v2

# 2010 Florida Building Code Wind Standard


**Building Codes and Standards**

Mo Madani, CBO, Technical Unit Manager

Joe Bigelow, Presentation Designer, Florida Building Code Staff


Florida Department of Business & Professional Regulation

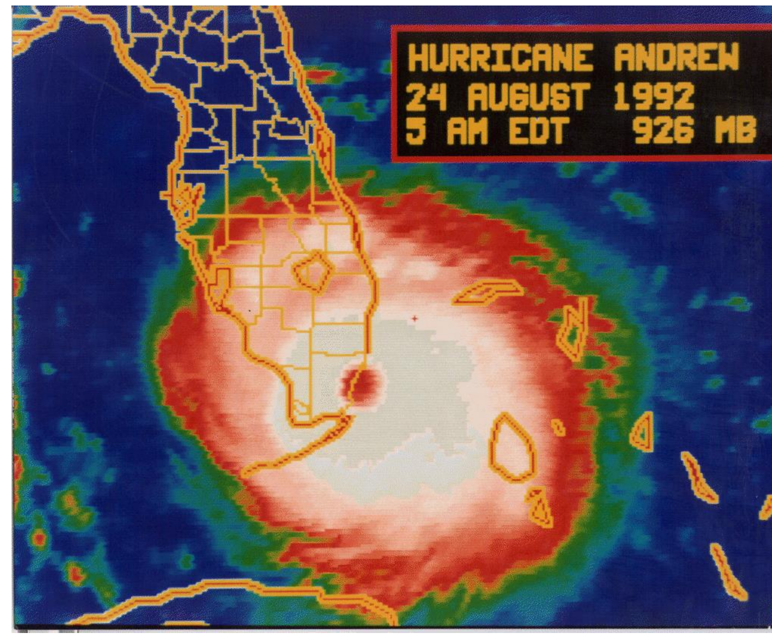
Building Codes and Standards

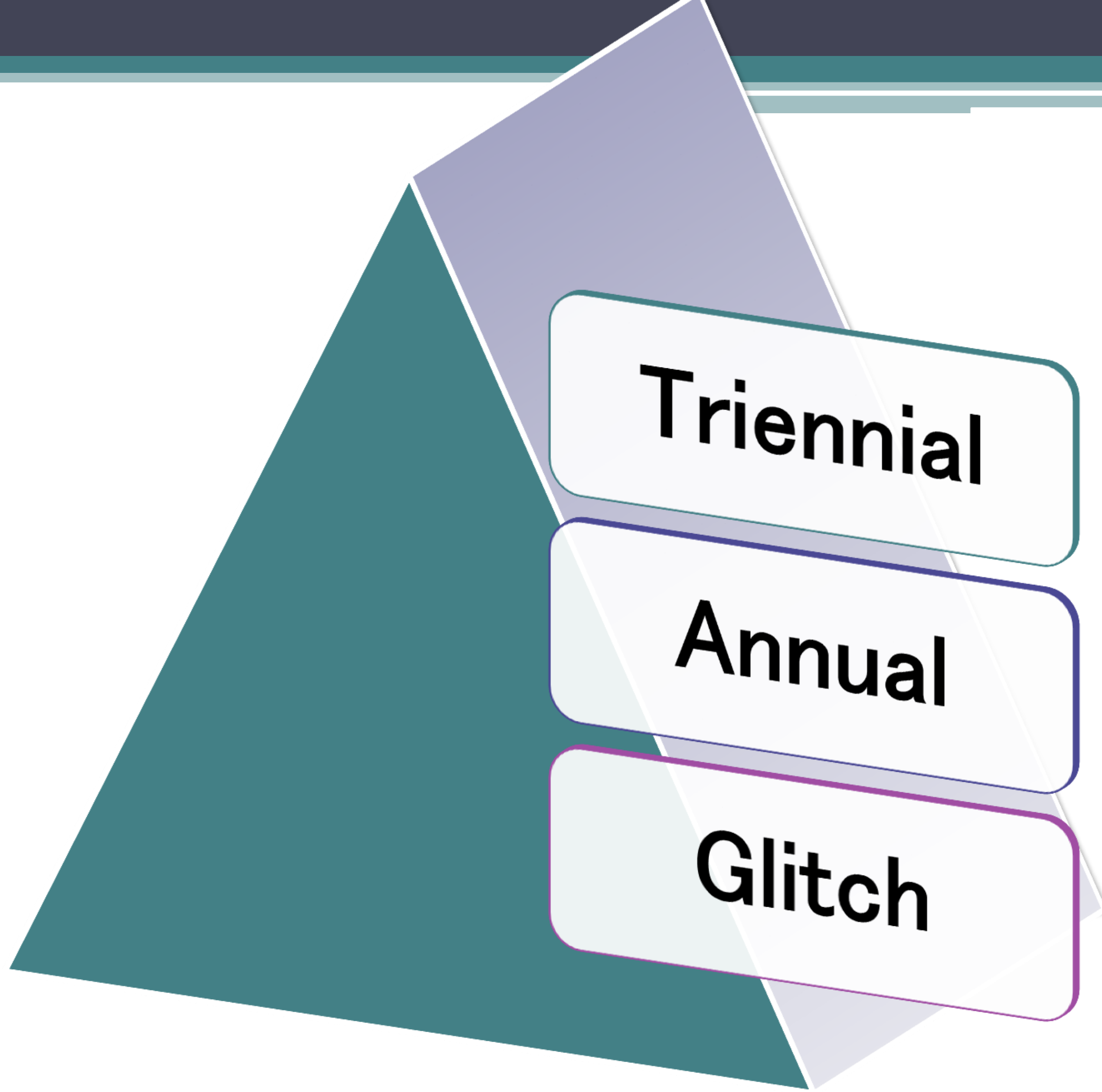
FBC
Florida Building Commission

- Contents
  - Part 1 – The Florida Building Code
  - Part 2 – Wind Speeds
  - Part 3 – Impact

# Part 1

## The Florida Building Code




Triennial
Annual
Glitch

- How to use the next chart

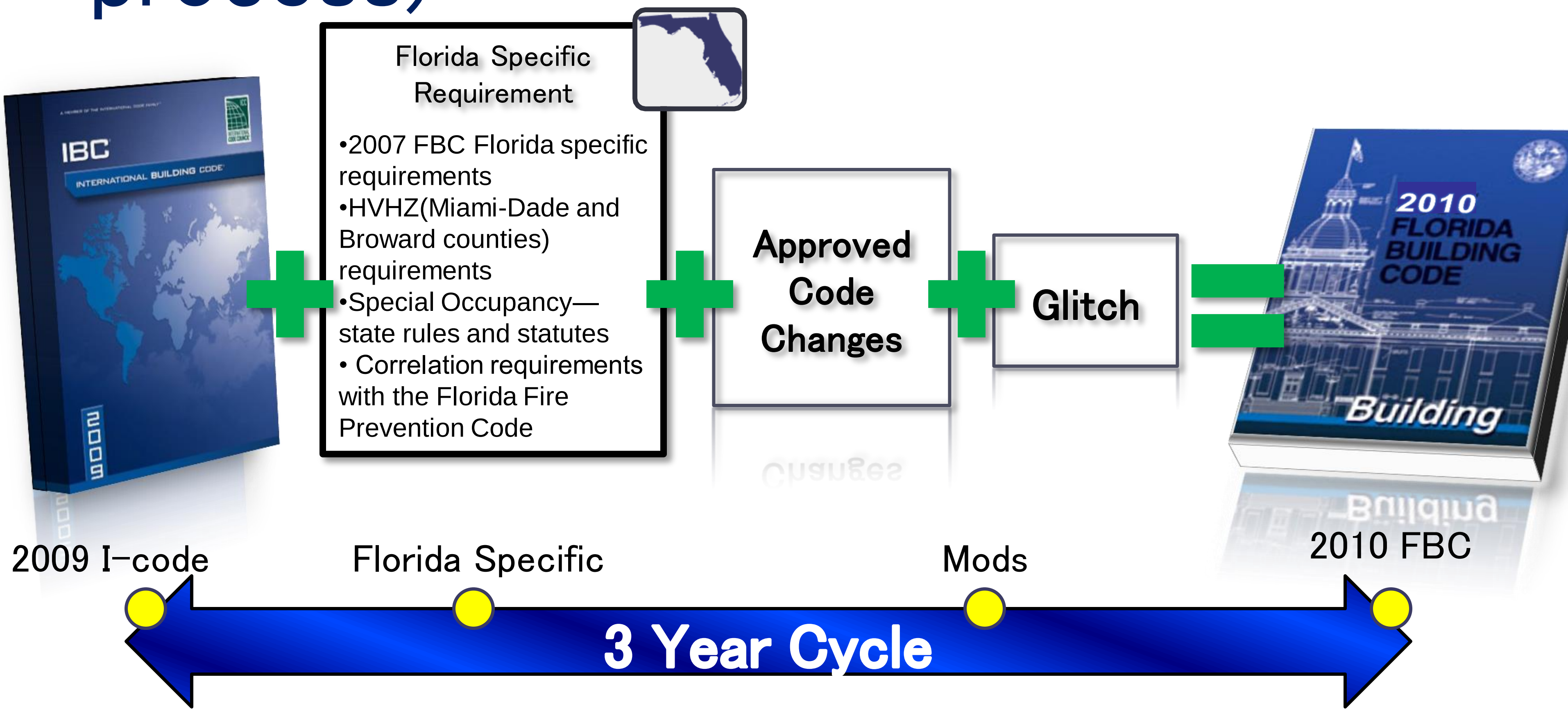
| | 2001 FBC | 2004 FBC |
|---|---|---|
| Effective Date | March 1, 2002 | October 1, 2005 |
| Building Volume – Based on... | 1997 Southern Building Code | 2003 IBC |
| Plumbing Volume – Based on... | 1997 IPC | 2003 IPC |
| Mechanical Volume – Based on... | 1998 [illegible] | 2003 IMC |

The 2004 Florida Building Code Mechanical Volume is based on the 2003 IMC (International Mechanical Code)

# Florida Building Code Editions

| | 2001 FBC | 2004 FBC | 2007 FBC | 2010 FBC |
|---|---|---|---|---|
| **Effective Date** | **March 1, 2002** | **October 1, 2005** | **March 1, 2009** | **March 15, 2012** |
| **Building** Volume – Based on… | 1997 Southern Building Code | 2003 IBC | 2006 IBC | 2009 IBC |
| **Plumbing** Volume – Based on… | 1997 IPC | 2003 IPC | 2006 IPC | 2009 IPC |
| **Mechanical** Volume – Based on… | 1998 IMC | 2003 IMC | 2006 IMC | 2009 IMC |
| **Fuel/Gas** Volume – Based on… | 1998 IFGC | 2003 IFGC | 2006 IFGC | 2009 IFGC |
| **Electrical** Volume – Based on… | 2002 NEC | 2005 NEC | 2005 NEC | 2008 NEC |
| **Fire** Volume – Based on… | 2001 FFPC | 2004 FFPC | 2007 FFPC | 2010 FFPC |
| **Existing Building** Volume – Based on… | --- | 2003 IEBC | 2006 IEBC | 2009 IEBC |
| **Residential** Volume – Based on… | --- | 2003 IRC | 2006 IRC | 2009 IRC |
| **Energy Conservation** Volume – Based on… | --- | --- | --- | 2009 IECC |
| **Accessibility** Volume – Based on… | --- | --- | --- | 2010 ADA |

# 2010 FBC Integration (current process)

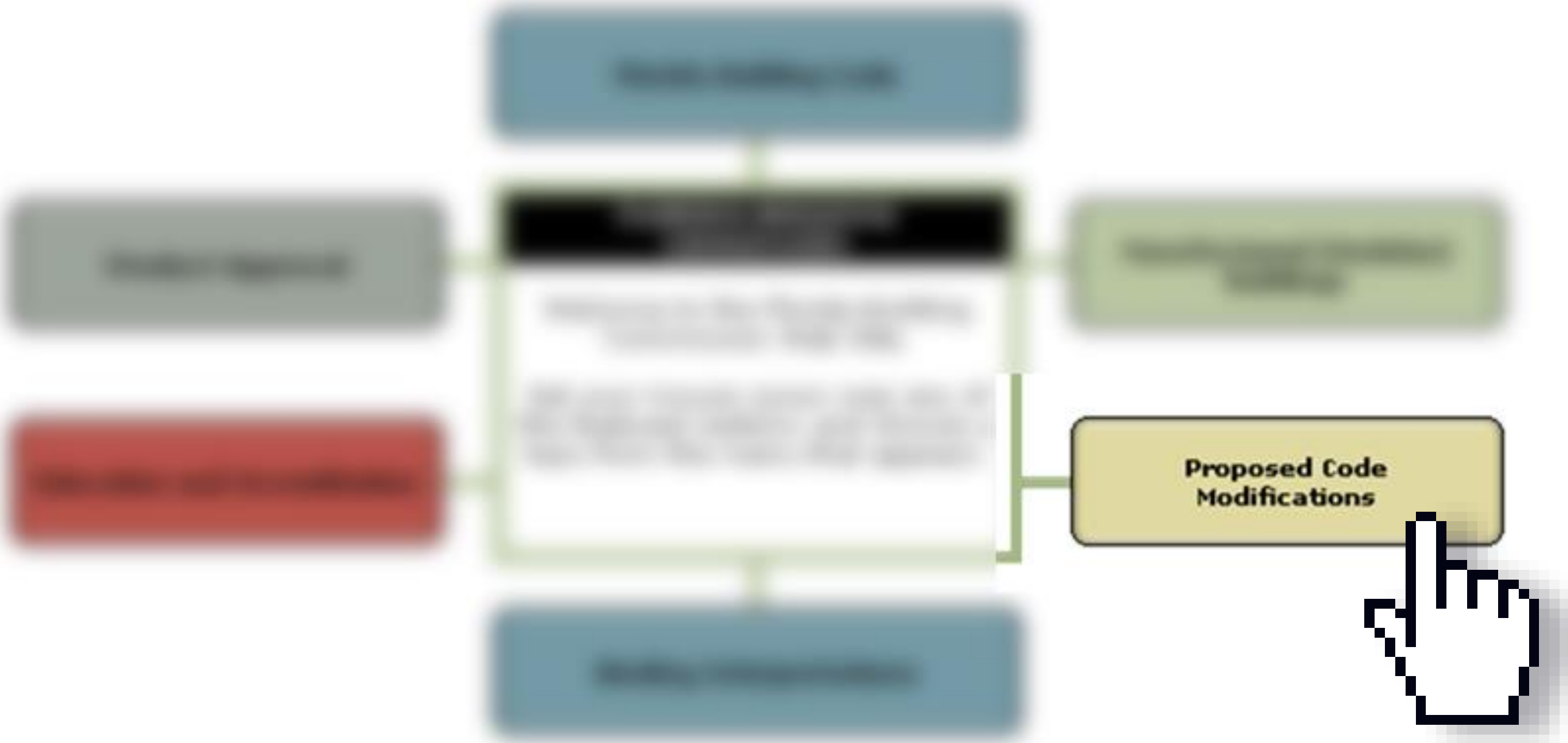


## Code development Schedule

| | |
|---|---|
| • 2009 I- Codes and FBC Supplement posted | 2/1/10 |
| • Proposed amendment due and closes | 4/2/10 |
| • Proposed amendment posted for comments | 4/15/10 |
| • 45 day comment period ends | 5/31/10 |
| • TAC review and make recommendations | 7/27-8/25 |
| • TAC recommendations posted | 9/3/10 |
| • 45 day comment period ends | 10/18/10 |
| • TAC review comments on recommendations | 11/15/10 |
| • Commission considers TAC recommendation | 12/7-8/10 |
| • Code amended to resolve glitches | 4/11-6/11 |
| • Code printed | 10/1/11 |
| • **Code implemented** | 3/15/12 |

# Materials Available

## BCIS www.floridabuilding.org
## Proposed Code Module –



# FBC 2010

- **1609.1.1 Determination of wind loads.** Wind loads on every building or structure shall be determined in accordance with Chapters 26 through 30 of ASCE 7 **or the provisions of the alternate all-heights method in Section 1609.6**. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.
-
- **Exceptions:**
- 1. ICC 600 for Group R-2 and R-3 buildings.
- 2. AF&PA WFCM.
- 3. AISI S230.
- 4. Designs using NAAMM FP 1001.
- 5. Designs using TIA-222 for antenna-supporting structures and antennas.
- 6. Wind tunnel tests in accordance with Section 6.6 of ASCE 7. subject to the limitations in Section 1609.1.1.2.

# FBC 201 continued

- The wind speeds in Figure 1609A, 1609B and 1609C shall be converted **to nominal wind speeds, $V_{asd}$,** in accordance with Section 1609.3.1 when the provisions of the standards referenced in Exceptions 1 through 5 **and 7** are used unless the wind provisions in the standards are based on Ultimate Wind Speeds **as specified** in ~~**accordance with**~~ Figures 1609A, 1609B, or 1609C **or Chapter 26 of ASCE 7.**
- [S4673]

# Part 2 – Wind Speeds

## ASCE 7-10

## TABLE 1604.5 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

| OCCUPANCY CATEGORY | NATURE OF OCCUPANCY |
|---|---|
| I | **Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:**<br>• Agricultural facilities.<br>• Certain temporary facilities.<br>• Minor storage facilities. |
| II | **Buildings and other structures except those listed in Occupancy Categories I, III and IV** |
| III | **Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:**<br>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.<br>• Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250.<br>• Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500.<br>• Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities.<br>• Group I-3 occupancies.<br>• Any other occupancy with an occupant load greater than 5,000[a].<br>• Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV.<br>• Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. |
| IV | **Buildings and other structures designated as essential facilities, including but not limited to:**<br>• Group I-2 occupancies having surgery or emergency treatment facilities.<br>• Fire, rescue, ambulance and police stations and emergency vehicle garages.<br>• Designated earthquake, hurricane or other emergency shelters.<br>• Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.<br>• Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures.<br>• Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2).<br>• Aviation control towers, air traffic control centers and emergency aircraft hangars.<br>• Buildings and other structures having critical national defense functions.<br>• Water storage facilities and pump structures required to maintain water pressure for fire suppression. |

# Risk Category of Buildings and Other Structures

| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| OCCUPANCY | CATEGORY NATURE OF OCCUPANCY |
| I  | Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:<br>· Agricultural facilities.<br>· Certain temporary facilities.<br>· Minor storage facilities. |

# Risk Category of Buildings and Other Structures Cont.

| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| OCCUPANCY | CATEGORY NATURE OF OCCUPANCY |
| II  | Buildings and other structures except those listed in Occupancy Categories I, III and IV |

# Risk Category of Buildings and Other Structures Cont

| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| OCCUPANCY | CATEGORY NATURE OF OCCUPANCY |
| III  | Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:<br>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. |

# Risk Category of Buildings and Other Structures Cont.

| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| **OCCUPANCY** | **CATEGORY NATURE OF OCCUPANCY** |
| **III**<br> | · Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities.<br>· Group I-3 occupancies.<br>· Any other occupancy with an occupant load greater than 5,000a.<br>· Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public<br>utility facilities not included in Occupancy Category IV.<br>· Buildings and other structures not included in Occupancy |

# Risk Category of Buildings and Other Structures Cont.

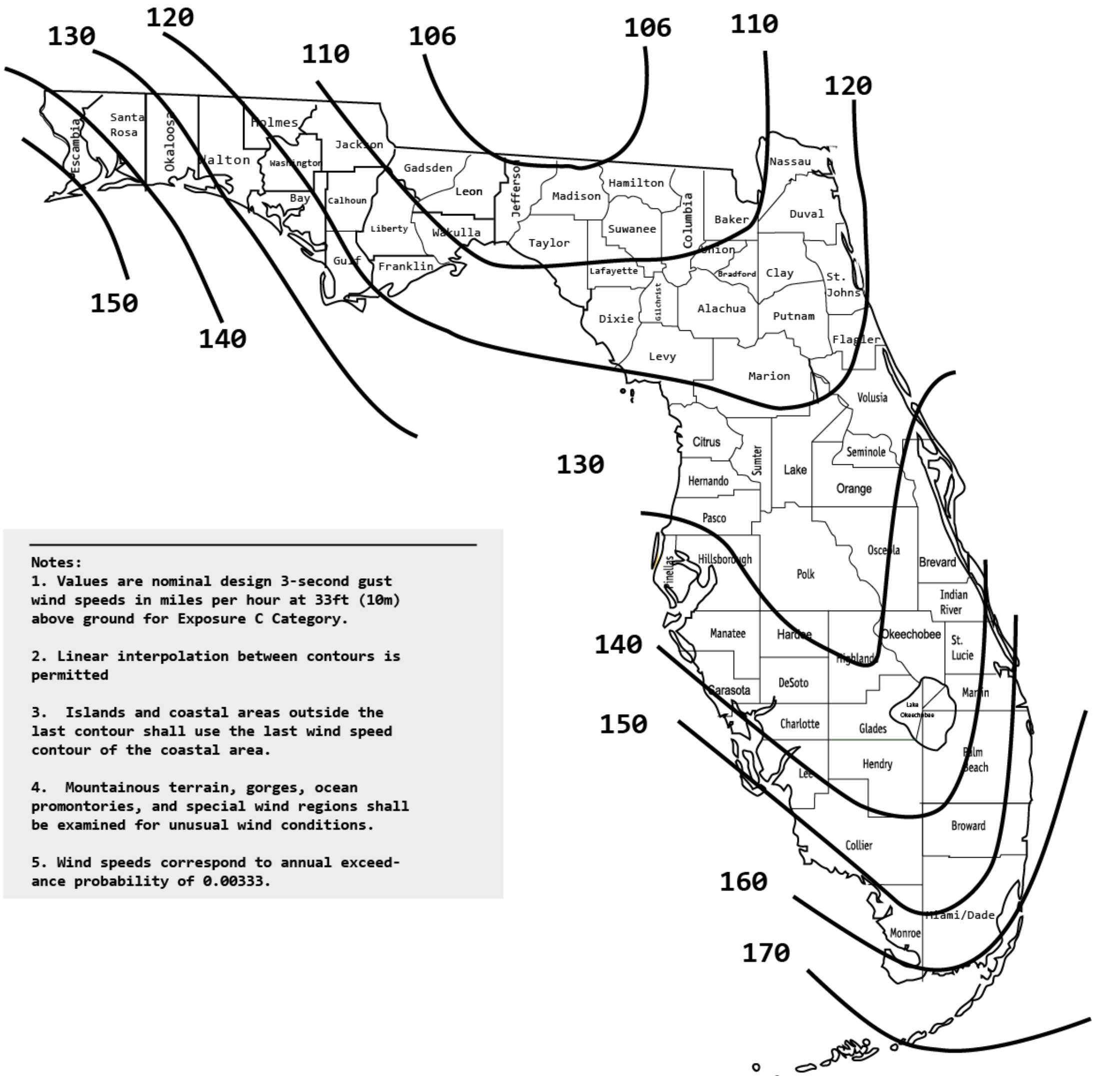
| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| OCCUPANCY | CATEGORY NATURE OF OCCUPANCY |
| **IV**  | Buildings and other structures designated as essential facilities, including but not limited to:<br>• Group I-2 occupancies having surgery or emergency treatment facilities.<br>• Fire, rescue, ambulance and police stations and emergency vehicle garages.<br>• Designated earthquake, hurricane or other emergency shelters. |

# Risk Category of Buildings and Other Structures Cont.

| | 2010 Florida Building Code – Projected to go into effect 12/31/2011 |
|---|---|
| OCCUPANCY | CATEGORY NATURE OF OCCUPANCY |
| IV  | · Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2).<br>· Aviation control towers, air traffic control centers and emergency aircraft hangars.<br>· Buildings and other structures having critical national defense functions.<br>· Water storage facilities and pump structures required to maintain water pressure for fire suppression. |

# 2010 FBC Figure C

**2010 FBC**
**State of Florida**
**Category I Building and Structures**
**In Miles Per Hour**
**Figure C**




**Notes:**
1. Values are nominal design 3-second gust wind speeds in miles per hour at 33ft (10m) above ground for Exposure C Category.

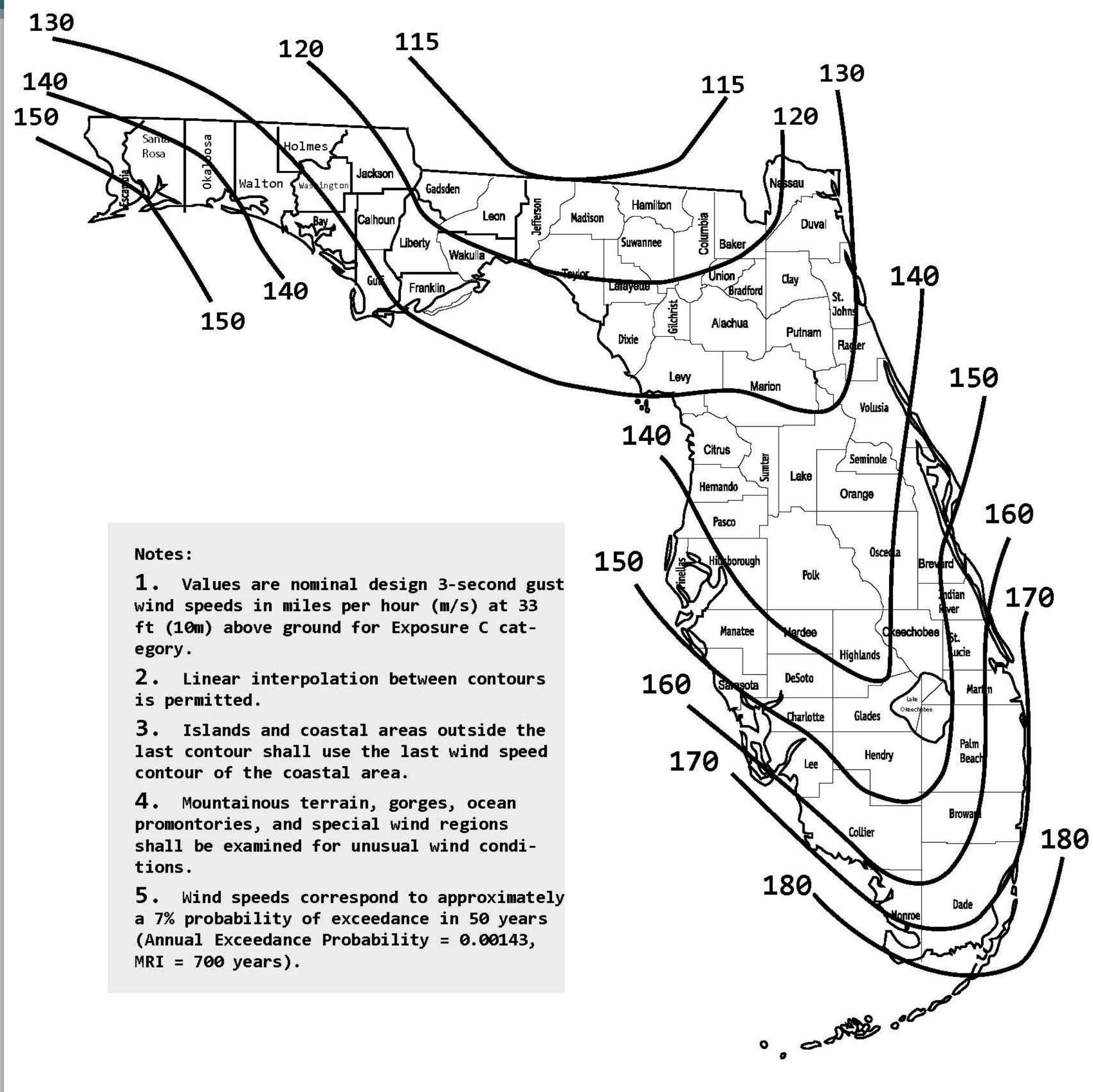
2. Linear interpolation between contours is permitted

3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.

4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

5. Wind speeds correspond to annual exceedance probability of 0.00333.

# FINAL FIGURE A




Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.

2. Linear interpolation between contours is permitted.

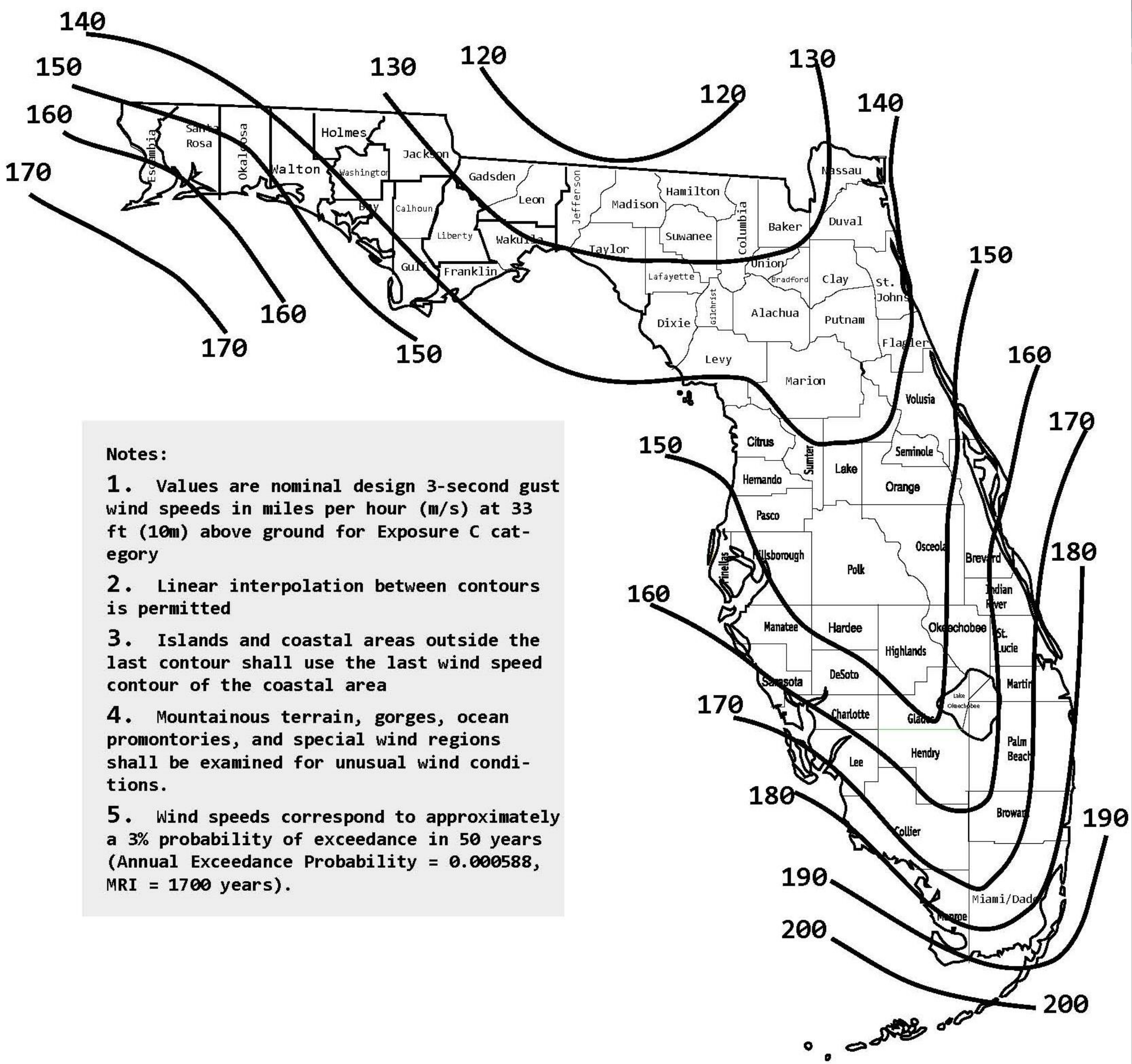
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.

4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 years).

Figure 1609A Ultimate Design Wind Speeds, $V_{ult}$, for Risk Category II Buildings and Other Structures

# FINAL FIGURE B




Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category
2. Linear interpolation between contours is permitted
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 years).

Figure 1609B Ultimate Design Wind Speeds, $V_{ult}$, for Risk Category III and IV Buildings and other Structures

**TABLE C26.5-6**
**Design Wind Speeds: ASCE 7-93 to ASCE 7-10**

| *ASCE 7-05 Design Wind Speed (3-sec gust in mph)* | *ASCE 7-10 Design Wind Speed (3-sec gust in mph)* | *ASCE 7-93 Design Wind Speed (fastest mile in mph)* |
|---|---|---|
| 85 | 110* | 71 |
| 90 | 115* | 76 |
| 100 | 126 | 85 |
| 105 | 133 | 90 |
| 110 | 139 | 95 |
| 120 | 152 | 104 |
| 130 | 164 | 114 |
| 140 | 177 | 123 |
| 145 | 183 | 128 |
| 150 | 190 | 133 |
| 170 | 215 | 152 |

* Wind speed values of 110 mph and 115 mph were rounded from the "exact" conversions of $85\sqrt{1.6} = 108$ and $90\sqrt{1.6} = 114$ mph, respectively.

# ASCE 7 – 05 VS. ASCE 7 – 10 Design Wind Loads

TABLE 1609.3.1

WIND SPEED CONVERSIONS[abc]

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $V_{ul}$ | 100 | 110 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 190 | 200 |
| $V_{asd}$ | 78 | 85 | 93 | 101 | 108 | 116 | 124 | 132 | 139 | 147 | 155 |

$V_{asd}$ = nominal design wind speed

$V_{ult}$ = ultimate design wind speed determined from Figures 1609A, 1609B, or 1609C

# ASCE 7 – 05 VS. ASCE 7 – 10 Design Wind Loads

The new maps, when used in combination with the 1.0 load factor on wind for strength design and the 0.6 factor on wind for allowable stress design, **result in a net decrease in design wind loads in Hurricane-Prone Regions.** Parts of southern Florida (due to the re-introduction of Exposure D for coastal areas) are approximately the same when compared to previous editions of the maps. **In the remainder of the Hurricane-Prone Regions of Florida, the design wind pressures are on average approximately *20% less* than the loads determined from ASCE 7-05.**

# ASCE 7 – 05 VS. ASCE 7 – 10

## Design Wind Loads

Nominal design wind speed "$V_{asd}$"– using Allowable Stress Design (ASCE 7 – 2005) – old maps

Ultimate design wind speed "$V_{ult}$" – using Strength Design (ASCE – 2010) – new maps

**In order to convert the load "dp/design pressure" from ultimate to nominal you multiply by a factor of .6.**

**x .6**

# ASCE 7 – 05 VS. ASCE 7 – 10 Design Wind Loads

2007 FBC ASCE 7-1 0 ASCE 7 – 10 ASCE 7 – 10
Cat. I Cat. B. II Cat. B III &
IV

__________________________________

Palm Beach 130 150 160
170
MPH __________________________________

Wall -cc 30.4/-33.0 40.5/-43.9 46.1/-50.0 51.1/-56
dp – psf __________________________________

(X .6) 24.3/-26.34 27.66/-30.0
30.66/-33.6

# HVHZ

- **1620.2** – Change to read as shown.
- 1620.2 Wind velocity (3-second gust) used in structural calculations shall be as follows:
- **Miami-Dade County**
- Risk Category I Buildings and Structures: 165 mph
- Risk Category II Buildings and Structures: 175 mph
- Risk Category III and IV Buildings and Structures: 186 mph
- Broward County
- Risk Category I Buildings and Structures: 156 mph
- Risk Category II Buildings and Structures: 170 mph
- Risk Category III and IV Buildings and Structures: 180 mph
- [S4799]

- Part 3 – Impact

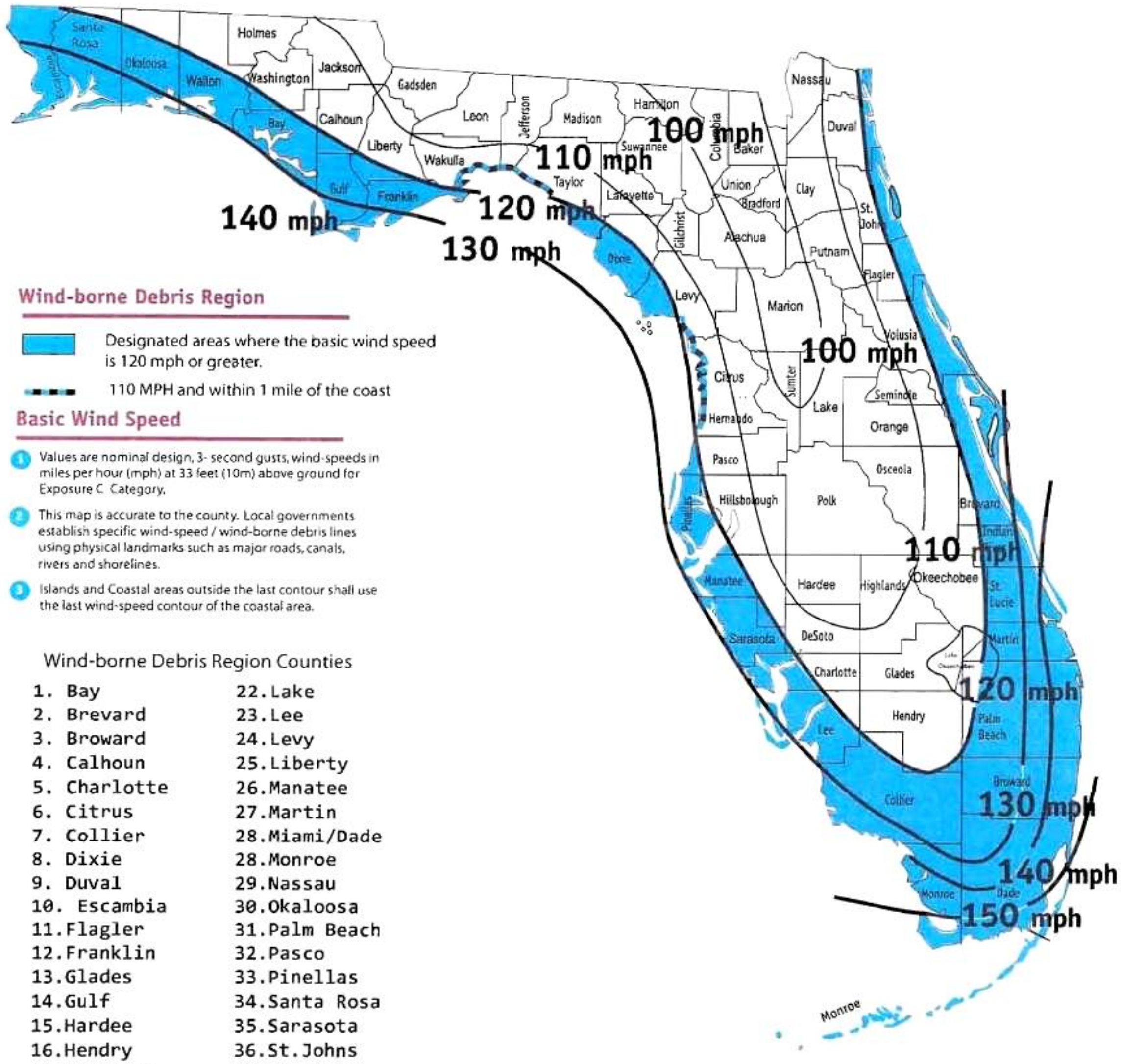
# Wind-Borne Debris Region

- Areas within *hurricane- prone regions* located:
- Within **1 mile** (1.61 km) of the coastal mean high water line where the ultimate design wind speed **$V_{ult}$ is 130** (48 m/s) or greater; or
- In areas where the ultimate design wind speed $V_{ult}$ is **140 mph** (53 m/s) or greater~~;~~
- For **Risk Category II** buildings and structures and occupancy **category III** buildings and structures, except health care facilities, the windborne debris region shall be based on **Figure 1609A**. For occupancy **category IV** buildings and structures and occupancy **category III** health care facilities, the windborne debris region shall be based on **Figure 1609B**.

# 2007 FBC

2007 FBC

## Wind-Borne Debris Region




### Wind-borne Debris Region

Designated areas where the basic wind speed is 120 mph or greater.

110 MPH and within 1 mile of the coast

### Basic Wind Speed

1. Values are nominal design, 3- second gusts, wind-speeds in miles per hour (mph) at 33 feet (10m) above ground for Exposure C Category.
2. This map is accurate to the county. Local governments establish specific wind-speed / wind-borne debris lines using physical landmarks such as major roads, canals, rivers and shorelines.
3. Islands and Coastal areas outside the last contour shall use the last wind-speed contour of the coastal area.

### Wind-borne Debris Region Counties

| | |
|---|---|
| 1. Bay | 22. Lake |
| 2. Brevard | 23. Lee |
| 3. Broward | 24. Levy |
| 4. Calhoun | 25. Liberty |
| 5. Charlotte | 26. Manatee |
| 6. Citrus | 27. Martin |
| 7. Collier | 28. Miami/Dade |
| 8. Dixie | 28. Monroe |
| 9. Duval | 29. Nassau |
| 10. Escambia | 30. Okaloosa |
| 11. Flagler | 31. Palm Beach |
| 12. Franklin | 32. Pasco |
| 13. Glades | 33. Pinellas |
| 14. Gulf | 34. Santa Rosa |
| 15. Hardee | 35. Sarasota |
| 16. Hendry | 36. St. Johns |
| 17. Hernando | 37. St. Lucie |
| 18. Highlands | 38. Taylor |
| 19. Hillsborough | 39. Volusia |
| 20. Indian River | 40. Walton |
| 21. Jefferson | 41. Washington |

www.florida building.org

# 2010 FBC Figure A

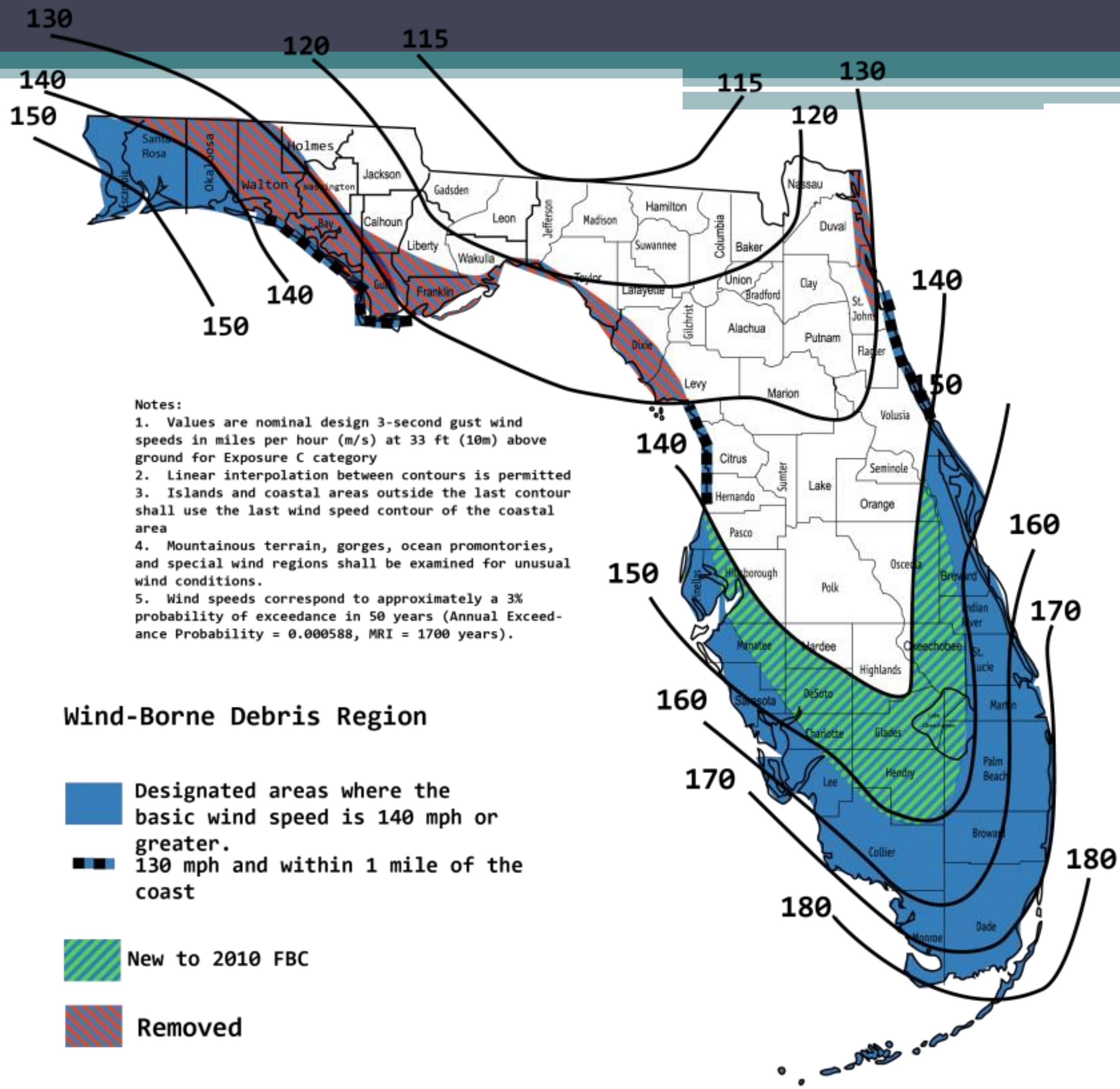



Figure 1609A Wind-Borne Debris Region, Category II and III
Buildings and Structures except health care facilities in Miles Per Hour

# 2010 FBC Figure B

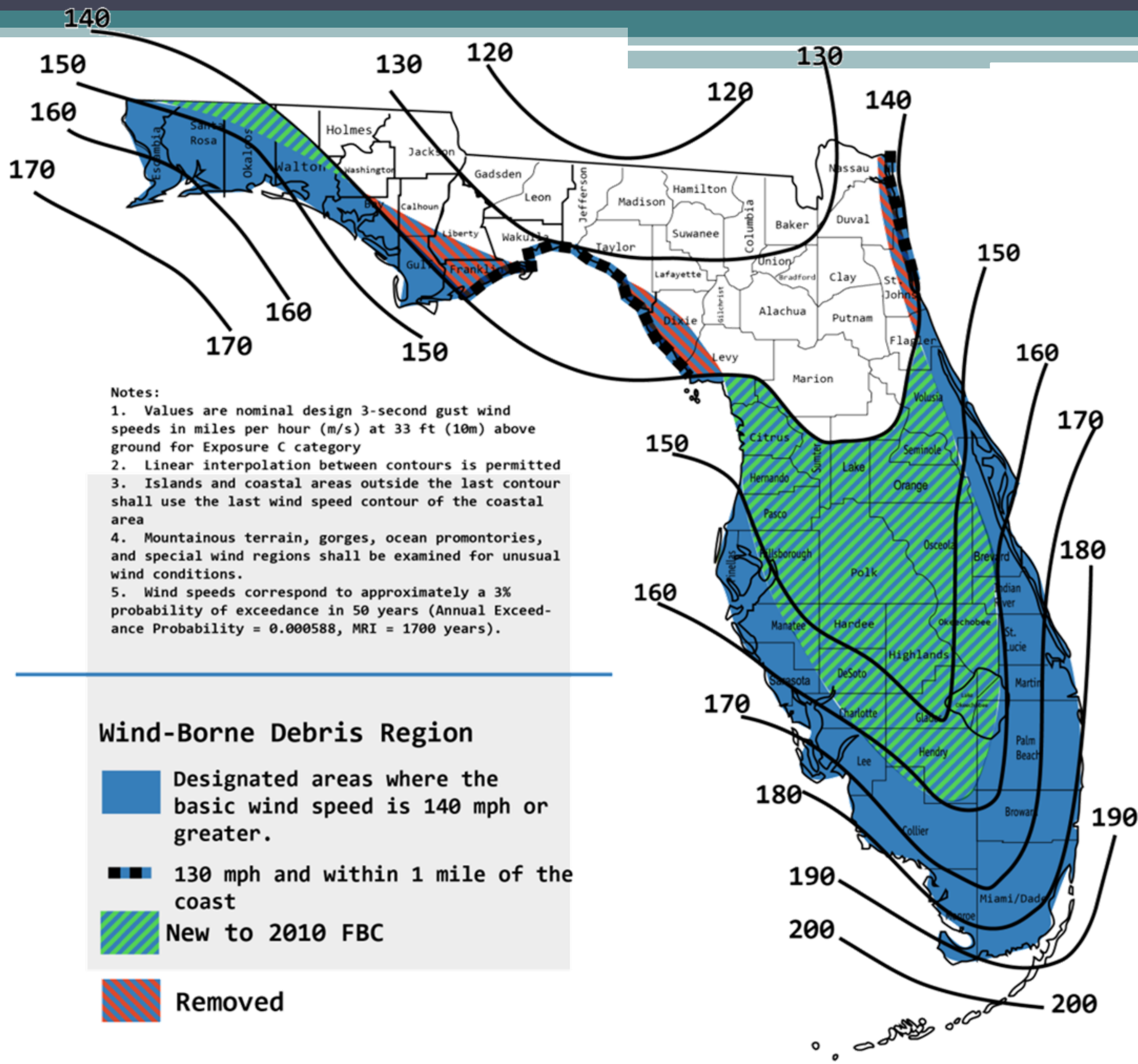



Figure 1609B Risk Category III and IV Buildings and other Structures and Category III healthcare facilities

# FINAL FIGURE R301.2(4)

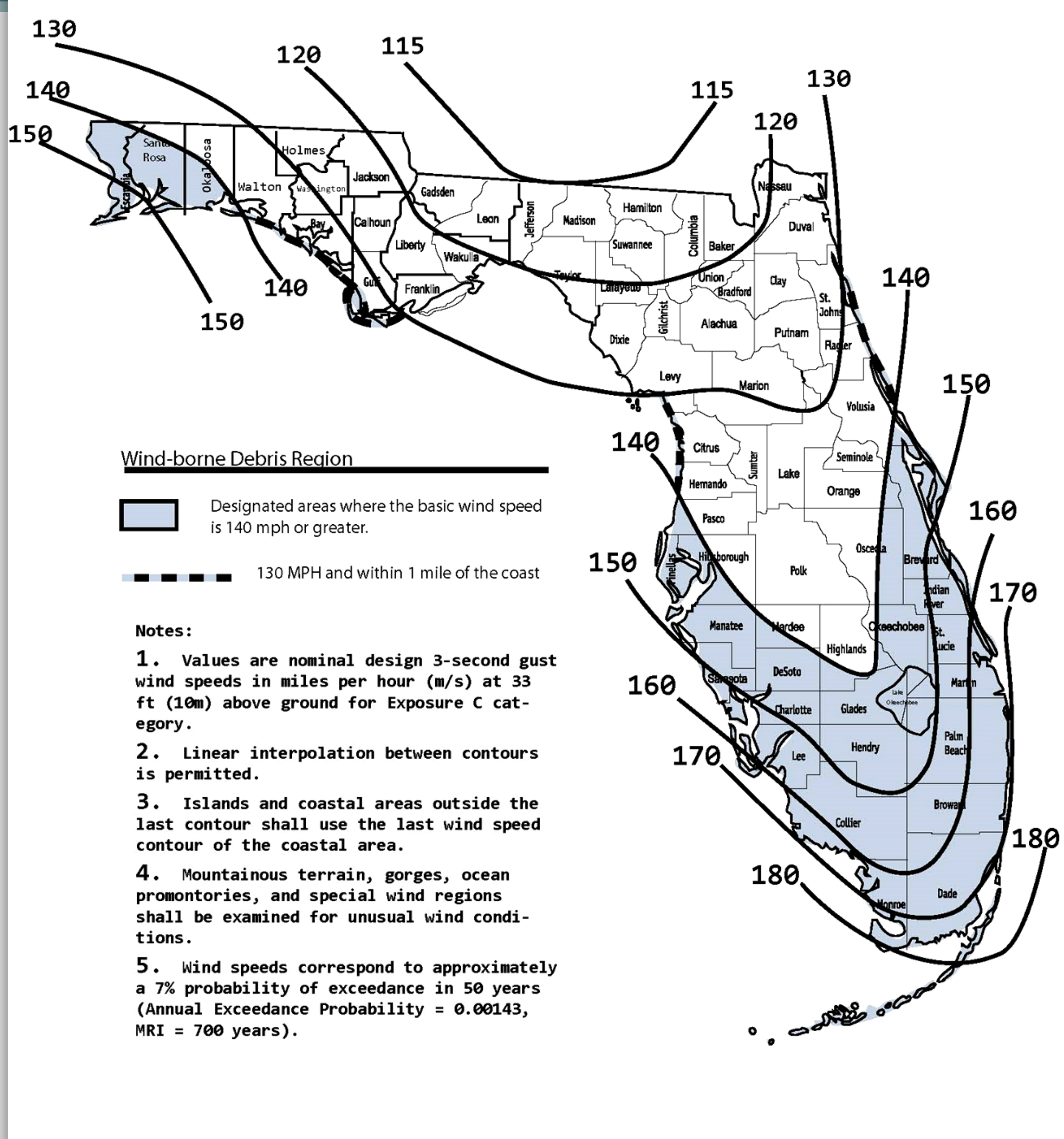
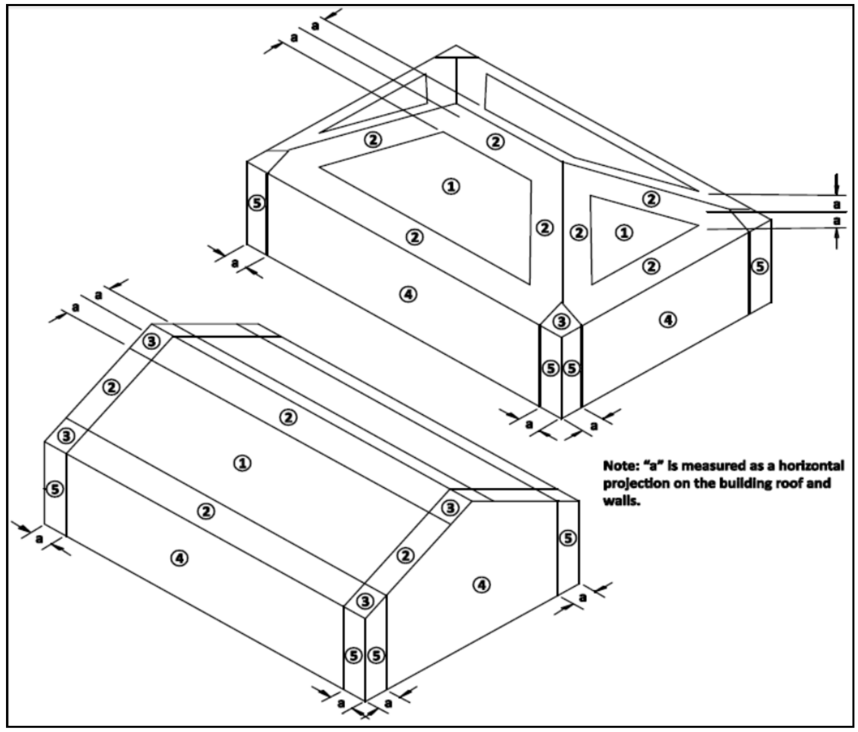



Figure R301.2(4) Ultimate Design Wind Speeds, $V_{ult}$

# Background Information

- Wind speed lines have changed because of the improved science (i.e. computer simulations).
- The updated maps are based on a new and more complete analysis of hurricane characteristics performed over the past 10 years.
- The wind speed indicated for each wind speed line is different for the 2010 standard due to change in wind speed calculation philosophy.
- The wind borne debris was changed from opening protection required for currently for 120 mph and higher wind speeds under current code to opening protection required for 110 mph equivalent current code wind speed (140 mph 2010 wind speeds) and higher wind speeds for the 2010 standard.


# CALCULATING WIND LOADS ON LOW-RISE STRUCTURES PER 2015 WFCM ENGINEERING PROVISIONS (STD342-1)

**John "Buddy" Showalter, P.E.**
**Vice President, Technology Transfer**
**American Wood Council**

CONTINUING EDUCATION AIA

ICC PREFERRED EDUCATION PROVIDER

## Description

The Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings (ANSI/AWC WFCM-2015) is referenced in the 2015 International Building Code and 2015 International Residential Code. For WFCM wind load calculations, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) is used. The 2015 WFCM includes design information for buildings located in regions with 700-year return period "three second gust" design wind speeds between 110 and 195 mph. ASD wind pressures for Main Wind-Force Resisting Systems (MWFRS) and Components and Cladding (C&C) are computed. Shear, uplift, and overturning loads are calculated for various building components. WFCM Chapter 2 provides minimum loads for the purpose of establishing specific resistance requirements for buildings within the scope of the document. This presentation will provide background and examples for calculation of these forces which will enable designers and code officials to quickly determine wind design loads for projects.

## Learning Objectives

Upon completion of this webinar, participants will:

1. Understand applicable wind loads from ASCE 7-10 for structures within the WFCM scope.
2. Be familiar with application of MWFRS versus C&C loads for various building components and systems.
3. Be familiar with shear, uplift, and overturning wind loads for various building components.
4. Be familiar with tabulated values and their basis in WFCM Chapter 2 for wind loads.

This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission of AWC is prohibited.

AMERICAN WOOD COUNCIL

© American Wood Council 2016

---

- The American Wood Council is a Registered Provider with The American Institute of Architects Continuing Education Systems (AIA/CES), Provider # 50111237.
- Credit(s) earned on completion of this course will be reported to AIA CES for AIA members. Certificates of Completion for both AIA members and non-AIA members are available upon request.

- This course is registered with AIA CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing, or dealing in any material or product.
- Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

# Polling Question

**What is your profession?**

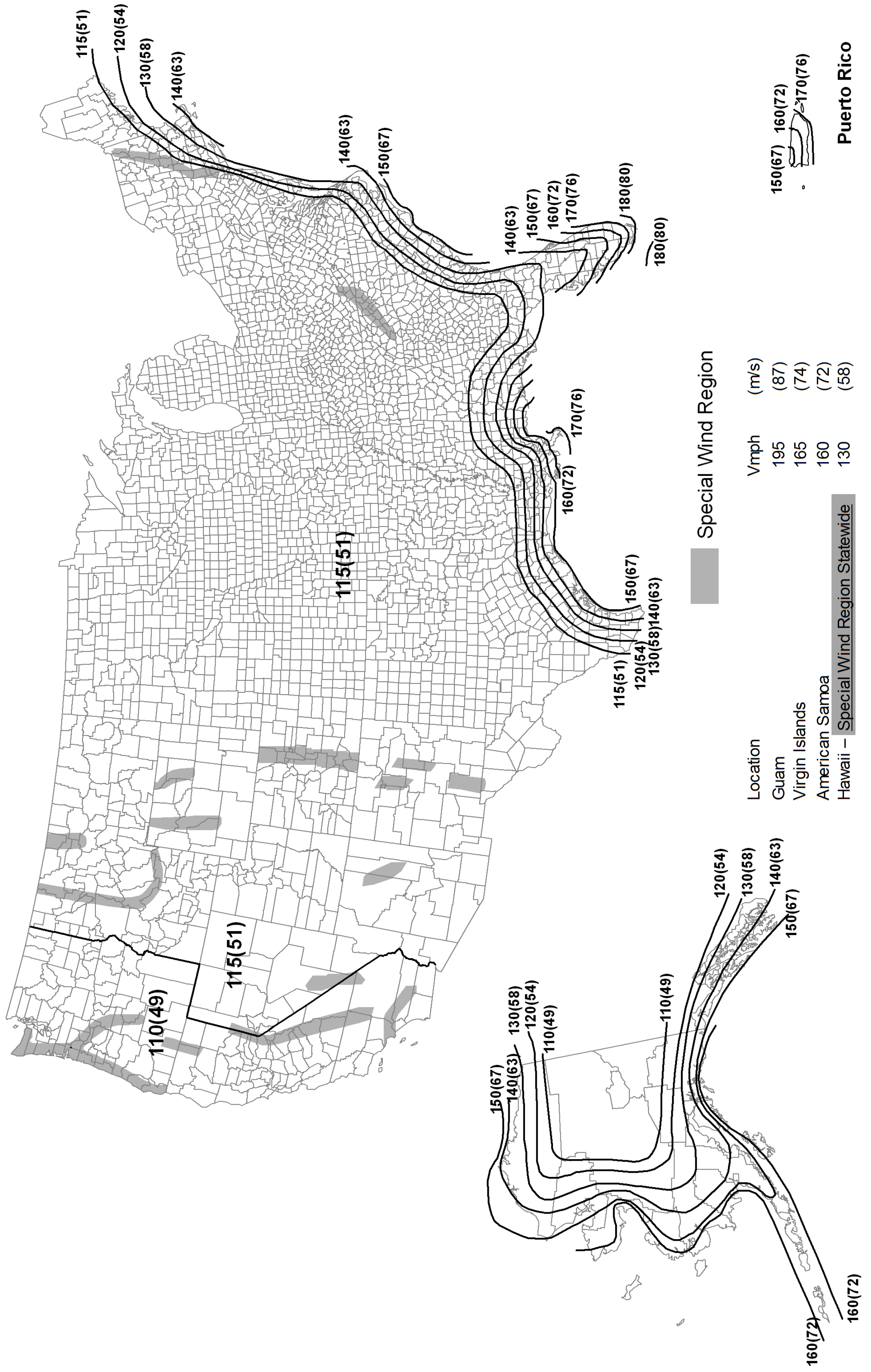
a) Architect
b) Engineer
c) Code Official
d) Builder
e) Other

## Table 1 Applicability Limitations

| Attribute | Limitation | Reference Section | Figures |
|---|---|---|---|
| **BUILDING DIMENSIONS** | | | |
| **Mean Roof Height (MRH)** | 33' | 1.1.3.1a | 1.2 |
| **Number of Stories** | 3 | 1.1.3.1a | - |
| **Building Length and Width** | 80' | 1.1.3.1b | - |
| **LOAD ASSUMPTIONS** (See Chapter 2 or Chapter 3 tables for load assumptions applicable to the specific tabulated requirement) | | | |
| **Load Type** | **Load Assumption** | | |
| Partition Dead Load | 0-8 psf of floor area | | |
| Wall Assembly Dead Load | 11-18 psf | | |
| Floor Dead Load | 10-20 psf | | |
| Roof/Ceiling Assembly Dead Load | 0-25 psf | | |
| Floor Live Load | 30-40 psf | | |
| Roof Live Load | 20 psf | | |
| Ceiling Live Load | 10-20 psf | | |
| Ground Snow Load | 0-70 psf | | |
| Wind Load | 110-195 mph wind speed (700-yr. return period, 3-second gust) Exposure B, C, and D | | |
| Seismic Load | Seismic Design Category (SDC) SDC A, B, C, $D_0$, $D_1$, and $D_2$ | | |

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

**Figure 1.1 Basic Wind Speeds for One- and Two-Family Dwellings Based on 700-yr Return Period 3-second Gust Basic Wind Speeds**




Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

(from ASCE 7-10 Figure 26.5-1A with permission from ASCE)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## 2.1 General Provisions

### 2.1.1 Engineered Requirements

The provisions of this Chapter provide minimum loads for the purpose of establishing specific resistance requirements for buildings within the scope of this document. This Chapter includes the results of engineering calculations for specific structural elements, in specific configurations, under specific loads. The tabulated information does not represent a complete engineering analysis as would be performed by a registered professional engineer, but is expected to result in significant time-savings for the design professional.

### 2.1.2 Continuous Load Path

A continuous load path shall be provided to transfer all lateral and vertical loads from the roof, wall, and floor systems to the foundation.

### 2.1.3 Engineered Design Limitations

Wood-frame buildings built in accordance with this document shall be limited to the conditions of this section (see Table 2). Structural conditions not complying with this section shall be designed in accordance with accepted engineering practice.

#### 2.1.3.1 Adjustment for Wind Exposure and Mean Roof Height

Tabulated wind requirements in this chapter are based on wind exposure category B and a mean roof height of 33 feet. The building shall neither exceed three stories nor a mean roof height of 33 feet, measured from average grade to average roof elevation (see Figure 1.2). Additional loads from habitable attics shall be considered for purposes of determining gravity and seismic loads. For buildings sited in exposure category C or D, wind-related tabulated values shall be increased in accordance with specific adjustments as provided in table footnotes or per Table 2.1.3.1.

**Table 2.1.3.1 Adjustment for Wind Exposure and Mean Roof Height**

| Mean Roof Height (feet) | Exposure B | Exposure C | Exposure D |
|---|---|---|---|
| 0-15 | 1.00 | 1.18 | 1.43 |
| 20 | 1.00 | 1.25 | 1.50 |
| 25 | 1.00 | 1.31 | 1.56 |
| 30 | 1.00 | 1.36 | 1.61 |
| 33 | 1.00 | 1.39 | 1.64 |

#### 2.1.3.2 Floor Systems

**a. Framing Member Spans** Single spans of floor framing members shall not exceed 26 feet for lumber joists, I-joists, and floor trusses. Design spans shall consider both strength and serviceability limits. For serviceability, the computed joist deflection under live load shall not exceed L/360 (span divided by 360) or more stringent criteria as specified by the manufacturer.

**b. Framing Member Spacings** Floor framing member spacings shall not exceed 24 inches on center for lumber joists, I-joists, and floor trusses.

**c. Cantilevers** Lumber floor joist cantilevers supporting loadbearing walls shall not exceed the depth, d, of the joists (see Figure 2.1a). Lumber floor joist cantilevers supporting non-loadbearing walls shall be limited to L/4 (see Figure 2.1b). I-joist cantilevers shall be in accordance with the manufacturer's code evaluation report. Truss cantilevers shall be in accordance with the truss design/placement plans. Lumber joists, I-joists, and trusses shall be located directly over studs when used in cantilever conditions, unless the top plate is designed to carry the load.

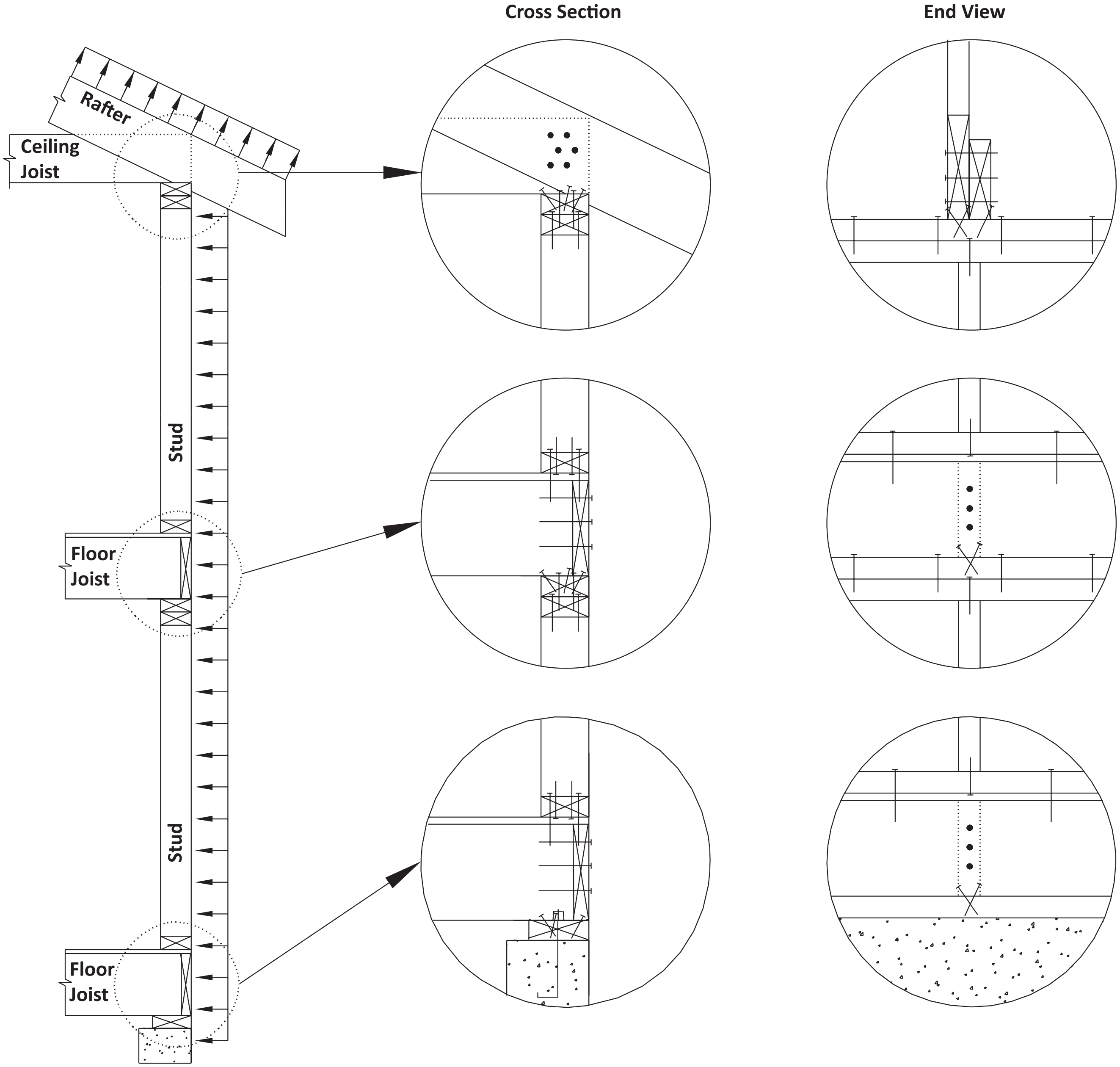
> **EXCEPTION:** Lumber floor joist cantilevers supporting loadbearing walls shall be permitted to exceed these limits when designed for the additional loading requirements, but in no case shall they exceed four times the depth (4d) of the member (see Figure 2.1c).

**d. Setbacks** Setbacks of loadbearing walls on lumber floor joist systems shall not exceed the depth, d, of the joists (see Figure 2.1d). Setbacks on I-joists shall be in accordance with the manufacturer's code evaluation report. Setbacks on floor trusses shall be in accordance with the truss design/placement plans. Lumber joists, I-joists, and trusses shall be located directly over studs when used in setback conditions supporting loadbearing walls, unless the top plate is designed to carry the load.

> **EXCEPTION:** Lumber floor joist setbacks supporting loadbearing walls shall be permit-

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

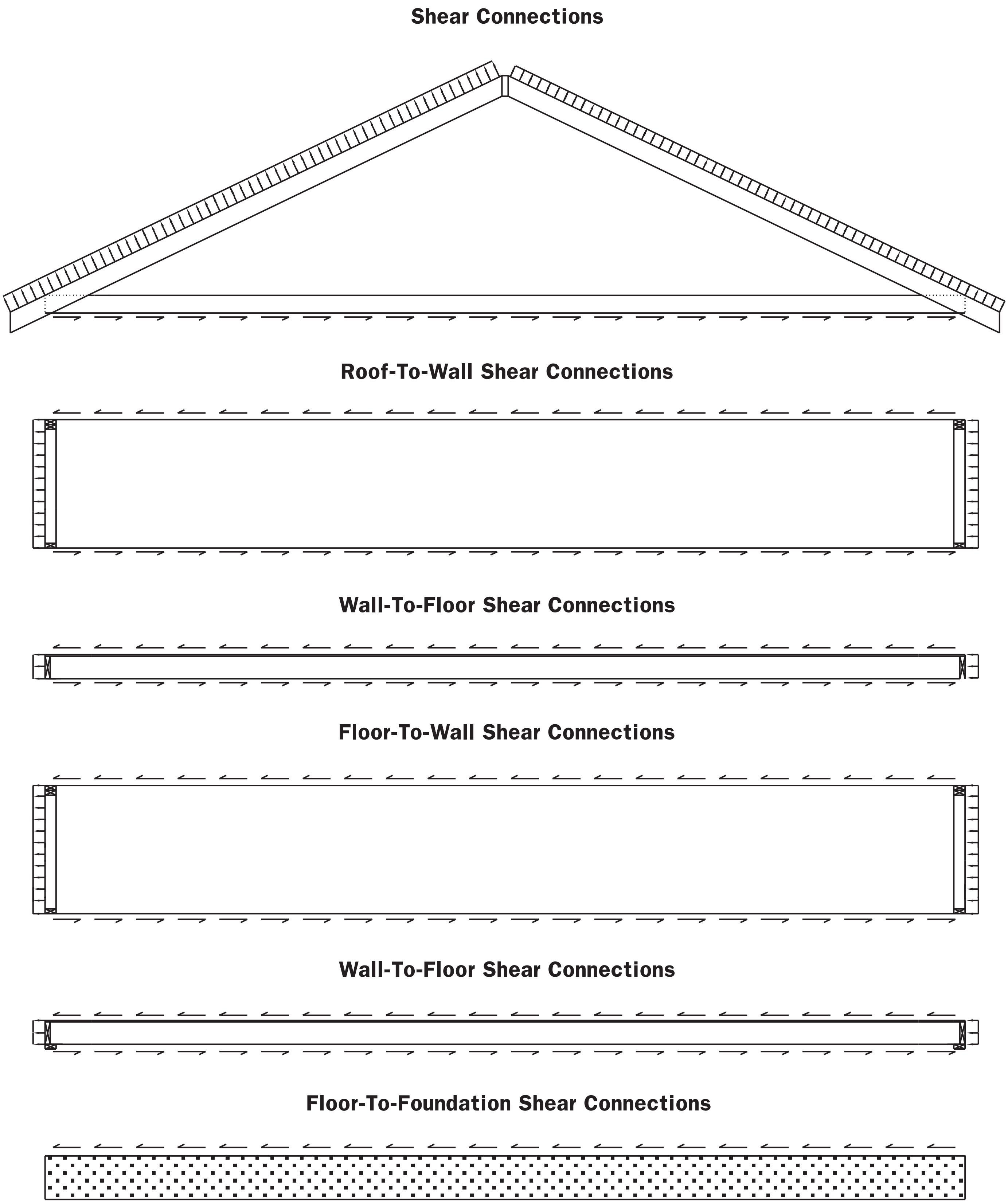
**Figure 2.2a Typical Lateral Framing Connections**





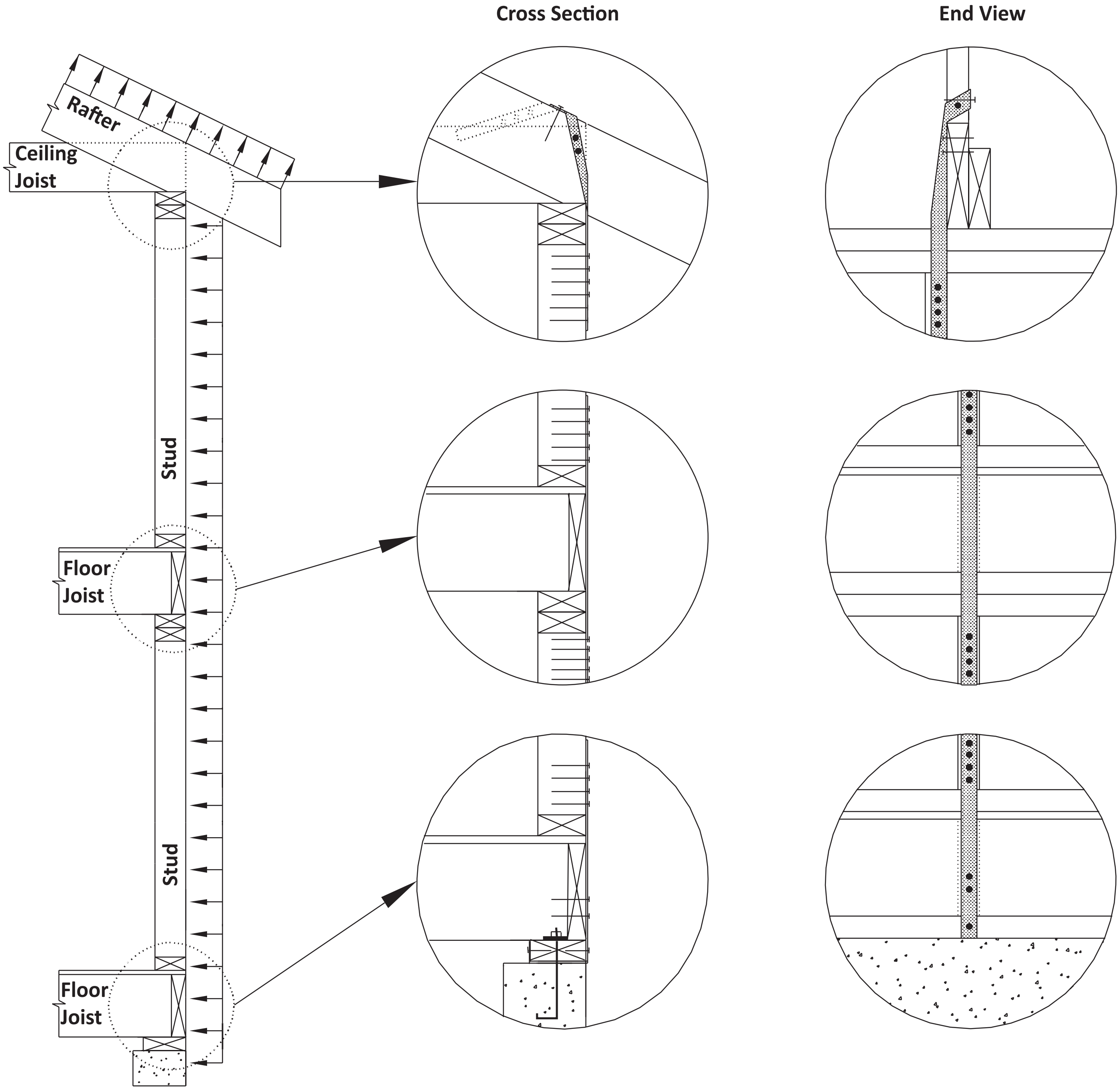

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

**Figure 2.2b Shear Connection Locations**




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

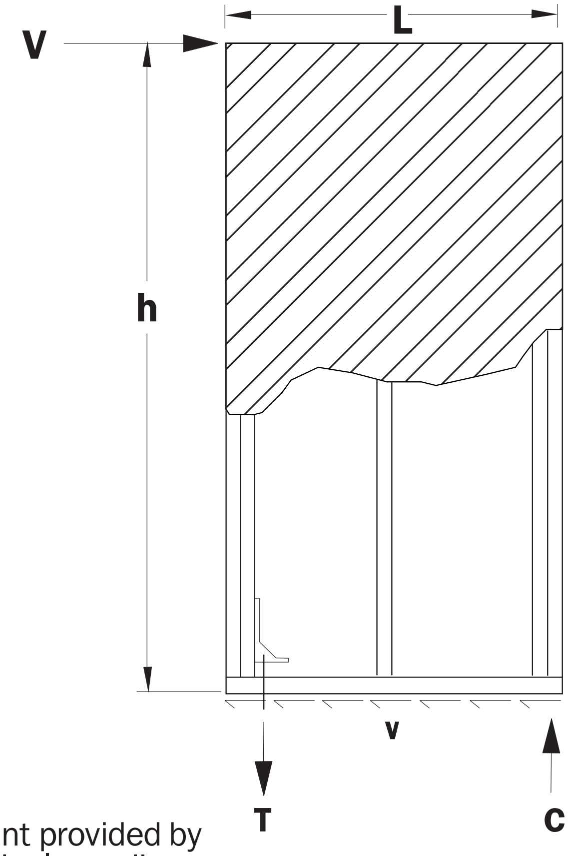
**Figure 2.2c Typical Wind Uplift Connections**






Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

**Figure 2.2d Overturning Detail**




**Tension End**
Overturning restraint provided by dead load, perpendicular walls, or mechanical anchorage in accordance with 2.2.4

**Compression End**
Compression force is resisted by stud end bearing on plate

| | | |
|---|---|---|
| V | = | Shear force (lbs) |
| L | = | Length (ft) |
| h | = | Height (ft) |
| v | = | Required unit shear capacity (plf) |
| C | = | Compression force in end member (lbs) |
| T | = | Tension force in end member<br>Required holddown capacity (lbs) |

$v = V/L$

$T = v \cdot h$

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Polling Question

**Adjustment of tabulated wind values for buildings is based on which of the following?**

a) Exposure category
b) Velocity pressure
c) Mean roof height
d) All of the above
e) a) and c)

# C1.1 Scope

## C1.1.1 General

The scope statement limits applicability of the provisions of the *Wood Frame Construction Manual* to one- and two-family dwellings. This limitation is related primarily to assumed design loads and to structural configurations. Code prescribed floor design loads for dwellings generally fall into the range of 30 to 40 psf, with few additional requirements such as concentrated load provisions. In these applications, use of closely spaced framing members covered by structural sheathing has proven to provide a reliable structural system.

## C1.1.2 Design Loads

Unless stated otherwise, all calculations are based on standard linear elastic analysis and Allowable Stress Design (ASD) load combinations using loads from *ASCE 7-10 Minimum Design Loads for Buildings and Other Structures.*

**Dead Loads**

Unless stated otherwise, tabulated values assume the following dead loads:

| | |
|---|---|
| Roof | 10 psf |
| Ceiling | 5 psf |
| Floor | 10 psf |
| | 12 psf (for Seismic) |
| Walls | 11 psf |
| Partitions | 8 psf (for Seismic) |

**Live Loads**

Unless stated otherwise, tabulated values assume the following live loads:

| | |
|---|---|
| Roof | 20 psf |
| Floor (sleeping areas) | 30 psf |
| Floor (living areas) | 40 psf |

**Wind Loads**

Wind forces are calculated assuming a "box-like" structure with wind loads acting perpendicular to wall and roof surfaces. Lateral loads flow into roof and floor diaphragms and are transferred to the foundation via shear walls. Roof uplift forces are transferred to the foundation by direct tension through the wall framing and tension straps or wall sheathing. Shear wall overturning forces are resisted by the structure's dead load and by supplemental hold down connections.

Implicit in the assumption of a "box-like" structure is a roughly rectangular shape, relatively uniform distribution of shear resistance throughout the structure, and no significant structural discontinuities. In addition, the buildings are assumed to be enclosed structures in which the structural elements are protected from the weather. Partially enclosed structures are subjected to loads that require further consideration.

For wind load calculations, *ASCE 7-10* is used. *ASCE 7-10* calculations are based on 700-year return period "three second gust" wind speeds corresponding to an approximate 7% probability of exceedence in 50 years, and use combined gust and pressure coefficients to translate these wind speeds into peak design pressures on the structure. The 2015 *WFCM* includes design information for buildings located in regions with 700-year return period "three second gust" design wind speeds between 110 and 195 mph.

**Basic Design Equations:**

ASD wind pressures, $p_{max}$, for Main Wind-Force Resisting Systems (MWFRS) and Components and Cladding (C&C) are computed by the following equations, taken from *ASCE 7-10*:

MWFRS – Envelope Procedure:

$$p_{max} = q[(GC_{pf}) - (GC_{pi})] \text{ (lbs/ft}^2)$$

**where:**

$q = 0.60\, q_h$

$q_h = 0.00256 K_z K_{zt} K_d V^2$ (*ASCE 7-10* Equation 28.3-1)

$GC_{pf}$ = external pressure coefficients (*ASCE 7-10* Figure 28.4-1)

$GC_{pi}$ = internal pressure coefficients (*ASCE 7-10* Table 26.11-1)

**C&C:**

$$p_{max} = q[(GC_p) - (GC_{pi})] \text{ (lbs/ft}^2)$$

**where:**

$q = 0.60 q_h$

$q_h = 0.00256 K_z K_{zt} K_d V^2$ (*ASCE 7-10* Equation 30.3-1)

$GC_p$ = external pressure coefficients (*ASCE7-10* Figures 30.4-1,30.4-2A, B, &C)

$GC_{pi}$ = internal pressure coefficients (*ASCE 7-10* Table 26.11-1)

The calculation of ASD velocity pressure, q, for various wind speeds and Exposures is shown in Table C1.1.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table C1.1 ASD Velocity Pressure, q (psf), for Exposures B, C, and D and 33' MRH

| Exposure Category | ASD Velocity Pressure, q (psf) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 700-yr. Wind Speed 3-second gust (mph) | | | | | | | | | |
| | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
| Exposure B | 11.37 | 12.43 | 13.54 | 15.89 | 18.42 | 21.15 | 24.06 | 27.17 | 30.46 | 35.74 |
| Exposure C | 15.80 | 17.27 | 18.80 | 22.06 | 25.59 | 29.38 | 33.42 | 37.73 | 42.30 | 49.65 |
| Exposure D | 18.64 | 20.37 | 22.18 | 26.04 | 30.20 | 34.66 | 39.44 | 44.52 | 49.92 | 58.58 |

$$q = 0.6\, q_h$$

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

**and**

$K_z\,(33\text{ ft}) = 0.72$ *ASCE 7-10* Table 28.3-1 (MWFRS), Table 30.3-1(C&C) at mean roof height (MRH) of 33 ft

$K_{zt} = 1.0$ No topographic effects

$K_d = 0.85$ *ASCE 7-10* Table 26.6-1

Design wind pressures in *ASCE 7-10* are based on an ultimate 700-year return period. Since the *WFCM* uses allowable stress design, forces calculated from design wind pressures are multiplied by 0.60 in accordance with load combination factors per *ASCE 7-10*.

For example, the ASD velocity pressure, q, at 150 mph for Exposure B is calculated as follows:

$$q = 0.6\,(0.00256)(0.72)(1.0)(0.85)(150)^2\ (\text{lbs/ft}^2)$$
$$= 21.15\ (\text{lbs/ft}^2)$$

In order to use the *2015 WFCM* with basic wind speeds from the *2015 International Residential Code (IRC)*, see the wind speed conversion Table C1.2 based on the following calculations:

Equating wind pressures calculated using *ASCE 7-10* wind speeds with those from the *2015 IRC*.

Velocity pressure for the *ASCE 7-05* basic wind speed of 90 mph (Exposure B) is calculated as follows:

$$q = 0.00256(0.72)(0.85)90^2 = 12.7\text{ psf}$$

ASD velocity pressure using the *ASCE 7-10* wind speed of 116 mph (Exposure B) is calculated as follows:

$$q = (0.60)[0.00256(0.72)(0.85)116^2] = 12.7\text{ psf}$$

On the basis of equating wind pressures, the 90 mph *ASCE 7-05* basic wind speed is "equivalent" to the 116 mph *ASCE 7-10* basic wind speed.

## Table C1.2 Wind Speed Conversion Table

| ASCE 7-05 Basic Wind Speeds (mph) | | | | | | | |
|---|---|---|---|---|---|---|---|
| 85 | 90 | 100 | 110 | 120 | 130 | 140 | 150 |
| **Equivalent ASCE 7-10 Basic Wind Speeds (mph)** | | | | | | | |
| 110 | 116 | 129 | 142 | 155 | 168 | 181 | 194 |

Wind speed contour maps in the *2015 IRC* show the 90 mph contour as covering approximately the same geographical area as that for the 115 mph wind speed contour in *ASCE 7-10*. The velocity pressure for the 115 mph (Exposure B) *ASCE 7-10* wind speed (12.4 psf) however, is slightly less than the velocity pressure corresponding to the 90 mph *2015 IRC* (Exposure B) wind speed (12.7 psf).

Note that the worst case of internal pressurization is used in design. Internal pressure and internal suction for MWFRS are outlined in *WFCM* Tables C1.3A and C1.3B, respectively. Pressure coefficients and loads for wind parallel and perpendicular to ridge are tabulated. Parallel to ridge coefficients are used to calculate wind loads acting perpendicular to end walls. Perpendicular-to-ridge coefficients are used to calculate wind loads acting perpendicular to side walls.

Pressures resulting in shear, uplift, and overturning forces are applied to the building as follows:

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

### Shear Calculations

The horizontal component of roof pressures is applied as a lateral load at the highest ceiling level (top of the uppermost wall).

Windward and leeward wall pressures are summed and applied (on a tributary area basis) as lateral loads at each horizontal diaphragm. For example, in typical two story construction, one-half of the height of the top wall goes to the roof or ceiling level, a full story height goes to intermediate floor diaphragms (one-half from above and one-half from below) and one-half of the bottom story goes directly into the foundation.

Lateral roof and wall pressures for determining diaphragm and shear wall loads are calculated using enveloped MWFRS coefficients. Spatially-averaged C&C coefficients are used for determining lateral framing loads, suction pressures on wall and roof sheathing, and exterior stud capacities.

### Uplift Calculations

Uplift for roof cladding is calculated using C&C loads. Uplift connections for roof framing members are calculated using enveloped MWFRS loads. The rationale for using MWFRS loads for computing the uplift of roof assemblies recognizes that the spatial and temporal pressure fluctuations that cause the higher coefficients for components and cladding are effectively averaged by wind effects on different roof surfaces. The uplift load minus sixty percent of the roof and/or ceiling dead load is applied at the top of the uppermost wall. As this load is carried down the wall, the wall dead load is included in the analysis. The dead load from floors framing into walls is not included, in order to eliminate the need for special framing details where floors do not directly frame into walls.

### Overturning Calculations

Overturning of the structure as a result of lateral loads is resisted at the ends of shear walls in accordance with general engineering practice, typically with hold downs or other framing anchorage systems. In the *WFCM*, overturning loads are differentiated from uplift loads. Overturning moments result from lateral loads which are resisted by shear walls. Uplift forces arise solely from uplift on the roof, and are transferred directly into the walls supporting the roof framing.

*ASCE 7-10* requires checking the MWFRS with a minimum 5 psf ASD lateral load on the vertical projected area of the roof and a 10 psf ASD lateral load on the wall. The *2015 WFCM* incorporates this design check.

### Snow Loads

The *2015 WFCM* includes design information for snow loads in accordance with *ASCE 7-10* for buildings located in regions with ground snow loads between 0 and 70 psf. Both balanced and unbalanced snow load conditions are considered in design.

### Seismic Loads

The *2015 WFCM* includes seismic design information in accordance with *ASCE 7-10* for buildings located in Seismic Design Categories A-D, as defined by the *2015 IRC*.

### C1.1.2.1 Torsion

Design for torsion is outside the scope of this document.

### C1.1.2.2 Sliding Snow

Design for sliding snow is outside the scope of this document.

## C1.1.3 Applicability

### C1.1.3.1 Building Dimensions

**a. Mean Roof Height** Building height restrictions limit the wind forces on the structure, and also provide assurance that the structure remains “low-rise” in the context of wind and seismic-related code requirements.

The tables in the *WFCM* are based on wind calculations assuming a 33 ft mean roof height, (MRH). This assumption permits table coverage up to a typical 3-story building. Footnotes have been provided to adjust tabulated requirements to lesser mean roof heights.

**b. Building Length and Width** Limiting the maximum building length and width to 80 feet is provided as a reasonable upper limit for purposes of tabulating requirements in the *WFCM*.

### C1.1.3.2 Floor, Wall, and Roof Systems

See C2.1.3.2 (Floor Systems), C2.1.3.3 (Wall Systems), and C2.1.3.4 (Roof Systems).

## C1.1.4 Foundation Provisions

Design of foundations and foundation systems is outside the scope of this document.

## C1.1.5 Protection of Openings

Wind pressure calculations in the *WFCM* assume that buildings are fully enclosed and that the building envelope is not breached. Interior pressure coefficients, GCpi, of +/-0.18 are used in the calculations per *ASCE 7-10* Table 26.11-1. Penetration of openings (e.g. windows and doors) due to flying debris can occur in sites subject to high winds with a significant debris field. Where these areas occur,

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

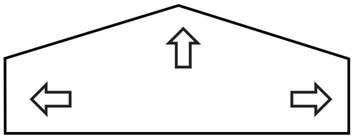
opening protection or special glazing requirements may be required by the local authority to ensure that the building envelope is maintained.

### C1.1.6 Ancillary Structures

Design of ancillary structures is outside the scope of this document.

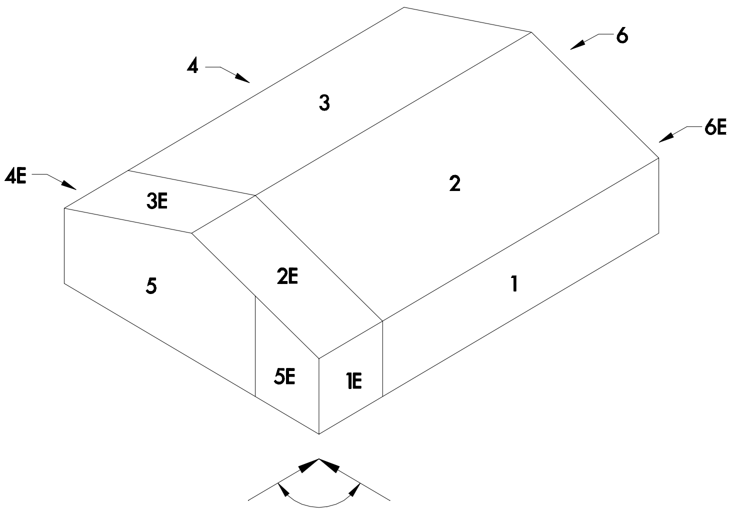
**Table C1.3A ASCE 7-10 Exposure B Main Wind-Force Resisting System (MWFRS) Loads, p (psf), for an Enclosed Building, 33' Mean Roof Height with Internal Pressure, 150 mph (700-yr. 3-second gust), Exposure B**

q = 21.15 psf (See Table C1.1)



with internal pressure

| | | Interior Zones | | | | | | Exterior Zones | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof Angle | | 1 | 2 | 3 | 4 | 5 | 6 | 1E | 2E | 3E | 4E | 5E | 6E |
| 0 - 5 | $GC_{pf}$ | 0.40 | -0.69 | -0.37 | -0.29 | 0.40 | -0.29 | 0.61 | -1.07 | -0.53 | -0.43 | 0.61 | -0.43 |
| | $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 |
| | p (psf) | 4.65 | -18.40 | -11.63 | -9.94 | 4.65 | -9.94 | 9.09 | -26.44 | -15.02 | -12.90 | 9.09 | -12.90 |
| 20 | $GC_{pf}$ | 0.53 | -0.69 | -0.48 | -0.43 | 0.40 | -0.29 | 0.80 | -1.07 | -0.69 | -0.64 | 0.61 | -0.43 |
| | $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 |
| | p (psf) | 7.40 | -18.40 | -13.96 | -12.90 | 4.65 | -9.94 | 13.11 | -26.44 | -18.40 | -17.34 | 9.09 | -12.90 |
| 26.6 (6:12) | $GC_{pf}$ | 0.55 | -0.10 | -0.45 | -0.39 | 0.40 | -0.29 | 0.73 | -0.19 | -0.58 | -0.53 | 0.61 | -0.43 |
| | $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 |
| | p (psf) | 7.82 | -5.84 | -13.26 | -12.06 | 4.65 | -9.94 | 11.58 | -7.73 | -16.17 | -15.11 | 9.09 | -12.90 |
| 30-45 | $GC_{pf}$ | 0.56 | 0.21 | -0.43 | -0.37 | 0.40 | -0.29 | 0.69 | 0.27 | -0.53 | -0.48 | 0.61 | -0.43 |
| | $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 |
| | p (psf) | 8.04 | 0.63 | -12.90 | -11.63 | 4.65 | -9.94 | 10.79 | 1.90 | -15.02 | -13.96 | 9.09 | -12.90 |
| 90 | $GC_{pf}$ | 0.56 | 0.56 | -0.37 | -0.37 | 0.40 | -0.29 | 0.69 | 0.69 | -0.48 | -0.48 | 0.61 | -0.43 |
| | $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 | 0.18 |
| | p (psf) | 8.04 | 8.04 | -11.63 | -11.63 | 4.65 | -9.94 | 10.79 | 10.79 | -13.96 | -13.96 | 9.09 | -12.90 |

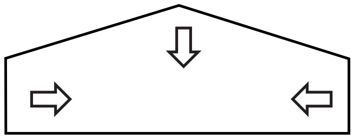



WIND DIRECTION RANGE

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

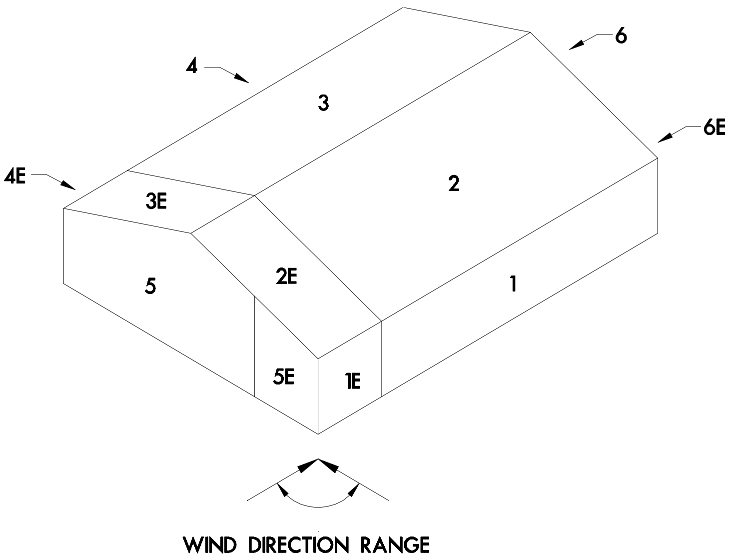
## Table C1.3B ASCE 7-10 Exposure B Main Wind-Force Resisting System (MWFRS) Loads, p (psf), for an Enclosed Building, 33' Mean Roof Height with Internal Suction, 150 mph (700-yr. 3-second gust), Exposure B

q = 21.15 psf (See Table C1.1)



with internal suction

| Roof Angle | | Interior Zones | | | | | | Exterior Zones | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 1E | 2E | 3E | 4E | 5E | 6E |
| 0 - 5 | $GC_{pf}$ | 0.40 | -0.69 | -0.37 | -0.29 | 0.4 | -0.29 | 0.61 | -1.07 | -0.53 | -0.43 | 0.61 | -0.43 |
| | $GC_{pi}$ | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| | p (psf) | 12.27 | -10.79 | -4.02 | -2.33 | 12.27 | -2.33 | 16.71 | -18.82 | -7.40 | -5.29 | 16.71 | -5.29 |
| 20 | $GC_{pf}$ | 0.53 | -0.69 | -0.48 | -0.43 | 0.4 | -0.29 | 0.8 | -1.07 | -0.69 | -0.64 | 0.61 | -0.43 |
| | $GC_{pi}$ | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| | p (psf) | 15.02 | -10.79 | -6.35 | -5.29 | 12.27 | -2.33 | 20.73 | -18.82 | -10.79 | -9.73 | 16.71 | -5.29 |
| 26.6 (6:12) | $GC_{pf}$ | 0.55 | -0.10 | -0.45 | -0.39 | 0.40 | -0.29 | 0.73 | -0.19 | -0.58 | -0.53 | 0.61 | -0.43 |
| | $GC_{pi}$ | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| | p (psf) | 15.44 | 1.78 | -5.65 | -4.45 | 12.27 | -2.33 | 19.19 | -0.12 | -8.55 | -7.50 | 16.71 | -5.29 |
| 30-45 | $GC_{pf}$ | 0.56 | 0.21 | -0.43 | -0.37 | 0.4 | -0.29 | 0.69 | 0.27 | -0.53 | -0.48 | 0.61 | -0.43 |
| | $GC_{pi}$ | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| | p (psf) | 15.65 | 8.25 | -5.29 | -4.02 | 12.27 | -2.33 | 18.40 | 9.52 | -7.40 | -6.35 | 16.71 | -5.29 |
| 90 | $GC_{pf}$ | 0.56 | 0.56 | -0.37 | -0.37 | 0.4 | -0.29 | 0.69 | 0.69 | -0.48 | -0.48 | 0.61 | -0.43 |
| | $GC_{pi}$ | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| | p (psf) | 15.65 | 15.65 | -4.02 | -4.02 | 12.27 | -2.33 | 18.40 | 18.40 | -6.35 | -6.35 | 16.71 | -5.29 |




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Polling Question

**Main wind force resisting system (MWFRS) loads are used for which of the following?**

a) Diaphragms

b) Shear walls

c) Roof framing uplift connectors

d) All of the above

e) a) and b) only

# List of Tables



2

ENGINEERED DESIGN

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.1 Lateral Framing Connection Loads from Wind

(For Roof-to-Plate, Plate-to-Plate, Plate-to-Stud, and Plate-to-Floor)

| 700-yr. Wind Speed 3-second gust (mph) | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|
| **Wall Height (ft)** | **Unit Framing Loads (plf)[1,2,3,4]** | | | | | | | | | |
| **8** | 67 | 73 | 79 | 93 | 108 | 124 | 141 | 159 | 178 | 209 |
| **10** | 79 | 87 | 94 | 111 | 129 | 148 | 168 | 190 | 212 | 249 |
| **12** | 91 | 100 | 109 | 128 | 148 | 170 | 193 | 218 | 245 | 287 |
| **14** | 103 | 112 | 122 | 144 | 167 | 191 | 218 | 246 | 275 | 323 |
| **16** | 114 | 124 | 135 | 159 | 184 | 212 | 241 | 272 | 305 | 358 |
| **18** | 124 | 136 | 148 | 174 | 201 | 231 | 263 | 297 | 333 | 391 |
| **20** | 135 | 147 | 160 | 188 | 218 | 250 | 285 | 321 | 360 | 423 |

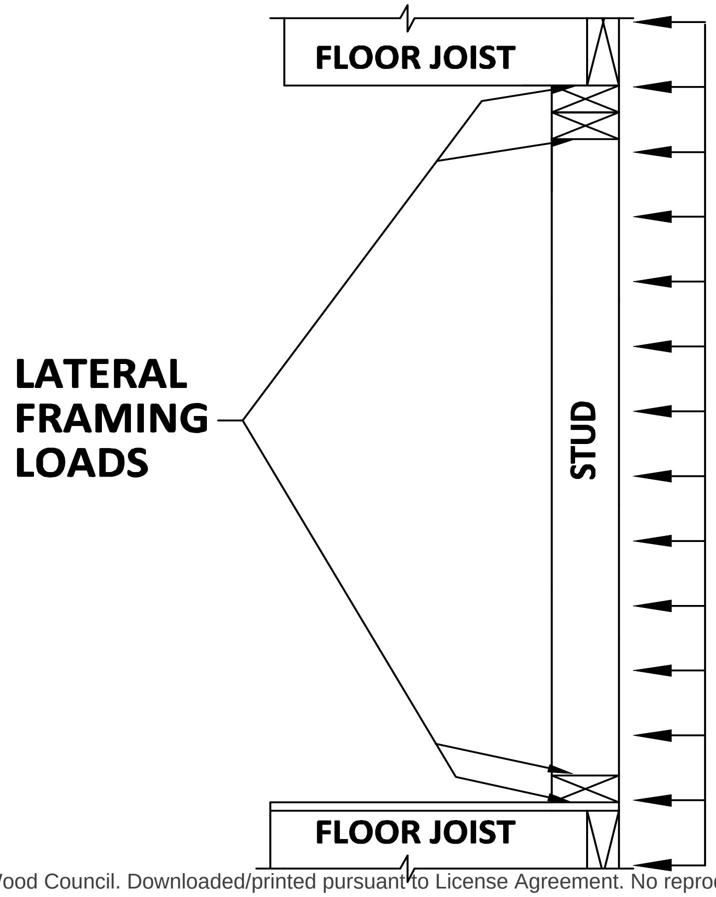
1 Tabulated framing loads shall be permitted to be multiplied by 0.92 for framing not located within 3 feet of corners for buildings less than 30 feet in width (W), or within W/10 of corners for buildings greater than 30 feet in width.

2 Tabulated framing loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

3 Tabulated framing loads are specified in pounds per linear foot of wall. To determine connection requirements, multiply the tabulated unit lateral framing load by the multiplier from the table below corresponding to the spacing of the connection:

| Connection Spacing (in.) | 12 | 16 | 19.2 | 24 | 48 |
|---|---|---|---|---|---|
| **Multiplier** | 1.00 | 1.33 | 1.60 | 2.00 | 4.00 |

4 When calculating lateral loads for ends of headers, girders, and window sills, multiply the tabulated unit lateral load by ½ of the header, girder, or sill span (ft).




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.
AMERICAN WOOD COUNCIL

## Table 2.1 Lateral Framing Connection Loads from Wind

***Description:*** Lateral framing connection loads at base and top of wall expressed in pounds per linear foot of wall length.

***Procedure:*** Compute the lateral framing connection load at the top and bottom of studs based on tributary wind loads, using external (end zone) components and cladding pressure coefficients and internal pressure coefficients for enclosed buildings.

***Background:*** Components and cladding (C&C) coefficients result in higher wind loads relative to main wind force resisting system (MWFRS) coefficients. When determining C&C pressure coefficients ($GC_p$), the effective wind area equals the tributary area of the framing member. 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. This results in lower average wind pressures. The increase in width applies only to calculation of wind force coefficients.

### Example:

Given -150 mph, Exposure B, 33' MRH, 10' wall height, 16" o.c. connection spacing.

$$p_{max} = qGC_p - qGC_{pi}$$

**where:**

$p_{max}$ = pressure on the wall

$q$ = 21.15 psf (See Table C1.1)

$GC_p$ = external pressure coefficients for C&C

$GC_{pi}$ = +/- 0.18 internal pressure coefficient for enclosed buildings

Stud tributary area equals 13.3 ft². The minimum required area for analysis is $h^2/3=33.3$ ft². The $GC_p$ equation is determined using *ASCE 7-10* Figure 30.4-1.

End Zones (See Zone 5 as shown in WFCM Table 2.4):

$GC_p$ = -1.4 for $A \leq 10$ ft²

$GC_p$ = -0.8 - 0.6[(log(A/500)) / (log (10/500))] for $10 < A \leq 500$ ft²

$GC_p$ = -0.8 for $A > 500$ ft²

**therefore:**

$GC_p$ = -0.8 - 0.6[(log(33.3/500)) / (log (10/500))]

$GC_p$ = -1.22

The internal pressure coefficient ($GC_{pi}$) is taken from *ASCE 7-10* Table 26.11-1.

$GC_{pi}$ = +/- 0.18

**therefore:**

$p_{max}$ = 21.15 (-1.22 - 0.18)

= -29.61 psf (Negative pressure denotes suction)

The pressure is multiplied by half the stud height to obtain the unit lateral framing connection load:

= -29.61(10/2)

= |-148 plf| (WFCM Table 2.1)

Required capacity of lateral framing connections spaced at 16" o.c. is:

= 148plf (16 in./12 in./ft)

= 197 lbs = 148 (1.33) (WFCM Table 2.1 Footnote 3)

### Footnote 1:

Lateral framing connection loads are based on End Zone Coefficients (Zone 5) per the figure of Table 2.4. Where Interior Zones (Zone 4) occur, connection loads may be reduced. Adjustment of tabulated loads are conservatively based on a 20' wall height where A = 133 ft².

End Zone

$GC_p$ = -0.8 - 0.6[(log(A/500)) / (log (10/500))]

= -0.8 - 0.6[(log(133/500)) / (log(10/500))]

= -1.00

Interior Zone

$GC_p$ = -0.8 - 0.3[(log(A/500)) / (log(10/500))]

= -0.8 - 0.3[(log(133/500)) / (log(10/500))]

= -0.9

The ratio of Zone 4 to Zone 5 loads is:

(-0.9-0.18) / (-1.0-0.18) = 0.92 (WFCM Table 2.1 Footnote 1)

Therefore, Interior Zone loads may be reduced to 92% of tabulated values.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2A Uplift Connection Loads from Wind

(For Roof-to-Wall, Wall-to-Wall, and Wall-to-Foundation)

| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof/Ceiling Assembly Design Dead Load | Roof Span (ft) | Unit Connection Loads (plf)[1,2,3,4,5,6,7] | | | | | | | | | |
| 0 psf [8] | 12 | 118 | 128 | 140 | 164 | 190 | 219 | 249 | 281 | 315 | 369 |
| | 24 | 195 | 213 | 232 | 272 | 315 | 362 | 412 | 465 | 521 | 612 |
| | 36 | 272 | 298 | 324 | 380 | 441 | 506 | 576 | 650 | 729 | 856 |
| | 48 | 350 | 383 | 417 | 489 | 567 | 651 | 741 | 836 | 938 | 1100 |
| | 60 | 428 | 468 | 509 | 598 | 693 | 796 | 906 | 1022 | 1146 | 1345 |
| 10 psf | 12 | 70 | 80 | 92 | 116 | 142 | 171 | 201 | 233 | 267 | 321 |
| | 24 | 111 | 129 | 148 | 188 | 231 | 278 | 328 | 381 | 437 | 528 |
| | 36 | 152 | 178 | 204 | 260 | 321 | 386 | 456 | 530 | 609 | 736 |
| | 48 | 194 | 227 | 261 | 333 | 411 | 495 | 585 | 680 | 782 | 944 |
| | 60 | 236 | 276 | 317 | 406 | 501 | 604 | 714 | 830 | 954 | 1153 |
| 15 psf | 12 | 46 | 56 | 68 | 92 | 118 | 147 | 177 | 209 | 243 | 297 |
| | 24 | 69 | 87 | 106 | 146 | 189 | 236 | 286 | 339 | 395 | 486 |
| | 36 | 92 | 118 | 144 | 200 | 261 | 326 | 396 | 470 | 549 | 676 |
| | 48 | 116 | 149 | 183 | 255 | 333 | 417 | 507 | 602 | 704 | 866 |
| | 60 | 140 | 180 | 221 | 310 | 405 | 508 | 618 | 734 | 858 | 1057 |
| 20 psf | 12 | 22 | 32 | 44 | 68 | 94 | 123 | 153 | 185 | 219 | 273 |
| | 24 | 27 | 45 | 64 | 104 | 147 | 194 | 244 | 297 | 353 | 444 |
| | 36 | 32 | 58 | 84 | 140 | 201 | 266 | 336 | 410 | 489 | 616 |
| | 48 | 38 | 71 | 105 | 177 | 255 | 339 | 429 | 524 | 626 | 788 |
| | 60 | 44 | 84 | 125 | 214 | 309 | 412 | 522 | 638 | 762 | 961 |
| 25 psf | 12 | - | 8 | 20 | 44 | 70 | 99 | 129 | 161 | 195 | 249 |
| | 24 | - | 3 | 22 | 62 | 105 | 152 | 202 | 255 | 311 | 402 |
| | 36 | - | - | 24 | 80 | 141 | 206 | 276 | 350 | 429 | 556 |
| | 48 | - | - | 27 | 99 | 177 | 261 | 351 | 446 | 548 | 710 |
| | 60 | - | - | 29 | 118 | 213 | 316 | 426 | 542 | 666 | 865 |

1 Tabulated unit uplift connection loads shall be permitted to be multiplied by 0.75 for framing not located within 6 feet of corners for buildings less than 30 feet in width (W), or W/5 for buildings greater than 30 feet in width.

2 Tabulated uplift loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values for 0 psf roof dead load shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1 then reduced by the appropriate design dead load.

3 Tabulated uplift loads are specified in pounds per linear foot of wall. To determine connection requirements, multiply the tabulated unit uplift load by the multiplier from the table below corresponding to the spacing of the connectors:

| Connection Spacing (in.) | 12 | 16 | 19.2 | 24 | 48 |
|---|---|---|---|---|---|
| Multiplier | 1.00 | 1.33 | 1.60 | 2.00 | 4.00 |

4 Tabulated uplift loads equal total uplift minus 0.6 of the roof/ceiling assembly design dead load.

5 Tabulated uplift loads are specified for roof-to-wall connections. When calculating uplift loads for wall-to-wall or wall-to-foundation connections, tabulated uplift values shall be permitted to be reduced by 73 plf (0.60 x 121 plf) for each full wall above.

6 When calculating uplift loads for ends of headers/girders, multiply the tabulated unit uplift load by 1/2 of the header/girder span (ft.). Cripple studs need only be attached per typical uplift requirements.

7 For jack rafter uplift connections, use a roof span equal to twice the jack rafter length. The jack rafter length includes the overhang length and the jack span.

8 Tabulated uplift loads for 0 psf design dead load are included for interpolation or use with actual roof dead loads.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2A Uplift Connection Loads from Wind

***Description:*** Uplift loads at the roof to wall connection.

***Procedure:*** Use Main Wind Force Resisting System (MWFRS) coefficients to calculate wind uplift forces. Sum moments to compute maximum uplift force at the roof to wall connection.

***Background:*** Per *ASCE 7-10*, worst case uplift loads occur at a 20 degree roof slope with wind perpendicular to the ridge and roof overhang uplift forces included. Roof/ceiling gravity loads are included to resist uplift forces.

**Example:**

Given - 150 mph, Exposure B, 33' MRH, 20 degree roof slope, 36' roof span, 2' overhangs, 15 psf roof/ceiling dead load, 16" o.c. connection spacing.

External pressure coefficients are taken from *ASCE 7-10* Figure 28.4-1. Internal pressure coefficients are taken from *ASCE 7-10* Table 26.11-1 and applied to the Windward Roof (WR) and Leeward Roof (LR). The pressure coefficient for the bottom surface of the Windward Overhang (WO) is computed using a gust factor (0.85) and a pressure coefficient (0.7) from sections *ASCE 7-10* sections 26.9.1 and 28.4.3, respectively. The positive internal pressure coefficient was applied to the bottom surface of the Leeward Overhang (LO) to model background pressure.

$p_{roof}$ = pressure on the roof portion

$p_{roof}$ = $q(GC_{pf} - GC_{pi})$, for each portion of the roof

where:

$q = 21.15$ psf (See Table C1.1)

$GC_{pf}$ = external pressure coefficients for MWFRS

$GC_{pi}$ = +/- 0.18 internal pressure coefficient for enclosed buildings

End Zone:

| | | | | |
|---|---|---|---|---|
| WO | $GC_{pf}$ = -1.07, | $GC_p$ = (0.85) (-0.7) = | -0.60 |
| WR | $GC_{pf}$ = -1.07, | $GC_{pi}$ = -0.18 | |
| LO | $GC_{pf}$ = -0.69, | $GC_{pi}$ = -0.18 | |
| LR | $GC_{pf}$ = -0.69, | $GC_{pi}$ = -0.18 | |

Substituting:

$p_{WO}$ = 21.15 (-1.07 - 0.60) = -35.2 psf

$p_{WR}$ = 21.15 (-1.07 - 0.18) = -26.4 psf

$p_{LR}$ = 21.15 (-0.69 - 0.18) = -18.4 psf

$p_{LO}$ = 21.15 (-0.69 - 0.18) = -18.4 psf

Since dead loads in this case are resisting uplift forces, they are multiplied by 0.6, per *ASCE 7-10* section 2.4.1. Thus roof/ceiling dead loads are equal to:

(15 psf) 0.6 = 9 psf

Summing moments about the leeward roof-to-wall intersection, the uplift forces are calculated for a 1' wide section through the building. To parallel the notation above, the forces retain the WR, WO, etc., notation and are preceded by a V (for vertical) or H (for horizontal):

| | | | |
|---|---|---|---|
| VWO | = -35.2(2) | = -70.4 lbs |
| VWR | = -26.4(36 / 2) | = -475.2 lbs |
| VLR | = -18.4(36 / 2) | = -331.2 lbs |
| VLO | = -18.4(2) | = -36.8 lbs |
| HWO | = -35.2(2)(tan(20°)) | = -25.7 lbs |
| HWR | = -26.4(36 / 2)(tan(20°)) | = -173.0 lbs |
| HLR | = -18.4(36 / 2)(tan(20°)) | = -120.5 lbs |
| HLO | = -18.4(2)(tan(20°)) | = -13.4 lbs |

The dead load of the roof, R, is also added:

| | | |
|---|---|---|
| RWO | = 9(2) | = 18 lbs |
| RWR | = 9(18) | = 162 lbs |
| RLO | = 9(2) | = 18 lbs |
| RLR | = 9(18) | = 162 lbs |

Summing moments about the leeward top of wall:

$$\Sigma M_L = 0 = [-70.4+18][1+36] + [-475.2+162][9+18] + [-331.2+162][9] - [-36.8+18][1] + [(-25.7)(0.364)] - [(-173.0)(3.276)] + [(-120.5)(3.276)] - [(-13.4)(0.364)] - 36F_W$$

Solving for $F_W$:

$F_W$ = -326 plf

Unit uplift connection load is:

= 326 plf (WFCM Table 2.2A)

Required uplift capacity of connections spaced at 16" o.c.

= 326 plf (16 in./12in./ft)

= 434 lbs = 326(1.33) (WFCM Table 2.2A Footnote 3)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

**Footnote 1:**
Tabulated loads are based on end zone pressures. Where the requirements of footnote 1 are met, tabulated loads may be decreased as follows:

Given - 110 mph, Exposure B, 33' MRH, 20 degree roof slope, 12' roof span, 2' overhangs, 0 psf roof/ceiling dead load, 12" o.c.

End Zone uplift force is:

118 plf

Interior Zone uplift force based on calculation similar to exterior zone is:

88 plf

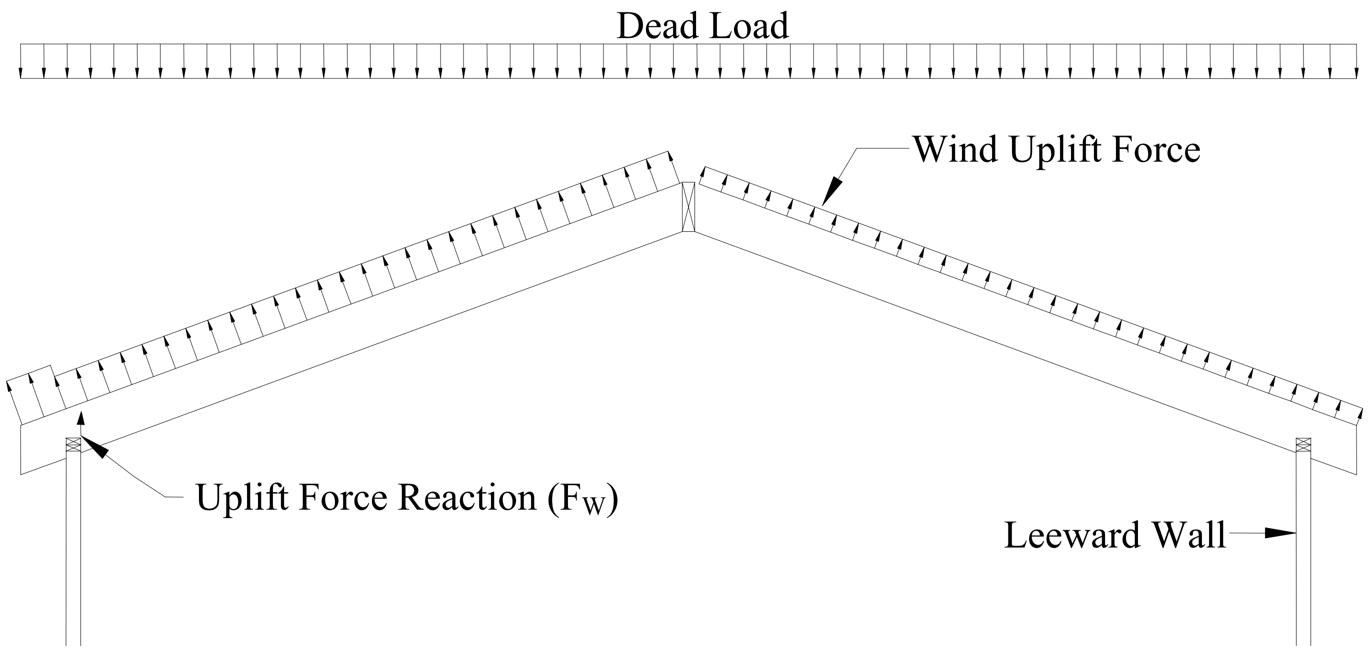
Reduction factor:

= (88 plf) / (118 plf)

= 0.75








Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2B Ridge Connection Loads from Wind

(Dead Load Assumptions: Roof Assembly DL = 10 psf)

| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Roof Pitch** | **Roof Span (ft)** | **Unit Connection Loads (plf)[1,2,3,4,5]** | | | | | | | | | |
| 3:12 | 12 | 77 | 91 | 105 | 136 | 170 | 205 | 243 | 284 | 327 | 397 |
| | 24 | 154 | 182 | 211 | 273 | 339 | 411 | 487 | 568 | 655 | 793 |
| | 36 | 231 | 273 | 316 | 409 | 509 | 616 | 730 | 852 | 982 | 1190 |
| | 48 | 308 | 364 | 422 | 545 | 678 | 821 | 974 | 1137 | 1309 | 1586 |
| | 60 | 386 | 455 | 527 | 681 | 848 | 1026 | 1217 | 1421 | 1636 | 1983 |
| 4:12 | 12 | 66 | 77 | 88 | 113 | 140 | 168 | 199 | 232 | 266 | 322 |
| | 24 | 131 | 153 | 177 | 226 | 280 | 337 | 398 | 463 | 533 | 644 |
| | 36 | 197 | 230 | 265 | 339 | 419 | 505 | 597 | 695 | 799 | 966 |
| | 48 | 262 | 307 | 353 | 452 | 559 | 674 | 796 | 927 | 1065 | 1288 |
| | 60 | 328 | 384 | 442 | 565 | 699 | 842 | 995 | 1159 | 1332 | 1610 |
| 5:12 | 12 | 51 | 60 | 69 | 88 | 109 | 132 | 156 | 182 | 209 | 253 |
| | 24 | 102 | 119 | 138 | 177 | 219 | 264 | 312 | 364 | 418 | 506 |
| | 36 | 153 | 179 | 207 | 265 | 328 | 396 | 468 | 545 | 627 | 758 |
| | 48 | 204 | 239 | 276 | 353 | 437 | 528 | 624 | 727 | 836 | 1011 |
| | 60 | 255 | 299 | 344 | 442 | 547 | 660 | 780 | 909 | 1045 | 1264 |
| 6:12 | 12 | 48 | 55 | 63 | 81 | 99 | 119 | 141 | 164 | 188 | 227 |
| | 24 | 95 | 111 | 127 | 162 | 199 | 239 | 282 | 327 | 376 | 453 |
| | 36 | 143 | 166 | 190 | 242 | 298 | 358 | 423 | 491 | 564 | 680 |
| | 48 | 190 | 221 | 254 | 323 | 398 | 478 | 564 | 655 | 751 | 907 |
| | 60 | 238 | 277 | 317 | 404 | 497 | 597 | 704 | 818 | 939 | 1134 |
| 7:12-12:12 | 12 | 49 | 55 | 62 | 76 | 94 | 112 | 132 | 153 | 175 | 211 |
| | 24 | 98 | 110 | 123 | 153 | 187 | 224 | 263 | 305 | 350 | 421 |
| | 36 | 147 | 165 | 185 | 229 | 281 | 336 | 395 | 458 | 525 | 632 |
| | 48 | 196 | 220 | 246 | 306 | 374 | 448 | 527 | 611 | 700 | 842 |
| | 60 | 244 | 275 | 308 | 382 | 468 | 560 | 659 | 763 | 874 | 1053 |

1 Tabulated ridge connection loads shall be permitted to be multiplied by 0.70 for framing not located within 6 feet of corners for buildings less than 30 feet in width (W), or W/5 for buildings greater than 30 feet in width.

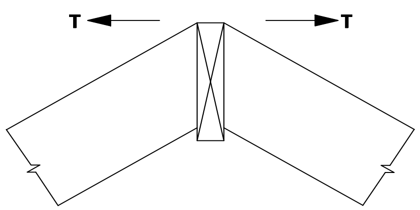
2 Tabulated ridge connection loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

3 Tabulated ridge connection loads are specified in pounds per linear foot of ridge. To determine connection requirements, multiply the tabulated ridge connection load by the multiplier from the table below corresponding to the spacing of the connectors:

| Ridge Connection Spacing (in.) | 12 | 16 | 19.2 | 24 | 48 |
|---|---|---|---|---|---|
| Multiplier | 1.00 | 1.33 | 1.60 | 2.00 | 4.00 |

4 Tabulated ridge connection loads assume 0.6 of the roof assembly design dead load (0.6 x 10 psf).

5 For buildings with roof slopes of less than 3:12, the roof framing members shall be attached to the ridge beam with connectors in accordance with Table 2.2A.

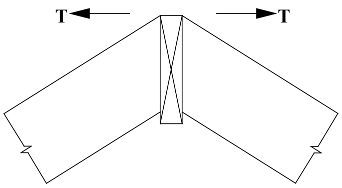



Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2B Ridge Connection Loads From Wind

***Description:*** Strap connection capacity required at ridge to resist separation due to wind uplift.

***Procedure:*** Compute the wind uplift force using MWFRS coefficients. Sum moments to compute maximum horizontal tension force (T).




***Background:*** Ridge straps restrain the ridge from separating under suction wind loads.

**Example:**

Given - 150 mph, Exposure B, 33' MRH, 7:12 roof pitch, 36' roof span, 10 psf roof dead load, 16" o.c.

Wind pressures at ridge:

$$p = qGC_{pf} - qGC_{pi}$$

**where:**

$p$ = pressure on individual roof section

$GC_{pf}$ = external pressure coefficient for that roof section. See Commentary Table 2.2A

$q$ = 21.15 psf (See Table C1.1)

$p_{WR}$ = 21.15[-1.07 - 0.18] = -26.4 psf

$p_{LR}$ = 21.15[-0.53 - 0.18] = -15.0 psf

**where:**

$p_{WR}$ = pressure on windward roof

$p_{LR}$ = pressure on leeward roof

Since dead loads in this case are resisting uplift forces, they are multiplied by 0.6, per *ASCE 7-10* section 2.4.1. Thus roof/ceiling dead loads are equal to:

$$(10 \text{ psf})\ 0.6 = 6 \text{ psf}$$

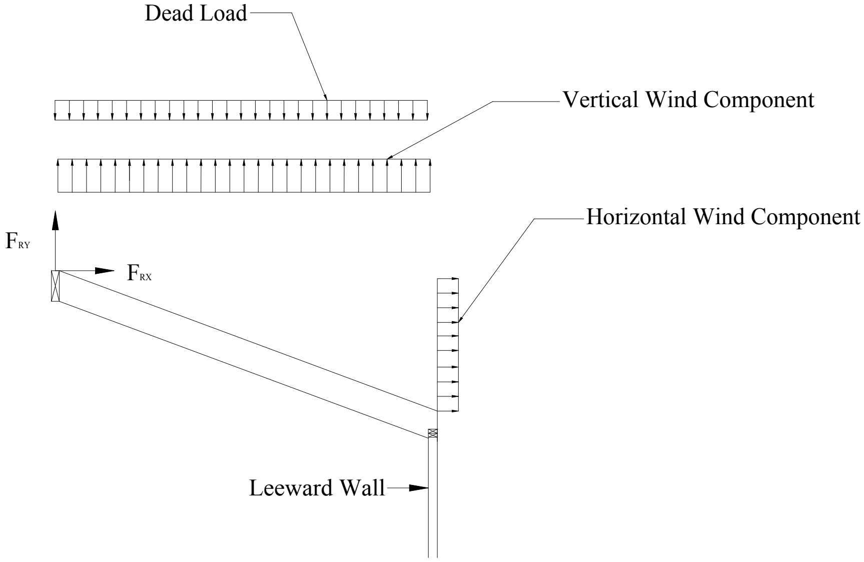
Determining horizontal wind force at ridge:

Separating the roof system at the ridge and summing moments about the windward and leeward roof-to-wall intersection separately, the horizontal ridge forces needed to counterbalance the applied wind loads, designated $F_{Rx\ (windward)}$ and $F_{Rx\ (leeward)}$, can be determined. As in the commentary for Table 2.2A, calculations are for a 1 foot wide section of roof and similar force notations; WR, WO, etc. are used. Subscripts V and H designate the direction of loads.

| | | | | |
|---|---|---|---|---|
| $WR_V$ | = | -26.4(36 / 2) | = | -475.2 lbs |
| $LR_V$ | = | -15.0(36 / 2) | = | -270.0 lbs |
| $WR_H$ | = | -26.4(36 / 2)(7/12) | = | -277.2 lbs |
| $LR_H$ | = | -15.0(36 / 2)(7/12) | = | -157.5 lbs |

Finally, the dead load R of each section of the roof is added. The dead loads act vertically:

| | | | | |
|---|---|---|---|---|
| $RWR_V$ | = | 6.0(18) | = | 108 lbs |
| $RLR_V$ | = | 6.0(18) | = | 108 lbs |




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

Summing moments about the windward top of wall:

$$\Sigma M_W = 0 = -[(-475.2) + (108)][9] - [-277.2][5.25] + 10.5F_{Rx} + 18F_{Ry}$$
$$= 3304.8 + 1455.3 + 10.5F_{Rx} + 18F_{Ry}$$

Summing moments about the leeward top of wall:

$$\Sigma M_L = 0 = [(-270.0) + (108)][9] + [-157.5][5.25] - 10.5F_{Rx} + 18F_{Ry}$$
$$= -1458.0 - 826.9 - 10.5F_{Rx} + 18F_{Ry}$$

Setting $18F_{Ry\,(leeward)} = 18F_{Ry\,(windward)}$

$$\Sigma M_W = 0 = 3304.8 + 1455.3 + 10.5F_{Rx} = -1458.0 - 826.9 - 10.5F_{Rx}$$

$$21F_{Rx} = 7045.0$$

Solving for $F_{Rx}$:

$$F_{Rx} = -336 \text{ plf}$$

Maximum horizontal tension force (T) per the figure of Table 2.2B:

$T = 336$ plf (WFCM Table 2.2B)

Required capacity of ridge connections spaced at 16" o.c.

$= 336$ plf (16 in./12in./ft)
$= 447$ lbs $= T(1.33)$ (WFCM Table 2.2B Footnote 3)

2

ENGINEERED DESIGN

**Footnote 1:**
For framing not located within W/5 or 6' of corners, wind pressures are reduced.

Horizontal force at ridge:

$$p_{WR} = 21.15[-0.69 - 0.18] = -18.4 \text{ psf}$$

$$p_{LR} = 21.15[-0.37 - 0.18] = -11.6 \text{ psf}$$

Using the reduced horizontal wind force and solving for $F_{Rx}$:

$$F_{Rx} = 218 \text{ plf}$$

Dividing the reduced load by the tabulated load:

$= 218 / 336$
$= 0.65$

WFCM Table 2.2B specifies a slightly conservative multiplier of 0.70 to cover all cases.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2C Rake Overhang Outlooker Uplift Connection Loads

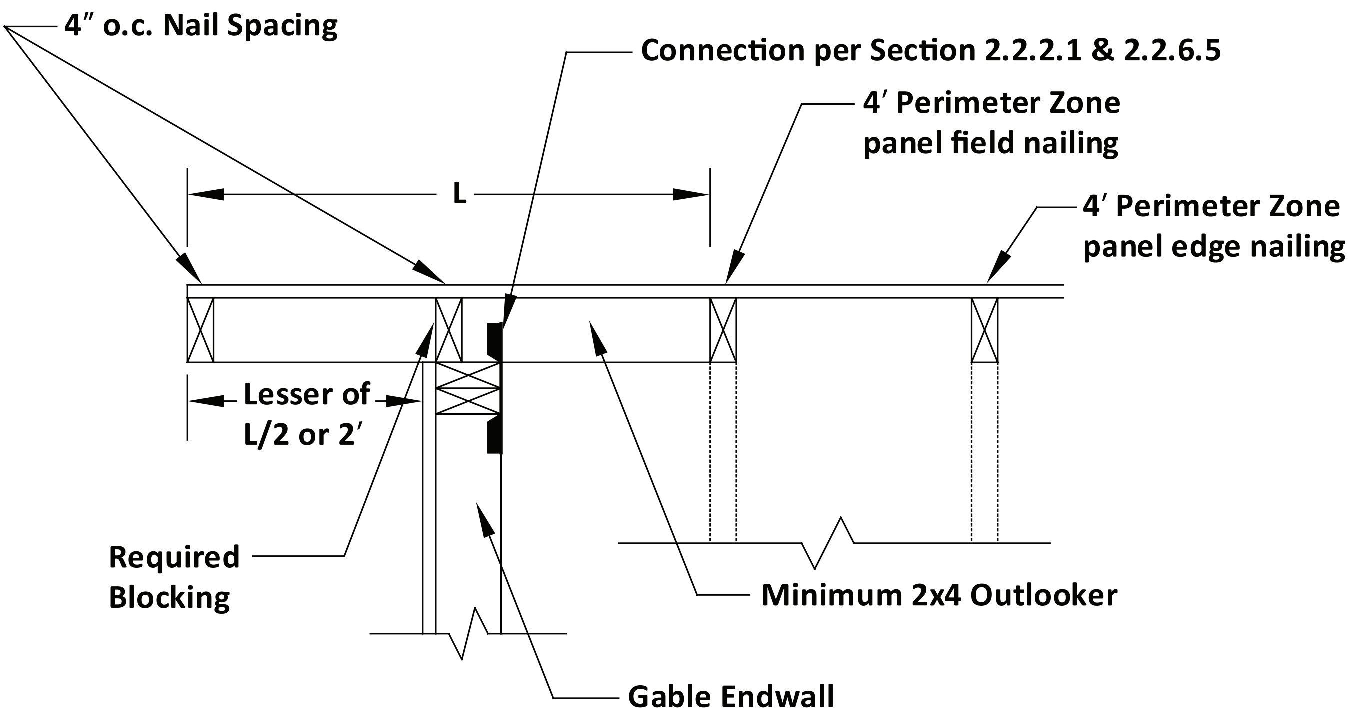
| 700-yr. Wind Speed 3-second gust (mph) | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|
| **Outlooker Spacing (in.)** | **Uplift Connection Loads (lbs)[1,2,3]** | | | | | | | | | |
| **12** | 187 | 205 | 223 | 262 | 304 | 349 | 397 | 448 | 502 | 589 |
| **16** | 250 | 273 | 298 | 349 | 405 | 465 | 529 | 597 | 669 | 786 |
| **24** | 375 | 410 | 446 | 524 | 607 | 697 | 793 | 896 [4] | 1004 [4] | 1178 [4] |

1 Tabulated outlooker uplift connection loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, or with mean roof heights less than 33 feet, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

2 Tabulated outlooker uplift connection loads are based on 2 foot overhangs. For overhangs less than 2 feet, tabulated values shall be permitted to be multiplied by $[(2' + OH) / 4']^2$ (OH measured in feet).

3 For overhangs located in Zone 2 per the figures of Table 2.4, tabulated uplift loads shall be permitted to be multiplied by 0.65.

4 Outlooker overhang length shall be limited to 20 inches. See footnote 2 to calculate reduced uplift connection load.




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.2C Rake Overhang Outlooker Uplift Connection Loads

***Description:*** Uplift loads at the connection of the rake overhang outlooker to endwall or rake truss.

***Procedure:*** Calculate wind pressures based on C&C pressure coefficients assuming Zone 3 and Zone 3 Overhang wind loads per the Figure of Table 2.4. Sum moments about the first interior truss to calculate the uplift connection load.

***Background:*** Outlooker connection loads are based on Table 2.4 suction pressures.

### Example:

Given - 150 mph, Exposure B, 24" o.c. outlooker spacing, 24" truss spacing, 2' overhang.

For Zone 3 Roof Overhangs the suction pressure is:

From WFCM Table 2.4:

78.3 psf

For Zone 3 Roof the suction pressure is:

From WFCM Table 2.4:

63.0 psf

Summing moments about the first interior truss, the uplift force at the connector:

$$\Sigma M = 0 = (78.3/12)(36\text{ in.})(24\text{ in.}) + (63.03/12)(24\text{ in.})(12\text{ in.}) - (U)(24\text{ in.} - 3.5\text{ in.})$$

$$U = [(78.26/12)(36\text{ in.})(24\text{ in.}) + (63.03/12)(24\text{ in.})(12\text{ in.})]/(24\text{ in.} - 3.5\text{ in.})$$

$$U = 349\text{ lbs/ft}$$

Required capacity of connections spaced at 24" o.c.

$$U = 349\text{ lbs/ft }(24\text{ in.}/12\text{ in./ft})$$

$$U = 697\text{ lbs} \qquad \text{(WFCM Table 2.2C)}$$

### Footnote 3:

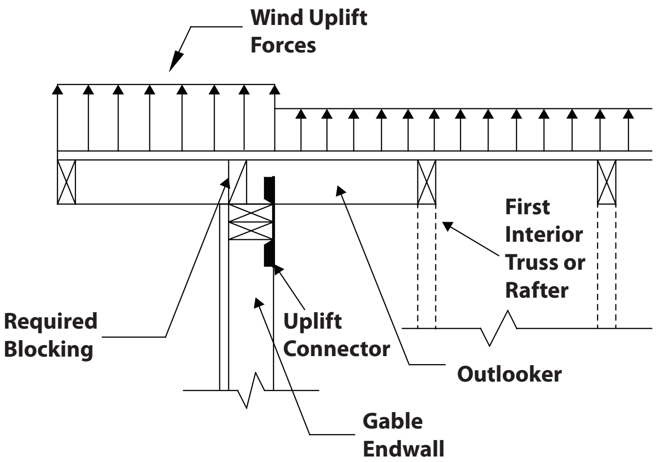
Tabulated loads may be reduced when they occur in Zone 2 per the Figure of Table 2.4. The required capacity for a connector in Zone 2 with Zone 2 overhang wind loads for the overhanging section is:

$$U_{zone\ 2} = 419\text{ lbs}$$

Thus, where Zone 2 occurs, tabulated loads may be multiplied by the following factor:

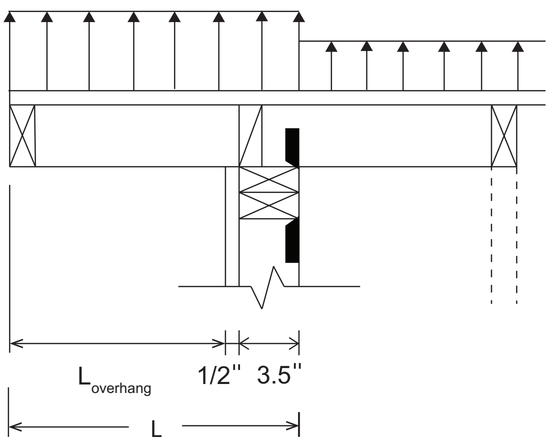
$$U_{zone\ 2}/U_{zone\ 3} = (419\text{ lbs})/(697\text{ lbs})$$

$U_{zone\ 2}/U_{zone\ 3}$ = 0.60 (a conservative multiplier of 0.65 is specified in Table 2.2 Footnote 3)




### Footnote 4:

For the footnoted cases in Table 2.2C, the outlooker overhang length, $L_{overhang}$, is limited to 20 inches based on the bending capacity of the outlooker.




$$M \leq F_b S_x$$

The bending moment, M, due to the uplift forces on the portion of the cantilevered outlooker is:

$$M = (wL^2)/2$$
$$= (132.3/12)(L^2)/2$$
$$= 5.51\,(L^2)\text{ in.-lbs/ft}$$

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

**where:**

$w$ = 132.3 psf (WFCM Table 2.4 Zone 3 Overhang @ 195 mph)

$L$ = $L_{overhang}$ + 1/2 in. (sheathing thickness) + 3.5 in. (width of stud)

$L_{overhang}$ = L - 4 in.

$F_b{}'S_x$ for a No. 2, 2x4 Southern Pine Outlooker is:

$$F_b{}'S_x = F_b C_D C_F C_r S_x$$
$$= (1{,}100 \text{ psi})(1.6)(1.0)(1.15)(3.0625)$$
$$= 6{,}199 \text{ in.-lbs}$$

(Note: In the NDS Supplement, bending design values for Southern Pine are already size-adjusted, so the size factor, $C_F$, is equal to 1.0.)

$$M_{Req.} \leq F_b{}'S_x$$

For outlookers spaced at 24" o.c.

$$(5.51)(L^2) \text{ in.-lbs/ft } (2\text{ft}) \leq 6{,}199 \text{ in.-lbs}$$
$$L \leq 23.7 \text{ in.}$$

Therefore $L_{overhang} \leq 23.7 - 4$ in.

$$= 19.7 \text{ in.} \sim 20 \text{ in.}$$

WFCM Table 2.2C limits outlooker overhang lengths to 20" for 24" outlooker spacing and the 195 mph, 180 mph, and 175 mph (700 yr, 3-second gust) Exposure B wind load cases.

NOTE: WFCM Table 3.4C provides additional outlooker overhang limits for Exposure C wind load cases.

2

ENGINEERED DESIGN

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

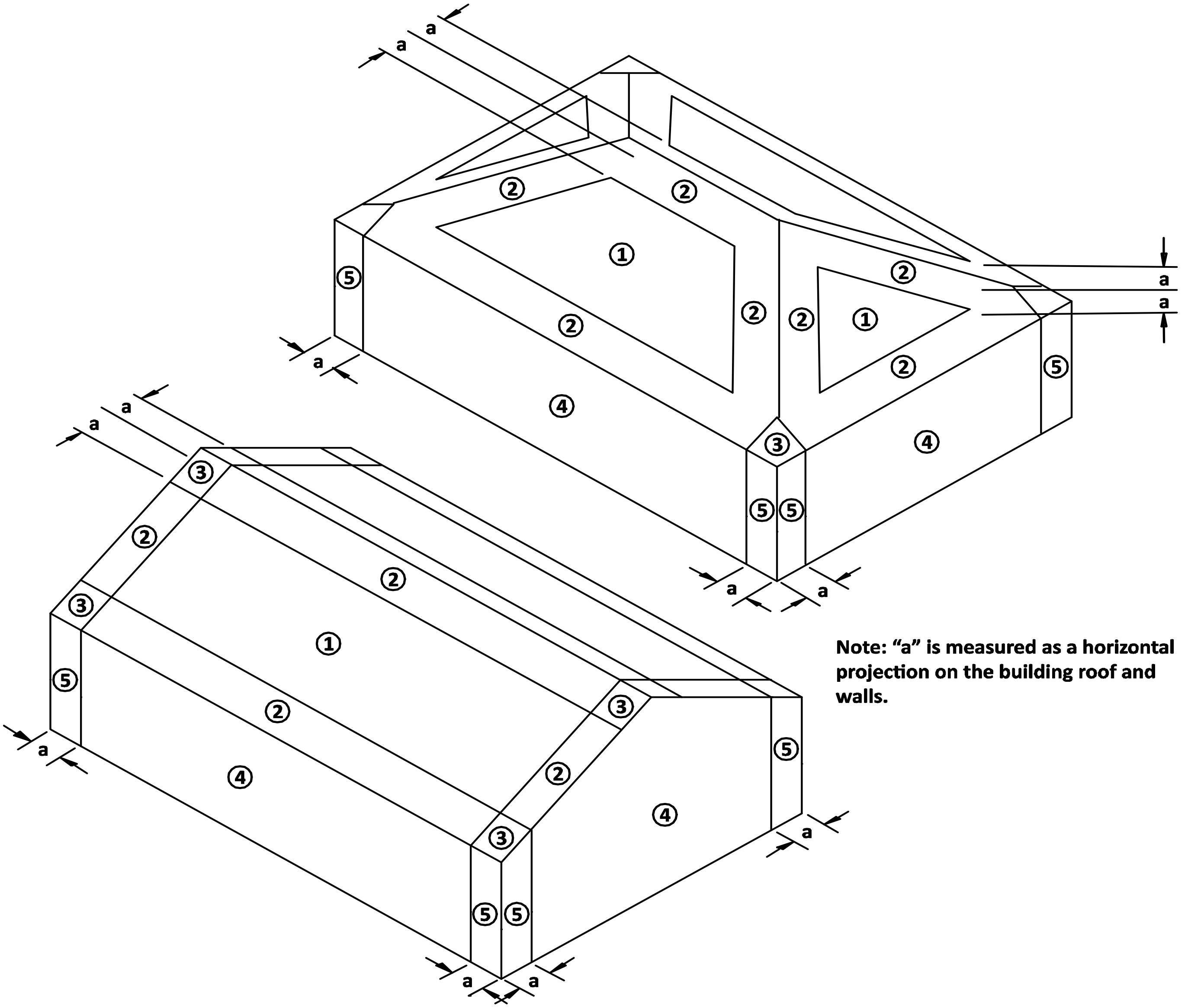
## Table 2.4 Roof and Wall Sheathing Suction Loads

(For Sheathing and Sheathing Attachment)

| 700-yr. Wind Speed 3-second gust (mph) | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|
| Sheathing Location[1] | Dual Slope Roof Suction Pressure (psf)[2] | | | | | | | | | |
| Zone 1 | 13.4 | 14.7 | 16.0 | 18.7 | 21.7 | 25.0 | 28.4 | 32.1 | 35.9 | 42.2 |
| Zone 2 | 22.5 | 24.6 | 26.8 | 31.5 | 36.5 | 41.9 | 47.6 | 53.8 | 60.3 | 70.8 |
| Zone 3 | 33.9 | 37.0 | 40.3 | 47.3 | 54.9 | 63.0 | 71.7 | 81.0 | 90.8 | 106.5 |
| Zone 3 Overhang | 42.1 | 46.0 | 50.1 | 58.8 | 68.2 | 78.3 | 89.0 | 100.5 | 112.7 | 132.3 |
| Zone 4 | 14.6 | 15.9 | 17.3 | 20.3 | 23.6 | 27.1 | 30.8 | 34.8 | 39.0 | 45.8 |
| Zone 5 | 18.0 | 19.6 | 21.4 | 25.1 | 29.1 | 33.4 | 38.0 | 42.9 | 48.1 | 56.5 |

1 The dimension, a, is measured as 10% of the minimum building dimension, but not less than 3 feet.

2 Tabulated framing loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.




Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.4 Roof and Wall Sheathing Suction Loads

***Description:*** Suction pressure on roof sheathing.

***Procedure:*** Calculate wind pressures based on C&C pressure coefficients.

***Background:*** Roof sheathing suction loads are tabulated for dual-slope roofs based on *ASCE 7-10* Figures 30.4-2A, B, and C. The roof slope creating the greatest suction loads for slopes between 7° and 45° are tabulated. Wall sheathing suction loads are tabulated based on *ASCE 7-10* Figure 30.4-1.

### Example:

150 mph, Exposure B, 33' MRH

$$p = q(GC_p - GC_{pi})$$

**where:**

$p$ = pressure on the roof/wall

$q$ = 21.15 psf (See Table C1.1)

$GC_p$ = external pressure coefficient for individual roof/wall areas

$GC_{pi}$ = +/- 0.18 internal pressure coefficient for enclosed buildings

*ASCE 7-10* states that for cladding and fasteners, the effective wind area shall not be greater than the tributary area for an individual fastener. Maximum loads occur for effective wind areas 10 ft² or less. Sheathing suction loads are therefore based on an effective wind area of 10 ft² for all roof and wall zones.

From *ASCE 7-10* Figures 30.4-2A, B, and C:

Zone 1:

$$GC_p = -0.9 - 0.1([\log(A/100)] / [\log(10/100)]) = -1.0$$

Zone 2:

$$GC_p = -1.1 - 0.7([\log(A/100)] / [\log(10/100)]) = -1.8$$

Zone 2 Overhang:

$$GC_p = -2.2$$

Zone 3:

$$GC_p = -1.7 - 1.1([\log(A/100)] / [\log(10/100)]) = -2.8$$

Zone 3 Overhang:

$$GC_p = -2.5 - 1.2([\log(A/100)] / [\log(10/100)]) = -3.7$$

For all Zones

$$GC_{pi} = +/- 0.18$$

### Calculate roof pressures:

(Negative pressures denote suction)

Zone 1:

$p$ = 21.15 (-1.0 - 0.18) = -25.0 psf (WFCM Table 2.4)

Zone 2:

$p$ = 21.15(-1.8 - 0.18) = -41.9 psf (WFCM Table 2.4)

Zone 2 Overhang:

$p$ = 21.15(-2.2) = -46.5 psf

Zone 3:

$p$ = 21.15(-2.8 - 0.18) = -63.0 psf (WFCM Table 2.4)

Zone 3 Overhang:

$p$ = 21.15(-3.7) = -78.3 psf (WFCM Table 2.4)

From *ASCE 7-10* Figure 30.4-1:

Zone 5:

$$GC_p = -0.8 - 0.6[(\log(A/100)) / (\log(10/100))] = -1.4$$

Zone 4:

$$GC_p = -0.8 - 0.3[(\log(A/100)) / (\log(10/100))] = -1.1$$

For all Zones

$$GC_{pi} = +/- 0.18$$

### Calculate wall pressures:

(Negative pressures denote suction)

Zone 4:

$p$ = 21.15(-1.1 - 0.18) = -27.1 psf (WFCM Table 2.4)

Zone 5:

$p$ = 21.15(-1.4 - 0.18) = -33.4 psf (WFCM Table 2.4)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

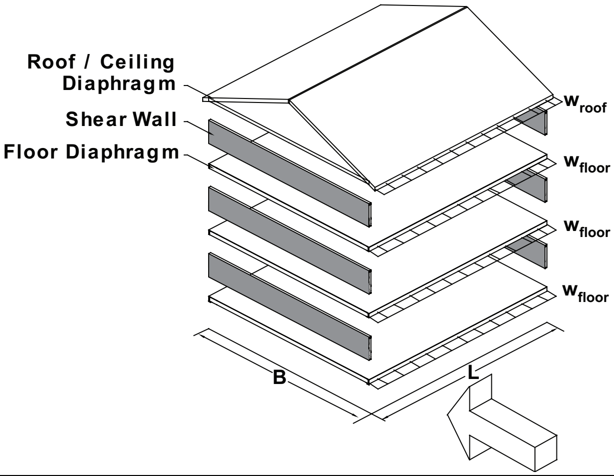
## Polling Question

**Worst case MWFRS uplift loads occur at what roof slope?**

a) 10 degrees
b) 20 degrees
c) 30 degrees
d) 45 degrees

## Table 2.5A Lateral Diaphragm Loads from Wind - Perpendicular to Ridge

(For Calculating In-Plane Shear in Roof and Floor Diaphragm)




| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Roof Pitch** | **Roof Span (ft)** | Unit Lateral Loads for Roof Diaphragm, $w_{roof\perp}$, (plf)[1,3,4,5] | | | | | | | | | |
| **0:12 - 1:12** | **24 - 60** | 60 | 60 | 60 | 62 | 72 | 82 | 94 | 106 | 118 | 139 |
| **2:12 - 3:12** | **24** | 62 | 62 | 65 | 76 | 88 | 101 | 115 | 130 | 146 | 172 |
| | **36** | 70 | 70 | 70 | 76 | 88 | 101 | 115 | 130 | 146 | 172 |
| | **48** | 77 | 77 | 77 | 77 | 88 | 101 | 115 | 130 | 146 | 172 |
| | **60** | 84 | 84 | 84 | 84 | 88 | 101 | 115 | 130 | 146 | 172 |
| **4:12** | **24** | 67 | 67 | 71 | 83 | 97 | 111 | 126 | 142 | 160 | 187 |
| | **36** | 77 | 77 | 77 | 83 | 97 | 111 | 126 | 142 | 160 | 187 |
| | **48** | 86 | 86 | 86 | 86 | 97 | 111 | 126 | 142 | 160 | 187 |
| | **60** | 96 | 96 | 96 | 96 | 97 | 111 | 126 | 142 | 160 | 187 |
| **5:12** | **24** | 72 | 72 | 72 | 84 | 97 | 112 | 127 | 143 | 161 | 189 |
| | **36** | 84 | 84 | 84 | 84 | 95 | 110 | 125 | 141 | 158 | 185 |
| | **48** | 96 | 96 | 96 | 96 | 96 | 110 | 125 | 141 | 158 | 185 |
| | **60** | 108 | 108 | 108 | 108 | 108 | 110 | 125 | 141 | 158 | 185 |
| **6:12** | **24** | 83 | 90 | 98 | 115 | 134 | 154 | 175 | 197 | 221 | 260 |
| | **36** | 94 | 102 | 112 | 131 | 152 | 174 | 198 | 224 | 251 | 295 |
| | **48** | 106 | 116 | 126 | 148 | 172 | 197 | 224 | 253 | 284 | 333 |
| | **60** | 120 | 129 | 141 | 165 | 191 | 220 | 250 | 282 | 316 | 371 |
| **7:12** | **24** | 110 | 121 | 131 | 154 | 179 | 205 | 234 | 264 | 296 | 347 |
| | **36** | 136 | 149 | 162 | 190 | 220 | 253 | 287 | 325 | 364 | 427 |
| | **48** | 163 | 178 | 194 | 227 | 263 | 302 | 344 | 388 | 435 | 511 |
| | **60** | 189 | 207 | 225 | 265 | 307 | 352 | 401 | 452 | 507 | 595 |
| **8:12** | **24** | 118 | 129 | 141 | 165 | 191 | 220 | 250 | 282 | 316 | 371 |
| | **36** | 147 | 161 | 175 | 206 | 239 | 274 | 312 | 352 | 395 | 463 |
| | **48** | 178 | 194 | 212 | 248 | 288 | 331 | 376 | 425 | 476 | 559 |
| | **60** | 208 | 228 | 248 | 291 | 338 | 388 | 441 | 498 | 558 | 655 |
| **9:12** | **24** | 126 | 138 | 150 | 176 | 204 | 234 | 266 | 301 | 337 | 396 |
| | **36** | 159 | 174 | 189 | 222 | 257 | 295 | 336 | 379 | 425 | 499 |
| | **48** | 193 | 211 | 230 | 270 | 313 | 359 | 409 | 461 | 517 | 607 |
| | **60** | 228 | 249 | 271 | 318 | 369 | 423 | 482 | 544 | 609 | 715 |
| **10:12** | **24** | 134 | 146 | 159 | 187 | 216 | 249 | 283 | 319 | 358 | 420 |
| | **36** | 170 | 186 | 203 | 238 | 276 | 317 | 360 | 407 | 456 | 535 |
| | **48** | 208 | 228 | 248 | 291 | 338 | 388 | 441 | 498 | 558 | 655 |
| | **60** | 247 | 270 | 294 | 345 | 400 | 459 | 522 | 589 | 661 | 775 |
| **11:12** | **24** | 141 | 155 | 168 | 197 | 229 | 263 | 299 | 338 | 379 | 444 |
| | **36** | 182 | 199 | 216 | 254 | 294 | 338 | 385 | 434 | 487 | 571 |
| | **48** | 224 | 245 | 266 | 313 | 362 | 416 | 473 | 534 | 599 | 703 |
| | **60** | 266 | 291 | 316 | 371 | 431 | 494 | 562 | 635 | 712 | 835 |
| **12:12** | **24** | 149 | 163 | 177 | 208 | 242 | 277 | 315 | 356 | 399 | 469 |
| | **36** | 193 | 211 | 230 | 270 | 313 | 359 | 409 | 461 | 517 | 607 |
| | **48** | 239 | 261 | 285 | 334 | 387 | 445 | 506 | 571 | 640 | 751 |
| | **60** | 285 | 311 | 339 | 398 | 462 | 530 | 603 | 681 | 763 | 895 |
| | | Unit Lateral Loads for Floor Diaphragm, $w_{floor\perp}$, (plf)[1,2,3,5] | | | | | | | | | |
| | | 135 | 148 | 161 | 189 | 219 | 251 | 286 | 323 | 362 | 425 |

See footnotes 1 - 5.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.5A Lateral Diaphragm Loads from Wind - Perpendicular to Ridge

(Roof and Floor Shear)

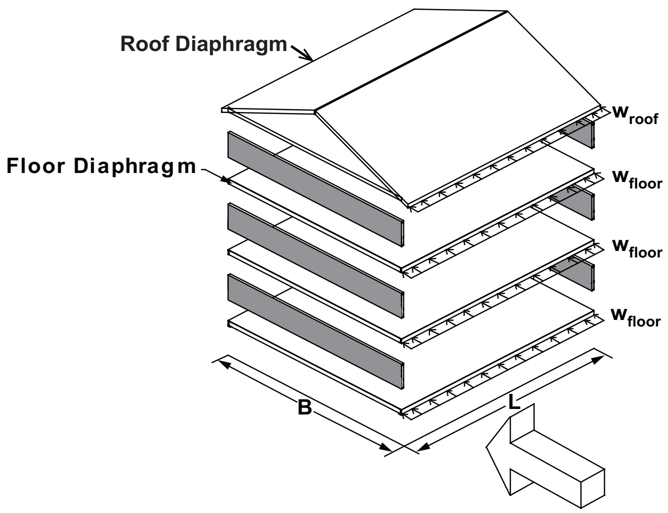
***Description:*** For calculating lateral loads in roof and floor diaphragms from wind forces acting perpendicular to the roof ridge.

***Procedure:*** Compute wall and roof wind pressures. Collect forces into diaphragms.

***Background:*** Lateral loads are based on MWFRS and are a function of roof slope. For loads into floor diaphragms, the MWFRS coefficients are based on a worst case 20 degree roof slope. Minimum 10 psf (ASD) wall pressure and minimum 5 psf (ASD) roof pressure specified by *ASCE 7-10* Section 28.4.4 are checked.

**Example:**

Given - 150 mph, Exposure B, 33' MRH, 6:12 roof slope, 24' roof span, 10' wall height.




Calculate wind forces in roof and floor diaphragms:

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

**where:**

p = pressure on the roof/walls

q = 21.15 psf (See Table C1.1)

Calculate wall pressures:

| | Interior Zone | | End Zone | |
|---|---|---|---|---|
| | Windward | Leeward | Windward | Leeward |
| $GC_{pf}$ | 0.55 | -0.39 | 0.73 | -0.53 |
| $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 |
| p (psf) | 7.8 | -12.1 | 11.6 | -15.1 |

Since these pressures act in the same direction, they will sum to 19.9 psf at the interior zone and 26.7 psf at the end zone.

Calculate the average pressure on the wall assuming a building length (parallel to ridge), L, equal to the roof span (24 ft):

$$p = [19.9(L\text{-}X) + 26.7(X)] / L$$
$$= [19.9 \text{ psf} (18 \text{ ft}) + 26.7 \text{ psf} (6 \text{ ft})] / 24$$
$$= 21.59 \text{ psf}$$

**where:**

L = Building Length (parallel to ridge)

X = End Zone Length

Calculate the roof pressures for a 6:12 roof pitch:

| | Interior Zone | | End Zone | |
|---|---|---|---|---|
| | Windward | Leeward | Windward | Leeward |
| $GC_{pf}$ | -0.10 | -0.45 | -0.19 | -0.58 |
| $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 |
| p (psf) | -5.9 | -13.3 | -7.9 | -16.3 |

Since these pressures act in opposite directions, they will sum to 7.4 psf at the interior zone and 8.4 psf at the end zone.

Calculate the average pressure on the roof:

$$p = [7.4(L\text{-}X) + 8.4(X)] / L$$
$$= [7.4(18 \text{ ft}) + 8.4(6 \text{ ft})] / 24$$
$$= 7.65 \text{ psf}$$

**where:**

W = Building Length (parallel to ridge)

X = End Zone Length

Calculate the lateral load on the roof diaphragm:

The roof diaphragm will take load from half the wall below and load directly applied to the vertical projection of the roof diaphragm.

$$w_{roof} = 21.59(10/2 \text{ ft}) + 7.65[(6/12)(24/2) \text{ ft}]$$

$$w_{roof} = 21.59(5 \text{ ft}) + 7.65(6 \text{ ft})$$
$$= 154 \text{ plf}$$ (WFCM Table 2.5A)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

Calculate average pressure on the wall given the maximum MWFRS coefficients occur at a 20 degree roof slope per *ASCE 7-10*.

| | Interior Zone | | End Zone | |
|---|---|---|---|---|
| | **Windward** | **Leeward** | **Windward** | **Leeward** |
| $GC_{pf}$ | 0.53 | -0.43 | 0.80 | -0.64 |
| $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 |
| p (psf) | 7.4 | -12.9 | 13.0 | -17.3 |

Since these pressures act in the same direction, they will sum to 20.3 psf at the interior zone and 30.3 psf at the end zone.

$$p = [20.3(18\text{ ft}) + 30.3(6\text{ ft}) +]/24$$
$$= 22.8\text{ psf}$$

Summing the wall pressure for a 10' high wall and adding an extra 1' to account for the depth of the floor joists, calculate the lateral load on the floor diaphragm:

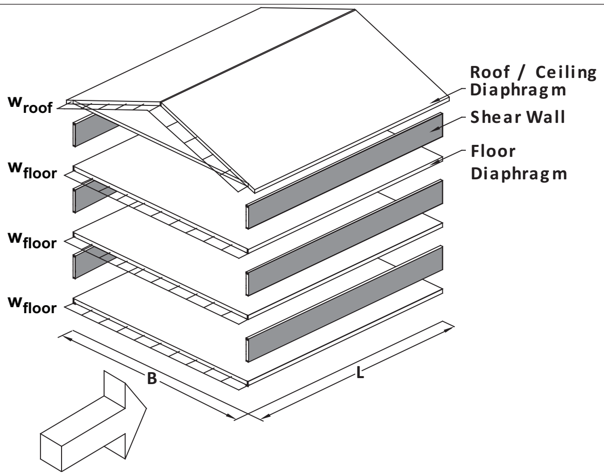
$$w_{floor} = 22.8(11)$$
$$= 251\text{ plf} \qquad \text{(WFCM Table 2.5A)}$$

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.5B Lateral Diaphragm Loads from Wind - Parallel to Ridge

(For Calculating In-Plane Shear in Roof and Floor Diaphragm)






| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Roof Pitch** | **Roof Span (ft)** | **Unit Lateral Loads for Roof Diaphragm, $w_{roof||}$, (plf)[1,3,4,5]** | | | | | | | | | |
| **0:12 - 1:12** | **24 -60** | 60 | 60 | 61 | 72 | 83 | 96 | 109 | 123 | 138 | 162 |
| **2:12 - 3:12** | **24** | 62 | 62 | 65 | 76 | 88 | 101 | 115 | 130 | 145 | 170 |
| | **36** | 70 | 70 | 71 | 84 | 97 | 111 | 126 | 143 | 160 | 188 |
| | **48** | 77 | 77 | 79 | 92 | 107 | 123 | 140 | 158 | 177 | 207 |
| | **60** | 84 | 84 | 86 | 101 | 117 | 134 | 153 | 172 | 193 | 227 |
| **4:12** | **24** | 67 | 67 | 70 | 82 | 95 | 109 | 124 | 140 | 156 | 184 |
| | **36** | 77 | 77 | 79 | 92 | 107 | 123 | 140 | 158 | 177 | 207 |
| | **48** | 86 | 86 | 88 | 104 | 120 | 138 | 157 | 177 | 199 | 233 |
| | **60** | 96 | 96 | 98 | 115 | 134 | 153 | 174 | 197 | 221 | 259 |
| **5:12** | **24** | 72 | 72 | 74 | 87 | 101 | 116 | 132 | 150 | 168 | 197 |
| | **36** | 84 | 84 | 86 | 101 | 117 | 134 | 153 | 172 | 193 | 227 |
| | **48** | 96 | 96 | 98 | 115 | 134 | 153 | 174 | 197 | 221 | 259 |
| | **60** | 108 | 108 | 110 | 130 | 150 | 173 | 196 | 222 | 248 | 292 |
| **6:12** | **24** | 77 | 77 | 79 | 93 | 108 | 124 | 141 | 159 | 179 | 210 |
| | **36** | 91 | 91 | 93 | 109 | 127 | 146 | 166 | 187 | 210 | 246 |
| | **48** | 106 | 106 | 108 | 127 | 147 | 169 | 192 | 217 | 243 | 285 |
| | **60** | 120 | 120 | 123 | 144 | 167 | 192 | 218 | 246 | 276 | 324 |
| **7:12** | **24** | 82 | 82 | 84 | 99 | 115 | 132 | 150 | 169 | 190 | 223 |
| | **36** | 98 | 98 | 101 | 118 | 137 | 157 | 179 | 202 | 226 | 266 |
| | **48** | 115 | 115 | 118 | 138 | 160 | 184 | 209 | 236 | 265 | 311 |
| | **60** | 132 | 132 | 135 | 158 | 184 | 211 | 240 | 271 | 304 | 356 |
| **8:12** | **24** | 86 | 86 | 89 | 105 | 122 | 140 | 159 | 179 | 201 | 236 |
| | **36** | 106 | 106 | 108 | 127 | 147 | 169 | 192 | 217 | 243 | 285 |
| | **48** | 125 | 125 | 128 | 150 | 174 | 199 | 227 | 256 | 287 | 337 |
| | **60** | 144 | 144 | 147 | 173 | 200 | 230 | 262 | 295 | 331 | 389 |
| **9:12** | **24** | 91 | 91 | 94 | 111 | 128 | 147 | 168 | 189 | 212 | 249 |
| | **36** | 113 | 113 | 115 | 135 | 157 | 180 | 205 | 231 | 259 | 305 |
| | **48** | 134 | 134 | 137 | 161 | 187 | 215 | 244 | 276 | 309 | 363 |
| | **60** | 156 | 156 | 159 | 187 | 217 | 249 | 284 | 320 | 359 | 421 |
| **10:12** | **24** | 96 | 96 | 99 | 117 | 135 | 155 | 177 | 199 | 223 | 262 |
| | **36** | 120 | 120 | 123 | 144 | 167 | 192 | 218 | 246 | 276 | 324 |
| | **48** | 144 | 144 | 147 | 173 | 200 | 230 | 262 | 295 | 331 | 389 |
| | **60** | 168 | 168 | 172 | 202 | 234 | 268 | 305 | 345 | 386 | 454 |
| **11:12** | **24** | 101 | 101 | 104 | 122 | 142 | 163 | 185 | 209 | 235 | 275 |
| | **36** | 127 | 127 | 130 | 153 | 177 | 203 | 231 | 261 | 293 | 343 |
| | **48** | 154 | 154 | 157 | 184 | 214 | 245 | 279 | 315 | 353 | 415 |
| | **60** | 180 | 180 | 184 | 216 | 250 | 288 | 327 | 369 | 414 | 486 |
| **12:12** | **24** | 106 | 106 | 109 | 128 | 149 | 171 | 194 | 219 | 246 | 289 |
| | **36** | 134 | 134 | 137 | 161 | 187 | 215 | 244 | 276 | 309 | 363 |
| | **48** | 163 | 163 | 167 | 196 | 227 | 261 | 297 | 335 | 375 | 441 |
| | **60** | 192 | 192 | 196 | 230 | 267 | 307 | 349 | 394 | 442 | 518 |
| | | **Unit Lateral Loads for Floor Diaphragm, $w_{floor||}$, (plf)[1,2,3,5]** | | | | | | | | | |
| | | 92 | 100 | 109 | 128 | 149 | 171 | 194 | 219 | 246 | 289 |

See footnotes 1 - 5.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.5B Lateral Diaphragm Loads from Wind - Parallel to Ridge
(Roof and Floor Shear)

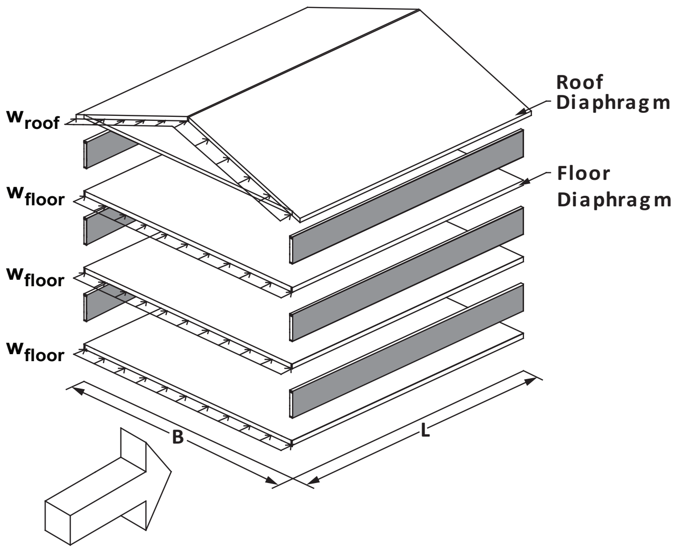
***Description:*** For calculating lateral loads in roof and floor diaphragms from wind forces acting parallel to the roof ridge.

***Procedure:*** Compute wall and roof wind pressures. Collect forces into diaphragms.

***Background:*** Lateral loads are based on MWFRS. For wind loads parallel to the ridge, MWFRS coefficients are not dependent on roof slope. Lateral forces into the roof diaphragm shall include the contribution from the triangular portion of the gable end wall in addition to the tributary portion of the wall below. Minimum 10 psf (ASD) wall pressure specified by *ASCE 7-10* Section 28.4.4 is checked.

**Example:**

Given - 150 mph, Exposure B, 33' MRH, 7:12 roof pitch, 24' roof span.




Calculate forces in roof and floor diaphragms:

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

**where:**

p = pressure on the wall

q = 21.15 psf (See Table C1.1)

Calculate wall pressures:

| | Interior Zone | | End Zone | |
|---|---|---|---|---|
| | Windward | Leeward | Windward | Leeward |
| $GC_{pf}$ | 0.40 | -0.29 | 0.61 | -0.43 |
| $GC_{pi}$ | 0.18 | 0.18 | 0.18 | 0.18 |
| p (psf) | 4.7 | -9.9 | 9.1 | -12.9 |

Since these forces act in the same direction, they will sum to 14.6 psf at the interior zone and 22.0 psf at the end zone.

Calculate the average pressure on the gable end wall.

$$p = [14.6(B\text{-}X) + 22.0(X)] / B$$
$$= [14.6(21\text{ ft}) + 22.0(3\text{ ft})] / (24\text{ ft.})$$
$$= 15.53\text{ psf}$$

**where:**

B = Building Width (perpendicular to the ridge)

X = End Zone Length

Calculate the lateral load on the roof diaphragm:

$$w_{roof} = [p_{wall}(A_{gable})/B] + [p_{wall}(H_{wall}/2)]$$
$$= [15.53(½)(24\text{ ft})(7\text{ ft})] / 24 + [15.53(10\text{ ft}/2)]$$
$$= 132\text{ plf} \quad \text{(WFCM Table 2.5B)}$$

Note: The calculation for $w_{roof}$, assumes there is no attic/floor ceiling diaphragm. If an attic floor/ceiling diaphragm is used, the lateral load on the roof diaphragm, $w_{roof}$, can be reduced by subtracting the lateral load acting on the attic floor/ceiling diaphragm per Table 2.5C.

Calculate the lateral load on the floor diaphragm:

Summing this pressure for a 10' high wall and adding an extra 1' to account for the depth of the floor joists, calculate the lateral load on the floor diaphragm:

$$w_{floor} = 15.53(11)$$
$$= 171\text{ plf} \quad \text{(WFCM Table 2.5B)}$$

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Footnotes to Tables 2.5A and 2.5B

1 The total shear load equals the tabulated unit lateral load multiplied by the building length perpendicular to the wind direction.

2 Tabulated unit lateral loads are based on 10 foot wall heights and a 1 foot floor depth. For other wall heights, H, tabulated values for floor diaphragms shall be permitted to be used when multiplied by (H+1)/11.

3 Tabulated unit lateral loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

4 Hip roof systems shall be designed using Table 2.5A for both orthogonal directions.

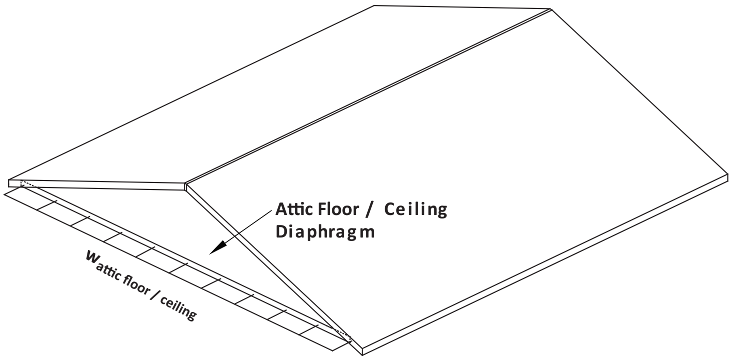
5 Shear capacity requirements for roof diaphragms, shear walls, and floor diaphragms shall be calculated as follows:

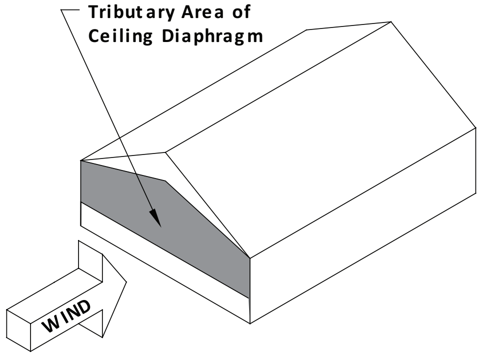
<table>
<tr><th colspan="2">Calculating Total Shear Capacity Requirements $V_{roof}$, $V_{floor}$<br><br>(lbs)</th><th colspan="2">Calculating Diaphragm Unit Shear Capacity Requirements $v_{roof}$, $v_{floor}$<br><br>(plf)</th><th colspan="2">Calculating Total Shear Wall Shear Capacity Requirements $V_{wall}$<br><br>(plf)</th></tr>
<tr><th>Wind Perpendicular to Ridge (“w” from Table 2.5A)</th><th>Wind Parallel to Ridge (“w” from Table 2.5B)</th><th>Wind Perpendicular to Ridge</th><th>Wind Parallel to Ridge</th><th>Wind Perpendicular to Ridge</th><th>Wind Parallel to Ridge</th></tr>
<tr><td rowspan="6">$V_{roof\perp} = w_{roof\perp}(L)$<br>$V_{floor\,(i)\perp} = w_{floor(i)\perp}(L)$</td><td rowspan="6">$V_{roof\|} = w_{roof\|}(B)$<br>$V_{floor\,(i)\|} = w_{floor(i)\|}(B)$</td><td rowspan="6">$v_{roof\perp} = \frac{V_{roof\perp}}{2(B)}$<br>$v_{floor(i)\perp} = \frac{V_{floor(i)\perp}}{2(B)}$</td><td rowspan="6">$v_{roof\|} = \frac{V_{roof\|}}{2(L)}$<br>$v_{floor(i)\|} = \frac{V_{floor(i)\|}}{2(L)}$</td><td colspan="2">Shear Wall Bracing Roof & Ceiling</td></tr>
<tr><td>$V_{wall\perp} = V_{roof\perp}$</td><td>$V_{wall\|} = V_{roof\|}$</td></tr>
<tr><td colspan="2">Shear Wall Bracing Roof/Ceiling & 1 Floor</td></tr>
<tr><td>$V_{wall\perp} = V_{roof\perp} + V_{floor(1)\perp}$</td><td>$V_{wall\|} = V_{roof\|} + V_{floor(1)\|}$</td></tr>
<tr><td colspan="2">Shear Wall Bracing Roof/Ceiling & 2 Floors</td></tr>
<tr><td>$V_{wall\perp} = V_{roof\perp} + V_{floor(1)\perp} + V_{floor(2)\perp}$</td><td>$V_{wall\|} = V_{roof\|} + V_{floor(1)\|} + V_{floor(2)\|}$</td></tr>
</table>

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.5C Lateral Diaphragm Loads from Wind - Parallel to Ridge

(For Attic Floor or Ceiling Diaphragm When Bracing Gable Endwall)




| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof Pitch | Roof Span (ft) | Unit Lateral Loads for Attic Floor/Ceiling Diaphragm, $w_{attic\ floor/ceiling}$ (plf)[1,2,3,4,5,6,7,8] | | | | | | | | | |
| 0:12 - 1:12 | 24 - 60 | 54 | 54 | 55 | 65 | 75 | 86 | 98 | 111 | 124 | 146 |
| 2:12-3:12 | 24 | 55 | 55 | 57 | 67 | 78 | 89 | 102 | 115 | 129 | 151 |
| | 36 | 59 | 59 | 60 | 71 | 82 | 94 | 107 | 121 | 135 | 159 |
| | 48 | 62 | 62 | 64 | 75 | 87 | 100 | 113 | 128 | 144 | 168 |
| | 60 | 66 | 66 | 67 | 79 | 92 | 105 | 120 | 135 | 152 | 178 |
| 4:12 | 24 | 58 | 58 | 60 | 70 | 81 | 93 | 106 | 120 | 134 | 157 |
| | 36 | 62 | 62 | 64 | 75 | 87 | 100 | 113 | 128 | 144 | 168 |
| | 48 | 67 | 67 | 69 | 81 | 94 | 107 | 122 | 138 | 155 | 181 |
| | 60 | 72 | 72 | 74 | 86 | 100 | 115 | 131 | 148 | 166 | 194 |
| 5:12 | 24 | 60 | 60 | 62 | 73 | 84 | 97 | 110 | 125 | 140 | 164 |
| | 36 | 66 | 66 | 67 | 79 | 92 | 105 | 120 | 135 | 152 | 178 |
| | 48 | 72 | 72 | 74 | 86 | 100 | 115 | 131 | 148 | 166 | 194 |
| | 60 | 78 | 78 | 80 | 94 | 109 | 125 | 142 | 160 | 179 | 211 |
| 6:12 | 24 | 62 | 62 | 65 | 76 | 88 | 101 | 115 | 130 | 145 | 170 |
| | 36 | 70 | 70 | 71 | 84 | 97 | 111 | 126 | 143 | 160 | 188 |
| | 48 | 77 | 77 | 79 | 92 | 107 | 123 | 140 | 158 | 177 | 207 |
| | 60 | 84 | 84 | 86 | 101 | 117 | 134 | 153 | 172 | 193 | 227 |
| 7:12 | 24 | 65 | 65 | 67 | 79 | 91 | 105 | 119 | 135 | 151 | 177 |
| | 36 | 73 | 73 | 75 | 88 | 102 | 117 | 133 | 150 | 168 | 198 |
| | 48 | 82 | 82 | 83 | 98 | 114 | 130 | 148 | 167 | 188 | 220 |
| | 60 | 90 | 90 | 92 | 108 | 125 | 144 | 164 | 185 | 207 | 243 |
| 8:12 | 24 | 67 | 67 | 70 | 82 | 95 | 109 | 124 | 140 | 156 | 184 |
| | 36 | 77 | 77 | 79 | 92 | 107 | 123 | 140 | 158 | 177 | 207 |
| | 48 | 86 | 86 | 88 | 104 | 120 | 138 | 157 | 177 | 199 | 233 |
| | 60 | 96 | 96 | 98 | 115 | 134 | 153 | 174 | 197 | 221 | 259 |
| 9:12 | 24 | 70 | 70 | 72 | 85 | 98 | 113 | 128 | 145 | 162 | 190 |
| | 36 | 80 | 80 | 82 | 96 | 112 | 128 | 146 | 165 | 185 | 217 |
| | 48 | 91 | 91 | 93 | 109 | 127 | 146 | 166 | 187 | 210 | 246 |
| | 60 | 102 | 102 | 104 | 122 | 142 | 163 | 185 | 209 | 235 | 275 |
| 10:12 | 24 | 72 | 72 | 74 | 87 | 101 | 116 | 132 | 150 | 168 | 197 |
| | 36 | 84 | 84 | 86 | 101 | 117 | 134 | 153 | 172 | 193 | 227 |
| | 48 | 96 | 96 | 98 | 115 | 134 | 153 | 174 | 197 | 221 | 259 |
| | 60 | 108 | 108 | 110 | 130 | 150 | 173 | 196 | 222 | 248 | 292 |
| 11:12 | 24 | 74 | 74 | 77 | 90 | 105 | 120 | 137 | 154 | 173 | 203 |
| | 36 | 88 | 88 | 90 | 105 | 122 | 140 | 159 | 180 | 201 | 236 |
| | 48 | 101 | 101 | 103 | 121 | 140 | 161 | 183 | 207 | 232 | 272 |
| | 60 | 114 | 114 | 117 | 137 | 159 | 182 | 207 | 234 | 262 | 308 |
| 12:12 | 24 | 77 | 77 | 79 | 93 | 108 | 124 | 141 | 159 | 179 | 210 |
| | 36 | 91 | 91 | 93 | 109 | 127 | 146 | 166 | 187 | 210 | 246 |
| | 48 | 106 | 106 | 108 | 127 | 147 | 169 | 192 | 217 | 243 | 285 |
| | 60 | 120 | 120 | 123 | 144 | 167 | 192 | 218 | 246 | 276 | 324 |

See footnotes 1 - 8.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Footnotes to Table 2.5C

1 The total shear load equals the tabulated unit lateral load multiplied by the endwall length.

2 Tabulated unit lateral loads are based on 10 foot wall heights.

3 Tabulated unit lateral loads assume the attic floor/ceiling diaphragm is continuous between endwalls. When the diaphragm only resists loads from one endwall, the tabulated unit lateral load shall be multiplied by 0.84.

4 Tabulated unit lateral loads assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

5 Attic floor or ceiling diaphragms shall not be used to brace gable endwalls used with cathedral ceilings.

6 Attic floor or ceiling diaphragms are not required for hip roof systems.

7 When a ceiling diaphragm is used to brace the gable endwalls, the unit lateral loads on the roof diaphragm, when the wind is parallel to the ridge, need not exceed the tabulated roof lateral load (from Table 2.5B) minus the ceiling lateral load (from Table 2.5C).

8 Shear capacity requirements for attic floor or ceiling diaphragms shall be calculated as follows:

| Calculating Total Shear Capacity Requirements $V_{roof}$, $V_{attic\ floor/ceiling}$ (lbs) | Calculating Diaphragm Unit Shear Capacity Requirements $v_{roof}$, $v_{attic\ floor/ceiling}$ (plf) |
|---|---|
| $V_{attic\ floor/ceiling} = w_{attic\ floor/ceiling}(B)$ <br> $V_{roof} = V_{roof\,\|\|(from\,Table\ 2.5B)} - V_{attic\ floor/ceiling}$ | $v_{attic\ floor/ceiling} = \frac{V_{attic\ floor/ceiling}}{2(L)}$ <br> $v_{roof} = \frac{V_{roof}}{2(L)}$ |

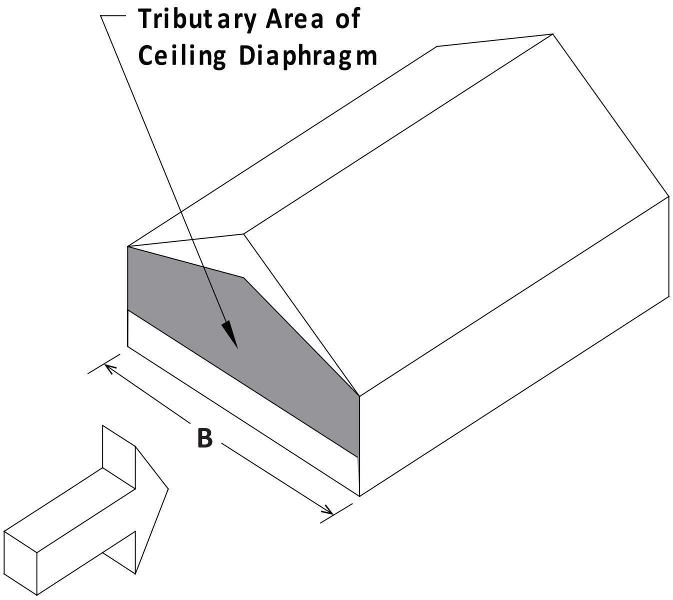
Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.5C Lateral Diaphragm Loads from Wind - Parallel to Ridge

***Description:*** For calculating lateral loads in attic floor or ceiling diaphragms when bracing gable end walls.

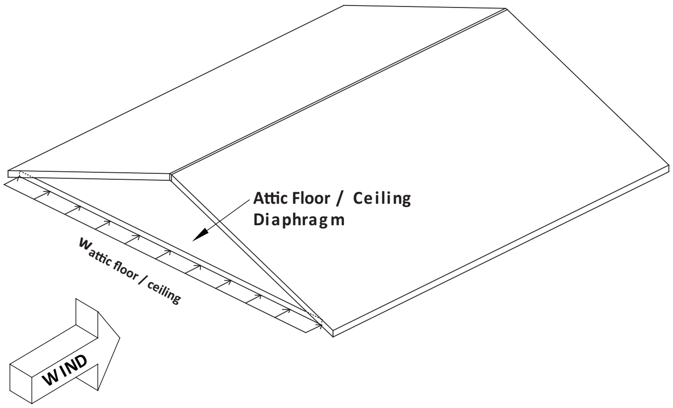
***Procedure:*** Compute loads on end walls tributary to the attic floor/ceiling diaphragm.

***Background:*** Lateral loads are based on MWFRS. For wind loads parallel to the ridge, MWFRS coefficients are not dependent on roof slope. Lateral forces into the attic floor/ceiling diaphragm shall include the tributary contribution from the triangular portion of the gable end wall in addition to the tributary portion of the wall below. Minimum 10 psf (ASD) wall pressure specified by *ASCE 7-10* Section 28.4.4 is checked.




### Example:

Given - 150 mph, Exposure B, 33' MRH, 6:12 roof slope, 24' building width (perpendicular to ridge).




Calculate the wall pressure:

p = 15.53 psf (See Commentary for Table 2.5B)

Calculate the lateral load on the attic floor/ceiling diaphragm:

When an attic floor or ceiling diaphragm is used to brace a gable end wall for wind, the loaded area equals half the area of the gable above plus half the wall below. The areas are calculated as:

$$A_{gable} = (½)[½(6\text{ ft})(24\text{ ft})] = 36\text{ ft}^2$$

$$A_{wall} = (5\text{ ft})(24\text{ ft}) = 120\text{ ft}^2$$

$$A_{total} = 36+120 = 156\text{ ft}^2$$

$$w_{attic\ floor/ceiling} = [p_{wall}(A_{total})] / W$$

$$= [15.53\text{ psf}(156\text{ ft}^2)] / 24\text{ ft}$$

$$= 101\text{ plf} \quad \text{(WFCM Table 2.5C)}$$

### Footnote 3:

Where the diaphragm resists loads from only one end wall the average pressure on the wall is a function of either the loads on the windward or leeward wall. For a gable end wall, the worst case MWFRS loads occur on the windward wall. Tabulated loads may be conservatively reduced by taking the ratio of the windward load for a single end wall to the windward and leeward wall loads. The table below shows the external MWFRS pressure coefficients and internal pressure coefficients for windward and leeward end walls. It can be shown that the most conservative case (least amount of reduction) when the attic/ceiling diaphragm resists loads from only one end wall, is to use a negative internal pressure for the interior zone as follows:

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

| | Interior Zone | | End Zone | |
|---|---|---|---|---|
| | Windward | Leeward | Windward | Leeward |
| $GC_{pf}$ | 0.40 | -0.29 | 0.61 | -0.43 |
| $GC_{pi}$ | ±0.18 | ±0.18 | ±0.18 | ±0.18 |
| positive internal pressure (psf) | 4.7 | -9.9 | 9.1 | -12.9 |
| negative internal pressure (psf) | 12.3 | -2.3 | 16.7 | -5.2 |

Reduction Factor = 12.3 / [12.3 - (-2.3)]
= 0.84

**Footnote 7:**
The lateral load on the roof diaphragm from Table 2.5B may be reduced as follows using this example:

$w_{roof}$ (with attic floor/ceiling diaphragm) = $w_{roof}$ (without attic floor/ceiling diaphragm) – $w_{attic\ floor/ceiling}$ = 124 – 101 = 23 plf.

2

ENGINEERED DESIGN

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Polling Question

**Which of the following is false regarding attic floor/ceiling diaphragms?**

a) Lateral loads are based on wind parallel to ridge

b) Assumed to be continuous between endwalls

c) Uses tributary area of the entire triangular gable

d) Are not required for hip roof systems

## Table 2.9A Exterior Wall Stud Bending Stresses from Wind Loads (Cont.)

| 700-yr. Wind Speed 3-second gust (mph) | | 150 | | | 160 | | | 170 | | | 180 | | | 195 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Stud Size | | 2x4 | 2x6 | 2x8 | 2x4 | 2x6 | 2x8 | 2x4 | 2x6 | 2x8 | 2x4 | 2x6 | 2x8 | 2x4 | 2x6 | 2x8 |
| Wall Height | Stud Spacing | Induced $f_b$ (psi)[1,2,3] | | | | | | | | | | | | | | |
| 8 ft | 12 in. | 903 | 366 | 211 | 1028 | 416 | 240 | 1160 | 470 | 270 | 1301 | 527 | 303 | 1527 | 618 | 356 |
| | 16 in. | 1205 | 488 | 281 | 1371 | 555 | 319 | 1547 | 627 | 361 | 1735 | 702 | 404 | 2036 | 824 | 474 |
| | 24 in. | 1807 | 732 | 421 | 2056 | 833 | 479 | 2321 | 940 | 541 | 2602 | 1054 | 606 | 3054 | 1237 | 712 |
| 10 ft | 12 in. | 1365 | 553 | 318 | 1553 | 629 | 362 | 1754 | 710 | 409 | 1966 | 796 | 458 | 2307 | 934 | 538 |
| | 16 in. | 1820 | 737 | 424 | 2071 | 839 | 483 | 2338 | 947 | 545 | 2621 | 1062 | 611 | 3077 | 1246 | 717 |
| | 24 in. | 2731 | 1106 | 636 | 3107 | 1258 | 724 | 3507 | 1420 | 817 | 3932 | 1592 | 916 | 4615 | 1869 | 1076 |
| 12 ft | 12 in. | 1906 | 772 | 444 | 2168 | 878 | 505 | 2448 | 991 | 570 | 2744 | 1111 | 640 | 3220 | 1304 | 751 |
| | 16 in. | 2541 | 1029 | 592 | 2891 | 1171 | 674 | 3263 | 1322 | 761 | 3659 | 1482 | 853 | 4294 | 1739 | 1001 |
| | 24 in. | 3811 | 1543 | 888 | 4336 | 1756 | 1011 | 4895 | 1982 | 1141 | 5488 | 2222 | 1279 | - | 2608 | 1501 |
| 14 ft | 12 in. | 2519 | 1020 | 587 | 2866 | 1161 | 668 | 3236 | 1310 | 754 | 3628 | 1469 | 845 | 4258 | 1724 | 992 |
| | 16 in. | 3359 | 1360 | 783 | 3822 | 1548 | 891 | 4314 | 1747 | 1006 | 4837 | 1959 | 1127 | 5677 | 2299 | 1323 |
| | 24 in. | 5039 | 2040 | 1174 | 5733 | 2322 | 1336 | - | 2621 | 1508 | - | 2938 | 1691 | - | 3448 | 1985 |
| 16 ft | 12 in. | 3203 | 1297 | 746 | 3644 | 1476 | 849 | 4113 | 1666 | 959 | 4612 | 1868 | 1075 | 5412 | 2192 | 1261 |
| | 16 in. | 4270 | 1729 | 995 | 4858 | 1967 | 1132 | 5485 | 2221 | 1278 | - | 2490 | 1433 | - | 2922 | 1682 |
| | 24 in. | - | 2594 | 1493 | - | 2951 | 1698 | - | 3332 | 1917 | - | 3735 | 2150 | - | 4383 | 2523 |
| 18 ft | 12 in. | 3952 | 1600 | 921 | 4496 | 1821 | 1048 | 5076 | 2056 | 1183 | 5691 | 2304 | 1326 | - | 2705 | 1556 |
| | 16 in. | 5269 | 2134 | 1228 | 5995 | 2428 | 1397 | - | 2741 | 1577 | - | 3073 | 1768 | - | 3606 | 2075 |
| | 24 in. | - | 3201 | 1842 | - | 3642 | 2096 | - | 4111 | 2366 | - | 4609 | 2652 | - | 5409 | 3113 |
| 20 ft | 12 in. | 4764 | 1929 | 1110 | 5420 | 2195 | 1263 | - | 2478 | 1426 | - | 2778 | 1599 | - | 3260 | 1876 |
| | 16 in. | - | 2572 | 1480 | - | 2927 | 1684 | - | 3304 | 1901 | - | 3704 | 2132 | - | 4347 | 2502 |
| | 24 in. | - | 3859 | 2221 | - | 4390 | 2527 | - | 4956 | 2852 | - | 5556 | 3198 | - | - | 3753 |

1 Tabulated bending stresses assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures, the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

2 Tabulated bending stresses shall be permitted to be multiplied by 0.92 for framing not located within 3 feet of corners for buildings less than 30 feet in width (W), or within W/10 of corners for buildings greater than 30 feet in width.

3 The tabulated bending stress ($f_b$) shall be less than or equal to the allowable bending design value ($F_b'$).

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.9A Exterior Wall Stud Bending Stresses from Wind Loads

***Description:*** Bending stress in wall studs due to wind load.

***Procedure:*** Compute wind pressures using C&C coefficients and calculate stud requirements.

***Background:*** As in Table 2.4, peak suction forces are very high. Defining the effective wind area and the tributary area of the wall stud is key to computing the design suction. Stud span equals the wall height minus the thickness of the top and bottom plates. For a nominal 8' wall, the height is: 97 1/8" - 4.5" = 92 3/8". Two cases have been checked in these tables. For C&C wind pressures, the bending stresses are computed independent of axial stresses. In addition, the case in which bending stresses from MWFRS pressures act in combination with axial stresses from wind and gravity loads must be analyzed. For buildings limited to the conditions in this Manual, the C&C loads control the stud design.

**Example:**
Given - 150 mph, Exposure B, 33' MRH, 10' wall height, 2x4 studs, 16" o.c. stud spacing.

Calculations from Table 2.10 for exterior wall induced moments from wind loads showed the applied bending moment for this case to be 464.6 ft-lbs.

Substituting this bending moment into a bending stress calculation:

$$
\begin{aligned}
f_b &= M_{(tabulated)}/S \\
&= 464.6\,(12)/3.0625 \\
&= 1{,}820 \text{ psi}
\end{aligned}
$$

$f_{b\,(Tabulated)}$ = 1,820 psi (WFCM Table 2.9A)

**Footnote 2:**
See Commentary Table 2.1 for calculation of footnotes.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.10 Exterior Wall Induced Moments from Wind Loads

| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall Height** | **Stud Spacing** | Induced Moment (ft-lbs)[1,2] | | | | | | | | | |
| **8 ft** | **12 in.** | 124 | 136 | 148 | 173 | 201 | 231 | 262 | 296 | 332 | 390 |
| | **16 in.** | 165 | 181 | 197 | 231 | 268 | 307 | 350 | 395 | 443 | 520 |
| | **24 in.** | 248 | 271 | 295 | 346 | 402 | 461 | 525 | 592 | 664 | 779 |
| **10 ft** | **12 in.** | 187 | 205 | 223 | 262 | 304 | 348 | 396 | 448 | 502 | 589 |
| | **16 in.** | 250 | 273 | 297 | 349 | 405 | 465 | 529 | 597 | 669 | 785 |
| | **24 in.** | 375 | 410 | 446 | 523 | 607 | 697 | 793 | 895 | 1004 | 1178 |
| **12 ft** | **12 in.** | 262 | 286 | 311 | 365 | 424 | 486 | 553 | 625 | 700 | 822 |
| | **16 in.** | 349 | 381 | 415 | 487 | 565 | 648 | 738 | 833 | 934 | 1096 |
| | **24 in.** | 523 | 572 | 622 | 731 | 847 | 973 | 1107 | 1249 | 1401 | 1644 |
| **14 ft** | **12 in.** | 346 | 378 | 411 | 483 | 560 | 643 | 732 | 826 | 926 | 1087 |
| | **16 in.** | 461 | 504 | 549 | 644 | 747 | 857 | 975 | 1101 | 1234 | 1449 |
| | **24 in.** | 692 | 756 | 823 | 966 | 1120 | 1286 | 1463 | 1652 | 1852 | 2173 |
| **16 ft** | **12 in.** | 440 | 480 | 523 | 614 | 712 | 817 | 930 | 1050 | 1177 | 1381 |
| | **16 in.** | 586 | 641 | 697 | 819 | 949 | 1090 | 1240 | 1400 | 1569 | 1842 |
| | **24 in.** | 879 | 961 | 1046 | 1228 | 1424 | 1635 | 1860 | 2100 | 2354 | 2763 |
| **18 ft** | **12 in.** | 542 | 593 | 645 | 758 | 879 | 1009 | 1147 | 1295 | 1452 | 1704 |
| | **16 in.** | 723 | 790 | 861 | 1010 | 1171 | 1345 | 1530 | 1727 | 1936 | 2273 |
| | **24 in.** | 1085 | 1186 | 1291 | 1515 | 1757 | 2017 | 2295 | 2591 | 2905 | 3409 |
| **20 ft** | **12 in.** | 654 | 715 | 778 | 913 | 1059 | 1216 | 1383 | 1562 | 1751 | 2055 |
| | **16 in.** | 872 | 953 | 1038 | 1218 | 1412 | 1621 | 1844 | 2082 | 2334 | 2740 |
| | **24 in.** | 1308 | 1429 | 1556 | 1826 | 2118 | 2432 | 2767 | 3123 | 3502 | 4110 |

1 Tabulated induced moments assume a building located in Exposure B with a mean roof height of 33 feet. For buildings located in other exposures , the tabulated values shall be multiplied by the appropriate adjustment factor in Section 2.1.3.1.

2 Tabulated induced moments shall be permitted to be multiplied by 0.92 for framing not located within 3 feet of corners for buildings less than 30 feet in width (W), or within W/10 of corners for buildings greater than 30 feet in width.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.10 Exterior Wall Induced Moments From Wind Loads

***Description:*** Applied moment on wall due to wind loads.

***Procedure:*** Calculate the applied moment based on C&C wind pressures.

***Background:*** Applied suction force is dependent on tributary areas.

**Example:**

Given - 150 mph, Exposure B, 33' MRH, 10' wall height, 16" o.c. stud spacing.

$p_{max}$ = 29.61 psf (See Commentary to Table 2.1)

$w = p_{max}$ (16 in./12in./ft)

= 39.48 plf

Substituting this uniform load, w, into a bending calculation for a simply supported member:

$$M = \frac{wL^2}{8}$$

$$= \frac{(39.48)(10 - \frac{3.375}{12})^2}{8}$$

$$= 466 \text{ ft} - \text{lbs}$$

$M_{(Tabulated)}$ = 466 ft-lbs

(A value of 465ft-lbs is shown in WFCM Table 2.10 – difference due to rounding in calculation of $p_{max}$)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Polling Question

**For calculating exterior wall stud stresses per the WFCM, which of the following is true?**

a) For C&C wind pressures, bending stresses are computed independent of axial stresses

b) Bending stresses from MWFRS pressures in combination with axial stresses from gravity and wind loads must be analyzed

c) For buildings limited to WFCM conditions, C&C loads control stud design

d) All of the above

## Table 2.14A Rafter Spans for 20 psf Live Load

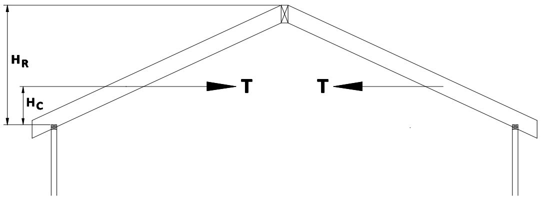
| Rafter Spacing | DL = 10 psf $f_b$ (psi) | DL = 20 psf $f_b$ (psi) | $L/\Delta_{LL}$ = 180 No Attached Ceiling $E$[4] (psi) | $L/\Delta_{LL}$ = 240 Attached Ceiling: Flexible Finish (including gypsum board) $E$[4] (psi) | $L/\Delta_{LL}$ = 360 Attached Ceiling: Brittle Finish (including plaster and stucco) $E$[4] (psi) | 2x4 Maximum Span[1,2,3] (ft-in.) | 2x6 Maximum Span[1,2,3] (ft-in.) | 2x8 Maximum Span[1,2,3] (ft-in.) | 2x10 Maximum Span[1,2,3] (ft-in.) | 2x12 Maximum Span[1,2,3] (ft-in.) |
|---|---|---|---|---|---|---|---|---|---|---|
| 12 in. | 200 | 267 | 63,246 | 84,327 | 126,491 | 3 - 8 | 5 - 10 | 7 - 8 | 9 - 9 | 11 - 10 |
| | 400 | 533 | 178,885 | 238,514 | 357,771 | 5 - 3 | 8 - 2 | 10 - 10 | 13 - 9 | 16 - 9 |
| | 600 | 800 | 328,634 | 438,178 | 657,267 | 6 - 5 | 10 - 0 | 13 - 3 | 16 - 11 | 20 - 6 |
| | 800 | 1,067 | 505,964 | 674,619 | 1,011,929 | 7 - 5 | 11 - 7 | 15 - 3 | 19 - 6 | 23 - 9 |
| | 1,000 | 1,333 | 707,107 | 942,809 | 1,414,214 | 8 - 3 | 13 - 0 | 17 - 1 | 21 - 10 | 26-0* |
| | 1,200 | 1,600 | 929,516 | 1,239,355 | 1,859,032 | 9 - 0 | 14 - 2 | 18 - 9 | 23 - 11 | 26-0* |
| | 1,400 | 1,867 | 1,171,324 | 1,561,765 | 2,342,648 | 9 - 9 | 15 - 4 | 20 - 3 | 25 - 10 | 26-0* |
| | 1,600 | 2,133 | 1,431,084 | 1,908,111 | 2,862,167 | 10 - 5 | 16 - 5 | 21 - 7 | 26-0* | 26-0* |
| | 1,800 | 2,400 | 1,707,630 | 2,276,840 | 3,415,260 | 11 - 1 | 17 - 5 | 22 - 11 | 26-0* | 26-0* |
| | 2,000 | 2,667 | 2,000,000 | 2,666,667 | 4,000,000 | 11 - 8 | 18 - 4 | 24 - 2 | 26-0* | 26-0* |
| | 2,200 | 2,933 | 2,307,379 | 3,076,506 | 4,614,759 | 12 - 3 | 19 - 3 | 25 - 4 | 26-0* | 26-0* |
| | 2,400 | 3,200 | 2,629,068 | 3,505,424 | 5,258,137 | 12 - 9 | 20 - 1 | 26-0* | 26-0* | 26-0* |
| | 2,600 | 3,467 | 2,964,456 | 3,952,608 | 5,928,912 | 13 - 4 | 20 - 11 | 26-0* | 26-0* | 26-0* |
| | 2,800 | 3,733 | 3,313,005 | 4,417,340 | 6,626,009 | 13 - 10 | 21 - 8 | 26-0* | 26-0* | 26-0* |
| | 3,000 | 4,000 | 3,674,235 | 4,898,979 | 7,348,469 | 14 - 3 | 22 - 5 | 26-0* | 26-0* | 26-0* |
| 16 in. | 200 | 267 | 54,772 | 73,030 | 109,545 | 3 - 2 | 5 - 0 | 6 - 7 | 8 - 5 | 10 - 3 |
| | 400 | 533 | 154,919 | 206,559 | 309,839 | 4 - 6 | 7 - 1 | 9 - 4 | 11 - 11 | 14 - 6 |
| | 600 | 800 | 284,605 | 379,473 | 569,210 | 5 - 6 | 8 - 8 | 11 - 6 | 14 - 8 | 17 - 9 |
| | 800 | 1,067 | 438,178 | 584,237 | 876,356 | 6 - 5 | 10 - 0 | 13 - 3 | 16 - 11 | 20 - 6 |
| | 1,000 | 1,333 | 612,372 | 816,497 | 1,224,745 | 7 - 2 | 11 - 3 | 14 - 10 | 18 - 11 | 23 - 0 |
| | 1,200 | 1,600 | 804,984 | 1,073,313 | 1,609,969 | 7 - 10 | 12 - 4 | 16 - 3 | 20 - 8 | 25 - 2 |
| | 1,400 | 1,867 | 1,014,396 | 1,352,528 | 2,028,793 | 8 - 5 | 13 - 3 | 17 - 6 | 22 - 4 | 26-0* |
| | 1,600 | 2,133 | 1,239,355 | 1,652,473 | 2,478,709 | 9 - 0 | 14 - 2 | 18 - 9 | 23 - 11 | 26-0* |
| | 1,800 | 2,400 | 1,478,851 | 1,971,801 | 2,957,702 | 9 - 7 | 15 - 1 | 19 - 10 | 25 - 4 | 26-0* |
| | 2,000 | 2,667 | 1,732,051 | 2,309,401 | 3,464,102 | 10 - 1 | 15 - 11 | 20 - 11 | 26-0* | 26-0* |
| | 2,200 | 2,933 | 1,998,249 | 2,664,332 | 3,996,498 | 10 - 7 | 16 - 8 | 22 - 0 | 26-0* | 26-0* |
| | 2,400 | 3,200 | 2,276,840 | 3,035,787 | 4,553,680 | 11 - 1 | 17 - 5 | 22 - 11 | 26-0* | 26-0* |
| | 2,600 | 3,467 | 2,567,294 | 3,423,059 | 5,134,589 | 11 - 6 | 18 - 1 | 23 - 10 | 26-0* | 26-0* |
| | 2,800 | 3,733 | 2,869,146 | 3,825,528 | 5,738,292 | 12 - 0 | 18 - 9 | 24 - 9 | 26-0* | 26-0* |
| | 3,000 | 4,000 | 3,181,981 | 4,242,641 | 6,363,961 | 12 - 4 | 19 - 5 | 25 - 8 | 26-0* | 26-0* |
| 19.2 in. | 200 | 267 | 50,000 | 66,667 | 100,000 | 2 - 11 | 4 - 7 | 6 - 1 | 7 - 9 | 9 - 5 |
| | 400 | 533 | 141,421 | 188,562 | 282,843 | 4 - 1 | 6 - 6 | 8 - 7 | 10 - 11 | 13 - 3 |
| | 600 | 800 | 259,808 | 346,410 | 519,615 | 5 - 1 | 7 - 11 | 10 - 6 | 13 - 4 | 16 - 3 |
| | 800 | 1,067 | 400,000 | 533,333 | 800,000 | 5 - 10 | 9 - 2 | 12 - 1 | 15 - 5 | 18 - 9 |
| | 1,000 | 1,333 | 559,017 | 745,356 | 1,118,034 | 6 - 6 | 10 - 3 | 13 - 6 | 17 - 3 | 21 - 0 |
| | 1,200 | 1,600 | 734,847 | 979,796 | 1,469,694 | 7 - 2 | 11 - 3 | 14 - 10 | 18 - 11 | 23 - 0 |
| | 1,400 | 1,867 | 926,013 | 1,234,684 | 1,852,026 | 7 - 9 | 12 - 2 | 16 - 0 | 20 - 5 | 24 - 10 |
| | 1,600 | 2,133 | 1,131,371 | 1,508,494 | 2,262,742 | 8 - 3 | 13 - 0 | 17 - 1 | 21 - 10 | 26-0* |
| | 1,800 | 2,400 | 1,350,000 | 1,800,000 | 2,700,000 | 8 - 9 | 13 - 9 | 18 - 2 | 23 - 2 | 26-0* |
| | 2,000 | 2,667 | 1,581,139 | 2,108,185 | 3,162,278 | 9 - 3 | 14 - 6 | 19 - 1 | 24 - 5 | 26-0* |
| | 2,200 | 2,933 | 1,824,144 | 2,432,192 | 3,648,287 | 9 - 8 | 15 - 2 | 20 - 0 | 25 - 7 | 26-0* |
| | 2,400 | 3,200 | 2,078,461 | 2,771,281 | 4,156,922 | 10 - 1 | 15 - 11 | 20 - 11 | 26-0* | 26-0* |
| | 2,600 | 3,467 | 2,343,608 | 3,124,811 | 4,687,217 | 10 - 6 | 16 - 6 | 21 - 9 | 26-0* | 26-0* |
| | 2,800 | 3,733 | 2,619,160 | 3,492,214 | 5,238,320 | 10 - 11 | 17 - 2 | 22 - 7 | 26-0* | 26-0* |
| | 3,000 | 4,000 | 2,904,738 | 3,872,983 | 5,809,475 | 11 - 4 | 17 - 9 | 23 - 5 | 26-0* | 26-0* |
| 24 in. | 200 | 267 | 44,721 | 59,628 | 89,443 | 2 - 7 | 4 - 1 | 5 - 5 | 6 - 11 | 8 - 5 |
| | 400 | 533 | 126,491 | 168,655 | 252,982 | 3 - 8 | 5 - 10 | 7 - 8 | 9 - 9 | 11 - 10 |
| | 600 | 800 | 232,379 | 309,839 | 464,758 | 4 - 6 | 7 - 1 | 9 - 4 | 11 - 11 | 14 - 6 |
| | 800 | 1,067 | 357,771 | 477,028 | 715,542 | 5 - 3 | 8 - 2 | 10 - 10 | 13 - 9 | 16 - 9 |
| | 1,000 | 1,333 | 500,000 | 666,667 | 1,000,000 | 5 - 10 | 9 - 2 | 12 - 1 | 15 - 5 | 18 - 9 |
| | 1,200 | 1,600 | 657,267 | 876,356 | 1,314,534 | 6 - 5 | 10 - 0 | 13 - 3 | 16 - 11 | 20 - 6 |
| | 1,400 | 1,867 | 828,251 | 1,104,335 | 1,656,502 | 6 - 11 | 10 - 10 | 14 - 4 | 18 - 3 | 22 - 2 |
| | 1,600 | 2,133 | 1,011,929 | 1,349,238 | 2,023,858 | 7 - 5 | 11 - 7 | 15 - 3 | 19 - 6 | 23 - 9 |
| | 1,800 | 2,400 | 1,207,477 | 1,609,969 | 2,414,953 | 7 - 10 | 12 - 4 | 16 - 3 | 20 - 8 | 25 - 2 |
| | 2,000 | 2,667 | 1,414,214 | 1,885,618 | 2,828,427 | 8 - 3 | 13 - 0 | 17 - 1 | 21 - 10 | 26-0* |
| | 2,200 | 2,933 | 1,631,564 | 2,175,418 | 3,263,127 | 8 - 8 | 13 - 7 | 17 - 11 | 22 - 10 | 26-0* |
| | 2,400 | 3,200 | 1,859,032 | 2,478,709 | 3,718,064 | 9 - 0 | 14 - 2 | 18 - 9 | 23 - 11 | 26-0* |
| | 2,600 | 3,467 | 2,096,187 | 2,794,916 | 4,192,374 | 9 - 5 | 14 - 9 | 19 - 6 | 24 - 10 | 26-0* |
| | 2,800 | 3,733 | 2,342,648 | 3,123,531 | 4,685,296 | 9 - 9 | 15 - 4 | 20 - 3 | 25 - 10 | 26-0* |
| | 3,000 | 4,000 | 2,598,076 | 3,464,102 | 5,196,152 | 10 - 1 | 15 - 11 | 20 - 11 | 26-0* | 26-0* |

* Spans (horizontal projection) are limited to 26 feet in length. Check sources for availability of lumber in lengths greater than 20 feet. See footnotes 1-4.

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Footnotes to Table 2.14A

1 Tabulated rafter spans assume ceiling joists or rafter ties are located at the bottom of the attic space to resist thrust. When ceiling joists or rafter ties are located higher in the attic space and are used to resist thrust, the rafter spans shall be reduced using the factors given in the following table:




| Ceiling Height/Top Plate-to-Roof Ridge Height ($H_C$ / $H_R$) | Rafter Span Adjustment Factors |
|---|---|
| **1/2** | 0.58 |
| **1/3** | 0.67 |
| **1/4** | 0.76 |
| **1/5** | 0.83 |
| **1/6** | 0.90 |
| **1/7.5 and less** | 1.00 |

**Note:** Lateral deflection of the rafter below the rafter ties may exceed 3/4 inch when rafter ties are located above one-third of the top plate-to- roof ridge height, $H_R$, or when $H_c$ is greater than 2 feet and may require additional consideration.

2 Tabulated rafter spans (horizontal projection) in Table 2.14A shall be permitted to be multiplied by the sloped roof adjustment factors in the following table for roof pitches greater than 4:12:

| | 10 psf Dead | 20 psf Dead |
|---|---|---|
| **Roof Pitch** | **Adjustment Factor For Sloped Roofs** | |
| **5:12** | 1.02 | 1.01 |
| **6:12** | 1.04 | 1.03 |
| **7:12** | 1.05 | 1.04 |
| **8:12** | 1.07 | 1.05 |
| **9:12** | 1.10 | 1.07 |
| **10:12** | 1.12 | 1.08 |
| **11:12** | 1.14 | 1.10 |
| **12:12** | 1.17 | 1.12 |

3 Tabulated rafter spans (horizontal projection) in Table 2.14A are based on roof dead and live loads only. To determine the maximum rafter span from wind loading, multiply the span from Table 2.14A by the appropriate wind uplift load span adjustment factor from the tables below as well as by the rafter span adjustment factor for ceiling joist/rafter tie location from Footnote 1 and the appropriate sloped roof adjustment factor from Footnote 2. The wind load span shall not exceed the live and dead load span.

**RAFTER SPAN ADJUSTMENT FOR EXPOSURE B WIND LOADS** **Exposure B**

| 700-yr. Wind Speed 3-second gust (mph) | | 110 | 115 | 120 | 130 | 140 | 150 | 160 | 170 | 180 | 195 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Roof Pitch** | | **Factor to adjust Table 2.14A tabulated rafter spans (once adjusted per Footnotes 1 & 2 as appropriate)** | | | | | | | | | |
| **4' End Zone** | **0:12 - 3:12** | 1.17 | 1.11 | 1.05 | 0.96 | 0.88 | 0.82 | 0.76 | 0.71 | 0.67 | 0.62 |
| | **4:12** | 1.15 | 1.09 | 1.04 | 0.94 | 0.87 | 0.80 | 0.75 | 0.70 | 0.66 | 0.61 |
| | **5:12** | 1.09 | 1.04 | 0.99 | 0.90 | 0.83 | 0.77 | 0.72 | 0.67 | 0.63 | 0.58 |
| | **6:12** | 1.03 | 0.98 | 0.93 | 0.85 | 0.79 | 0.73 | 0.68 | 0.64 | 0.60 | 0.55 |
| **Interior Zone** | **0:12 - 3:12** | 1.52 | 1.43 | 1.35 | 1.22 | 1.12 | 1.03 | 0.96 | 0.89 | 0.84 | 0.77 |
| | **4:12** | 1.47 | 1.39 | 1.31 | 1.19 | 1.09 | 1.00 | 0.93 | 0.87 | 0.82 | 0.75 |
| | **5:12** | 1.39 | 1.32 | 1.25 | 1.13 | 1.04 | 0.96 | 0.89 | 0.83 | 0.78 | 0.71 |
| | **6:12** | 1.31 | 1.24 | 1.18 | 1.07 | 0.98 | 0.91 | 0.84 | 0.79 | 0.74 | 0.68 |
| **4' End & Interior Zone** | **7:12** | 1.52 | 1.43 | 1.35 | 1.22 | 1.11 | 1.02 | 0.95 | 0.88 | 0.83 | 0.76 |
| | **8:12** | 1.41 | 1.33 | 1.26 | 1.14 | 1.04 | 0.96 | 0.89 | 0.83 | 0.78 | 0.71 |
| | **9:12** | 1.31 | 1.24 | 1.17 | 1.06 | 0.97 | 0.90 | 0.84 | 0.78 | 0.73 | 0.67 |
| | **10:12** | 1.22 | 1.15 | 1.09 | 0.99 | 0.91 | 0.84 | 0.78 | 0.73 | 0.69 | 0.63 |
| | **11:12** | 1.13 | 1.07 | 1.02 | 0.93 | 0.85 | 0.79 | 0.73 | 0.68 | 0.64 | 0.59 |
| | **12:12** | 1.05 | 1.00 | 0.95 | 0.86 | 0.79 | 0.73 | 0.68 | 0.64 | 0.60 | 0.55 |

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

## Table 2.14A Rafter Spans for 20 psf Roof Live Load

***Description:*** Calculation of maximum permissible spans for lumber rafters.

***Procedure:*** Perform span calculation for a given set of E and $f_b$ properties. Adjust spans for rafter tie locations, roof pitch, and wind uplift.

***Background:*** Based on simple bending calculations, assuming the rafter is simply supported at each end. Span is assumed to be equal to the horizontal projection of the rafter.

A more sophisticated approach would take the following into account:

- Compression stresses in a rafter when a ridge board replaces a ridge beam.
- Additional bending and compression load capacity provided by roof sheathing.
- Increased length of a sloped rafter relative to the horizontal projection.
- Reduced loads using a sloped rafter length relative to the horizontal projection.

The magnitude of error introduced by ignoring additional compression is a function of load magnitude, span, and roof slope.

**Example:**
Given - 2x8 rafter, 16" o.c. rafter spacing, $f_b$ = 1,600 psi, 10 psf dead load, $\Delta_{LL} \leq L/180$ (no attached ceiling).

$w_{dead}$ = 10 psf(16 in./12) = 13.33 plf
$w_{live}$ = 20 psf(16 in./12) = 26.67 plf
$w_{total}$ = 3.33 + 26.67 = 40 plf

Calculate the moment-limited span:

$$f_b \geq \frac{w_{total} L^2}{8S}$$

$$L = \sqrt{\frac{8S(f_b)}{w_{total}}}$$

$$= \sqrt{\frac{(8)(13.14)(1,600)}{40/12}}$$

$$= 225 \text{ in.} = 18 \text{ ft } 9 \text{ in.}$$

L = 18 ft-9 in. (WFCM Table 2.14A)

Calculate the modulus of elasticity required to limit live load deflection:

$$\Delta = \frac{L}{180} \leq \frac{5 w_{live} L^4}{384\,EI}$$

$$E = \frac{(5)(180) w_{live} L^3}{384 I}$$

$$= \frac{(5)(180)\left(\frac{26.67}{12}\right)(224.6)^3}{384(47.63)}$$

$$= 1.239 \text{ x } 10^6 \text{ psi}$$

E = $1.239 \times 10^6$ psi (WFCM Table 2.14A)

**Footnote 1:**
When ceiling joists are located higher in the attic space, the rafter span shall be reduced. Assuming the maximum moment occurs at the rafter tie location, the maximum applied moment is:

$M_{max}$ = $wLx - wx^2/2$

**where:**

x = the horizontal distance from the edge of the top plate to the location of the rafter tie

Based on similar triangles:

$x/H_C = L/H_R$

Substituting x into the equation and solving for maximum moment yields:

$$M_{max} = wL\left(\frac{H_C L}{H_R}\right) - \frac{w}{2}\left(\frac{H_C L}{H_R}\right)^2$$

The maximum tabulated moment is:

$$M_{max\ tabulated} = w\frac{L^2}{8}$$

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

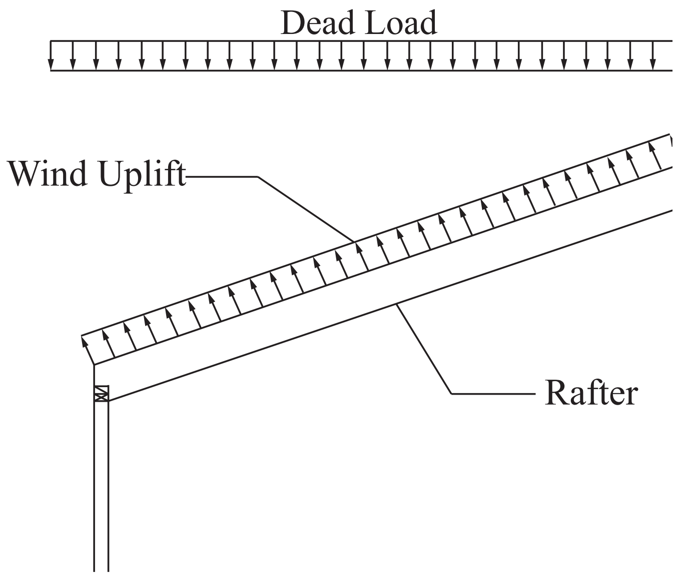

2

ENGINEERED DESIGN

The equation for horizontal deflections does not account for dead load contributions to deflection. The following amplification factor may be used to account for dead loads.

$$A = \frac{(DL + LL)}{LL}$$

**Footnote 3:**
For rafters, the tabulated spans in Table 2.14A shall be adjusted for wind uplift loads. Adjustments for wind uplift loads shall be applied after adjustments are applied for Footnote 1 (for ceiling joist/rafter tie location), and Footnote 2 (for roof pitch).




The following examples are shown for a rafter span adjustment factor for 150 mph Exposure B, 33' MRH, for the 4' End Zone, Interior Zone, and 4' End & Interior Zone, respectively for a 2x8 rafter, 16" o.c., $f_b$ = 1,600 psi, and 10 psf dead load. Components and cladding (C&C) wind exposure coefficients are used:

**Example for 4' End Zone (6:12 roof pitch):**

$p = q(GC_p - GC_{pi})$

**where:**

$q$ = 21.15 psf (See Table C1.1)

$GC_p$ = -2.6 (*ASCE 7-10* Figure 30.4-2B)

$GC_{pi}$ = +/-0.18 (internal pressure coefficient for enclosed buildings)

p = 21.15(-2.6-0.18) psf
= -58.8 psf (negative pressure denotes suction)

Wind pressure, p, acts normal to the roof surface along the actual roof span. Since the horizontal projected span is used as the basis of the adjustments, the wind load is increased as follows in order to maintain the same effect:

p = [-58.8 psf] / [(cos(6/12))(cos(6/12))]
= 73.5 psf

Adjusting for 16 in. rafter spacing:

p = -73.5(16 in./12)
= -98.0 plf

Accounting for dead load:

$w_{DL}$ = 10 psf(0.6)(16 in./12)
= 8 plf

Determining net uplift load:

= p - $w_{DL}$
= 98.0 plf – 8 plf
= 90.0 plf

Load duration factors are used to account for the difference in roof live and wind loads.

$$f_b \geq \frac{w_{total} L^2}{8S}$$

$$L = \sqrt{\frac{8S(f_b)}{w_{total}}}$$

$$= \sqrt{\frac{(8)(13.14)(1,600)(\frac{1.6}{1.25})}{90.0/12}}$$

$$= 169.42 \text{ in.}$$

$$= 14.12 \text{ ft}$$

From Table 2.14A, the maximum tabulated rafter span is 18ft 9 in. or 18.75 ft. The adjustment factor for a 10psf dead load and 6:12 roof pitch is 1.04 per Footnote 2, therefore the adjusted rafter span is calculated as follows:

Max. span = 18.75 ft (1.04)
= 19.5 ft

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

Dividing the maximum span for wind loading by the maximum span adjusted for roof slope:

(14.12 ft) / (19.5 ft)

= 0.72 (WFCM Table 2.14A Footnote 3 for "4' End Zone" lists a multiplier of 0.73. The difference results from rounding in intermediate calculations.)

**Example for Interior Zone (6:12 roof pitch):**

p = q($GC_p$ - $GC_{pi}$)

**where:**

q = 21.15 psf (See Table C1.1)

$GC_p$ = -1.7 (*ASCE 7-10* Figure 30.4-2B)

$GC_{pi}$ = +/-0.18 (internal pressure coefficient for enclosed buildings)

p = 21.15(-1.7-0.18) psf

= -39.8 psf (negative pressure denotes suction)

Wind pressure, p, acts normal to the roof surface along the actual roof span. Since the horizontal projected span is used as the basis of the adjustments, the wind load is increased as follows in order to maintain the same effect:

p = [-39.8 psf] / [(cos(6/12))(cos(6/12))]

= -49.75 psf

Adjusting for 16 in. rafter spacing:

p = -49.75(16 in./12)

= -66.33 plf

Accounting for dead load:

$w_{DL}$ = 10 psf(0.6)(16 in./12)

= 8 plf

Determining net uplift load:

= p - $w_{DL}$

= 66.33 plf – 8 plf

= 58.33 plf

Load duration factors are used to account for the difference in roof live and wind loads.

$$f_b \geq \frac{w_{total} L^2}{8S}$$

$$L = \sqrt{\frac{8S(f_b)}{w_{total}}}$$

$$= \sqrt{\frac{(8)(13.14)(1,600)(\frac{1.6}{1.25})}{58.33/12}}$$

$$= 210.45 \text{ in.}$$

$$= 17.5 \text{ft}$$

From Table 2.14A, the maximum tabulated rafter span is 18 ft 9 in. or 18.75 ft. The adjustment factor for a 10 psf dead load and 6:12 roof pitch is 1.04 per Footnote 2, therefore the adjusted rafter span is calculated as follows:

Max. span = 18.75 ft (1.04)

= 19.5 ft

Dividing the maximum span for wind loading by the maximum span adjusted for roof slope:

(17.5 ft) / (19.5 ft)

= 0.90 (WFCM Table 2.14A Footnote 3 for "Interior Zone" lists a multiplier of 0.91. The difference results from rounding in intermediate calculations.)

**Example for 4' End & Interior Zone (10:12) roof pitch:**

p = q($GC_p$ - $GC_{pi}$)

**where:**

q = 21.15 psf (See Table C1.1)

$GC_p$ = -1.2 (*ASCE 7-10* Figure 30.4-2C)

$GC_{pi}$ = +/-0.18 (internal pressure coefficient for enclosed buildings)

p = 21.15(-1.2-0.18) psf

= -29.2 psf (negative pressure denotes suction)

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

Wind pressure, p, acts normal to the roof surface along the actual roof span. Since the horizontal projected span is used as the basis of the adjustments, the wind load is increased as follows in order to maintain the same effect:

$$p = [-29.2\text{ psf}] / [(\cos(10/12))(\cos(10/12))]$$
$$= -49.48\text{ psf}$$

Adjusting for 16 in. rafter spacing:

$$p = -49.48(16\text{ in.}/12)$$
$$= -65.97\text{ plf}$$

Accounting for dead load:

$$w_{DL} = 10\text{ psf }(0.6)(16\text{ in.}/12)$$
$$= 8\text{ plf}$$

Determining net uplift load:

$$= p - w_{DL}$$
$$= 65.97\text{ plf} - 8\text{ plf}$$
$$= 57.97\text{ plf}$$

Load duration factors are used to account for the difference in roof live and wind loads.

$$f_b \geq \frac{w_{total} L^2}{8S}$$

$$L = \sqrt{\frac{8S(f_b)}{w_{total}}}$$

$$= \sqrt{\frac{(8)(13.14)(1{,}600)(\frac{1.6}{1.25})}{57.97/12}}$$

$$= 211.10\ in.$$

$$= 17.6\ ft$$

From Table 2.14A, the maximum tabulated rafter span is 18 ft 9 in. or 18.75 ft. The adjustment factor for a 10psf dead load and 10:12 roof pitch is 1.12 per Footnote 2, therefore the adjusted rafter span is calculated as follows:

Max. span = 18.75 ft (1.12)
= 21.0 ft

Dividing the maximum span for wind loading by the maximum span adjusted for roof slope:

(17.6 ft) / (21.0 ft) = 0.84 (WFCM Table 2.14A Footnote 3 for “4' End & Interior Zone”)

2

ENGINEERED DESIGN

Copyright © American Wood Council. Downloaded/printed pursuant to License Agreement. No reproduction or transfer authorized.

- **This concludes The American Institute of Architects Continuing Education Systems Course**

American Wood Council
info@awc.org
www.awc.org

AMERICAN WOOD COUNCIL