum of 3 ft at each end of the wall. For long or tall walls, end zone lengths are based on 10 percent of the least horizontal dimension or 40 percent of the mean roof height, whichever is smaller, but not less than either 4 percent of the least horizontal dimension or 3 ft at each end of the wall. The end zone width where the pressures are applied is 3 ft.

The average pressure on the wall is:

$$P = \frac{[34.4 \text{ psf}(6 \text{ ft}) + 27.3 \text{ psf}(24 \text{ ft} - 6 \text{ ft})]}{24 \text{ ft}} = 29.1 \text{ psf}$$

where:

24 ft = building length assumed to be equal to the roof span for purposes of accounting for average effects of pressure differences at end zones and interior zones

Roof interior zone

$p_{Ww} = 29.38 \text{ psf}(0.21 - 0.18) = 0.88 \text{ psf}$

$p_{Lw} = 29.38 \text{ psf}(-0.43 - 0.18) = -17.92 \text{ psf}$

$Sum = 0.88 \text{ psf} + |-17.92 \text{ psf}\ | = 18.8 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Roof end zone

$p_{Ww} = 29.38 \text{ psf}(0.27 - 0.18) = 2.64 \text{ psf}$

EXAMPLE 8.6. LATERAL DIAPHRAGM LOADS FROM WIND PERPENDICULAR TO RIDGE (concluded)

$p_{Lw} = 29.38 \text{ psf}(-0.53 - 0.18) = -20.86 \text{ psf}$

$Sum = 2.64 \text{ psf} + |-20.86 \text{ psf}| = 23.5 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

The average pressure on the roof is:

$$P = \frac{23.5 \text{ psf}(6 \text{ ft}) + 18.8 \text{ psf}(24 \text{ ft} - 6 \text{ ft})}{24 \text{ ft}} = 19.98 \text{ psf}$$

The roof diaphragm will take its load plus half the load of the 8-ft-tall wall below.

$$w_{roof} = \frac{1}{2}(29.1 \text{ psf})(8 \text{ ft}) + 19.98 \text{ psf}(7 \text{ ft}) = \mathbf{256.3 \text{ plf}}$$

Step 2: Floor Diaphragm

- The floor diaphragm loads are based on the maximum MWRFS coefficients associated with a 20-degree roof slope. It is assumed that the floor diaphragm tributary area is the height of one 8-ft wall plus 1 ft to account for floor framing depth.

Wall interior zone

$p_{Ww} = 29.38 \text{ psf}(0.53 - 0.18) = 10.28 \text{ psf}$

$p_{Lw} = 29.38 \text{ psf}(-0.43 - 0.18) = -17.92 \text{ psf}$

$Sum = 10.28 \text{ psf} + |-17.92 \text{ psf}| = 28.2 \text{ psf}$ (note that leeward and windward forces are acting in the same direction)

Wall end zone

$p_{Ww} = 29.38 \text{ psf}(0.80 - 0.18) = 18.22 \text{ psf}$

$p_{Lw} = 29.38 \text{ psf}(-0.64 - 0.18) = -24.09 \text{ psf}$

$Sum = 18.22 psf + |-24.09 psf| = 42.3 psf$ (note that leeward and windward forces are acting in the same direction)

The average pressure on the wall is:

$$P = \frac{42.3 \text{ psf}(6 \text{ ft}) + 28.2 \text{ psf}(24 \text{ ft} - 6 \text{ ft})}{24 \text{ ft}} = 31.73 \text{ psf}$$

The floor diaphragm load is based on a 9-ft tributary height obtained from adding the height of one 8-ft wall plus 1 ft to account for the floor framing depth.

$$w_{floor} = 31.73 \text{ psf}(9 \text{ ft}) = \mathbf{286 \text{ plf}}$$

Note: *This solution matches the information in Table 8-7.*

8.7.3 Components and Cladding

ASCE 7-10 defines components and cladding (C&C) as "... elements of the building envelope that do not qualify as part of the MWFRS." These elements include roof sheathing, roof coverings, exterior siding, windows, doors, soffits, fascia, and chimneys. The design and installation of the roof sheathing attachment may be the most critical consideration because the attachment location for the sheathing is where the uplift load path begins (load path is described more fully in Chapter 9 of this Manual).

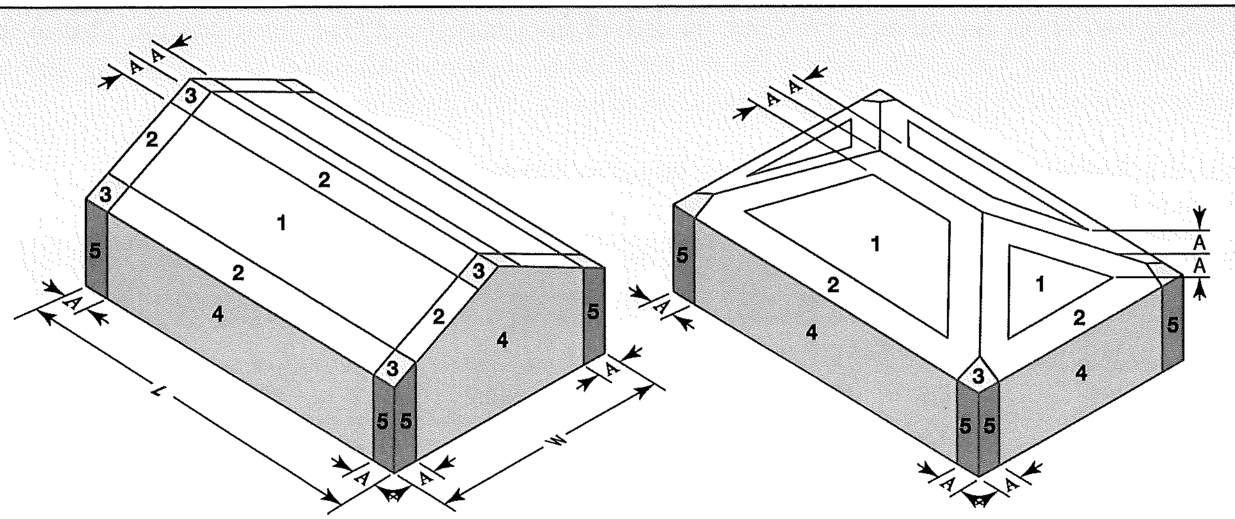
C&C pressures are determined for various "zones" of the building. ASCE 7-10 includes illustrations of those zones for both roofs and walls. Illustrations for gable, monoslope, and hip roof shapes are presented. The pressure coefficients vary according to roof pitch (from 0 degrees to 45 degrees) and effective wind area (defined in ASCE 7-10).

C&C loads act on all elements exposed to wind. These elements and their attachments must be designed to resist these forces to prevent failure that could lead to breach of the building envelope and create sources of flying debris. Examples of building elements and their connections subject to C&C loads include the following:

- Exterior siding
- Roof sheathing
- Roof framing
- Wall sheathing
- Wall framing (e.g., studs, headers)
- Wall framing connections (e.g., stud-to-plate, header-to-stud)
- Roof coverings
- Soffits and overhangs
- Windows and window frames
- Skylights
- Doors and door frames, including garage doors
- Wind-borne debris protection systems
- Any attachments to the building (e.g., antennas, chimneys, roof and ridge vents, roof turbines)

Furthermore, individual MWFRS elements of shear walls and roof diaphragms (studs and chords) may also act as components and should also be analyzed under the loading requirements of C&C.

Figure 8-20 shows the locations of varying localized pressures on wall and roof surfaces. The magnitude of roof uplift and wall suction pressures is based on the most conservative wind pressures in each location for given roof types and slopes in accordance with Figure 30.4 of ASCE 7-10. As noted previously, prescriptive



NOTE

Edge zone dimension, A, is measured as the horizontal projection on the building roof and walls.

A = the smaller of 10 percent of the least horizontal dimension of the building (i.e. either L or W) or 40 percent of the mean roof height (MRH), but not less than either 4 percent of the least horizontal dimension or 3 feet.

L = length

W = width

Figure 8-20.
Components and cladding wind pressures

standards can be used to simplify the calculation of C&C design loads. Examples of prescriptive C&C design load tables for purposes of illustration are included in this Manual as follows:

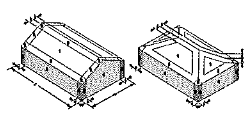
- **Roof and wall suction pressures** (see Table 8-8). Application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.7. Suction pressures are used to size connections between sheathing and framing and to size the sheathing material itself for wind induced bending. In ASCE 7-10, there is no adjustment for effective wind areas less than 10 square feet; therefore, sheathing suction loads are based on an effective wind area of 10 square feet.

- **Lateral connector loads from wind and building end zones** (see Table 8-9). Application of ASCE 7-10 provisions and typical assumptions used to derive the tabulated load values are addressed in Example 8.8. Lateral connector loads from wind are used to size the connection from wall stud-to-plate, wall plate-to-floor, and wall plate-to-roof connections to ensure that higher C&C loads acting over smaller wall areas can be adequately resisted and transferred into the roof or floor diaphragm. In ASCE 7-10, the effective wind area for a member is calculated as the span length times an effective width of not less than one-third the span length. For example, the effective area for analysis is calculated as $h^2/3$ where h represents the span (or height) of the wall stud.

Example load tables and example problems are derived from more wind load procedures provided in the WFCM-2012 load Tables 8-8 and 8-9 are not intended to replace requirements of the building code or reference standard for the actual design of C&C attachments for a building.

Table 8-8. Roof and Wall Sheathing Suction Loads (based on ASD design), psf (33-ft mean roof height, Exposure C)

Sheathing Location	Wind Speed[a] (mph) 110	115	120	130	140	150	160	170	180
	Roof, suction pressure[b][c] (psf)								
Zone 1	18.6	20.4	22.2	26.0	30.2	34.7	39.4	44.5	44.9
Zone 2	31.3	34.2	37.2	43.7	50.7	58.2	66.2	74.7	83.8
Zone 2 Overhang	34.8	38.0	41.4	48.5	56.3	64.6	73.5	83.0	93.1
Zone 3	47.1	51.5	56.0	65.8	76.3	87.5	99.6	112.4	126.1
Zone 3 Overhang	58.5	63.9	69.6	81.6	94.7	108.7	123.7	139.6	156.5
	Wall, suction pressure[b][c] (psf)								
Zone 4	20.2	22.1	24.1	28.2	32.8	37.6	42.8	48.3	54.1
Zone 5	25.0	27.3	29.7	34.9	40.4	46.4	52.8	59.6	66.8



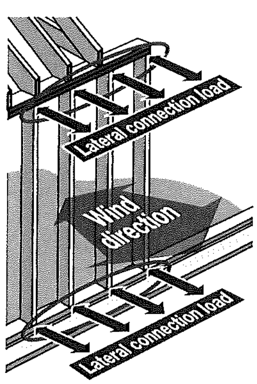
(a) 700-year wind speed, 3-sec gust.

(b) Roof and wall sheathing suction loads are based on 33-ft mean roof height and Exposure C (see Example 8.7).

(c) Loads based on minimum effective area of 10 ft^2

Table 8-9. Lateral Connector Loads from Wind at Building End Zones (Based on ASD Design), plf (33-ft mean roof height, Exposure C)

Wall Height (ft)	Wind Speed[a] (mph) 110	115	120	130	140	150	160	170	180
	Lateral connector loads[b][c][d] for wall zone 5 (plf)								
8	92	101	110	129	150	172	196	221	248
10	110	120	131	154	179	205	233	263	295
12	127	139	151	177	206	236	269	303	340
14	143	156	170	200	231	266	302	341	383
16	158	173	188	221	256	294	335	378	423



(a) 700-year wind speed, 3-sec gust.

(b) Lateral connector loads are based on 33-ft mean roof height and Exposure C (see Example 8.8).

(c) Lateral connector loads are tabulated in pounds per linear ft of wall. Individual connector loads can be calculated for various spacing of connectors (e.g., for spacing of connectors at 2 ft o.c., the individual connector load would be 2 ft times the tabulated value).

(d) Loads based on minimum area of (wall height)$^{2/3}$

EXAMPLE 8.7. ROOF SHEATHING SUCTION LOADS

Given:

- The wind load is based on 150 mph, Exposure C at 33-ft mean roof height
- The building is enclosed
- From Example 8.4, for the same site condition, the ASD velocity pressure q = 29.38 psf
- The internal pressure coefficient for roof and wall sheathing is GC_{pi}= 0.18

Find: Roof sheathing and wall sheathing suction loads using the C&C coefficients specified in Figure 30.4 of ASCE 7-10.

For cladding and fasteners, the effective wind area should not be greater than the area that is tributary to an individual fastener. In ASCE 7-10, there is no adjustment for wind areas less than 10 ft^2; therefore, sheathing suction loads are based on an effective wind area of 10 ft^2 for different zones on the roof.

Solution: The roof sheathing and wall sheathing suction loads can be determined using the C&C coefficients specified in Figure 30.4 of ASCE 7-10, as follows:

- The design wind pressure is determined from Equation 8.14 (where $q = q_h$ in this case) as follows:

  $$p = q \,|\, GC_{pf} - GC_{pi} \,|$$

- Determine the roof sheathing suction load pressure coefficients using Figure 30.4 of ASCE 7-10 as follows:

Step 1: Roof sheathing suction loads pressure coefficients

Pressure coefficient equations developed from C&C, Figure 30.4, graphs of ASCE 7-10 coefficients:

$$\text{Zone 1: } GC_{pf} = 0.9 - 0.1\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -1.0$$

$$\text{Zone 2: } GC_{pf} = -1.1 - 0.7\left[\frac{\log\left(\frac{A}{100}\right)}{\text{og}\left(\frac{10}{100}\right)}\right] = -1.8$$

$$\text{Zone 2 Overhang: } GC_{pf} = -2.2$$

EXAMPLE 8.7. ROOF SHEATHING SUCTION LOADS (concluded)

$$\text{Zone 3: } GC_{pf} = -1.7 - 1.1\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -2.8$$

$$\text{Zone 3 Overhang: } GC_{pf} = 2.5 - 1.2\left[\frac{\log\left(\frac{A}{100}\right)}{\log\left(\frac{10}{100}\right)}\right] = -3.7$$

Step 2: Wall sheathing suction loads pressure coefficient

Pressure coefficient equations developed from C&C, Figure 30.4 graphs of ASCE 7-10 coefficients:

$$\text{Zone 4: } GC_{pf} = -0.8 - 0.3\left[\frac{\log\left(\frac{A}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.1$$

$$\text{Zone 5: } GC_{pf} = -0.8 - 0.6\left[\frac{\log\left(\frac{A}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.4$$

Step 3: For all zones – internal pressure coefficient

GC_{pi} = +/-0.18

Step 4: Calculate roof sheathing and wall sheathing suction pressures for all zones using Equation 8.14

Zone 1: $p = 29.38\text{ psf}(-1 - 0.18) = \mathbf{-34.7\text{ psf}}$

Zone 2: $p = 29.38\text{ psf}(-1.8 - 0.18) = \mathbf{-58.2\text{ psf}}$

Zone 2 Overhang: $p = 29.38\text{ psf}(-2.2) = \mathbf{-64.6\text{ psf}}$

Zone 3: $p = 29.38\text{ psf}(-2.8 - 0.18) = \mathbf{-87.5\text{ psf}}$

Zone 3 Overhang: $p = 29.38\text{ psf}(-3.7) = \mathbf{-108.7\text{ psf}}$

Zone 4: $p = 29.38\text{ psf}(-1.1 - 0.18) = \mathbf{-37.6\text{ psf}}$

Zone 5: $p = 29.38\text{ psf}(-1.4 - 0.18) = \mathbf{-46.4\text{ psf}}$

Note: *This solution matches the information in Table 8-8.*

EXAMPLE 8.8. LATERAL CONNECTION FRAMING LOADS FROM WIND

Given:

- Wind load is based on 150 mph, Exposure C at 33-ft mean roof height, and wall and diaphragm framing as shown in Illustration A
- Building is enclosed
- Wall height is 10 ft
- Stud spacing is 16 in. o.c.
- Sheathing effective area is 10 ft^2
- ASD velocity pressure q = 29.38 psf (from Example 8.5)
- Wall suction equations for Zone 4 and Zone 5 are provided in Example 8.7
- Internal pressure coefficient for wall sheathing is GC_{pi} = +/- 0.18

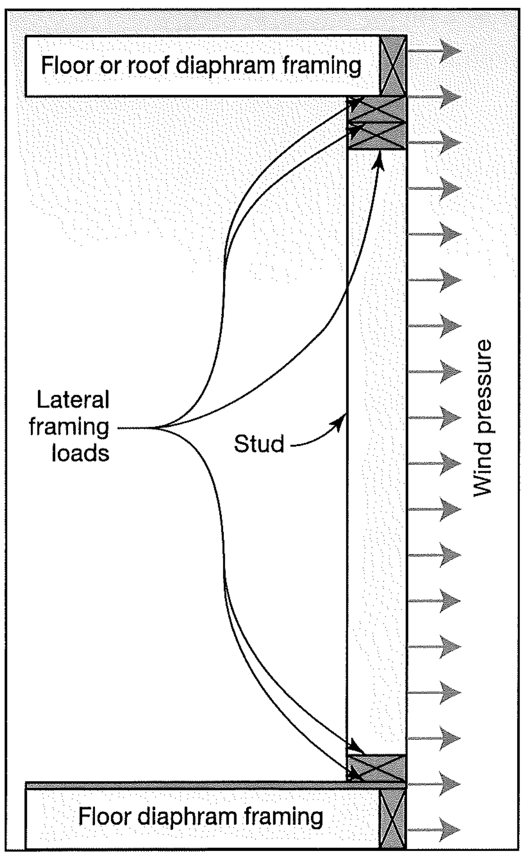


Illustration A. Lateral connector loads for wall-to-roof and wall-to-floor connections

Find: Framing connection requirements at the top and base of the wall.

Solution: The connector load can be determined as follows:

EXAMPLE 8.8. LATERAL CONNECTION FRAMING LOADS FROM WIND (concluded)

- C&C coefficients are used
- When determining C&C pressure coefficients, the effective wind area equals the tributary area of the framing members
- For long and narrow tributary areas, the area width may be increased to one-third the framing member span to account for actual load distributions
- The larger area results in lower average wind pressures
- The increase in width for long and narrow tributary areas applies only to calculation of wind pressure coefficients
- Determine the tributary area and pressure coefficient GC_{pf} for the wall sheathing: Stud effective wind area equals 13.33 ft^2. The minimum required area for analysis is $h^2/3$=33.3 ft^2, where h is 10 ft
- In accordance with ASCE 7-10, the pressure coefficient, GC_{pf}, for wall sheathing can be determined based on a minimum effective wind area of 33.3 ft^2 as follows:

$$\text{Zone 5: } GC_{pf} = -0.8 - 0.6\left[\frac{\log\left(\frac{33.3}{500}\right)}{\log\left(\frac{10}{500}\right)}\right] = -1.22$$

The design wind pressure is determined as follows from Equation 8.14: $p = q(GC_{pf} - GC_{pi})$

$$\text{Zone 5: } p = 29.38 \text{ psf}(-1.22 - 0.18) = -41.13 \text{ psf}$$

The required capacity of connectors assuming load is based on half the wall height:

$$\text{Zone 5: } w = 41.13 \text{ psf}\left(\frac{10 \text{ ft}}{2}\right) = \mathbf{205\ plf}$$

Note: *This solution matches the information in Table 8-9.*

8.8 Tornado Loads

Tornadoes have wind speeds that vary based on the magnitude of the event; more severe tornadoes have wind speeds that are significantly greater than the minimum design wind speeds required by the building code. Designing an entire building to resist tornado-force winds of EF3 or greater based on the Enhanced Fujita tornado damage scale (in EF2 tornadoes, large trees are snapped or uprooted) may be beyond the realm

WARNING

Safe rooms should be located outside known flood-prone areas, including the 500-year floodplain, and away from any potential large debris sources. See Figure 5-2 of FEMA 320 for more direction regarding recommended siting for a safe room.

of practicality and cost-effectiveness, but this does not mean that solutions that provide life-safety protection cannot be achieved while maintaining cost-effectiveness.

A more practical approach is to construct an interior room or space that is "hardened" to resist not only tornado-force winds but also the impact of wind-borne missiles. FEMA guidance on safe rooms can be found in FEMA 320, *Taking Shelter from the Storm: Building a Safe Room for Your Home or Small Business* (FEMA 2008c), which provides prescriptive design solutions for safe rooms of up to 14 feet x 14 feet. These solutions can be incorporated into a structure or constructed as a nearby stand-alone safe room to provide occupants with a place of near-absolute protection. The designs in FEMA 320 are based on wind pressure calculations that are described in FEMA 361, *Design and Construction Guidance for Community Safe Rooms* (FEMA 2008a). FEMA 361 focuses on larger community safe rooms, but the process of design and many of the variables are the same for smaller residential safe rooms.

An additional reference, ANSI/ICC 500-2008 complements the information in FEMA 320 and FEMA 361 and is referenced in the 2012 IBC and 2012 IRC.

Safe rooms can be designed to resist both tornado and hurricane hazards, and though many residents of coastal areas are more concerned with hurricanes, tornadoes can be as prevalent in coastal areas as they are in inland areas such as Oklahoma, Kansas, and Missouri. Constructing to minimum requirements of the building code does not include the protection of life-safety or property of occupants from a direct hit of large tornado events. Safe rooms are not recommended in flood hazard areas.

8.9 Seismic Loads

This Manual uses the seismic provisions of ASCE 7-10 to illustrate a method for calculating the seismic base shear. To simplify design, the effect of dynamic seismic ground motion accelerations can be considered an equivalent static lateral force applied to the building. The magnitude of dynamic motion, and therefore the magnitude of the equivalent static design force, depends on the building characteristics, and the spectral response acceleration parameter at the specific site location.

The structural configuration in Figure 8-21 is called an "inverted pendulum" or "cantilevered column" system. This configuration occurs in elevated pile-supported buildings where almost all of the weight is at the top of the piles. For a timber frame cantilever column system, ASCE 7-10 assigns a response modification factor (R) equal to 1.5 (e.g., R = 1.5). For wood frame, wood structural panel shear walls, ASCE 7-10 assigns an R factor equal to 6.5. The R factor of 1.5 can be conservatively used to determine shear for the design of all elements and connections of the structure. An R factor of 1.5 is not permitted for use in Seismic Design Categories E and F per ASCE 7-10.

ASCE 7-10 contains procedures for the seismic design of structures with different structural systems stacked vertically within a single structure. Rules for vertical combinations can be applied to enable the base of the structure to be designed for shear forces associated with R=1.5 and the upper wood frame, wood structural panel shear wall structure to be designed for reduced shear forces associated with R=6.5.

ASCE 7-10 also provides R factors for cantilever column systems using steel and concrete columns. A small reduction in shear forces for steel piles or concrete columns could be obtained by using what ASCE 7-10 calls a "steel special cantilever column system" or a "special reinforced concrete moment frame," both of which have an R = 2.5. However, these systems call for additional calculations, connection design, and

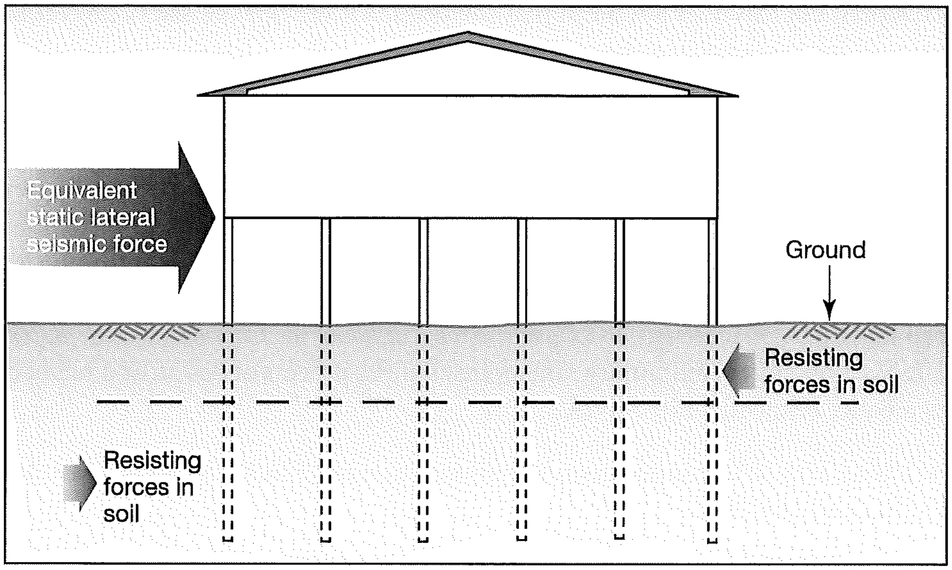


Figure 8-21. Effect of seismic forces on supporting piles

detailing, which are not commonly done for low-rise residential buildings. An engineer experienced in seismic design should be retained for this work, and builders should expect larger pile and column sizes and more reinforcing than is normally be required in a low-seismic area.

Total seismic base shear can be calculated using the Equivalent Lateral Force (ELF) procedure of ASCE 7-10 in accordance with Equation 8.15.

EQUATION 8.15. SEISMIC BASE SHEAR BY EQUIVALENT LATERAL FORCE PROCEDURE

$$V = C_s W \quad \text{(Eq. 8.15a)}$$

$$C_s = \frac{S_{DS}}{(R/I)} \quad \text{(Eq. 8.15b)}$$

where:

V = seismic base shear

C_s = seismic response coefficient

S_{DS} = design spectral response acceleration parameter in the short period range

R = response modification factor

I = occupancy importance factor

W = effective seismic weight

Lateral seismic forces are distributed vertically through the structure in accordance with Equation 8.16, taken from ASCE 7-10.

EQUATION 8.16. VERTICAL DISTRIBUTION OF SEISMIC FORCES

$$F_x = C_{vx} V \quad \text{(Eq. 8.16a)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(Eq. 8.16b)}$$

where:

F_x = lateral seismic force induced at any level

C_{vx} = vertical distribution factor

V = seismic base shear

w_i *and* w_x = portion of the total effective seismic weight of the structure (w) located or assigned to level i or x

h_i *and* h_x = height from the base to Level i or x

k = exponent related to the structure period; for structures having a period of 0.5 sec or less, k=1

The calculated seismic force at each story must be distributed into the building frame. The horizontal shear forces and related overturning moments are taken into the foundation through a load path of horizontal floor and roof diaphragms, shear walls, and their connections to supporting structural elements. A complete seismic analysis includes evaluating the structure for vertical and plan irregularities, designing elements and their connections in accordance with special seismic detailing, and considering structural system drift criteria. Example 8.9 illustrates the use of basic seismic calculations.

EXAMPLE 8.9. SEISMIC LOAD

Given:

- S_{DS} for the site = $2/3F_aS_s$, which is determined to be 2/3(1.2)(0.50) = 0.4
- The building structure as shown in Illustration A. Dead load for the building is as follows:

  Roof and ceiling = 10 lb/ft^2

  Exterior walls = 10 lb/ft^2

  Interior Walls = 8 lb/ft^2

  Floor = 10 lb/ft^2

  Piles = 409 lb each

EXAMPLE 8.9. SEISMIC LOAD (continued)

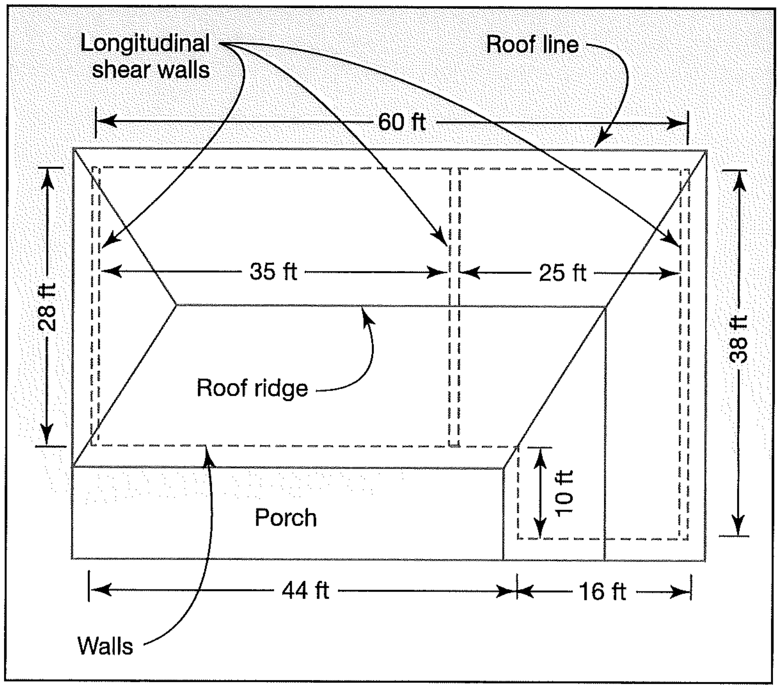


Illustration A. Building elevation and plan view of roof showing longitudinal shearwalls; dimensions are wall-to-wall and do not include the 2-ft roof overhang

Find (using ASCE 7-10 ELF procedure):

1. The total shear wall force
2. The shear force at the top of the pile foundation

Solution for #1: The total shear wall force using the ASCE 7-10 ELF procedure can be determined as follows:

- Calculate effective seismic weight:

  Roof/ceiling: (10 lb/ft^2)(2,390 ft^2) = 23,900 lb

  Exterior walls: (10 lb/ft^2)(1,960 ft^2) = 19,600 lb

  Interior partitions: (8 lb/ft^2)(2,000 ft^2) = 16,000 lb

  Floor = (10 lb/ft^2)(2,160 ft^2) = 21,600 lb

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

  Total effective seismic weight:
  W = 23,900 + 19,600 lb + 16,000 lb + 21,600 lb + 12,679 lb = 93,454 lb

EXAMPLE 8.9. SEISMIC LOAD (continued)

- Seismic forces are distributed vertically as follows:

  Roof level:
  Effective seismic weight, $w_{x\,roof}$ = 23,900 lb + (0.5)(19,600) lb + (0.5)(16,000/2) lb = 41,700 lb

  Height from base, $h_{x\,roof}$ = 18 ft

  $w_{x\,roof}(h_{x\,roof})$ = 750,600 ft-lb

  Floor level:
  Effective seismic weight, $w_{x\,floor}$ = 19,600/2 lb + 16,000/2 lb + 21,600 lb + 12,679 lb = 52,079 lb

  Height from base: $h_{x\,floor}$ = 8 ft

  $w_{x\,floor}(h_{x\,floor})$ = (52,079 lb)(8 ft) = 416,632 ft-lb

  $$C_{vx,roof} = \frac{750{,}600 \text{ ft-lb}}{750{,}600 \text{ ft-lb} + 416{,}632 \text{ ft-lb}} = 0.64 \text{ from Equation 8.16}$$

  $$C_{vx,floor} = \frac{416{,}632 \text{ ft-lb}}{750{,}000 \text{ ft-lb} + 416{,}632 \text{ ft-lb}} = 0.36 \text{ from Equation 8.16}$$

  The force in the shear walls and at the top of the piles will vary by the R factor for the shear wall system and the pile system (e.g., cantilevered column system).

- Lateral seismic force at the roof level for design of wood-frame shear walls (R = 6.5):

  $$F_{x\,roof} = C_{vx\,roof}V = C_{vx\,roof}\frac{S_{DS}}{R/I}W \text{ using Equation 8.15 for } V \text{ substituted into Equation 8.16}$$

  $$F_{x,roof} = \left[\frac{(0.64)(0.4)}{\frac{6.5}{1.0}}\right](93{,}454 \text{ lb}) = 3{,}681 \text{ lb}$$

where:

R = 6.5 for light-frame walls with plywood

I = 1.0 for residential structure

The design shear force for the shear walls is based on the lateral seismic force at the roof level. Total seismic force for shear wall design is **3,681 lb**

Solution for #2: The shear force to the top of the pile foundation (i.e., cantilevered column system, R = 1.5) can be determined as follows:

- Roof level

  $$F_{x\,roof} = C_{vx\,roof}V = C_{vx\,roof}\frac{S_{DS}}{R/I}W \text{ using Equation 8.15 for } V \text{ substituted into Equation 8.16}$$

EXAMPLE 8.9. SEISMIC LOAD (concluded)

$$F_{x,roof} = \left[\frac{\frac{(0.64)(0.4)}{1.5}}{1.0}\right](93{,}454\ \text{lb}) = 15{,}949\ \text{lb}$$

- Floor level

$F_{x\ floor} = C_{vx\ floor} V = C_{vx\ floor} \dfrac{S_{DS}}{R/I} W$ using Equation 8.15 for V substituted into Equation 8.16

$$F_{x,floor} = \left[\frac{\frac{(0.36)(0.4)}{1.5}}{1.0}\right](93{,}454\ \text{lb}) = 8{,}972\ \text{lb}$$

where:

R = 1.5 for cantilevered column system. For vertically mixed seismic-force-resisting systems, ASCE 7-10 allows a lower R to be used below a higher R value.

I = 1.0 for a residential structure

Total shear at the floor is based on the sum of the force at the roof level and the floor level:

F_{floor} = 15,949 lb + 8,972 lb = **24,921 lb**

8.10 Load Combinations

It is possible for more than one type of natural hazard to occur at the same time. Floods can occur at the same time as a high-wind event, which happens during most hurricanes. Heavy rain, high winds, and flooding conditions can occur simultaneously. ASCE 7-10 addresses the various load combination possibilities.

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

D = dead load

L = live load

E = earthquake load

F = load due to fluids with well-defined pressures and maximum heights (e.g., fluid load in tank)

F_a = flood load

H = loads due to weight and lateral pressures of soil and water in soil

L_r = roof live load

S = snow load

R = rain load

T = self-straining force

W = wind load

Loads combined using the ASD method are considered to act in the following combinations for buildings in Zone V and Coastal A Zone (Section 2.4.1 of ASCE 7-10), whichever produces the most unfavorable effect on the building or building element:

Combination No. 1: D

Combination No. 2: $D + L$

Combination No. 3: $D + (Lr \text{ or } S \text{ or } R)$

Combination No. 4: $D + 0.75L + 0.75(Lr \text{ or } S \text{ or } R)$

Combination No. 5: $D + (0.6W \text{ or } 0.7E)$

Combination No. 6a: $D + 0.75L + 0.75(0.6W) + 0.75(Lr \text{ or } S \text{ or } R)$

Combination No. 6b: $D + 0.75L + 0.75(0.7E) + 0.75S$

Combination No. 7: $0.6D + 0.6W$

Combination No. 8: $0.6D + 0.7E$

When a structure is located in a flood zone, the following load combinations should be considered in addition to the basic combinations in Section 2.4.1 of ASCE 7-10:

- In Zone V or Coastal A Zone, $1.5F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6
- In the portion of Zone A landward of the LiMWA, $0.75F_a$ should be added to combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6.

The ASCE 7-10 *Commentary* states "Wind and earthquake loads need not be assumed to act simultaneously. However, the most unfavorable effects of each should be considered in design, where appropriate."

The designer is cautioned that F is intended for fluid loads in tanks, not hydrostatic loads. F_a should be used for all flood loads, including hydrostatic loads, and should include the various components of flood loads as recommended in Section 8.5.11 in this chapter. It is important to note that wind and seismic loads acting on a building produce effects in both the vertical and horizontal directions. The load combinations discussed in this section must be evaluated carefully, with consideration given to whether a component of the wind or seismic load acts in the same vertical or horizontal direction as other loads in the combination. In some cases, gravity loads (dead and live loads) may counteract the effect of the wind or seismic load, either vertically or horizontally. Building elements submerged in water have a reduced effective weight due to buoyancy. Example 8.10 illustrates the use of load combinations for determining design loads.

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM

Given:

Use the flood loads from Example 8.3:

- $F_{sta} = 0$
- $F_{dyn} = 909$ lb
- $F_{brkp} = 625$ lb
- $F_i = 2{,}440$ lb
- $d_s = 4.6$ ft

Use for wind loads:

- Roof span = 28 ft
- Roof pitch = 7:12
- Wall height = 10 ft
- Wind uplift load = 33,913 lb (pre-factored with 0.6)
- Exposure Category D (multiply Exposure C wind loads by 1.18 at 33 ft mean roof height)
  - 1.18 is a conservative value because while the higher Exposure Category D has been factored, the lower roof height (24 ft versus 33 ft) has not. Refer to ASCE 7-10, Figure 28.6-1 for guidance.

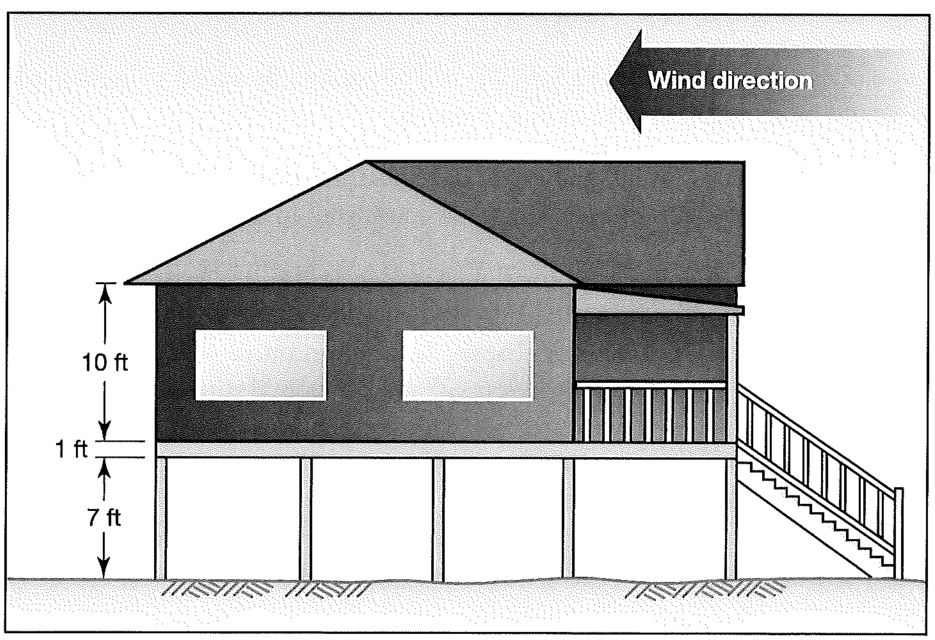


Illustration A. Side view of building

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

Use for dead load:

- 95,090 lb for house and piles

Use for buoyancy load:

- 9,663 lb

The locations given in Illustration B for the forces.

Find:

1. Calculate maximum horizontal wind load that occurs perpendicular to the ridge and the floor for the example building
2. Find the horizontal load required for foundation design
3. Calculate global overturning moment due to horizontal loads and wind uplift (see Illustration B)

Solution for #1: To determine the horizontal wind load perpendicular to the ridge, use the projected area method as follows:

- For wind perpendicular to the ridge of a roof with a span of 28 ft (using Table 8-7), 7:12 roof pitch and wall height of 10 ft, Category D as shown in Illustration A, the lateral roof diaphragm load, w_{roof}, can be found by interpolation between the 24 ft and 32 ft roof span w_{roof} values in Table 8.7:

  w_{roof} = (256 plf + 299 plf)(0.5) = 278 plf

To adjust w_{roof} for Exposure Category D due to the fact Table 8-7 assumes Exposure Category C:

$$w_{roof} = 1.18(278 \text{ plf}) = 328 \text{ plf}$$

where

1.18 = Exposure D adjustment factor (33 ft mean roof height)

To adjust w_{roof} for a wall height of 10 ft due to the fact Table 8.7 assumes a wall height of 8 ft:

$$w_{roof} = (328 \text{ plf})\left(\frac{10 \text{ ft}}{8 \text{ ft}}\right) = 410 \text{ plf}$$

- Determine lateral floor diaphragm load, w_{floor} from Table 8-7. Once more, this value needs to be adjusted for Exposure Category D from the assumed Exposure Category C:

  $$w_{floor} = 1.18(286 \text{ plf}) = 338 \text{ plf}$$

where

1.18 = Exposure D adjustment factor (33 ft mean roof height)

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

To adjust w_{roof} for a wall height of 10 ft due to the fact Table 8.7 assumes a wall height of 8 ft:

$$w_{floor} = (338\text{ plf})\left(\frac{10\text{ ft}}{8\text{ ft}}\right) = 423\text{ plf}$$

Finally, adjust this value to account for the reference case in Table 8-7 assuming the lateral floor diaphragm load is from wind pressures on the lower half of the wall above and the upper half of the wall below the floor diaphragm. Because the structure is open below the floor diaphragm level, adjust w_{floor} to account for the presence of only half of the wall area used in the reference case for Table 8-7 (e.g., structure is open below first floor diaphragm):

$$w_{floor} = 0.5(423\text{ plf}) = 212\text{ plf}$$

- For building length = 60 ft, total horizontal shear at the top of the foundation is:

  $W_{foundation}$ = (410 plf + 212 plf)(60 ft) = **37,320 lb**

Solution for #2: The horizontal load required for foundation design, can be determined using the following calculations of the load combination equations given in Section 8.10:

- Zone V and Coastal A Zone

  5. $D + 0.6W + 1.5F_a$

  6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) + 1.5F_a$

  6b. $D + 0.75L + 0.75S + 1.5F_a$

  7. $0.6D + 0.6W + 1.5F_a$

  Load combination No. 5 produces the maximum shear at the foundation for the loads considered. This load combination includes a wind load factor adjustment of 0.6. Because the value of $W_{foundation}$ from Solution #1 has already been adjusted by 0.6 for ASD, it will not be further adjusted in the calculations that follow.

  For flood load, the value of F_a is determined in accordance with Table 8-5. The hydrodynamic load is greater than breaking wave load, therefore, F_a for an individual pile and the foundation as a whole (i.e., global) is calculated as:

  $F_{a(individual)} = F_i + F_{dyn}$ = 2,440 lb + 909 lb = 3,349 lb

  $F_{a(global)}$ = (1 pile)F_i + (35 piles)F_{dyn} = 34,255 lb

  5. Total shear: $37,320\text{ lb} + 1.5(34,255\text{ lb}) = \mathbf{88,703\text{ lb}}$

- Portion of Zone A landward of the LiMWA

  5. $D + 0.6W + 0.75F_a$

  6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) + 0.75F_a$

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (continued)

6b. $D + 0.75L + 0.75S + 0.75F_a$

7. $0.6D + 0.6W + 0.75F_a$

Load combination 5 produces the maximum shear at the foundation for the loads considered.

5. Total shear: $37{,}320 + 0.75(34{,}255 \text{ lb}) = \mathbf{63{,}011\ lb}$

Note: *Considering seismic force from Example 8.8, ASD shear force at the foundation is determined by load combination No. 8:*

8. Total seismic base shear = 0.7(24,921 lb) = **17,444 lb**

Solution for #3: To determine the factored global overturning moment due to the factored loads on the building, take the moments about the pivot point in Illustration B.

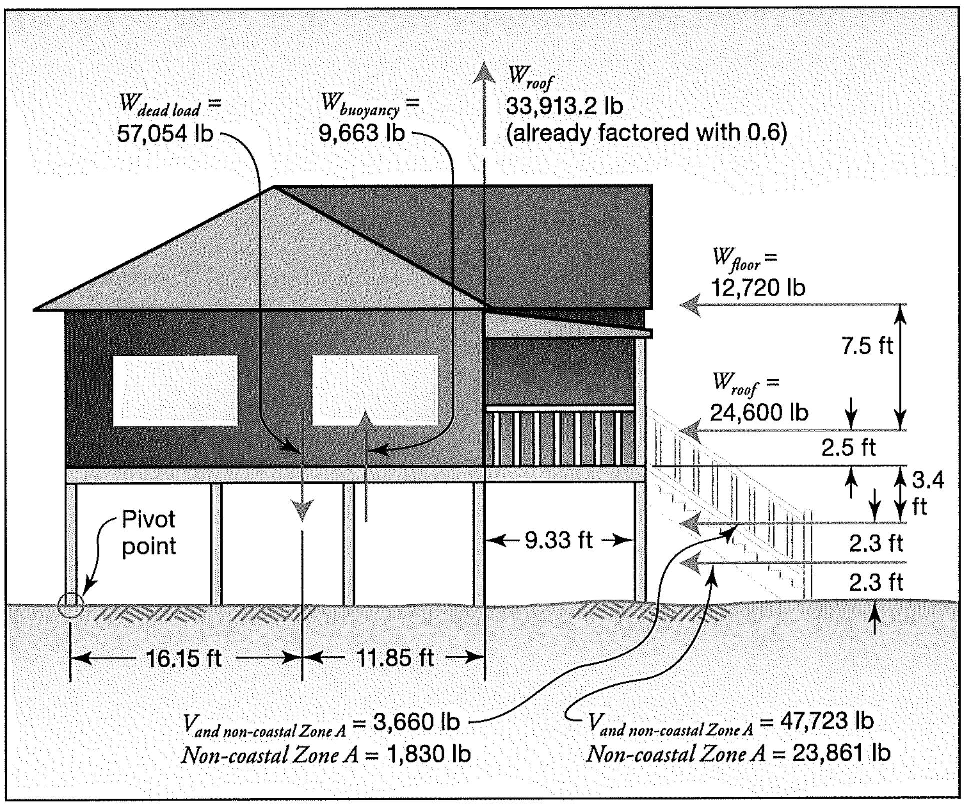


Illustration B. Loads on building for global overturning moment calculation

Load Combination 7 produces the maximum overturning at the foundation for the loads considered. Factored global overturning moment can be calculated from the factored loads and their location of application as shown in Illustration B.

- Zone V and Coastal A Zone

EXAMPLE 8.10. LOAD COMBINATION EXAMPLE PROBLEM (concluded)

7. $0.6D + 0.6W + 1.5F_a$ gives the appropriate factors to be used in calculating the factored global overturning moment

From Illustration B:

$$M_{global} = (0.6)w_{roof}(18\text{ ft}) + (0.6)w_{floor}(10.5\text{ ft}) + (1.5)F_i(4.6\text{ ft}) + (1.5)F_{dyn}(2.3\text{ ft}) + W_{uplift}(28\text{ ft}) - (0.6)DL(16.15\text{ ft}) + (1.5)F_b(19\text{ ft})$$

$$M_{global} = (0.6)(24{,}600\text{ lb})(18\text{ ft}) + (0.6)(12{,}720\text{ lb})(10.5\text{ ft}) + (1.5)(2{,}440\text{ lb})(4.6\text{ ft}) + (1.5)(31{,}815\text{ lb})(2.3\text{ ft}) + (33{,}913\text{ lb})(28\text{ ft}) - (0.6)(95{,}090\text{ lb})(16.15\text{ ft}) + (1.5)(9{,}663\text{ lb})(19\text{ ft}) = \mathbf{776{,}000\text{ ft-lb}}$$

- The portion of Zone A landward of the LiMWA

7. $0.6D + 0.6W + 0.75F_a$ gives the appropriate factors to be used in calculating the factored global overturning moment

From Illustration B:

$$M_{global} = (0.6)w_{roof}(18\text{ ft}) + (0.6)w_{floor}(10.5\text{ ft}) + (0.75)F_i(4.6\text{ ft}) + (0.75)F_{dyn}(2.3\text{ ft}) + W_{uplift}(28\text{ ft}) - (0.6)DL(16.15\text{ ft}) + (.75)F_b(19\text{ ft})$$

$$M_{global} = (0.6)(24{,}600\text{ lb})(18\text{ ft}) + (0.6)(12{,}720\text{ lb})(10.5\text{ ft}) + (0.75)(2{,}440\text{ lb})(4.6\text{ ft}) + (0.75)(31{,}815\text{ lb})(2.3\text{ ft}) + (33{,}913\text{ lb})(28\text{ ft}) - (0.6)(95{,}090\text{ lb})(16.15\text{ ft}) + (0.75)(9{,}663\text{ lb})(19\text{ ft}) = \mathbf{575{,}000\text{ ft-lb}}$$

Note: *In this example, the required uplift capacity to resist overturning is estimated by evaluating the skin friction capacity of the piles. The total pile uplift capacity is approximately 908,000 ft-lb. which exceeds both calculated overturning moments and is based on the horizontal distance to each row of piles from the pivot point and the following assumptions:*

- Pile embedment: 19.33 ft
- Pile size: 10 in.
- Coefficient of friction: 0.4 for wood piles
- Density of sand: 128 lb/ft^3
- Coefficient of lateral pressure: 09.5
- Critical depth fir sand: 15 ft
- Angle of internal friction: 38°
- Scour depth: 5 ft
- Factor of safety: 2

The following worksheet can be used to facilitate load combination computations.

Worksheet 3. Load Combination Computation

Load Combination Computation Worksheet

OWNER'S NAME: ______________________ PREPARED BY: ______________________

ADDRESS: ______________________ DATE: ______________________

PROPERTY LOCATION: ______________________

Variables

D (dead load) =

E (earthquake load) =

L (live load) =

F (fluid load) =

F_a (flood load) =

H (lateral soil and water in soil load) =

L_r (roof live load) =

S (snow load) =

R (rain load) =

T (self-straining force) =

W (wind load) =

Summary of Load Combinations:

1.

2.

3.

4.

5.

6a.

6b.

7.

8.

Combination No. 1

$D =$

Combination No. 2

$D + L =$

Combination No. 3

$D + (L_r \text{ or } S \text{ or } R) =$

Worksheet 3. Load Combination Computation (concluded)

Combination No. 4
$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) =$
Combination No. 5
$D + (0.6W \text{ or } 0.7E) =$
Combination No. 6a
$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R) =$
Combination No. 6b
$D + 0.75L + 0.75(0.7E) + 0.75S =$
Combination No. 7
$0.6D + 0.6W =$
Combination No. 8
$0.6D + 0.7E =$
When a structure is located in a flood zone, the following load combinations should be considered in addition to the basic combinations:
- In Zone V or Coastal A Zone, $1.5F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6.
- In the portion of Zone A landward of the LiMWA, $0.75F_a$ should be added to load combinations Nos. 5, 6, and 7, and E should be set equal to zero in Nos. 5 and 6.

8.11 References

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9 Designing the Building

This chapter provides guidance on design considerations for buildings in coastal environments. The topics discussed in this chapter are developing a load path through elements of the building structure, considerations for selecting building materials, requirements for breakaway walls, and considerations for designing appurtenances. Examples of problems for the development of the load path for specific building elements are provided, as well as guidance on requirements for breakaway walls, selection of building materials, and appurtenances.

CROSS REFERENCE

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

9.1 Continuous Load Path

In hazard-resistant construction, the ability of the elements of a building, from the roof to the foundation, to carry or resist loads is critical. Loads include lateral and uplift loads. A critical aspect of hazard-resistant construction is the capability of a building or structure to carry and resist all loads—including lateral and uplift loads—from the roof, walls, and other elements to the foundation and into the ground. The term "continuous load path" refers to the structural condition required to resist loads acting on a building. A load path can be thought of as a chain running through the building. A building may contain hundreds of continuous load paths. The continuous load path starts at the point or surface where loads are applied, moves through the building, continues through the foundation, and terminates where the loads are transferred to the soils that support the building. Because all applied loads must be transferred to the foundation, the load path must connect to the foundation. To be effective, each link in the load path chain must be strong enough to transfer loads without breaking.

Buildings that lack strong and continuous load paths may fail when exposed to forces from coastal hazards, thus causing a breach in the building envelope or the collapse of the building. The ability of a building to resist these forces depends largely on whether the building's construction provides a continuous load path and materials that are appropriate for the harsh coastal environment. The history of storm damage is replete with instances of failures in load paths. Figures 9-1 through 9-5 show instances of load path failure.

Figure 9-1.
Load path failure at gable end, Hurricane Andrew (Dade County, FL, 1992)

Figure 9-2.
Load path failure in connection between home and its foundation, Hurricane Fran (North Carolina, 1996)

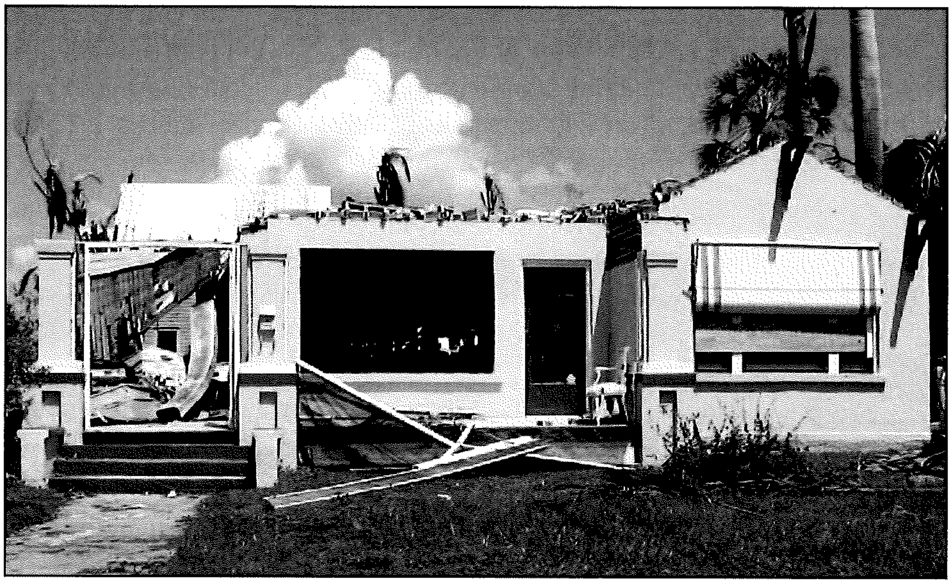


Figure 9-3.
Roof framing damage and loss due to load path failure at top of wall/roof structure connection, Hurricane Charley (Punta Gorda, FL, 2004)

Figure 9-4.
Load path failure in connections between roof decking and roof framing, Hurricane Charley (Punta Gorda, FL, 2004)

Most load path failures have been observed to occur at connections as opposed to the failure of an individual structural member (e.g. roof rafter or wall stud). Improvements in codes, design, and materials over the past decade have resulted in improved performance of structural systems. As the structural systems perform better, other issues related to load path—such as building envelope issues—become apparent.

CROSS REFERENCE

For a discussion of building envelope issues, see Chapter 11 of this Manual.

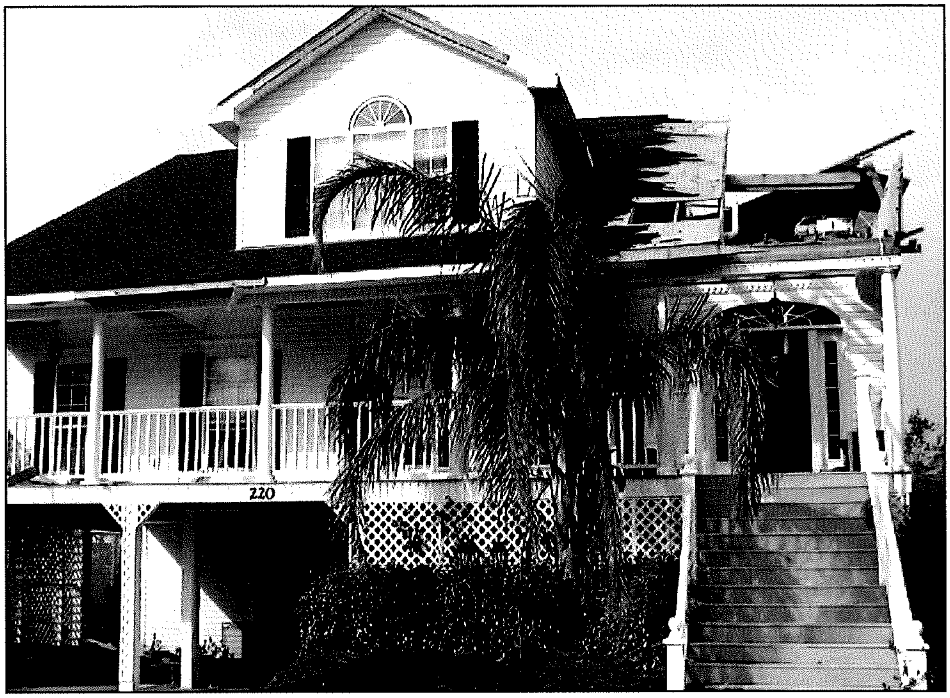


Figure 9-5.
Newer home damaged from internal pressurization and inadequate connections, Hurricane Katrina (Pass Christian, MS, 2005)

Load path guidance in this chapter is focused primarily on elements of the building structure, excluding foundation elements. Foundation elements are addressed in Chapter 10. Examples are provided primarily to illustrate how the load path resists wind uplift forces, but a complete building design includes a consideration of numerous other forces on the load path, including those from gravity loads and lateral loads. The examples illustrate important concepts and best practices in accordance with building codes and standards but do not represent an exhaustive collection of load calculation methods. See the applicable building code, standard, or design manual for more detailed guidance.

Figure 9-6 shows a load path for wind uplift beginning with the connection of roof sheathing to roof framing and ending with the resistance of the pile to wind uplift. Links #1 through #8 in the figure show connections that have been observed during investigations after high-wind events to be vulnerable to localized failure. However, the load path does not end with the resistance of the pile to wind uplift. The end of transfer through the load path occurs when the loads from the building are transferred into the soil (see Chapter 10 for information about the interaction of foundations and soils). Adequately sizing and detailing every link is important for overall performance because even a small localized failure can lead to a progressive failure of the building structure. The links shown in Figure 9-6 are discussed in more detail in Sections 9.1.1 through 9.1.8. For additional illustration of the concept of load path, see Fact Sheet 4.1, *Load Paths*, in FEMA P-499 (FEMA 2010b).

9.1.1 Roof Sheathing to Framing Connection (Link #1)

Link #1 is the nailed connection of the roof sheathing to the roof framing (see Figures 9-6 and 9-7). Design considerations include ensuring the connection has adequate strength to resist both the withdrawal of the nail shank from the roof framing and the sheathing's pulling over the head of the fastener (also referred to as "head pull-through"). Because of the potential for head pull-through and the required minimum nailing for diaphragm shear capacity, fastener spacing is typically not increased even where shank withdrawal strength

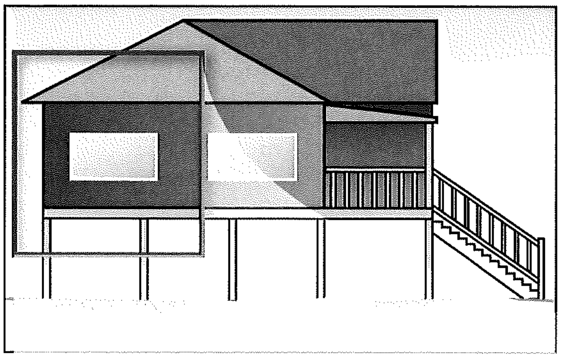
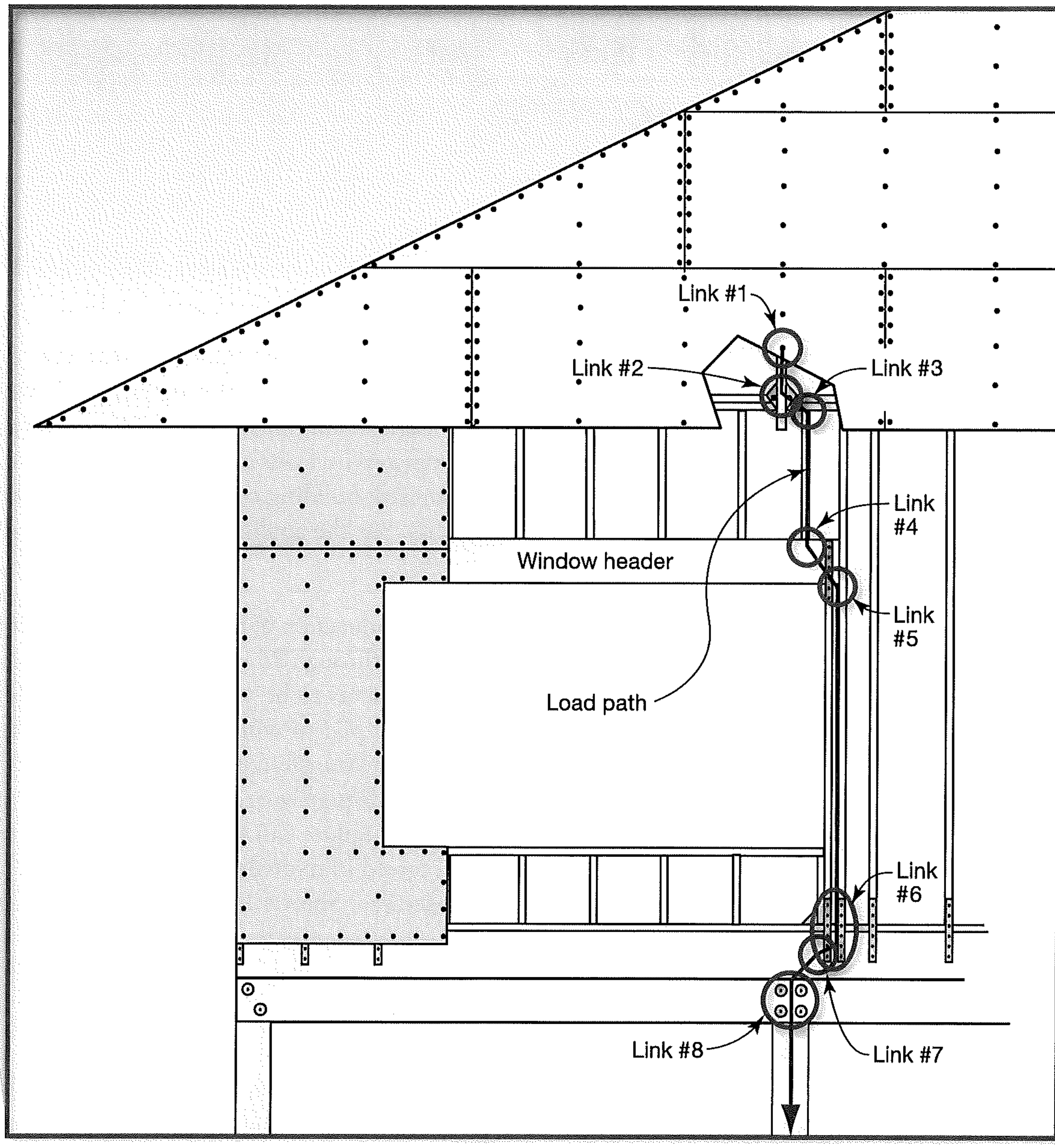


Figure 9-6.
Example load path for case study building



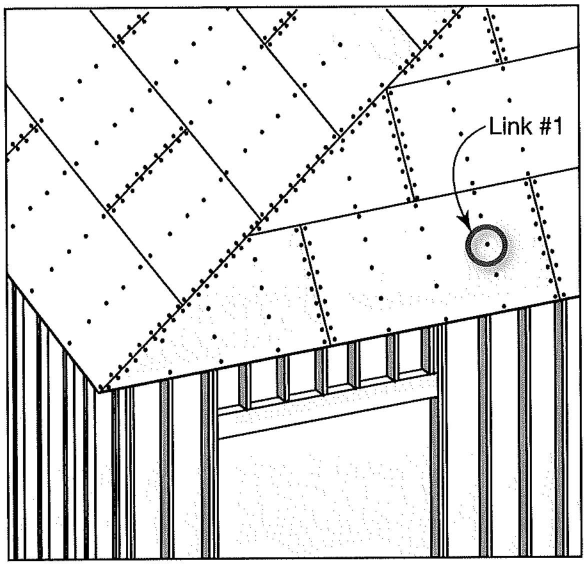


Figure 9-7. Connection of the roof sheathing to the roof framing (Link #1)

is significantly greater than that provided by a smooth shank nail. Additional strength can be added by using ring shank nails, also called deformed shank nails. The grooves and ridges along the shank act as wedges, giving the nail more withdrawal strength than a typical smooth shank nail.

Fastener attachment requirements for roof sheathing to roof framing are available in building codes and design standards and are presented in terms of nailing schedules dependent on nail diameter and length, framing spacing, specific gravity of framing lumber, and wind speed. Common assumptions for calculating nailing schedules to resist wind uplift are provided in Example 9.1. Minimum roof sheathing attachment prescribed in building codes and reference prescriptive standards is 6 inches o.c. at panel edges and 12 inches o.c. in the field of the panel.

EXAMPLE 9.1. ROOF SHEATHING NAIL SPACING FOR WIND UPLIFT

Given:

- Refer to Figure 9-7
- Wind speed = 150 mph (700-year wind speed, 3-sec gust), Exposure Category D
- Roof sheathing = 7/16-in. oriented strand board (OSB)
- Roof framing specific gravity, G = 0.42
- 8d common nail has withdrawal capacity of 66 lb/nail per the NDS

Find:

1. Nail spacing for the perimeter edge zone for rafter spacing of 16 in. o.c.
2. Nail spacing for the perimeter edge zone for rafter spacing of 24 in. o.c.

EXAMPLE 9.1. ROOF SHEATHING NAIL SPACING FOR WIND UPLIFT (continued)

Solution for #1: The following calculations are used to determine the nail spacing:

- From Table 8-8, the maximum wind suction pressure (based on ASD design) is:

  p = 108.7 psf acting normal to the roof surface (Zone 3 overhang) for Exposure Category C

  The maximum wind suction pressure for Exposure D is:

  p = 108.7 psf (1.18)=128.3 psf

where:

1.18 = the adjustment factor from Exposure C to Exposure D at 33-ft mean roof height (see Example 8.10)

- The assumed minimum tributary area for calculation of this pressure is 10 ft^2 in accordance with Example 8.7
- For framing at 16 in. o.c., roof suction loads in plf are:

$$P = 128.3\ \text{psf}\ \frac{16\ \text{in.}}{12\ \text{in./ft}} = 171.0\ \text{plf}$$

- Nail spacing:

$$\text{Spacing} = \frac{66\ \text{lb/nail}}{171.0\ \text{plf}} = 0.386\ \text{ft} = 4.6\ \text{in.}$$

  Rounding down to next typical spacing value, specify **4-in. spacing**

Solution for #2: The following calculations are used to determine the nail spacing:

- From Table 8-8, the maximum wind suction pressure is:

  p = 108.7 psf acting normal to the roof surface (Zone 3 overhang) for Exposure Category C

  See Figure 8-18 and Table 8-8.

The maximum wind suction pressure for Exposure D is:

  p = 108.7 psf (1.18)=128.3 psf

where:

1.18 = adjustment factor from Exposure C to Exposure D at 33-ft mean roof height (see Example 8.10).

- The assumed minimum tributary area for calculation of this pressure is 10 ft^2 in accordance with Example 8.7.

EXAMPLE 9.1. ROOF SHEATHING NAIL SPACING FOR WIND UPLIFT (concluded)

- For framing at 24 in. o.c., roof suction loads on a plf basis is:

$$P = 128.3 \text{ psf} \frac{24 \text{ in.}}{12 \text{ in./ft}} = 256.5 \text{ plf}$$

- Nail spacing:

$$\text{Spacing} = \frac{66 \text{ lb/nail}}{256.5 \text{ plf}} = 0.26 \text{ ft} = 3.09 \text{ in.}$$

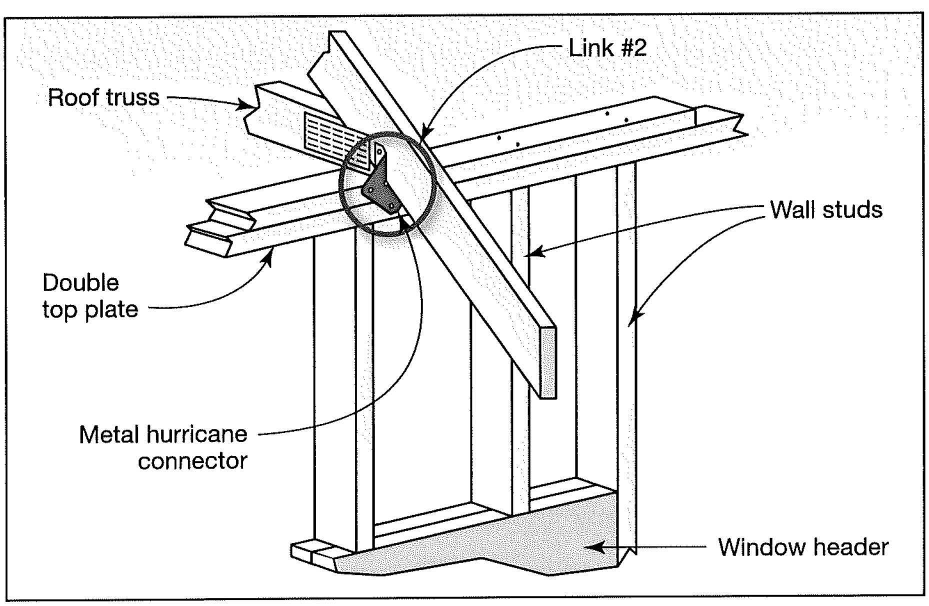
Rounding down to next typical spacing value, specify **3-in. spacing**

Note: *Edge zone nail spacing associated with Zone 3 OH pressures is conservative for other edge zone locations. Although increased nail spacing may be calculated for an edge zone away from the building corners, it is recommended that the same nailing schedule be used throughout all edge zones.*

9.1.2 Roof Framing to Exterior Wall (Link #2)

Link #2 is the connection between the roof framing member (truss or rafter) and the top of the wall below (see Figures 9-6 and 9-8) for resistance to wind uplift. Metal connectors are typically used where uplift forces are large. A variety of metal connectors are available for attaching roof framing to the wall. Manufacturers' literature should be consulted for proper use of the connector and allowable capacities for resistance to uplift. Prescriptive solutions for the connection of the roof framing to the wall top plates are available in building codes and wind design standards. One method of sizing the connection between the roof framing and the exterior wall is provided in Example 9.2.

Figure 9-8. Connection of roof framing to exterior wall (Link #2)



EXAMPLE 9.2. ROOF-TO-WALL CONNECTION FOR UPLIFT

Given:

- Refer to Figure 9-8 and Illustration A
- Wind speed = 150 mph, Exposure D
- Mean roof height = 24 ft
- Rafter spacing = 24 in. o.c.
- Hip rafter span = 14 ft
- Roof pitch = 7:12
- Roof dead load = 10 psf
- Wall height = 10 ft

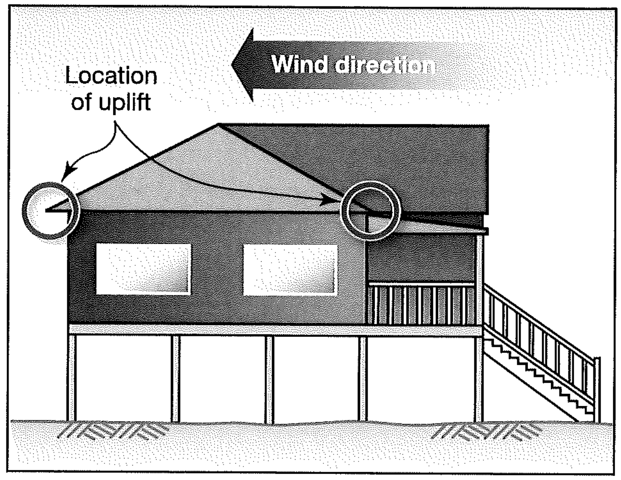


Illustration A. Location of uplift connection on hip roof

Find:

Determine the required connector size for wind uplift using prescriptive tables for wind uplift loads (i.e., find the uplift and lateral loads for the connector).

Solution: The required connector size using wind uplift prescriptive tables can be determined as follows:

Uplift

- For this example, the maximum hip rafter span = 14 ft
- To use Table 8-6, the uplift strap connector load should be obtained for a 28-ft roof width (e.g., 28 ft is 2 times the 14-ft maximum hip rafter span; see the note at the end of this Example)
- Interpolating between the 24-ft and 32-ft roof span uplift strap connector loads for 150 mph wind speed in Exposure C is:

$$\frac{(424 \text{ plf} + 534 \text{ plf})}{2} = 479 \text{ plf}$$

Adjust to Exposure Category D by multiplying by 1.18 (see Example 8.10)

$$1.18(479 \text{ plf}) = 565.2 \text{ plf}$$

- For rafter framing at 2 ft on center, the uplift connector force is:

$$(565.2 \text{ plf})(2 \text{ ft}) = \mathbf{1{,}131\ lb}$$

Lateral

- The lateral load on the connector is = 205 plf (see Table 8-9) for Exposure Category C
- Adjusting for Exposure Category D

EXAMPLE 9.2. ROOF-TO-WALL CONNECTION FOR UPLIFT (concluded)

- 1.18(205 plf) = 241.9 plf for rafter framing at 2 ft o.c., the lateral connector force at each rafter is:

  (241.9 plf)(2 ft) = **484 lb**

Note: *Although the connector forces shown in Table 8-9 assume a gable roof, requirements can be conservatively applied for attaching the hip rafter to the wall. See Table 2.5A, Wood Frame Construction Manual for One- and Two-Family Dwellings (AF&PA 2012). Note that the example roof uses both a gable roof and hip roof framing. For simplicity, the same rafter connection is often used at each connection between the rafter and wall framing. In addition, although smaller forces are developed in shorter hip roof rafter members, the same connector is typically used at all hip rafters.*

Figure 9-9 shows truss-to-wood wall connections made with metal connectors. Figure 9-10 shows a rafter-to-masonry wall connector that is embedded into the concrete-filled or grouted masonry cell.

Figure 9-9.
Connection of truss to wood-frame wall

9.1.3 Wall Top Plate to Wall Studs (Link #3)

Link #3 is the connection between the wall top plates and the wall stud over the window header (see Figures 9-6 and 9-11). The connection provides resistance to the same uplift force as used for the roof framing to the exterior connection minus the weight of the top plates. An option for maintaining the uplift load path is the use of metal connectors between the top plates and wall studs or wood structural panel sheathing (see Figure 9-12). The uplift load path can be made with wood structural panel wall sheathing, particularly when the uplift and shear forces in the wall are not very high. Guidance on using wood structural panel wall sheathing for resisting wind uplift is provided in ANSI/AF&PA SDPWS-08. The lateral load path (e.g., out-of-plane wall loads) is maintained by stud-to-top plate nailing.

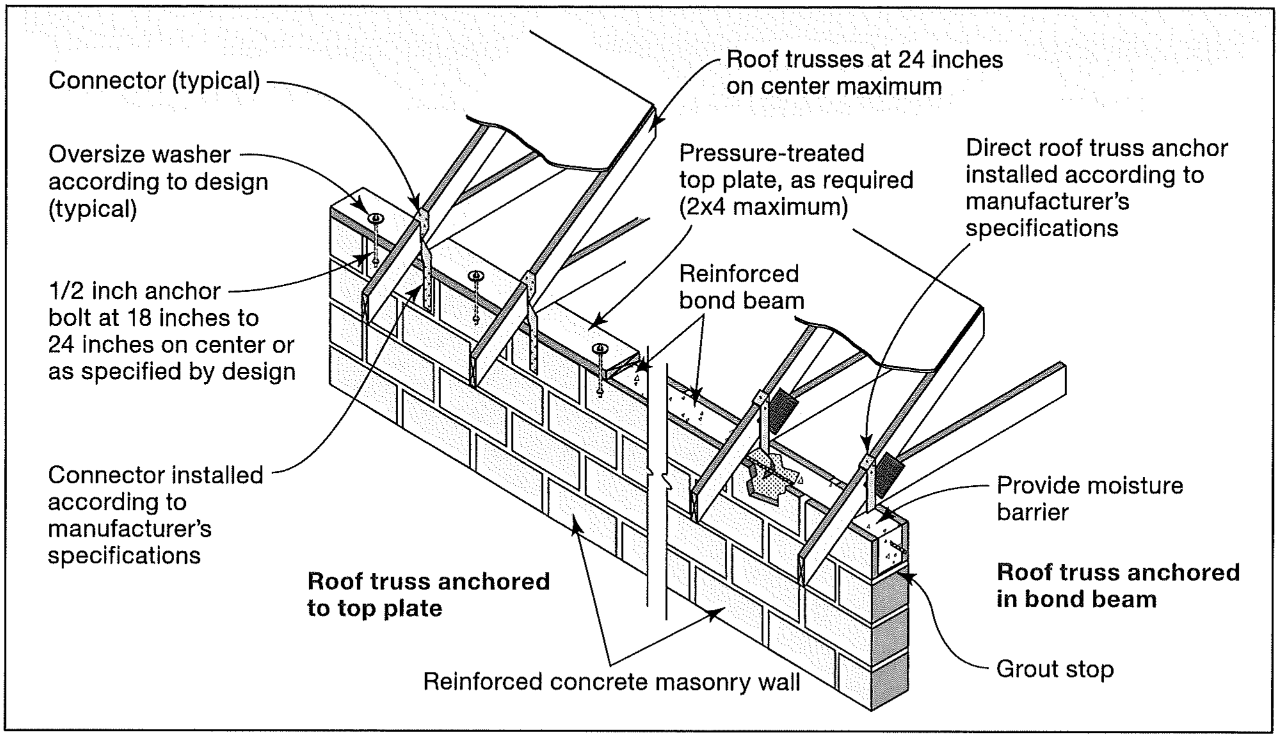


Figure 9-10.
Roof truss-to-masonry wall connectors embedded into concrete-filled or grouted masonry cell (left-hand side image has a top plate installed while the right-hand side does not)

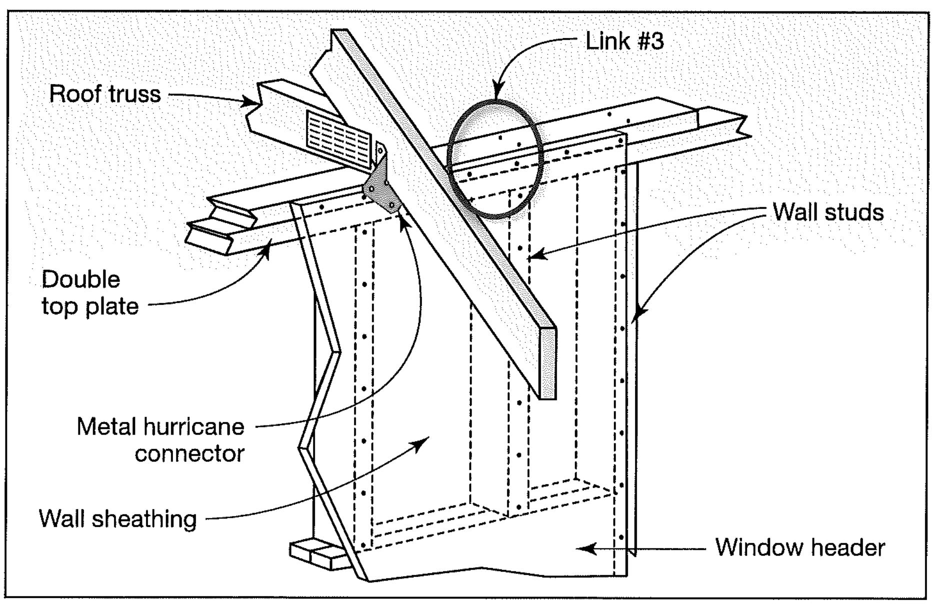
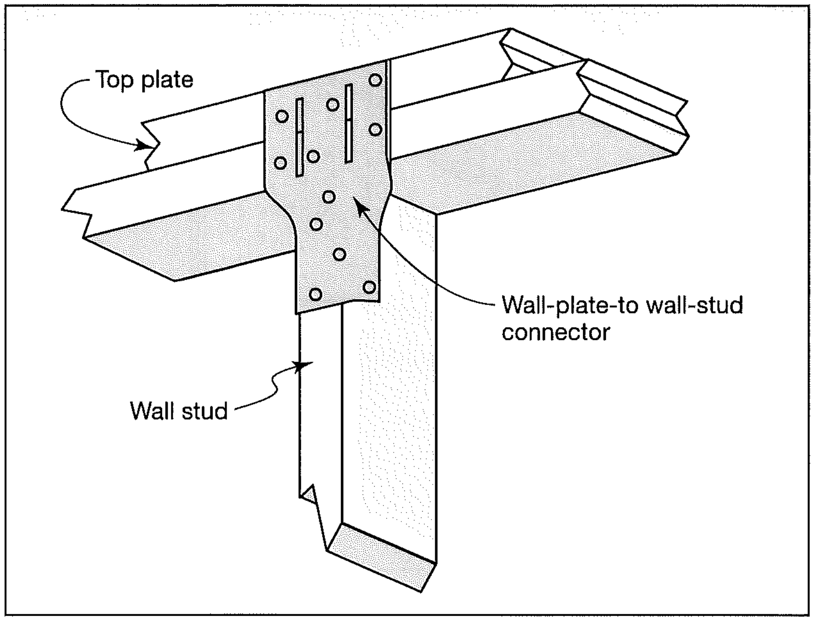


Figure 9-11.
Connection of wall top plate-to-wall stud (Link #3)

For masonry or concrete walls, the wood sill plate is typically connected by anchor bolts, cast-in straps, or other approved fasteners capable of maintaining a load path for uplift, lateral, and shear loads. Anchorage spacing varies based on the anchorage resistance to pullout, the resistance of the plate to bending, and strength of the anchorage in shear. Anchorage must be spaced to resist pullout, and the plate must resist bending and splitting. Placing anchor bolts close together assists in reducing the bending stress in the plate.

Figure 9-12.
Wall top plate-to-wall stud metal connector



9.1.4 Wall Sheathing to Window Header (Link #4)

Link #4 is the connection between the wood structural panel wall sheathing and the window header (see Figures 9-6 and 9-13). The connection maintains the uplift load path from the wall top plates for the same force as determined for the roof connection to the wall minus additional dead load from the wall. Options for maintaining the uplift load path include using metal connectors between the wall studs and header or wood structural panel sheathing (see Figures 9-13 and 9-14). The uplift load path is frequently made with wood structural panel wall sheathing, particularly when the uplift and shear forces in the wall are not very high. Additional design considerations include the resistance of the window header to bending from gravity loads, wind uplift, and out-of-plane bending loads from wind.

In masonry construction, a masonry or concrete bond beam, or a pre-cast concrete or masonry header, is often used over the window opening. Design considerations for this beam include resistance to bending in both the plane of the wall and normal to the wall. Resistance to bending is accomplished by placing reinforcing steel in the bond beam. Reinforcing steel must be placed in the bond beam in order for the beam to adequately resist bending stresses. The design of these members is beyond the scope of this Manual; therefore, the prescriptive methods presented in ICC 600-2008, or concrete and masonry references should be used.

9.1.5 Window Header to Exterior Wall (Link #5)

Link #5 is the connection from the window header to the adjacent wall framing (see Figures 9-6 and 9-14). Link #5 provides resistance to wind uplift and often consists of a metal strap or end-nailing the stud to the header. The total uplift force is based on the uplift forces tributary to the header. Maintaining the load path for the out-of-plane forces at this location includes consideration of both the positive (inward) and the negative (outward) pressures from wind. This load path is commonly developed by the stud-to-header nailing. One method of sizing the connection between the window header and the exterior wall is provided in Example 9.3.

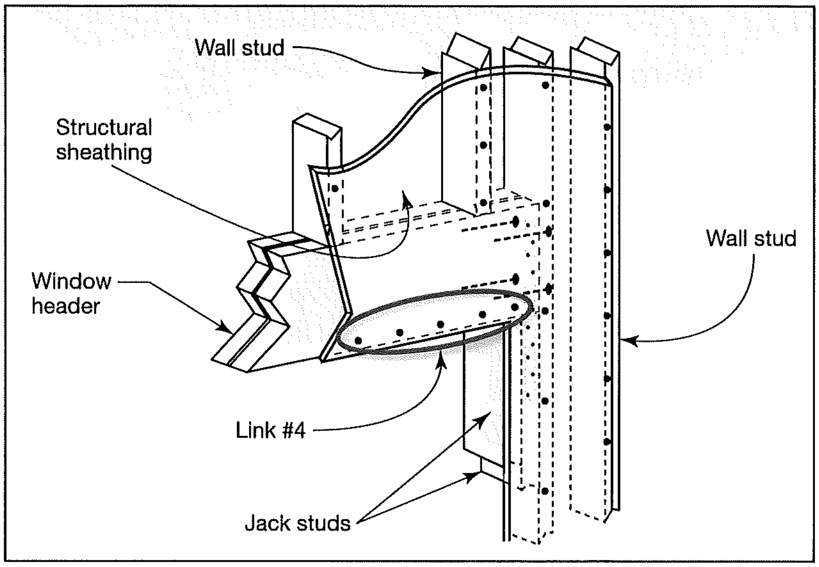


Figure 9-13. Connection of wall sheathing to window header (Link #4)

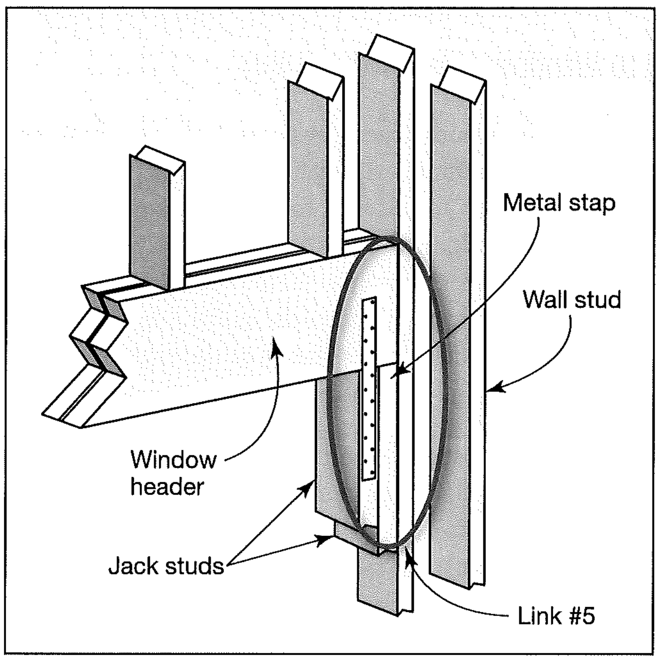


Figure 9-14. Connection of window header to exterior wall (Link #5)

EXAMPLE 9.3. UPLIFT AND LATERAL LOAD PATH AT WINDOW HEADER

Given:

- Refer to Figure 9-14 and Illustration A
- Unit uplift load on window header = 565.2 plf (from Example 9.2)
- Unit lateral load on header = 241.9 plf (from Example 9.2)
- Header span = 14 ft

Find:

- Uplift and lateral load for connection of the header to the wall framing.

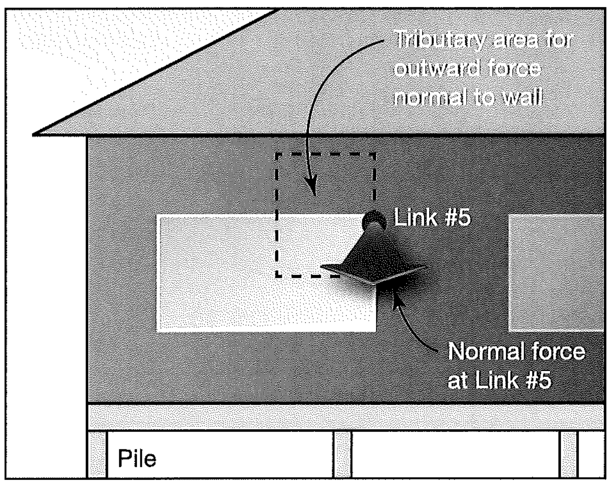


Illustration A. Tributary area for wind force normal to wall (Link #5)

Solution: The uplift and lateral forces can be determined as follows:

Uplift

- Ignore the contribution of the wall's dead load for resistance to uplift because the amount of wall dead load above the header connection is small

$$\text{Uplift load} = \frac{(565.2\text{ plf})(\text{header span})}{2} = (479\text{ plf})(7\text{ ft}) = \mathbf{3{,}955\ lb}$$

$$\text{Lateral load} = \frac{(241.9\text{ plf})(\text{header span})}{2} = (241.9\text{ plf})(7\text{ ft}) = \mathbf{1{,}694\ lb}$$

9.1.6 Wall to Floor Framing (Link #6)

Link #6 is the connection of the wall framing to the floor framing (see Figures 9-6 and 9-15) for resistance to wind uplift. This connection often includes use of metal connectors between the wall studs and the band joist or wood structural panel sheathing. In addition to uplift, connections between wall and floor framing can be used to maintain the load path for out-of-plane wall forces from positive and negative wind pressures and forces in the plane of the wall from shear. One method of sizing the wind uplift and lateral connections between the wall framing and the floor framing is provided in Example 9.4.

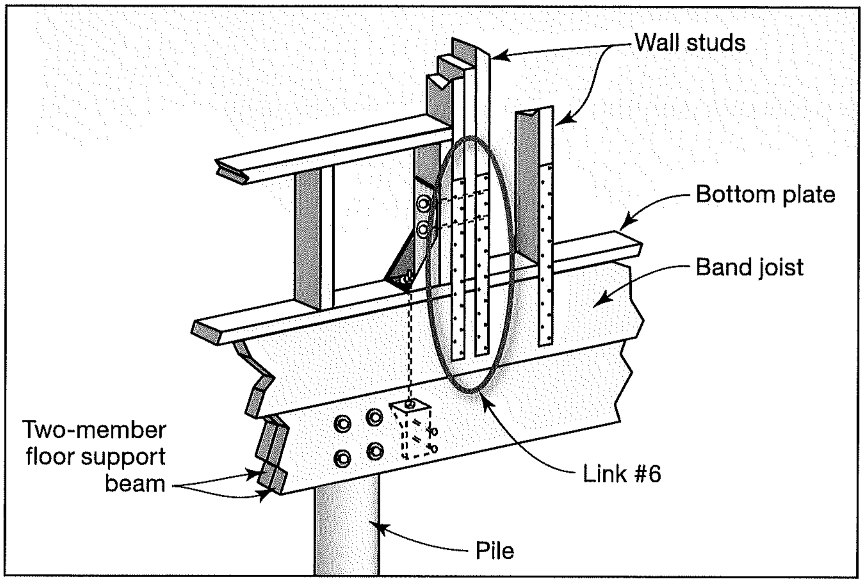


Figure 9-15. Connection of wall to floor framing (Link #6)

EXAMPLE 9.4. UPLIFT AND LATERAL LOAD PATH AT WALL-TO-FLOOR FRAMING

Given:

- Refer to Figure 9-15
- Unit uplift load at top of wall 565.2 plf (from Example 9.2)
- Unit lateral load = 241.9 plf (from Example 9.2)
- Wall dead load = 10 psf
- Wall height = 10 ft
- Wood specific gravity, G = 0.42
- Three 16d common stud-to-plate nails per stud to provide resistance to lateral loads
- Two 16d common plate-to-band joist nails per ft to provide resistance to lateral loads

EXAMPLE 9.4. UPLIFT AND LATERAL LOAD PATH AT WALL-TO-FLOOR FRAMING (concluded)

Find:

- Uplift load for wall-to-floor framing connections and if framing connections are adequate to resist the lateral loads.

Solution: Determine the uplift and lateral load for the wall-to-floor framing connections as follows:

Uplift:

Wall dead load = (10 psf)(10-ft wall height) = 100 plf

Uplift load at top of wall = 565.2 plf

Uplift load at the base of the wall = 565.2 plf – 0.6(100 plf) = 505.2 plf

where:

0.6 = load factor on dead load used to resist uplift forces

For connectors spaced at 16 in. o.c., the minimum uplift load per connector is:

$$\text{Uplift load per connector} = (505.2\ \text{plf})\frac{16\ \text{in.}}{12\ \text{in./ft}} = \mathbf{674\ lb}$$

Lateral:

- Stud-to-plate nail resistance to lateral loads can be calculated as:

Lateral resistance = (3 nails/ft)(120 lb/nail)(1.6)(0.67) = 386 lb

where:

1.6 = NDS load duration factor

0.67 = NDS end grain factor

Because studs are at 16 inches o.c., unit lateral load resistance is:

$$\text{Lateral resistance} = (386\ \text{lb})\frac{12\ \text{in./ft}}{16\ \text{in.}} = \mathbf{289\ lb}$$

289 plf > 241.9 plf ✓

- Plate-to-band joist nail resistance to lateral can be calculated as:

Lateral resistance = (2 nails/ft)(120 lb/nail)(1.6) = 384 plf

where:

1.6 = NDS load duration factor

384 plf > 241.9 plf ✓

The wall-to-floor framing connections provide adequate resistance to lateral forces.

9.1.7 Floor Framing to Support Beam (Link #7)

Link #7 is the connection between the floor framing and the floor support beam (see Figures 9-6, 9-16, 9-17, and 9-18). The connection transfers the uplift forces that are calculated in Example 9.4. Options for maintaining the uplift load path for wind uplift include using metal connectors (see Figures 9-16 and 9-17) between the floor joist and the band joist or wood blocking (see Figure 9-18). Connections are also necessary to maintain a load path for lateral and shear forces from the floor and wall framing into the support beam. One method of sizing the wind uplift connections between the floor framing and support beam is provided in Example 9.5.

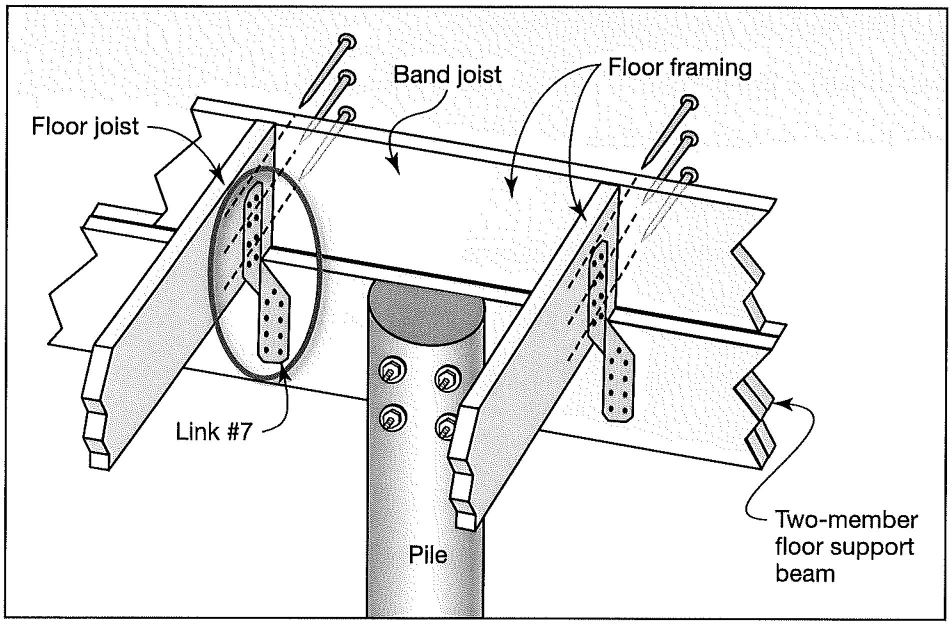


Figure 9-16. Connection of floor framing to support beam (Link #7) (band joist nailing to the floor joist is adequate to resist uplift forces)

Figure 9-17. Metal joist-to-beam connector

EXAMPLE 9.5. UPLIFT LOAD PATH AT FLOOR TO SUPPORT BEAM FRAMING

Given:

- Refer to Figure 9-16
- Unit uplift load at top of wall 565.2 plf (from Example 9.2)
- Wall dead load = 10 psf
- Floor dead load = 10 psf
- Wall height = 10 ft

Find:

- Uplift load for floor framing to beam connections

Solution: The uplift load for the floor framing to beam connections can be determined as follows:

Uplift:

Wall dead load = (10 psf)(10 ft wall height) = 100 plf

$$\text{Floor dead load} = 10 \text{ psf } \frac{14 \text{ ft}}{2} = 70 \text{ plf}$$

Uplift load at the base of the floor = 565.2 plf – 0.6 (100 plf + 70 plf) = 463.2 plf

where:

0.6 = load factor on dead load used to resist uplift forces

For connectors spaced at 16 in. o.c., the minimum uplift load per connector is:

$$\text{Uplift load per connector} = \left(463.2 \text{ plf}\right)\frac{16 \text{ in.}}{12 \text{ in./ft}} = \mathbf{618\ lb}$$

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

Link #8 is the connection of the floor support beam to the top of the pile (see Figures 9-6 and 9-18). Link #8 resists wind uplift forces, and the connection often consist of bolts in the beam-to-pile connection or holddown connectors attached from wall studs above to the pile. One method of sizing the wind uplift connections between the floor support beam and piles is provided in Example 9.6.

The connection of the beam to the pile is also designed to maintain load path for lateral and shear forces. It is typically assumed that lateral and shear forces are transferred through the floor diaphragm and can therefore be distributed to other support beam-to-pile connections. Stiffening of the diaphragm can be achieved by installing braces at each corner pile between the floor support beam in the plane of the floor (see Figure 9-19) or sheathing the underside of the floor framing. Stiffening also reduces pile cap rotation. The load path, however, does not end at Link #8. The load path ends with the transfer of loads from the foundation into the soil. See Chapter 10 for considerations that must be taken into account with regard to the interaction between the foundation members and soil in the load path.

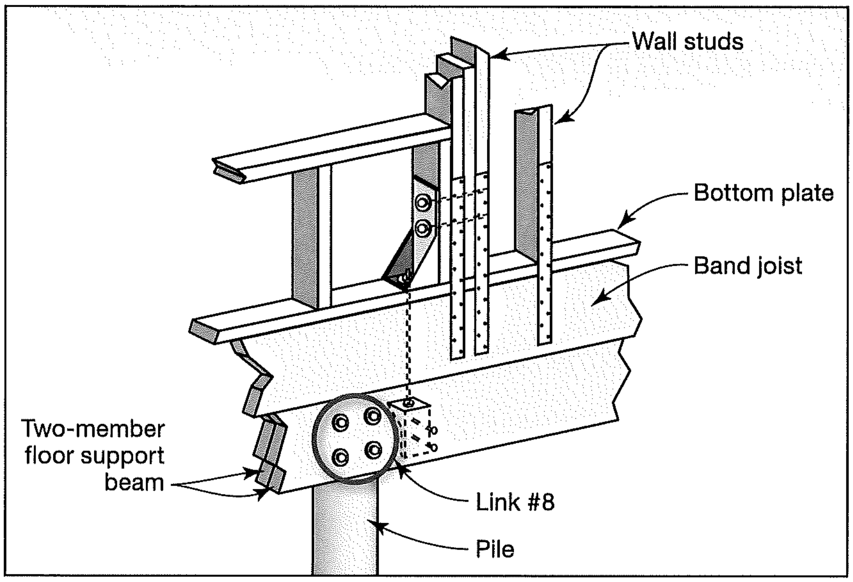


Figure 9-18. Connection of floor support beam to foundation (Link #8)

EXAMPLE 9.6. UPLIFT LOAD PATH FOR SUPPORT BEAM TO PILE

Given:

- Refer to Figure 9-18
- Unit uplift load at top of floor beams = 463.2 plf (from Example 9.5)
- Pile spacing = 9.33 ft
- Continuous beam of 28-ft length at end wall
- ASD capacity for 1-in. diameter bolt in beam-to-pile connection = 1,792 lb (where wood specific gravity (*G*) = 0.42, 3.5-in. side member, and 5.25-in. main member

Find:

1. Uplift load for support beam-to-pile connections.
2. Number of bolts required for support beam-to-pile connections for wind uplift.

Solution for #1: The uplift load for the support beam-to-pile connections can be determined as follows:

Uplift:

Tributary length of center pile connection = 9.33 ft

EXAMPLE 9.6. UPLIFT LOAD PATH FOR SUPPORT BEAM-TO-PILE (concluded)

Uplift load at center pile connection = (9.33 ft)(463.2 plf)= **4,322 lb**

$$\text{Tributary length of end pile connection} = \frac{9.33 \text{ ft}}{2} = 4.67 \text{ ft}$$

Uplift load at end pile connection = (4.67 ft)(463.2 plf) = **2,163 lb**

Solution for #2: The number of bolts required for the support beam-to-pile connections can be determined as follows:

Connection at center pile (number of bolts) =

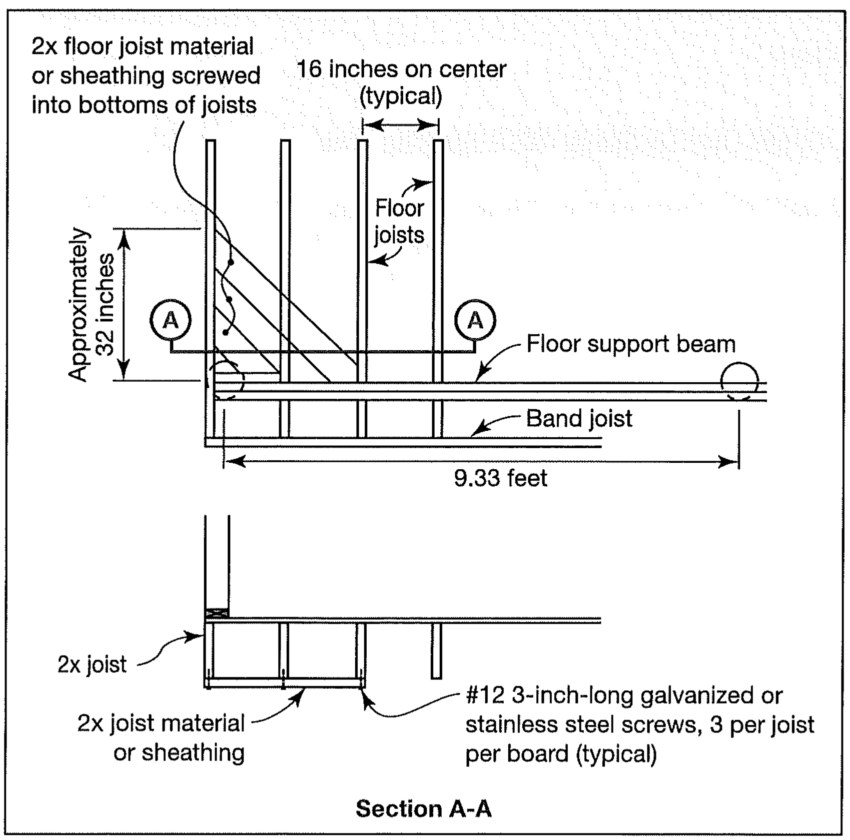
$$\text{Connection at center pile (number of bolts)} = \frac{4{,}322 \text{ lb}}{1{,}792 \text{ lb/bolt}} = 2.41 \text{ bolts} =$$

3 bolts at support beam-to-pile connection

$$\text{Connection at end pile (number of bolts)} = \frac{2{,}163 \text{ lb}}{1{,}792 \text{ lb/bolt}} = 1.21 \text{ bolts} =$$

2 bolt at support beam-to-pile connection

Figure 9-19.
Diaphragm stiffening and corner pile bracing to reduce pile cap rotation



9.2 Other Load Path Considerations

Several additional design considerations must be investigated in order for a 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.

Using the example of the building shown in Example 9.3, Illustration A, the following load paths should also be investigated:

- Load paths for shear transfer between shear walls and diaphragms including uplift due to shear wall overturning
- Gable wall support for lateral wind loads
- Uplift of the front porch roof
- Uplift of the main roof section that spans the width of the building

Other factors that influence the building design and its performance are:

- Connection choices
- Building eccentricities
- Framing system
- Roof shape

9.2.1 Uplift Due to Shear Wall Overturning

The shear wall that contains Link #6 includes connections designed to resist overturning forces from wind acting perpendicular to the ridge (see Example 9.7, Illustration A). Calculation of the overturning induced uplift and compressive forces are given in Example 9.7.

EXAMPLE 9.7. UPLIFT AND COMPRESSION DUE TO SHEAR WALL OVERTURNING

Given:

- Refer to Illustration A
- Wind speed = 150 mph, Exposure D
- Mean roof height = 33 ft
- Roof span perpendicular to ridge = 28 ft
- Roof pitch = 7:12
- Wall height = 10 ft

EXAMPLE 9.7. UPLIFT AND COMPRESSION DUE TO SHEAR WALL OVERTURNING (continued)

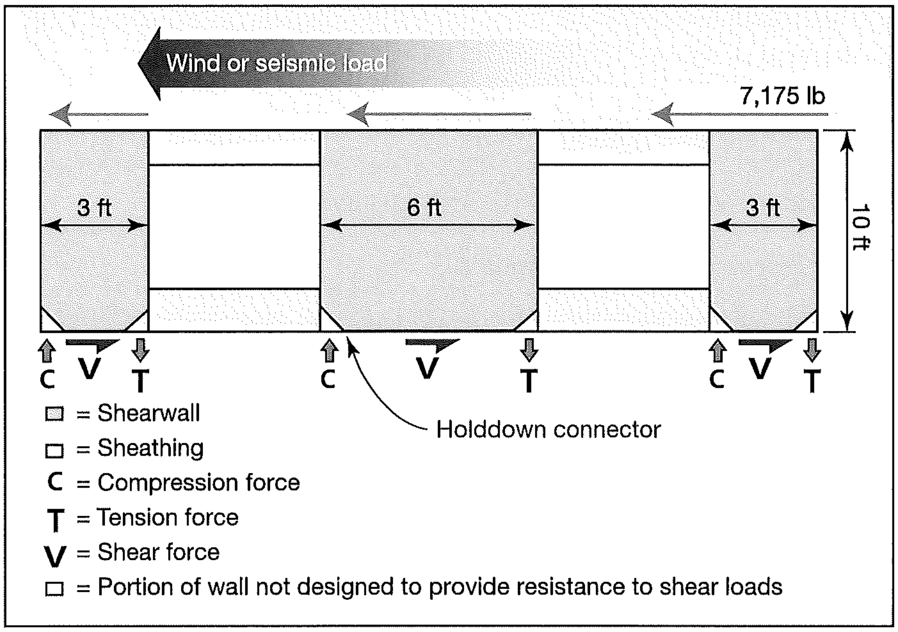


Illustration A. Loads on south shear wall

Find: Uplift and compressive force due to shear wall overturning.

Solution: The uplift and compressive force due to shear wall overturning can be determined as follows:

- The total shear force due to wind acting perpendicular to the ridge is determined for the 28-ft roof span by interpolation from Table 8-7:

  Roof diaphragm load for 24-ft roof span = 256 plf

  Roof diaphragm load for 32-ft roof span = 299 plf

  $$\text{Roof diaphragm load for 28-ft roof span} = \frac{(256 \text{ plf} + 299 \text{ plf})}{2} = 278 \text{ plf}$$

  Adjusting the roof diaphragm load to account for the building being located in Exposure Category D:

  1.18(278 plf) = 328 plf

  To adjust w_{roof} for a wall height of 10 ft because Table 8.7 assumes a wall height of 8 ft

  $$328 \text{ plf}\left(\frac{10 \text{ ft}}{8 \text{ ft}}\right) = 410 \text{ plf}$$

EXAMPLE 9.7. UPLIFT AND COMPRESSION DUE TO SHEAR WALL OVERTURNING (concluded)

- The total shear load for south wall assuming flexible diaphragm distribution of roof diaphragm load is calculated as follows:

$$\text{Length tributary to shear walls} = \frac{35\text{ ft}}{2} = 17.5\text{ ft (see Example 9.3, Illustration A)}$$

Shear load in south shear walls = (17.5 ft)(410 plf) = 7,175 lb

Shear wall segment aspect ratio (see Illustration B):

- Each shear wall segment must meet the requirements for shear wall aspect ratio in order to be considered as a shear resisting element. For wood structural panel shear walls, the maximum ratio of height to length (e.g., aspect ratio, h/L) is 3.5:1.
- The aspect ratio for shear wall segments in Illustration A can be calculated as follows:

$$\text{Aspect ratio of 6-ft long shear wall segment: } \frac{10\text{ ft}}{6\text{ ft}} = 1.67 < 3.5\ \checkmark$$

$$\text{Aspect ratio of 3-ft long shear wall segment: } \frac{10\text{ ft}}{3\text{ ft}} = 3.33 < 3.5\ \checkmark$$

$$\text{Unit shear, } v = \text{total shear load/shear wall length} = \frac{7{,}175\text{ lb}}{(6\text{ ft} + 3\text{ ft} + 3\text{ ft})} = 598\text{ plf}$$

Uplift (T) and compressive force (C) at shear wall ends due to overturning
= (598 plf)(10-ft wall height) = **5,980 lb**

Note: *As seen in this example, tension and compression forces due to shear wall overturning can be large. Alignment of shear wall end posts with piles below facilitates use of standard connectors and manufacturers' allowable design values. A check of the pile uplift and compressive capacity in soil is needed to ensure an adequate load path for overturning forces.*

Because of the magnitude of overturning induced uplift and compression forces, it is desirable to align shear wall ends with piles to provide direct vertical support and to minimize offset of the tension or compression load path from the axis of the pile. Where shear wall end posts are aligned with piles below, detailing that allows connection of the shear wall end post holddown directly to the pile is desirable to minimize forces transferred through other members such as the support beams. Where direct transfer of overturning induced uplift and compression forces into the pile is not possible, minimizing the offset distance reduces bending stresses in the primary support beam (see Figure 9-20). For the holddown connection shown in Figure 9-20, the manufacturers' listed allowable load will be reduced because the bolted connection to the wood beam is loaded perpendicular to grain rather than parallel to grain.

Figure 9-20.
Shear wall holddown connector with bracket attached to a wood beam

9.2.2 Gable Wall Support

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

- A gable wall that is not braced into the structure collapses, and the roof framing falls over (see Figure 9-21)
- An unsupported rake outrigger 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

The need for and type of bracing at gable-end frames depend on the method used to construct the gable end. Recommendations for installing rafter outriggers at overhangs for resistance to wind loads are provided in the *Wood Frame Construction Manual* (American Wood Council, 2001). In addition to using the gable-end truss bracing shown in Figure 9-22, installing permanent lateral bracing on all roof truss systems is recommended. Gable-end trusses and conventionally framed gable-end walls should be designed, constructed, and sheathed as individual components to withstand the pressures associated with the established basic wind speed.

9.2.3 Connection Choices

Alternatives for joining building elements include:

- Mechanical connectors such as those available from a variety of manufacturers
- Fasteners such as nails, screws, bolts, and reinforcing steel

CROSS REFERENCE

For recommendations on corrosion-resistant connectors, see Table 1 in NFIP Technical Bulletin 8, *Corrosion Protection for Metal Connectors in Coastal Areas* (FEMA 1996).

Figure 9-21.
Gable-end failure, Hurricane Andrew (Dade County, FL, 1992)

- Connectors such as wood blocks
- Alternative materials such as adhesives and strapping

Most commercially available mechanical connectors recognized in product evaluation reports are fabricated metal devices formed into shapes designed to fit snugly around elements such as studs, rafters, and wall plates. To provide their rated load, these devices must be attached as specified by the manufacturer. Mechanical connectors are typically provided with various levels of corrosion resistance such as levels of hot-dip galvanizing and stainless steel. Hot-dip galvanizing may be applied before or after fabrication. Thicker galvanized coatings can consist of 1 to 2 ounces of zinc per square foot. Thicker coatings provide greater protection against corrosion. Welded steel products generally have a hot-dip galvanized zinc coating or are painted for corrosion protection. Stainless steel (A304 and A316) connectors also provide corrosion resistance. Because exposed metal fasteners (even when galvanized) can corrode in coastal areas within a few years of installation, stainless steel is recommended where rapid corrosion is expected. According to FEMA NFIP Technical Bulletin 8-96, the amount of salt spray in the air is greatest near breaking waves and declines with increasing distance away from the shoreline. The decline may be rapid in the first 300 to 3,000 feet. FEMA P-499 recommends using stainless steel within 3,000 feet of the coast (including sounds and back bays).

Metal connectors must be used in accordance with the manufacturer's installation instructions in order for the product to provide the desired strength rating and to ensure that the product is suitable for a particular application. Particular attention should be given to the following information in the installation instructions:

- Preservative treatments used for wood framing
- Level of corrosion protection
- Wood species or lumber type used in framing (e.g., sawn lumber, pre-fabricated wood I-joists, laminated veneer lumber)

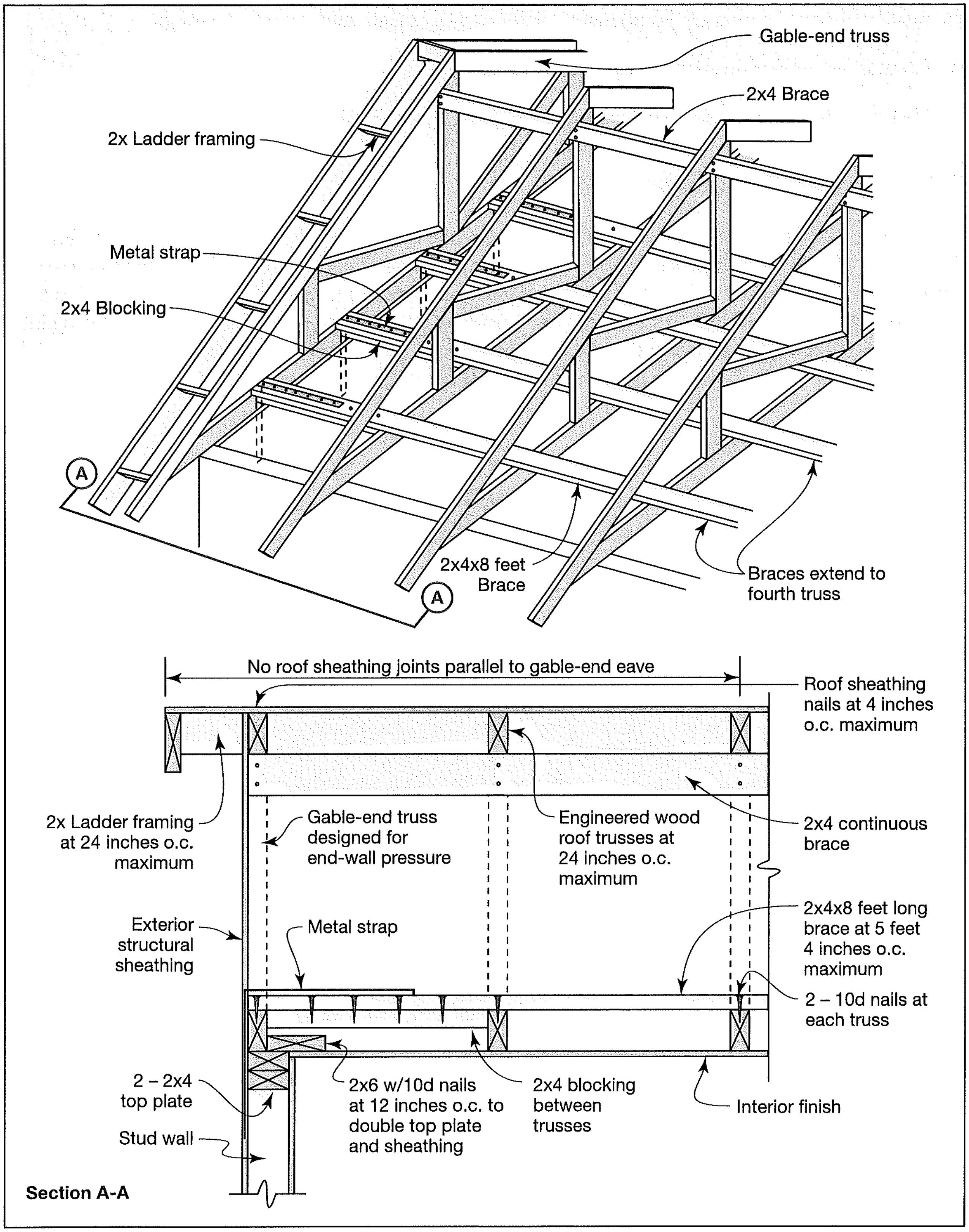


Figure 9-22.
Gable-end bracing detail; nailing schedule, strap specification, brace spacing, and overhang limits should be adapted for the applicable basic wind speed

- Rated capacity of connector for all modes of failure (e.g., shear, uplift, gravity loading)
- Level of corrosion protection for nails, bolts, and/or screws
- Nail, bolt, and/or screw size and type required to achieve rated loads

9.2.4 Building Eccentricities

The L-shaped building configuration produces stress concentrations in the re-entrant corner of the building structure. Additionally, differences between the center of rotation and the center of mass produce torsional forces that must be transferred by the diaphragms and accounted for in the design of shear walls. Provisions for torsional response are different for wind and seismic hazards. Design methods to account for building eccentricities is beyond the scope of this Manual; therefore, the user is referred to building code requirements and provisions of ASCE 7-10 and applicable material design standards.

9.2.5 Framing System

Methods used for maintaining a load path throughout the structure depend on the framing system or structural system that makes up the building structure. Specifics related to platform framing, concrete/masonry construction, and moment-resistant framing are provided below.

9.2.5.1 Platform Framing

Across the United States, platform framing 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 9-23 is an example of platform framing in a two-story building. This method is commonly used on all types of foundation systems, including walls, piles, piers, and columns consisting of wood, masonry, and concrete materials. Less common framing methods in wood-frame construction are balloon framing in which wall studs are continuous from the foundation to the roof and post-and-beam framing in which a structure of beams and columns is constructed first, including the floors and roof, and then walls are built inside the beam and column structure.

9.2.5.2 Concrete/Masonry

Masonry exterior 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 required in high-wind and seismic hazard areas. Floor framing is normally supported by a ledger board fastened to a bond beam in the masonry, and the roof is anchored to a bond beam at the top of the wall. Connections can be via a top plate as shown in Figure 9-24 or direct embedded truss anchors in the bond beam as shown in

NOTE

Masonry frames typically require continuous footings. However, continuous footings are not allowed in Zone V or Coastal A Zones and are not recommended in Zone A.

Figure 9-23.
Example of two-story platform framing on a pile-and-beam foundation

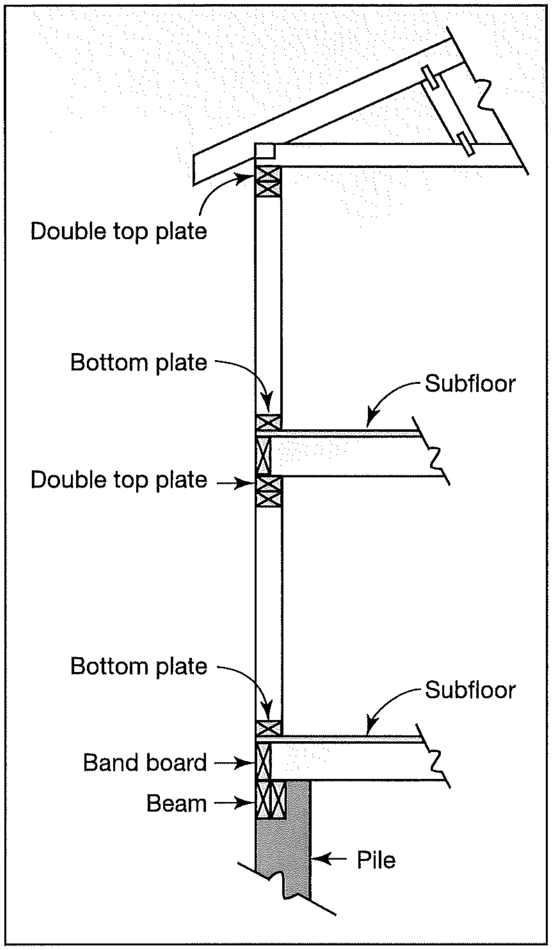


Figure 9-6. Options for end walls are hip roofs, continuous masonry gables, and braced gable frames. Details and design tables for all of the above can be found in ICC 600-2008. Figure 9-24 is an example of masonry wall construction in a two-story building.

9.2.5.3 Moment-Resisting Frames

Over the past few decades, an increasing number of moment-resisting frames have been built and installed in coastal homes (Hamilton 1997). The need for this special design is a result of 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 9-25 shows a typical steel moment frame.

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

- Large openings in shear walls
- Large deflection in shear walls
- Difficulties in distributing the shear load to the foundation

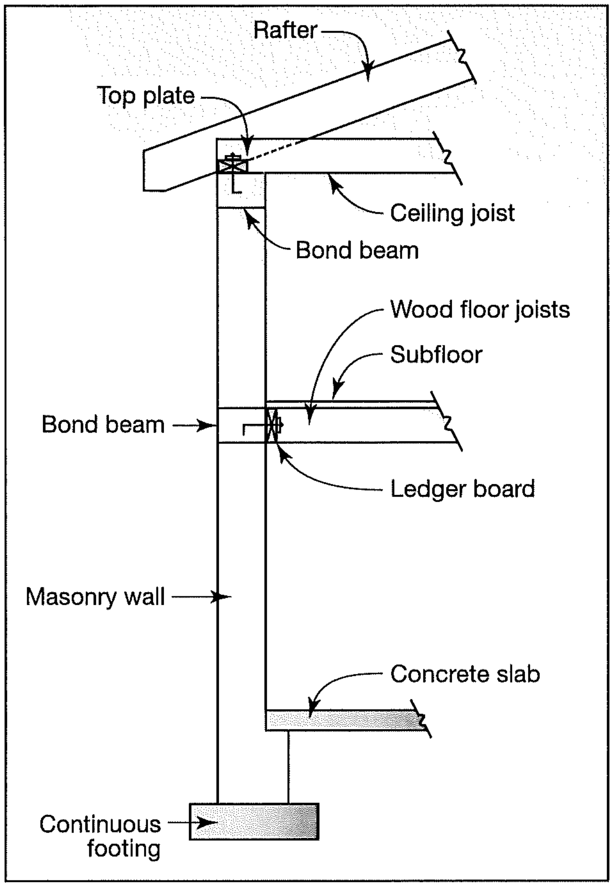


Figure 9-24.
Two-story masonry wall with wood floor and roof framing

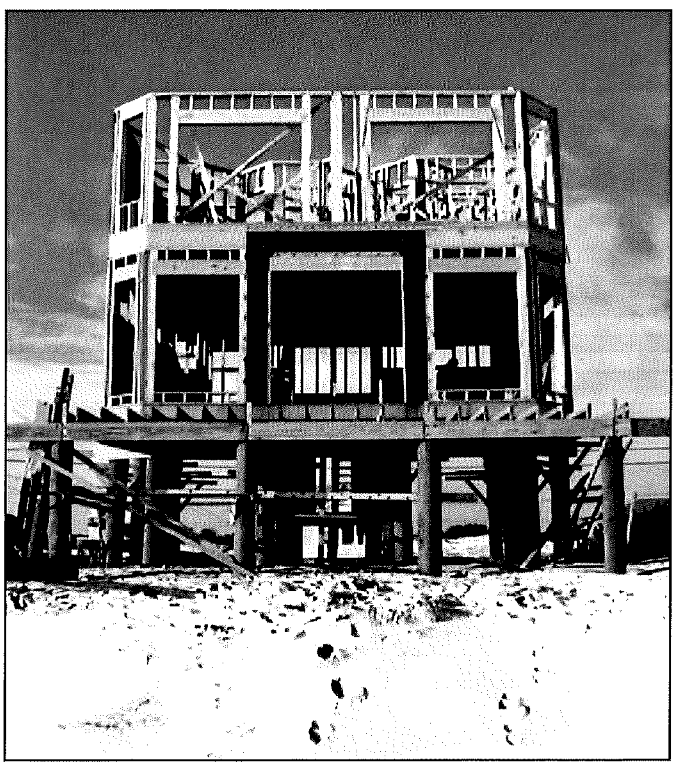


Figure 9-25.
Steel moment frame with large opening

- A moment-resisting frame usually resists shear by taking the lateral load into the top of the frame thus creating a moment at the base of the frame. The design professional must design a moment connection at the base between the steel frame and the wood, masonry, or concrete foundation.

In residential construction, moment frames are frequently tubular steel. Tubular steel shapes that are close to the size of nominal framing lumber can be selected. 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 because all connections in the frame are welded. There are a number of pre-manufactured moment frame products on the market now that have been designed for a variety of lateral forces to fit a variety of wall lengths and heights.

9.2.6 Roof Shape

Roof shape, both the structural aspect and the covering, plays a significant role in roof performance. 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. Steeply pitched roofs usually perform better than flat roofs. Figures 9-26 and 9-27 show two types of roofs in areas of approximately similar terrain that experienced the winds of Hurricane Marilyn. The gable roof in Figure 9-26 failed, while the hip roof in Figure 9-27 survived the same storm with little to no damage. Whether the roof is a gabled roof or hip roof, proper design and construction are necessary for successful performance in high-wind events.

9.3 Breakaway Wall Enclosures

In Zone 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 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 strikes 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, focused at the stillwater level. The pressures can be extreme. For example, a 5-foot wave height can produce a peak force of 4,500 pounds/square foot, roughly 100 times the force caused by a 170-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 2012 IBC and the 2012 IRC require that new, substantially damaged, and substantially improved buildings in Zone V be elevated above the BFE on an open foundation (e.g., pile, post, column, pier).

The 2012 IBC and 2012 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 (termed "breakaway walls"). Because such enclosures fail under flood forces, they do not transfer additional significant loads to the foundation. Regulatory requirements and design criteria concerning enclosures and breakaway walls below elevated buildings in Zone V are discussed in FEMA NFIP Technical Bulletin 9 (FEMA 2008a). Additional guidance is contained in Fact

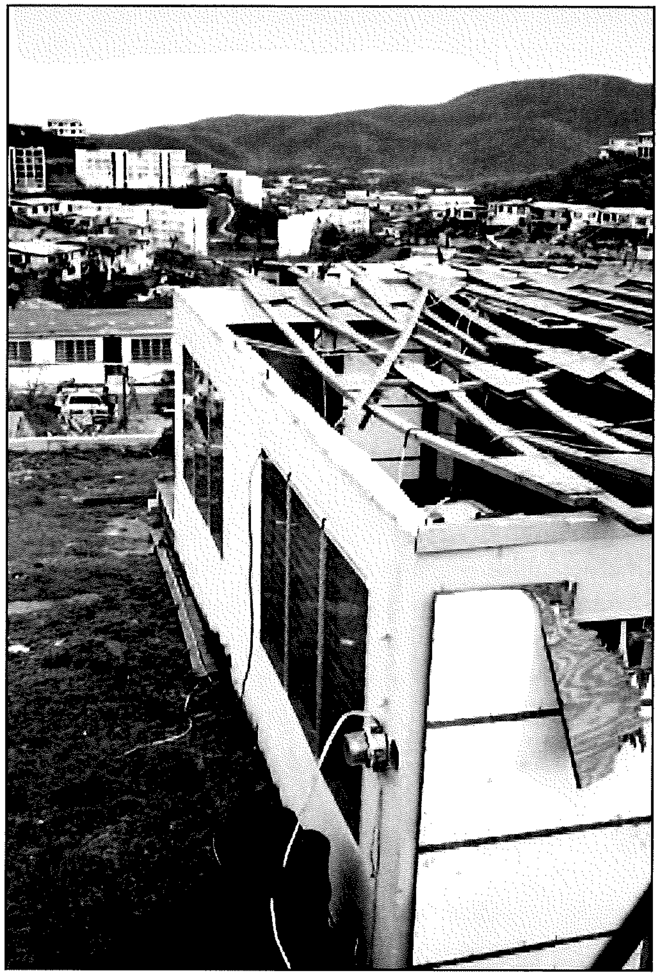


Figure 9-26.
Gable-end failure caused by high winds, Hurricane Marilyn (U.S. Virgin Islands, 1995)

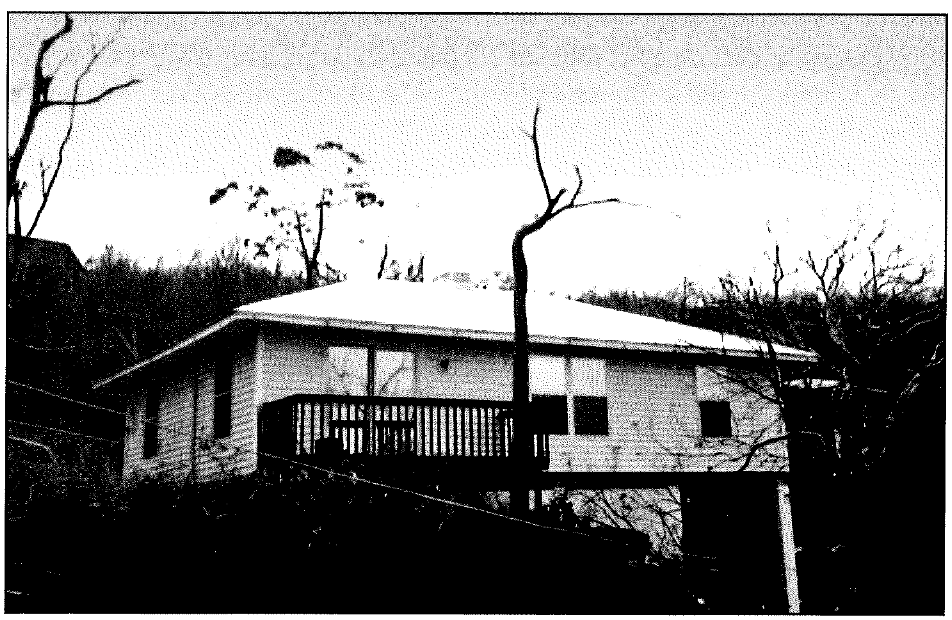


Figure 9-27.
Hip roof that survived high winds with little to no damage, Hurricane Marilyn (U.S. Virgin Islands, 1995)

Sheet No. 8.1, *Enclosures and Breakaway Walls* in FEMA P-499. Breakaway walls may be of wood- or metal-frame or masonry construction.

Figure 9-28 shows how a failure begins in a wood-frame breakaway wall. Note the failure of the connection between the bottom plate of the wall and the floor of the enclosed area. Figure 9-29 shows a situation in which utility components placed on and through a breakaway wall prevented it from breaking away cleanly.

To increase the likelihood of collapse as intended, it is recommended that the vertical framing members (such as 2x4s) on which the screen or lattice work is mounted be spaced at least 2 feet apart. Either metal or synthetic screening is acceptable. Wood and plastic lattice is available in 4-foot x 8-foot sheets. The material used to fabricate the lattice should be no thicker than 1/2 inch, and the finished sheet should have an opening ratio of at least 40 percent. Figure 9-30 shows lattice used to enclose an area below an elevated building.

Figure 9-28.
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, Hurricane Hugo (South Carolina, 1989)

Figure 9-29.
Breakaway wall panel prevented from breaking away cleanly by utility penetrations, Hurricane Opal (Florida, 1995)

Figure 9-30. Lattice beneath an elevated house in Zone V

9.4 Building Materials

The choice of materials is influenced by many considerations, including whether the materials will be used above or below the DFE. Below the DFE, design considerations include the risk of inundation by seawater, and the forces to be considered include those from wave action, water velocity, and waterborne debris impact. Materials intermittently wetted by floodwater below the BFE are subject to corrosion and decay.

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

Each type of commonly used material (wood, concrete, steel, and masonry) has both characteristics that can be advantageous and that can require special consideration when the materials are used in the coastal environment (see Table 9-1). A coastal residential structure usually has a combination of these materials.

Table 9-1. General Guidance for Selection of Materials

Material	Advantages	Special Considerations
Wood	• 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 • Some species are naturally decay-resistant	• Easily over-cut, over-notched, and over-nailed • 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	• Used for forces that are larger than wood can resist • Can span long distances • Can be coated to resist corrosion	• Not corrosion-resistant • Heavy and not easily handled and fabricated by carpenters • May require special connections such as welding

Table 9-1. General Guidance for Selection of Materials (concluded)

Material	Advantages	Special Considerations
Reinforced Concrete	• 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	• Saltwater infiltration into concrete cracks causes reinforcing steel corrosion • Pre-stressed members require special handling • Water intrusion and freeze-thaw cause deterioration and spalling
Masonry	• Resistant to corrosion if reinforcing is properly protected • Good material for compressive loads • Commonly used in residential construction	• Not good for beams and girders • Water infiltration into cracks causes reinforcing steel corrosion • Requires reinforcement to resist loads in coastal areas

9.4.1 Materials Below the DFE

The use of flood-resistant materials below the BFE is discussed in FEMA NFIP Technical Bulletin 2 (FEMA 2008b). According to the bulletin, "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 2012 IBC and 2012 IRC require that all new construction and substantial improvements in the SFHA be constructed with materials that are resistant to flood damage. Compliance with these requirements in coastal areas means that the only building elements below the BFE are:

NOTE

Although NFIP regulations, 2012 IBC, and 2012 IRC specify that flood-resistant materials be used below the BFE, in this Manual, flood-resistant materials below the DFE are recommended.

- Foundations – 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
- Access stairs

CROSS REFERENCE

For NFIP compliance provisions as described in the 2012 IBC and the 2012 IRC, see Chapter 5 of this Manual.

CROSS REFERENCE

For examples of flood insurance premiums for buildings in which the lowest floor is above the BFE and in which there is an enclosure below the BFE, see Table 7-2 in Chapter 7.

Material choices for these elements are limited to materials that meet the requirements provided in FEMA NFIP Technical Bulletin 2. Even for materials meeting those requirements, characteristics of various materials can be advantageous or may require special consideration when the materials are used for different building elements. Additional information about material selection for various locations and uses in a building is included in "Material Durability in Coastal Environments," available on the Residential Coastal Construction Web site (http://www.fema.gov/rebuild/mat/fema55.shtm).

9.4.2 Materials Above the DFE

Long-term durability, architectural, and structural considerations are normally the most important factors in material selection. Material that will be used in a coastal environment will be subjected to weathering, corrosion, termite damage, and decay from water infiltration, in addition to the stresses induced by loads from natural hazard events. These influences are among the considerations for selecting appropriate materials. "Material Durability in Coastal Environments" contains additional information about a variety of wood products and the considerations that are important in their selection and use.

9.4.3 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 design professional working with of coastal buildings, important design considerations when combining materials include:

1. The compatibility of metals is a design consideration because dissimilar metals that are in contact with each other may corrode in the presence of salt and moisture. "Material Durability in Coastal Environments" addresses a possible problem when galvanized fasteners and hardware are in contact with certain types of treated wood.

2. Connecting the materials together is crucial. Proper embedment of connectors (if into concrete or masonry) and proper placement of connectors are necessary for continuity of the vertical or horizontal load path. Altering a connector location after it has been cast into concrete or grout is a difficult and expensive task.

3. Combining different types of material in the same building adds to construction complexity and necessitates additional skills to construct the project. Figure 9-31 shows a coastal house being constructed with preservative-treated wood piles that support a welded steel frame, resulting in metal coming into direct contact with treated wood.

4. Material properties, such as stiffness of one material relative to another, affect movement or deflection of one material relative to the other.

Figure 9-31.
House being constructed with a steel frame on wood piles

9.4.4 Fire Safety Considerations

Designing and constructing townhouses and low-rise multi-family coastal buildings to withstand natural hazards and meet the building code requirements for adequate fire separation presents some challenges. Although fire separation provisions of the 2012 IBC and 2012 IRC differ, they both require that the common walls between living units be constructed of materials that provide a minimum fire resistance rating. The intent is for units to be constructed so that if a fire occurs 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. Beams that are parallel to the front and rear exterior walls are typically used to provide support for these walls as well as the floor framing. Figure 9-32 illustrates a framing system for a series of townhouses in which floor beams are perpendicular to the primary direction of flood forces. Design issues include the following:

5. 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.

6. The fire separation between townhouse units limits options for structural connections between units, making the transfer of lateral loads to the foundation more difficult to achieve.

7. The exposed undersides of buildings elevated on an open foundation (e.g., pile, pier, post, column) must be protected with a fire-rated material. Typically, this is accomplished with use of fire-resistant gypsum board; however, gypsum board is not a flood-damage-resistant material. An alternative approach is to use other materials such as cement-fiber board (with appropriate fire rating), which has a greater resistance to damage from floodwaters, and fire retardant treated wood. Other alternative materials or methods of protection that are flood-damage-resistant may be required in order to meet the competing demands of flood- and fire-resistance.

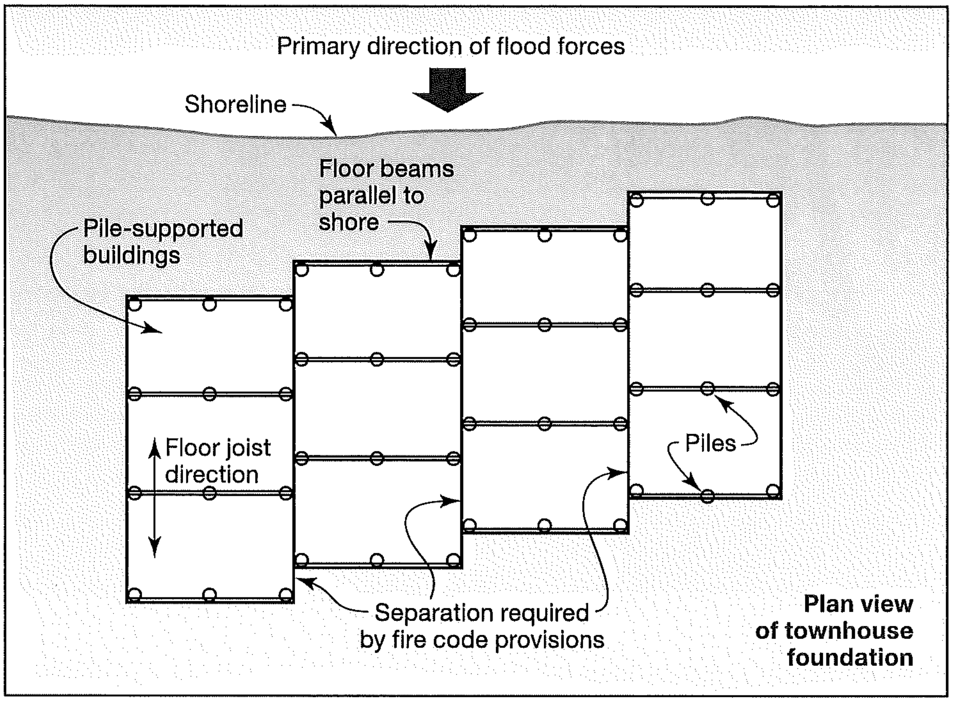


Figure 9-32.
Townhouse framing system

8. The requirement for separation of the foundation elements between townhouse units makes structural rigidity in the direction parallel to the shore more difficult to achieve. If the houses in Figure 9-32 were in a seismic hazard area, the designer could decide to place diagonal bracing parallel to the shore (i.e., perpendicular to the primary flood flow direction) or use more closely spaced and larger piles. Diagonal bracing would provide rigidity but would also create an obstruction below the DFE. The design professional should consult FEMA NFIP Technical Bulletin 5 (FEMA 2008c) for information about the types of construction that constitute an obstruction.

One solution to some of the issues illustrated by Figure 9-32 would be to use two parallel independent walls to provide the required fire separation between units. Each wall could be attached to the framing system of the unit on one side of the separation and supported by a beam running perpendicular to the shore and bearing on the open foundation of that unit.

9.4.5 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 that are comparable to those of the other materials used for construction. 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 element with the proper corrosion-resistant material.

CROSS REFERENCE

For additional information about corrosion of metal connectors in coastal construction, see FEMA NFIP Technical Bulletin 8-96.

9.5 Appurtenances

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" (44 CFR § 59.1). In this Manual, "appurtenant structure" means any other building or constructed element on the same property as the primary building, such as decks, covered porches, access to elevated buildings, pools, and hot tubs.

CROSS REFERENCE

For additional information about the types of building elements that are allowed below the BFE and for respective site development issues, see FEMA NFIP Technical Bulletin 5.

9.5.1 Decks 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 designing decks and other exterior attached structures:

- If a deck is structurally attached to a structure, the bottom of the lowest horizontal supporting member of the deck must be at or above the BFE. Deck supports that extend below the BFE (e.g., pilings, bracing) must comply with Zone V design and construction requirements. The structure must be designed to accommodate any increased loads resulting from the attached deck.
- Some attached decks are located above the BFE but rely on support elements that extend below the BFE. These supports must comply with Zone V design and construction requirements.
- If a deck or patio (not counting its supports) lies in whole or in part below the BFE, it must be structurally independent from the structure and its foundation system.
- If the deck surface is constructed at floor level, the deck surface/floor level joint provides a point of entry for wind-driven rain. This problem can be eliminated by lowering the deck surface below the floor level.
- If deck dimensions can be accommodated with cantilevering from the building, this 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 house framing. Chapter 11 discusses construction techniques for flashing cantilever decks that 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.

WARNING

Decks should not cantilever over bulkheads or retaining walls where waves can run up the vertical wall and under the deck.

9.5.1.1 Handrails

To minimize the effects of wind pressure, flood forces, and wave impacts, deck handrails should be open and have slender vertical or horizontal members spaced in accordance with the locally adopted building code. Many deck designs include solid panels (some made of impact-resistant glazing) between the top of the deck handrail and the deck. These solid panels must be able to resist the design wind and flood loads (below the DFE) or they will become debris.

9.5.1.2 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 floodwater through the stair stringers and anchor the stringers to a permanent foundation by using, for example, 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 reduces the likelihood of the stairs becoming debris.

9.5.2 Access to Elevated Buildings

The first floor of buildings in the SFHA is elevated from a few feet to many feet above the exterior grade in order to protect the building and its contents from flood damage. Buildings in Zone A may be only a few feet above grade; buildings in Zone V 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. The 2012 IBC and 2012 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 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 are generally not 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 one- to four-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 may be below the BFE. FEMA's NFIP Technical Bulletin 4 (FEMA 2010a) provides guidance on how to install elevators so that damage to elevator elements is minimized during a flood.

CROSS REFERENCE

For more information about elevator installation in buildings located in SFHAs, see FEMA NFIP Technical Bulletin 4.

9.5.3 Pools and Hot Tubs

Many homes at or near the coast have a swimming pool or hot tub as an accessory. Some of the pools are fiberglass and are installed on a pile-supported structural frame. Others are in-ground concrete pools. The design professional 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 Zone V 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 of the building must remain unenclosed.
- 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 Zone V 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 designer must assure community officials that a pool beneath or adjacent to an elevated Zone V 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 floodwaters, 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.

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.

NOTE

The construction of pools below or adjacent to buildings in coastal high hazard areas must meet the requirements presented in FEMA NFIP Technical Bulletin 5. In general, pools must 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.

- Pools and hot tubs have water pumps, piping, heaters, filters, and other equipment that is expensive and that can be damaged by floodwaters 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, tanks must not be susceptible to erosion and scour and thus failure of the anchoring system.

The design intent for concrete pools includes the following:

- Elevation of an in-ground pool should be such that scour will not permit the pool to fail from either normal internal loads of the filled pool or from exterior loads imposed by the flood forces.
- The pool should be located as far landward as possible and should be oriented in such a way that flood forces are minimized. One way to minimize flood forces includes placing the pool with the narrowest dimension facing the direction of flow, orienting the pool so there is little to no angle of attack from floodwater, and installing a pool with rounded instead of square corners. All of these design choices reduce the amount of scour around the pool and improve the chances the pool will survive a storm. These concepts are illustrated in Figure 9-33.
- A concrete pool deck should be frangible so that flood forces create concrete fragments that help reduce scour. The concrete deck should be installed with no reinforcing and should have contraction joints placed at 4-foot squares to "encourage" failure. See Figure 9-34 for details on constructing a frangible concrete pad.
- Pools should not be installed on fill in or near Zone V. Otherwise, a pool failure may result from scour of the fill material.

For concrete pools, buoyancy failure is also possible when floodwaters 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.

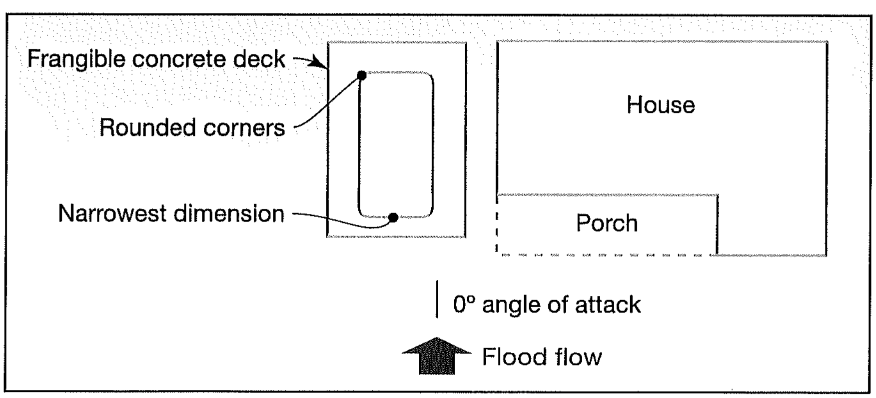
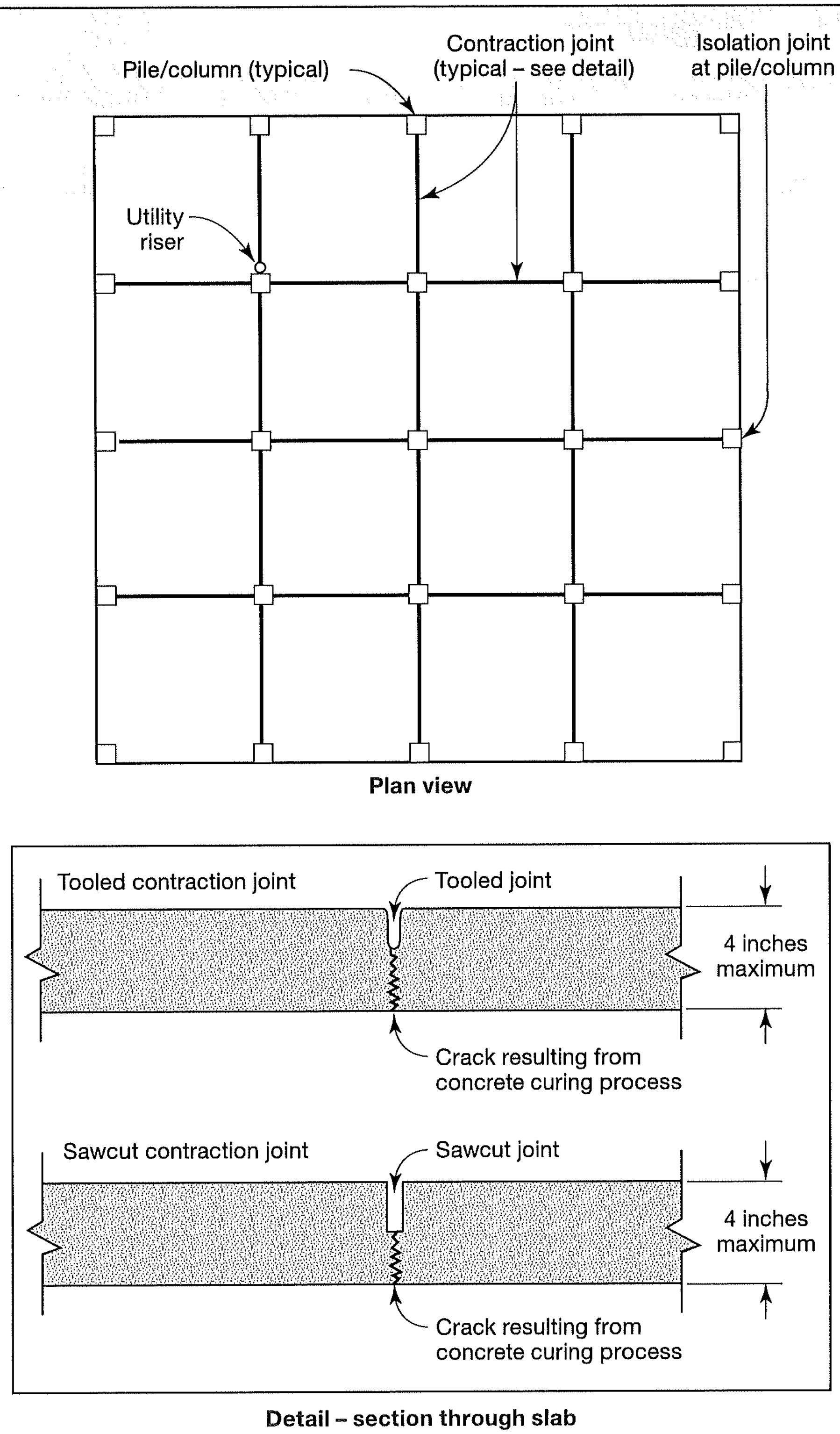


Figure 9-33. Recommendations for orientation of in-ground pools

Figure 9-34.
Recommended contraction joint layout for frangible slab-on-grade below elevated building



Note: Install expansion and isolation joints as appropriate in accordance with standard practice or as required by state and local codes.

9.6 References

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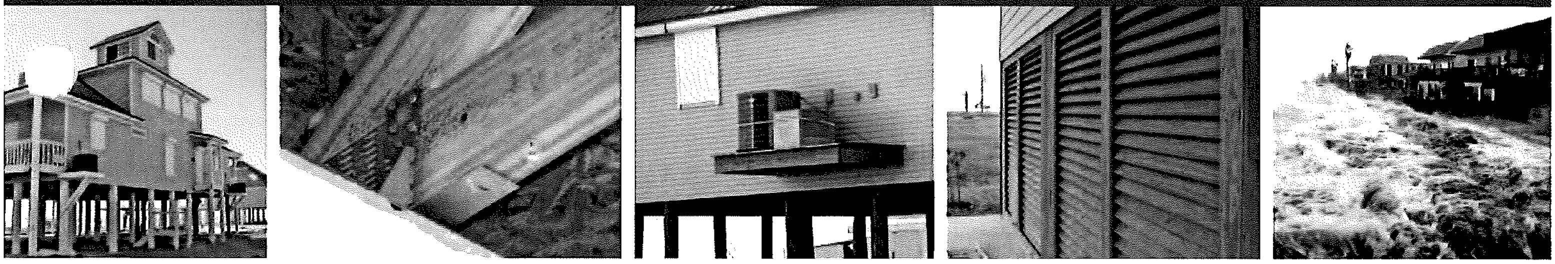

Designing the Foundation

This chapter provides guidance on designing foundations, including selecting appropriate materials, in coastal areas. It provides general guidance on designing foundations in a coastal environment and is not intended to provide complete guidance on designing foundations in every coastal area. Design professionals should consult other guidance documents, codes, and standards as needed.

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

Design considerations for foundations in coastal environments are in many ways similar to those in inland areas. Like all foundations, coastal foundations must support gravity loads, resist uplift and lateral loads, and maintain lateral and vertical load path continuity from the elevated building to the soils below. Foundations in coastal areas are different in that they must generally resist higher winds, function in a corrosive environment, and withstand the environmental aspects that are unique to coastal areas: storm surges, rapidly moving floodwaters, wave action, and scour and erosion. These aspects can make coastal flooding more damaging than inland flooding.

Like many design processes, foundation design is an iterative process. First, the loads on the elevated structure are determined (see Chapter 9). Then a preliminary foundation design is considered, flood loads on the preliminary design are determined, and foundation style is chosen and the respective elements are sized to resist those loads. With information on foundation size, the design professional can accurately determine flood loads on the foundation and can, through iteration, develop an efficient final design.

Because flood loads depend greatly on the foundation design criteria, the discussion of foundation design begins there. The appropriate styles of foundation are then discussed and how the styles can be selected to reduce vulnerability to natural hazards.

The distinction between *code requirements* and *best practices* is described throughout the chapter.

10.1 Foundation Design Criteria

Foundations should be designed in accordance with the latest edition of the 2012 IBC or the 2012 IRC and must address any locally adopted building ordinances. Designers will find that other resources will likely be needed in addition to the building codes in order to properly design a coastal foundation. These resources are listed at the end of this chapter. Properly designed and constructed foundations are expected to:

- Support the elevated building and resist all loads expected to be imposed on the building and its foundation during a design flood, wind, or seismic event
- In SFHAs, prevent flotation, collapse, and lateral movement of the building
- Function after being exposed to scour and erosion

In addition, the foundation must be constructed with flood-resistant materials below the BFE. See Technical Bulletin 2, *Flood Damage-Resistant Materials Requirements* (FEMA 2008a), and Fact Sheet 1.7, *Coastal Building Materials,* in FEMA P-499 (FEMA 2011).

Some coastal areas mapped as Zone A are referred to as "Coastal A Zones." Following Hurricane Katrina (2005), Coastal A Zones have also been referred to as areas with a Limit of Moderate Wave Action (LiMWA). Buildings in Coastal A Zones may be subjected to damaging waves and erosion and, when constructed to minimum NFIP requirements for Zone A, may sustain major damage or be destroyed during the base flood. Therefore, in this Manual, foundations for buildings in Coastal A Zones are strongly recommended to be designed and constructed with foundations that resist the damaging effects of waves.

TERMINOLOGY: LiMWA AND COASTAL A ZONE

Limit of Moderate Wave Action (LiMWA) is an advisory line indicating the limit of the 1.5-foot wave height during the base flood. FEMA requires new flood studies in coastal areas to delineate the LiMWA.

10.2 Foundation Styles

In this Manual, foundations are described as open or closed and shallow or deep. The open and closed descriptions refer to the above-grade portion of the foundation. The shallow and deep descriptions refer to the below-grade portion. Foundations can be open and deep, open and shallow, or closed and shallow. Foundations can also be closed and deep, but these foundations are relatively rare and generally found only in areas where (1) soils near the surface are relatively weak (700 pounds/square foot bearing capacity or less), (2) soils near the surface contain expansive clays (also called shrink/swell soils) that shrink when dry and swell when wet, or (3) other soil conditions exist that necessitate foundations that extend into deep soil strata to provide sufficient strength to resist gravity and lateral loads.

Open, closed, deep, and shallow foundations are described in the following subsections.

10.2.1 Open Foundations

An open foundation allows water to pass through the foundation of an elevated building, reducing the lateral flood loads the foundation must resist. Examples of open foundations are pile, pier, and column foundations. An open foundation is designed and constructed to minimize the amount of vertical surface area that is exposed to damaging flood forces. Open foundations have the added benefit of being less susceptible than closed foundations to damage from flood-borne debris because debris is less likely to be trapped.

Open foundations are required in Zone V and recommended in Coastal A Zone. Table 10-1 shows the recommended practices in Coastal A Zone and Zone V.

Table 10-1. Foundation Styles in Coastal Areas

Foundation Style	Zone V	Coastal A Zone (LiMWA)	Zone A
Open/deep	Acceptable	Acceptable	Acceptable
Open/shallow	Not permitted	Acceptable[(a)]	Acceptable
Closed/shallow	Not permitted	Not recommended	Acceptable
Closed/deep	Not permitted	Not recommended	Acceptable

LiMWA = Limit of Moderate Wave Action

(a) Shallow foundations in Coastal A Zone are acceptable only if the maximum predicted depth of scour and erosion can be accurately predicted and foundations can be constructed to extend below that depth.

10.2.2 Closed Foundations

A closed foundation is typically constructed using continuous perimeter foundation walls. Examples of closed foundations are crawlspace foundations and stem wall foundations,[1] which are usually filled with compacted soil. Slab-on-grade foundations are also considered closed.

A closed foundation does not allow water to pass easily through the foundation elements below an elevated building. Thus, these types of foundations obstruct floodwater flows and present a large surface area upon which waves and flood forces act. Closed foundations are prohibited in Zone V and are not recommended in Coastal A Zones. If perimeter walls enclose space below the DFE, they must be equipped with openings that allow floodwaters to flow in and out of the area enclosed by the walls (see Figure 2-19). The entry and exit of floodwater equalizes the water pressure on both sides of the wall and reduces the likelihood that the wall will fail. See Fact Sheet No. 3.5, *Foundation Walls,* in FEMA P-499, *Home Builder's Guide to Coastal Construction Technical Fact Sheet Series* (FEMA 2010).

Closed foundations also create much larger obstructions to moving floodwaters than open foundations, which significantly increases localized scour. Scour, with and without generalized erosion, can remove soils that support a building and can undermine the foundation and its footings. Once undermined, shallow footings readily fail (see Figure 10-1).

1 Stem wall foundations (in some areas, referred to as chain wall foundations) are similar to crawlspace foundations where the area enclosed by the perimeter walls are filled with compacted soil. Most stem wall foundations use a concrete slab-on-grade for the first floor. The NFIP requires flood vents in crawlspace foundations but not in stem wall foundations (see Section 6.1.1.1 and Section 7.6.1.1.5).

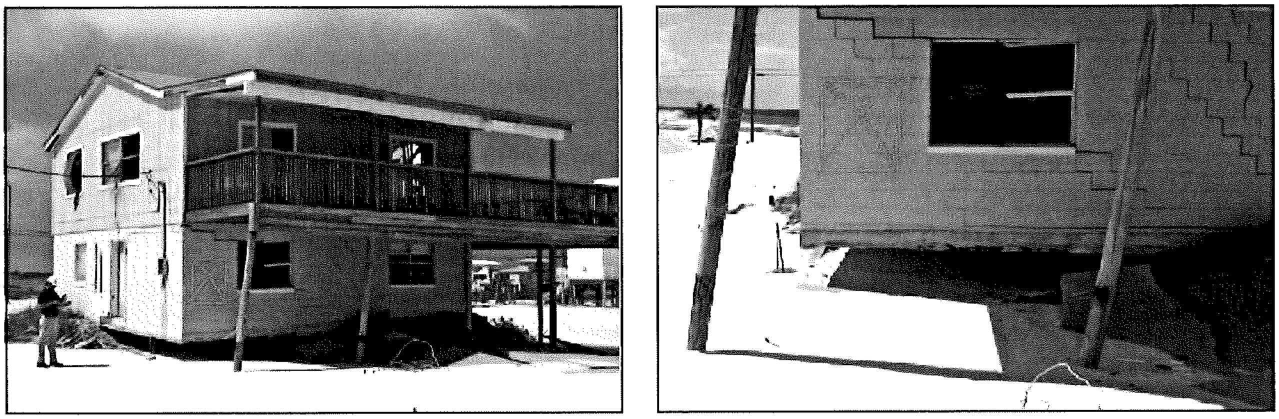

Figure 10-1.
Closed foundation failure due to erosion and scour undermining; photograph on right shows a close-up view of the foundation failure and damaged house wall, Hurricane Dennis (Navarre Beach, FL, 2005)

10.2.3 Deep Foundations

Buildings constructed on deep foundations are supported by soils that are not near grade. Deep foundations include driven timber, concrete or steel piles, and caissons.

Deep foundations are much more resistant to the effects of localized scour and generalized erosion than shallow foundations. Because of that, deep foundations are required in Zone V where scour and erosion effects can be extreme. Open/deep foundations are recommended in Coastal A Zones and in some riverine areas where scour and erosion can undermine foundations.

10.2.4 Shallow Foundations

Buildings constructed on shallow foundations are supported by soils that are relatively close to the ground surface. Shallow foundations include perimeter strip footings, monolithic slabs, discrete pad footings, and some mat foundations. Because of their proximity to grade, shallow foundations are vulnerable to damage from scour and erosion, and because of that, they are not allowed in Zone V and are not recommended in Coastal A Zones unless they extend below the maximum predicted scour and erosion depth.

In colder regions, foundations are typically designed to extend below the frost depth, which can exceed several feet below grade. Extending the foundation below the frost depth is done to prevent the foundation from heaving when water in the soils freeze and to provide adequate protection from scour and erosion. However, scour and erosion depths still need to be investigated to ensure that the foundation is not vulnerable to undermining.

10.3 Foundation Design Requirements and Recommendations

Foundations in coastal areas must elevate the home to satisfy NFIP criteria. NFIP criteria vary for Zone V and Zone A. In Zone V, the NFIP requires that the building be elevated so that the bottom of the lowest horizontal structural member is elevated to the BFE. In Zone A, the NFIP requires that the home be constructed such that the top of the lowest floor is elevated to the BFE.

In addition to elevation, the NFIP contains other requirements regarding foundations. Because of the increased flood, wave, flood-borne debris, and erosion hazards in Zone V, the NFIP requires homes to be elevated on open/deep foundations that are designed to withstand flood forces, wind forces, and forces for flood-borne debris impact. They must also resist scour and erosion.

10.3.1 Foundation Style Selection

Many foundation designs can be used to elevate buildings to the DFE. Table 10-1 shows which foundation styles are acceptable, not recommended, or not permitted in Zone V, Coastal A Zone, and Zone A. Additional information concerning foundation performance can be found in Fact Sheet 3.1, *Foundations in Coastal Areas,* in FEMA P-499.

A best practices approach in the design and construction of coastal foundations is warranted because of the extreme environmental conditions in coastal areas, the vulnerability of shallow foundations to scour and erosion, the fact that the flood loads on open foundations are much lower than those on closed foundations, and foundation failures typically result in extensive damage to or total destruction of the elevated building.

Structural fill can also be used to elevate and support stem wall, crawlspace, solid wall, slab-on-grade, pier, and column foundations in areas not subject to damaging wave action, erosion, and scour. The NFIP precludes the use of structural fill in Zone V. For more information, see FEMA Technical Bulletin 5, *Free-of-Obstruction Requirements* (FEMA 2008b).

10.3.2 Site Considerations

The selected foundation design should be based on the characteristics of the building site. A site characteristic study should include the following:

- **Design flood conditions.** Determine which flood zone the site is located in—Zone V, Coastal A Zone, or Zone A. Flood zones have different hazards and design and construction requirements.
- **Site elevation.** The site elevation and DFE determine how far the foundation needs to extend above grade.
- **Long- and short-term erosion.** Erosion patterns (along with scour) dictate whether a deep foundation is required. Erosion depth affects not only foundation design but also flood loads by virtue of its effect on design stillwater depth (see Section 8.5).
- **Site soils.** A soils investigation report determines the soils that exist on the site and whether certain styles of foundations are acceptable.

10.3.3 Soils Data

Accurate soils data are extremely important in the design of flood-resistant foundations in coastal areas. Although many smaller or less complex commercial buildings and most homes in non-coastal areas are designed without the benefit of specific soils data, all buildings in coastal sites, particularly those in Zone V, should have a thorough investigation of the soils at the construction site. Soils data are available in numerous publications and from onsite soils tests.

10.3.3.1 Sources of Published Soils Data

Numerous sources of soil information are available. Section 12.2 of the *Timber Pile Design and Construction Manual* (Collin 2002) lists the following:

- Topographic maps from the U.S. Geologic Survey (USGS)
- Topographic maps from the Army Map Service
- Topographic maps from the U.S. Coast and Geodetic Survey
- Topographic information from the USACE for some rivers and adjacent shores and for the Great Lakes and their connecting waterways
- Nautical and aeronautical charts from the Hydrographic Office of the Department of the Navy
- Geologic information from State and local governmental agencies, the Association of Engineering Geologists, the Geological Society of America, the Geo-Institute of the American Society of Civil Engineers, and local universities
- Soil survey maps from the Soil Conservation Service of the U.S. Department of Agriculture

10.3.3.2 Soils Data from Site Investigations

Site investigations for soils include surface and subsurface investigations. Surface investigations can identify evidence of landslides, areas affected by erosion or scour, and accessibility for equipment needed for subsurface testing and for equipment needed in construction. Surface investigations can also help identify the suitability or unsuitability of particular foundation styles based on the past performance of existing structures. However, caution should be used when basing the selection of a foundation style solely on the performance of existing structures because the structures may not have experienced a design event.

The 2012 IBC requires that geotechnical investigations be conducted by Registered Design Professionals. Section 1803.2 allows building officials to waive geotechnical investigations where satisfactory data are available from adjacent areas and demonstrate that investigations are not required. The 2012 IRC requires building officials to determine whether soils tests are needed where "quantifiable data created by accepted soil science methodologies indicate expansive, compressible, shifting or other questionable soil characteristics are likely to be present." Because of the hazards in coastal areas, a best practices approach is to follow the 2012 IBC requirements.

Subsurface exploration provides invaluable data on soils at and below grade. The data are both qualitative (e.g., soil classification) and quantitative (e.g., bearing capacity). Although some aspects of subsurface exploration are discussed here, subsurface exploration is too complicated and site-dependent to be covered fully in one document. Consulting with geotechnical engineers familiar with the site is strongly recommended.

Subsurface exploration typically consists of boring or creating test pits, soils sampling, and laboratory tests. The *Timber Pile Design and Construction Manual* (Collin 2002) recommends a minimum of one boring per structure, a minimum of one boring for every 1,000 square feet of building footprint, and a minimum of two borings for structures that are more than 100 feet wide. Areas with varying soil structure and profile dictate more than the minimum number of borings. Again, local geotechnical engineers should be consulted.

The following five types of data from subsurface exploration are discussed in the subsections below: soil classification, bearing capacity, compressive strength, angle of internal friction, and subgrade modulus.

Soil Classification

Soil classification qualifies the types of soils present along the boring depth. ASTM D2487-10 is a consensus standard for soil classification. Soil classification is based on whether soils are cohesive (silts and clays) or non-cohesive (composed of granular soils particles). The degree of cohesiveness affects foundation design. Coupled with other tests such as the plasticity/Atterburg Limits soil classification can identify unsuitable or potentially problematic soils. Table 10-2 contains the soil classifications from ASTM D2487-10. ASTM D2488-09a is a simplified standard for soil classification that may be used when directed by a design professional.

Bearing Capacity

Bearing capacity is a measure of the ability of soil to support gravity loads without soil failure or excessive settlement. Bearing capacity is generally measured in pounds/square foot and occasionally in tons/square foot. Soil bearing capacity typically ranges from 1,000 pounds/square foot (relatively weak soils) to more than 10,000 pounds/square foot (bedrock).

Bearing capacity has a direct effect on the design of shallow foundations. Soils with lower bearing capacities require proportionately larger foundations to effectively distribute gravity loads to the supporting soils. For deep foundations, like piles, bearing capacity has less effect on the ability of the foundation to support gravity loads because most of the resistance to gravity loads is developed by shear forces along the pile.

Presumptive allowable load bearing values of soils are provided in the 2012 IBC and the 2012 IRC. Frequently, designs are initially prepared based on presumed bearing capacities. The builder's responsibility is to verify that the actual site conditions agree with the presumed bearing capacities. As a ***best practices approach***, the actual soil bearing capacity should be determined to allow the building design to properly account for soil capacities and characteristics.

Compressive Strength

Compressive strength is typically determined by Standard Penetration Tests. Compressive strength controls the design of shallow foundations via bearing capacity and deep foundations via the soil's resistance to lateral loads. Compressive strength is also considered when determining the capacity of piles to resist vertical loads.

Compressive strength is determined by advancing a probe, 2 inches in diameter, into the bottom of the boring by dropping a 140-pound slide hammer a height of 30 inches. The number of drops, or blows, required to advance the probe 6 inches is recorded. Blow counts are then correlated to soil properties.

Table 10-2. ASTM D2487-10 Soil Classifications

Major Divisions			Group Symbol	Typical Names	Classification Criteria		
Coarse-grained soils more than 50% retained on No. 200 sieve	Gravels: 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines: • Less than 5% pass No. 200 sieve: GW, GP, SW, SP • More than 12% pass No. 200 sieve: GM, GC, SM, SC • 5% to 12% pass No. 200 sieve: borderline classification requiring dual symbols	$C_u = \frac{D_{60}}{C_{10}}$ greater than 4 $C_Z = \frac{(D_{30})^2}{(D_{10})(D_{60})}$ between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits plot above "A" line or plasticity index less than 7	
	Sands: More than 50% of coarse fraction passes No. 4 sieve	Clean sands	SW	Well-graded sands and gravelly sands, little or no fines		$C_u = \frac{D_{60}}{C_{10}}$ greater than 6 $C_Z = \frac{(D_{30})^2}{(D_{10})(D_{60})}$ between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with fines	SM	Silty sands, sand-silt mixtures		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.
			SC	Clayey sands, sand-clay mixtures		Atterberg limits plot above "A" line or plasticity index greater than 7	