WFCM

Wood Frame Construction Manual for One- and Two-Family Dwellings

COMMENTARY

2012 EDITION

AMERICAN WOOD COUNCIL

Updates and Errata

While every precaution has been taken to ensure the accuracy of this document, errors may have occurred during development. Updates or Errata are posted to the American Wood Council website at www.awc.org. Technical inquiries may be addressed to info@awc.org.

The American Wood Council (AWC) is the voice of North American traditional and engineered wood products. From a renewable resource that absorbs and sequesters carbon, the wood products industry makes products that are essential to everyday life. AWC's engineers, technologists, scientists, and building code experts develop state-of-the-art engineering data, technology, and standards on structural wood products for use by design professionals, building officials, and wood products manufacturers to assure the safe and efficient design and use of wood structural components.

WFCM

Wood Frame Construction Manual for One- and Two-Family Dwellings

COMMENTARY

2012 EDITION

AMERICAN WOOD COUNCIL

Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings Commentary 2012 Edition

First Web Version: August 2013

ISBN 978-1-940383-00-2

Copyright Permission
American Wood Council
222 Catoctin Circle, SE, Suite 201
Leesburg, VA 20175
info@awc.org

Printed in the United States of America

Foreword

The *Wood Frame Construction Manual for One- and Two-Family Dwellings* (*WFCM*) provides engineered and prescriptive design requirements for wood frame one- and two-family dwellings.

This Commentary to the *WFCM* has been requested by architects, engineers, and others, to provide background information and example calculations for various sections and tables of the *WFCM*. The *WFCM Commentary* is intended to respond to these user needs.

The *WFCM Commentary* follows the same organization as the *WFCM*. Discussion of a particular provision in the *WFCM* is found in the *WFCM Commentary* by locating the same section or subsection number found in the *WFCM*. Not every section of the *WFCM* has a corresponding commentary section. The *Commentary* provides background information intended to give the reader an understanding of the data and/or experience upon which the provision is based. One or more examples of the calculation procedures used to produce the tables are given to illustrate the scope of conditions covered by the table.

In developing the provisions of the *WFCM*, the most recent reliable data available from laboratory tests and experience with structures in-service have been carefully analyzed and evaluated for the purpose of providing a consistent standard of practice. It is intended that this document be used in conjunction with competent engineering design, accurate fabrication, and adequate supervision of construction. Therefore, AWC does not assume any responsibility for errors or omissions in the *WFCM* and *WFCM Commentary*, nor for engineering designs or plans prepared from it.

Inquiries, comments, and suggestions from readers of this document are invited.

American Wood Council

Table of Contents

GENERAL INFORMATION

AMERICAN WOOD COUNCIL

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 2012 *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:

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

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:

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

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.

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 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 *2012 WFCM* with basic wind speeds from the *2012 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 *2012 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 *2012 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 *2012 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:

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 *2012 WFCM* incorporates this design check.

Snow Loads

The *2012 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 *2012 WFCM* includes seismic design information in accordance with *ASCE 7-10* for buildings located in Seismic Design Categories A-D, as defined by the *2012 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

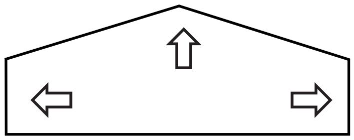
Wind pressure calculations in the *WFCM* assume that buildings are fully enclosed and that the building envelope is not breached. Interior pressure coefficients, GCpi, of +/-0.18 are used in the calculations per *ASCE 7-10* Table 26.11-1. Penetration of openings (e.g. windows and doors) due to flying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

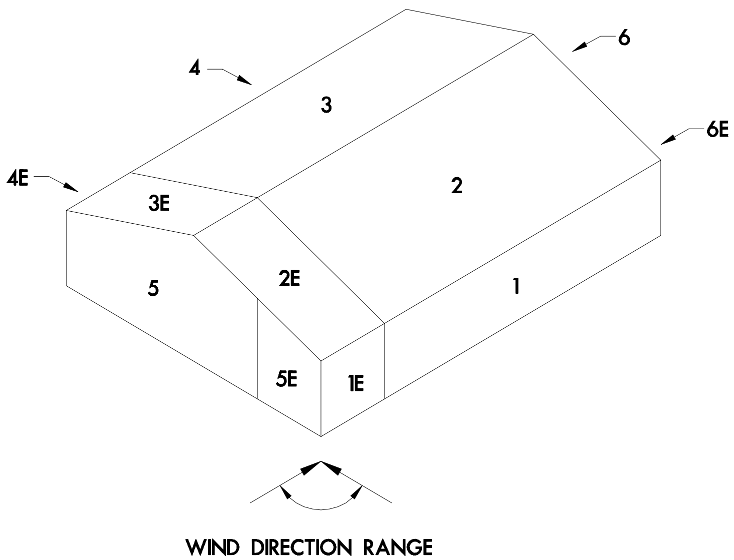
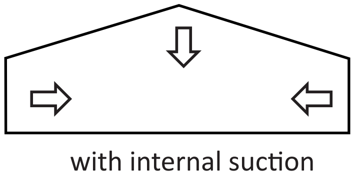
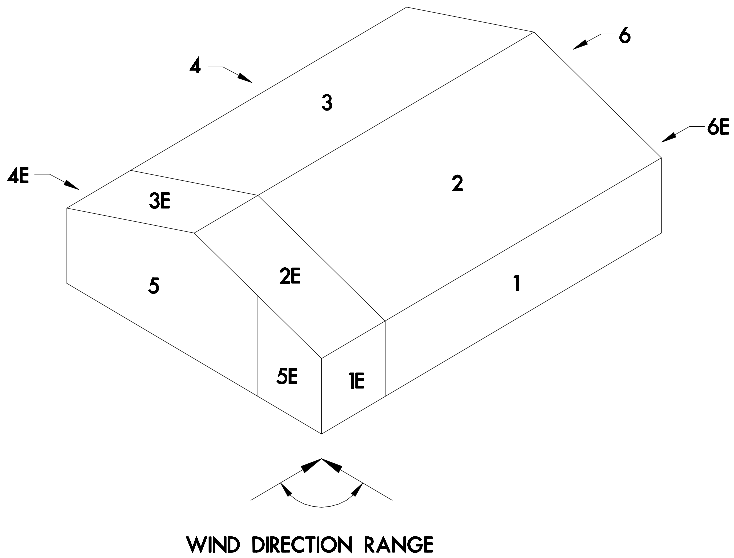


Table C1.3B ASCE 7-10 Exposure B Main Wind-Force Resisting System (MWFRS) Loads, p (psf), for an Enclosed Building, 33' Mean Roof Height with Internal Suction, 150 mph (700-yr. 3-second gust), Exposure B

q = 21.15 psf (See Table C1.1)



Roof Angle		Interior Zones 1	2	3	4	5	6	Exterior Zones 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



2

ENGINEERED DESIGN

AMERICAN WOOD COUNCIL

C2.1 General Provisions

C2.1.1 Engineered Requirements

Chapter 2 provides minimum loads for the purpose of establishing specific resistance requirements for buildings within the scope of this document. Note that Chapter 2 also contains some necessary construction details. These are duplicated from typical code provisions and are included in Chapter 2 for the convenience of the designer. The provisions of Chapter 2 are provided so that the designer can select acceptable materials or alternatives to the specific prescriptive solutions of Chapter 3.

C2.1.3 Engineered Design Limitations

Limits set forth in this Manual attempt to define boundary conditions typically encountered in residential construction. Where these limits are exceeded, accepted engineering practice in accordance with the local authority having jurisdiction, shall be used.

C2.1.3.1 Adjustment for Wind Exposure and Mean Roof Height Chapter 2 provisions in the *2012 WFCM* use a default wind Exposure Category B and assume a 33 ft mean roof height (MRH). Adjustment factors as shown in Table 2.1.3.1 *Adjustment for Wind Exposure and Mean Roof Height* are provided to adjust tabular values in Chapter 2 for buildings located in wind Exposure C or D and for mean roof heights less than 33 feet. Tabular wind requirements in Chapter 3 of the *2012 WFCM* are provided for both wind Exposure Categories B and C.

C2.1.3.2 Floor Systems

C2.1.3.2a Framing Member Spans Framing member spans are limited to 26 feet for floors based on the bending capacity of the double top plate supporting floor framing members. The worst case assumption is that a floor framing member bears directly between two studs creating a concentrated load at mid-span of the top plates. Section 2.1.3.3g requires bandjoists, blocking, or other methods to transfer roof, wall, and/or floor loads from upper stories to alleviate the concern of additional loads being transferred through the floor framing members into the top plate.

Framing member spans are based on L/360 for serviceability, however some product manufacturers recommend more conservative criteria than L/360 for floor live load deflection based on feedback from users related to floor performance in the field.

C2.1.3.2c Cantilevers Limiting floor cantilevers to the depth of the joist for lumber joists supporting a loadbearing non-shear wall is based on joist strength considerations.

Limiting floor cantilevers to L/4 for lumber joists supporting a non-loadbearing non-shear wall is based on deflection considerations.

The requirement to have floor framing members line up directly over studs in cantilever conditions supporting a loadbearing non-shear wall, avoids the case where a floor framing member bears between two studs creating a concentrated load at mid-span of the top plates. In this case the floor framing member would also be carrying roof, ceiling, wall, and potentially additional floor loads to the top plate which would exceed the bending capacity of the wall double top plates.

C2.1.3.2d Setbacks Limiting setbacks to the depth of the joist for lumber joists supporting a loadbearing wall is based on joist strength considerations. Similar to cantilevers, floor framing members shall line up over studs in setback conditions.

C2.1.3.2e Vertical Floor Offsets Prescriptively limiting vertical floor offsets to the depth of the floor framing member allows the offset to be connected with a shear connector as shown in Figure 2.1i. The floor discontinuity restriction keeps the vertical distance between adjacent diaphragms small enough to be reasonably connected. Larger vertical discontinuities would require special detailing for moment resisting elements at the connection, which is beyond the scope of this document. Floor discontinuities are not limited when the individual floor levels are considered as separate structures. In such cases, each separate floor diaphragm should be designed, detailed, and supported in accordance with all applicable provisions of the *WFCM*.

C2.1.3.2f Diaphragm Aspect Ratio Limiting the roof and floor diaphragm aspect ratio to 4:1, is based on maximum limitations for blocked horizontal diaphragms in the *Special Design Provisions for Wind and Seismic (SDPWS)* standard.

C2.1.3.2g Diaphragm Openings Diaphragm openings are limited to the lesser of 12 feet or 50% of the building dimension. Where larger openings are used, design of the diaphragm should be in accordance with accepted engineering practice.

C2.1.3.3 Wall Systems

C2.1.3.3a Wall Heights The 10 foot height limit is the maximum prescriptive span permitted in the model building codes for loadbearing walls.

Limiting non-loadbearing walls to 20 feet in height is a practical approach since typical lumber lengths do not exceed this limit.

C2.1.3.3c Shear Wall Line Offsets Where shear walls are offset less than 4 feet in plan view, the walls shall be assumed to act along the same shear wall line for purposes of collecting diaphragm lateral forces.

C2.1.3.3d Shear Wall Story Offsets Shear wall story offsets are not permitted unless supporting members and connections are specifically designed and detailed to provide a continuous load path for induced forces.

C2.1.3.3e Shear Wall Aspect Ratio Shear wall aspect ratio limitations vary by shear wall sheathing type and loading and are specified in *SDPWS*.

C2.1.3.3g Load Transfer The requirement for band-joists, blocking, or other methods to transfer roof, wall, and or floor loads from upper stories is based on considerations for compression perpendicular to grain and bending of top plates if loads from upper levels are carried into these members as concentrated loads through the floor framing members.

C2.1.3.4 Roof Systems

C2.1.3.4a Framing Spans Limiting lumber rafter spans (horizontal projection) to 26 feet is a practical limitation based on availability of those lengths for lumber. Limiting I-joist rafter spans to 26 feet is a carry-over from limits for floor I-joist span limitations (see Commentary 2.1.3.2a). Longer I-joists are manufactured.

C2.1.3.4c Overhang Lengths The rafter overhang length limitation of L/3 is based on moment capacity of the rafters due to notching for bearing at the top of the wall (birdsmouth notch). See Figure C2.5.1.4.

C2.1.3.4d Slope Roof slope is limited to 12:12 or less consistent with the limited scope of tabular requirements that are dependent on roof slope.

C2.1.3.4e Diaphragm Aspect Ratio See 2.1.3.2f.

C2.1.4 Interpolation

Tabulated values in Chapter 2 address a range of possible results using the engineered provisions. Unless otherwise noted, tabulated information is provided such that interpolation can be used to determine requirements for conditions not explicitly tabulated.

C2.1.5 Design Values

C2.1.5.1 Sawn Lumber Tabulated values in *WFCM* Chapter 2 Tables are based on properties required by engineering analysis. When selecting the material or product that will satisfy a given tabulated requirement, design values should be adjusted by all applicable adjustment factors per the *National Design Specification® (NDS®) for Wood Construction*. For example, if a rafter table requires a 1,400 psi bending design value to satisfy a given case, a minimum "adjusted" design value from the *2012 NDS* of 1,400 psi is required. The adjusted design value includes all size adjustments, repetitive member adjustments, and load duration adjustments. Thus, if checking a No. 2 Douglas Fir-Larch 2x6, with a tabulated "reference" bending design value of 900 psi, a size factor of 1.3, repetitive member factor of 1.15, and load duration factor of 1.25 (construction live load) applies. The "adjusted" design value for this example is equal to 1,682 psi (e.g. 900x1.3x1.15x1.25 = 1,682 psi) and satisfies the 1,400 psi requirement.

C2.2 Connections

This section provides capacity requirements for lateral, shear, uplift, overturning, and special connections.

C2.2.6.1 Ridge Connections Ridge straps must be installed to resist uplift/opening of the ridge. In cases where the ridge uplift loads are relatively small, collar ties, also called collar beams, can be installed in the upper third of the attic space to resist the ridge uplift/opening forces.

C2.3 Floor Systems

This section provides recommendations for three types of floor framing systems - lumber joists, prefabricated wood I-joists, and prefabricated wood trusses.

C2.3.1 Wood Joist Systems

Recommendations for lumber floor joists are consistent with standard practice and with common code requirements.

C2.3.2 Wood I-Joist Systems

The section on prefabricated wood I-joists reminds the user that product properties and installation recommendations are manufacturer-specific. Thus, while provisions common to all I-joists are included in this section, many specific sections refer the user to consult the manufacturer for installation or use instructions.

C2.3.3 Wood Floor Truss Systems

Prefabricated wood trusses are not only manufacturer-specific, but are also "job-specific." Prefabricated wood trusses are custom-manufactured to meet a unique set of design requirements. They must be used in the application for which they were designed, and must be installed per the truss installation instructions.

C2.4 Wall Systems

This section provides both prescriptive and performance-oriented requirements. Many of the prescriptive requirements (double top plate, minimum splice length, etc.) are intended to adequately tie together various portions of the structure – reflecting both common practice and basic engineering requirements. Performance requirements (stud properties, shear wall configurations, hold down requirements) are based on engineering considerations. Stud tables are based on component and cladding loads. For component and cladding wind pressures, the bending stresses are computed independent of axial stresses. In addition, the case in which bending stresses from main wind-force resisting system pressures act in combination with axial stresses from wind and gravity loads have been analyzed. For buildings limited to the conditions in this Manual, the component and cladding loads typically control the stud design.

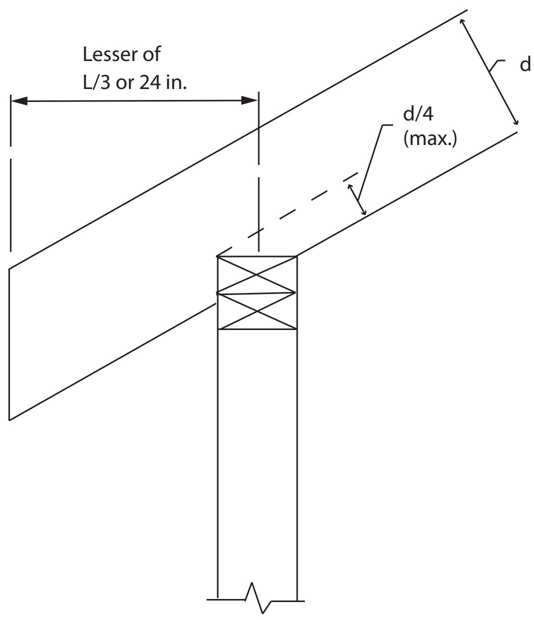
C2.5 Roof Systems

C2.5.1.4 Ridge Beams

This section provides guidance for two types of roof framing systems. The first system uses a ridge beam – defined as a structural member sufficient to carry rafter loads at the ridge, and supported in a manner sufficient to carry loads to the foundation. The ridge beam system is permitted in all uses that are within the scope of this Manual. The ridge beam framing system eliminates the horizontal thrust component acting on bearing walls.

The second system uses a ridge board – defined as a nonstructural member that merely provides a common fastening point for rafters from both sides of the ridge. The ridge board framing system relies on the inherently stable triangle, formed by the rafters and ceiling joist to resist the thrust component acting on the bearing walls. Special attention to fastening details is required. The ridge board system is not permitted for products other than lumber rafter systems. Due to the increase in thrust forces at lower slopes, the Manual limits the use of this system to slopes of 3 in 12 or greater. This system is not permitted for use with prefabricated wood I-joists. For sawn lumber roof rafters that utilize a birdsmouth notch as depicted in Figure C2.5.1.4, code provisions typically limit the depth of notch to 25% the depth of the solid sawn rafter where the depth of the cut is measured perpendicular to the length dimension of the rafter. For engineered wood products, the user should refer to the manufacturer's recommendations.

Figure C2.5.1.4 Birdsmouth Notch Limitations



C2.5.1.6 Ceiling Joists

When using a ridge board system, significant thrust loads are developed at the top of the walls supporting the rafters. Ceiling joists are typically used to resist this thrust force. When ceiling joists are not present, or when they run normal to the rafter direction, rafter ties must be installed in the lower third of the attic space to resist the thrust, or a ridge beam must be installed to resist the vertical loads at the ridge.

List of Tables

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 ft^2. The minimum required area for analysis is $h^2/3=33.3$ ft^2. 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 $A \leq 10$ ft^2

$GC_p = -0.8 - 0.6[(\log(A/500)) / (\log(10/500))]$ for $10 < A \leq 500$ ft^2

$GC_p = -0.8$ for $A > 500$ ft^2

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 ft^2.

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.

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)

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) / (118plf)
= 0.75

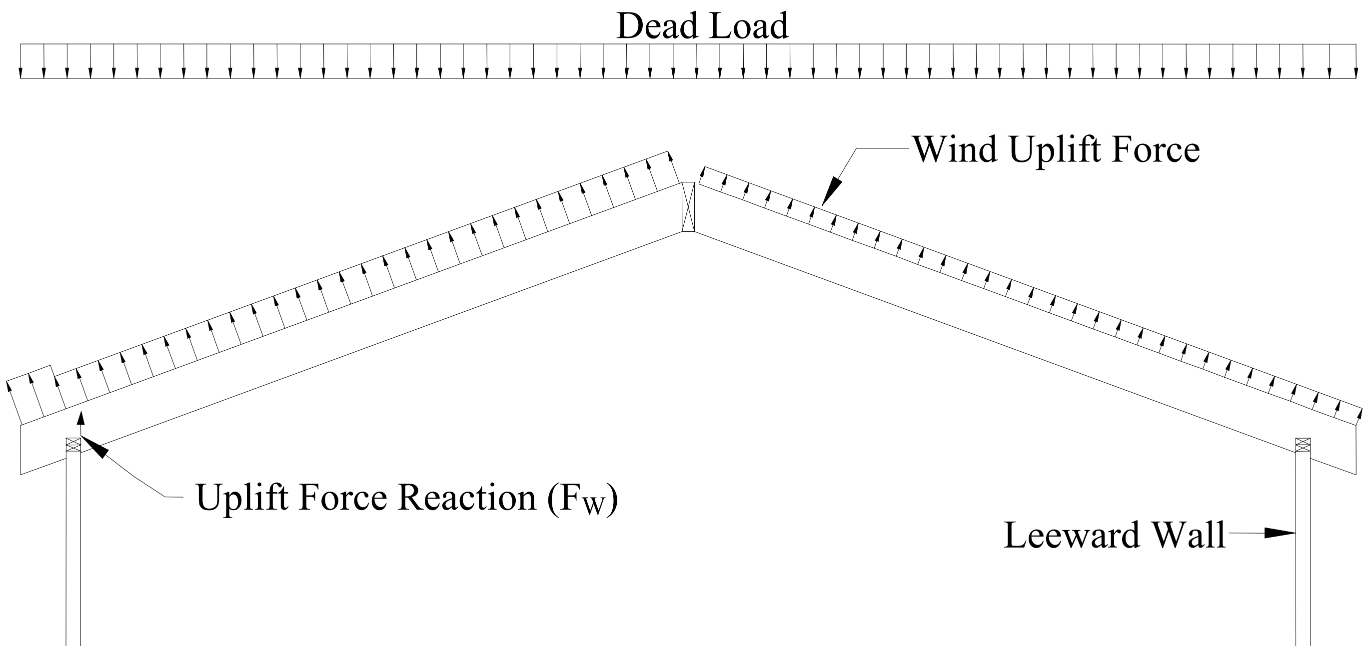
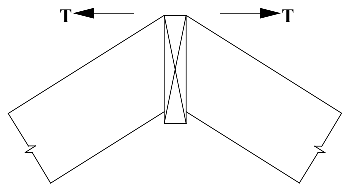


Table 2.2B Ridge Connection Loads From Wind

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:

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

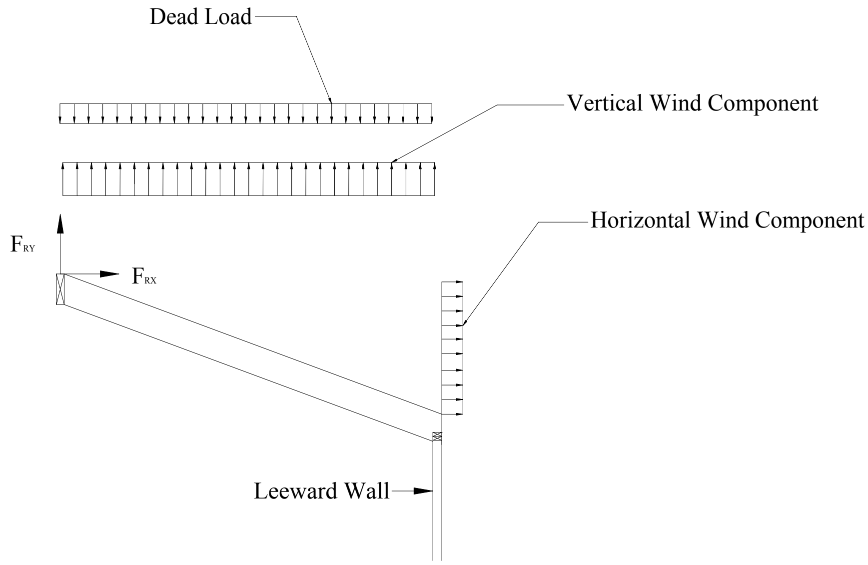
Determining horizontal wind force at ridge:

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



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:

$$T = 336 \text{ plf} \qquad \text{(WFCM Table 2.2B)}$$

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

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

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.

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:

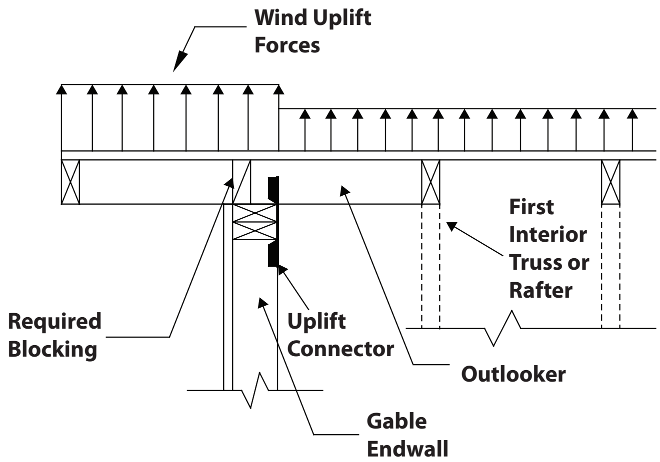
Tabulated loads may be reduced when they occur in Zone 2 per the Figure of Table 2.4. The required capacity for a connector in Zone 2 with Zone 2 overhang wind loads for the overhanging section is:

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

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

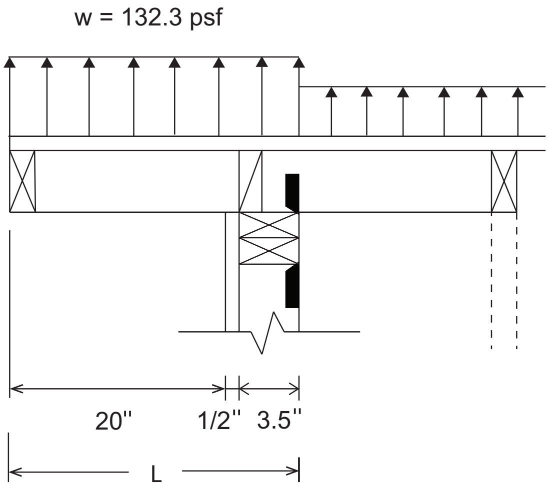
$$U_{zone\ 2}/U_{zone\ 3} = (419\text{ lbs})/(697\text{ lbs})$$

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



Footnote 4:

For the footnoted cases, outlooker overhangs shall be limited to 20 inches based on the bending capacity of the outlooker.



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

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

$$M_{Req.} = (wL^2)/2$$
$$= (132.3/12)(24^2)/2$$
$$= 3{,}175\text{ in.-lbs/ft}$$

where:

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

L = 20 in.(overhang) + 1/2 in.(sheathing thickness) + 3.5 in.(width of stud)
= 24 in.

For outlookers spaced 24" o.c.

$M_{Req.}$ = (3,175 in.-lbs/ft)(2 ft)
6,350 in.-lbs

$F_b' S_x$ for a No. 2, 2x4 Hem Fir Outlooker is:

$F_b' S_x = F_b C_D C_F C_r S_x$
= (850 psi)(1.6)(1.5)(1.15)(3.0625)
= 7,185 in.-lbs

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

6,350 in.-lbs < 7,185 in.-lbs

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

Table 2.3 Thrust Connection Loads

Description: Thrust connection loads are " tie forces" that develop at the heel joint connection between rafters and ceiling joists.

Procedure: Under live (or snow) plus dead load, sum moments about one heel joint to determine forces at the opposite joint. Using forces from the opposite heal joint, sum moments about the ridge joint to calculate thrust at the heel joint.

Background: Note that connection of a rafter to a ceiling joist is generally a single shear connection. For cases in which the forces are very high, the eccentricity of the load at this connection can be substantial and should be considered in the design. Overhangs are ignored in the connection calculation as they reduce the amount of thrust.

Example:

Given - 4:12 slope, 36' roof span, 16" o.c., 10 psf roof dead load, 30 psf ground snow load. *ASCE 7-10* is used to convert ground snow load to roof snow load.

Roof Dead Load = 10 psf

Roof Snow Load $= 0.7(p_g)(C_t)$ (*ASCE 7-10*)

$= 0.7(30)(1.1)$

$= 23.1$ psf

Total Load = Dead Load + Snow Load

= 33.1 psf

Maximum loads into heel joint connections result from balanced snow and dead loads. The thrust connection loads equal the forces that develop at the heel joints.

Summing moments about the right heel joint:

$$\Sigma M_{right} = 0$$

$$(10 + 23.1)(36)(18) - 36R_{Left} = 0$$

$$R_{Left} = 596 \text{ plf}$$

Given the reaction at the left wall R_{Left}, tensile forces can be determined by cutting a section through the ridge and summing moments about the ridge.

Summing moments about the ridge:

$$\Sigma M_{ridge} = 0$$

$$18R_{Left} - [(10 + 23.1)(18)(9)] - 6T = 0$$

Solving for the tensile heel joint force (T) per the figure of Table 2.3:

$$T = [18(596) - (33.1(18)(9))] / 6$$

$$= 894 \text{ plf} \quad \text{(WFCM Table 2.3)}$$

Required capacity of thrust connections spaced at 16" o.c.:

$$= T(16 \text{ in.}/12 \text{ in./ft})$$

$$= 1{,}192 \text{ lbs} = T(1.33) \quad \text{(WFCM Table 2.3 Footnote 3)}$$

Footnote 5:

When rafter ties are located above the top plate, additional thrust forces will occur. The increase in loads is a function of the thrust connector location and may be quantified using a ratio of moment arms (see Table 2.3 Figure).

$$T_{req}(H_R - H_C) = T_{tabulated}(H_R)$$

The required thrust connector force is:

$$T_{req} = T_{tabulated}(H_R) / (H_R - H_C)$$

Dividing through by H_R yields:

$$T_{req} = T_{tabulated} / [1 - (H_C/H_R)] \quad \text{(WFCM Table 2.3 Footnote 5)}$$

Note:

See Commentary for Table 2.14A for calculation of heel joint deflections when rafter ties are located above the top plates.

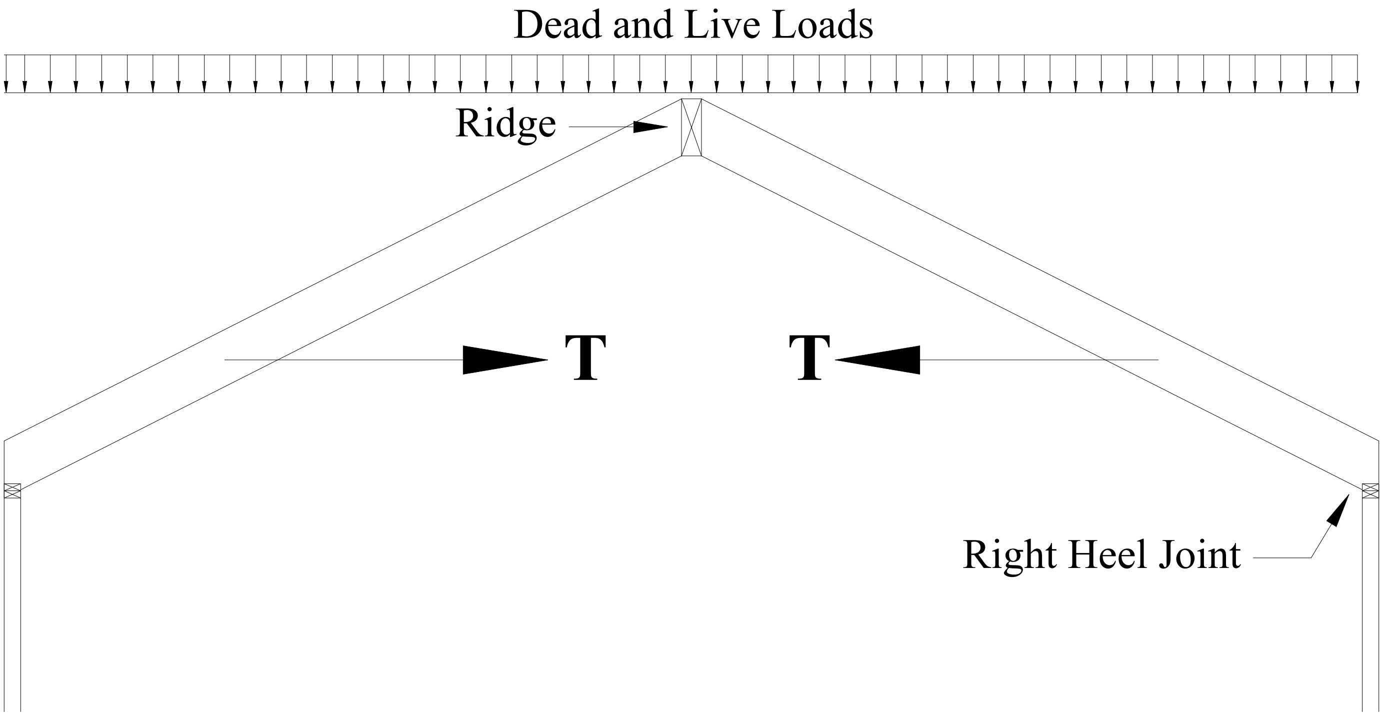

Dead and Live Loads
Ridge
T
T
Right Heel Joint

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)

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

(Roof and Floor Shear)

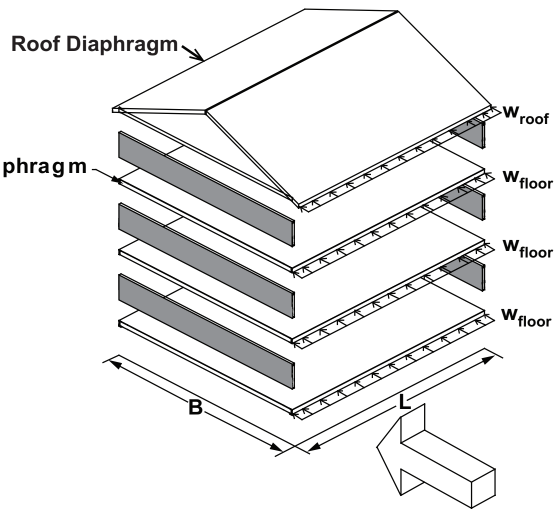
Description: For calculating lateral loads in roof and floor diaphragms from wind forces acting perpendicular to the roof ridge.

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):

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

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:

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

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)

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:

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

Table 2.5B Lateral Diaphragm Loads from Wind – Parallel to Ridge

(Roof and Floor Shear)

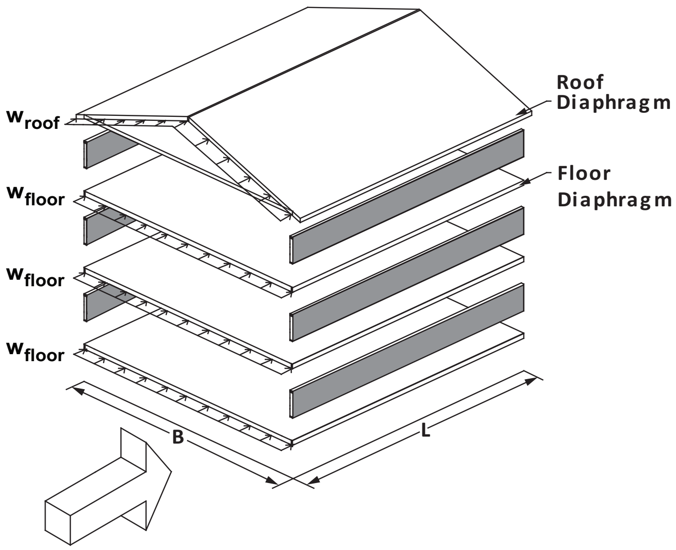
Description: For calculating lateral loads in roof and floor diaphragms from wind forces acting parallel to the roof ridge.

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:

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

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

2

ENGINEERED DESIGN

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

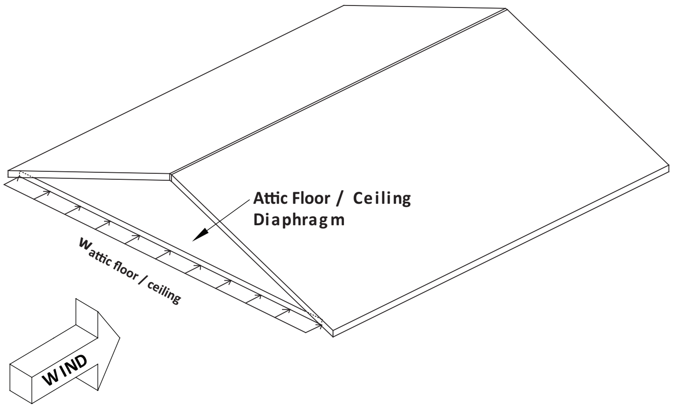
Description: For calculating lateral loads in attic floor or ceiling diaphragms when bracing gable end walls.

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).

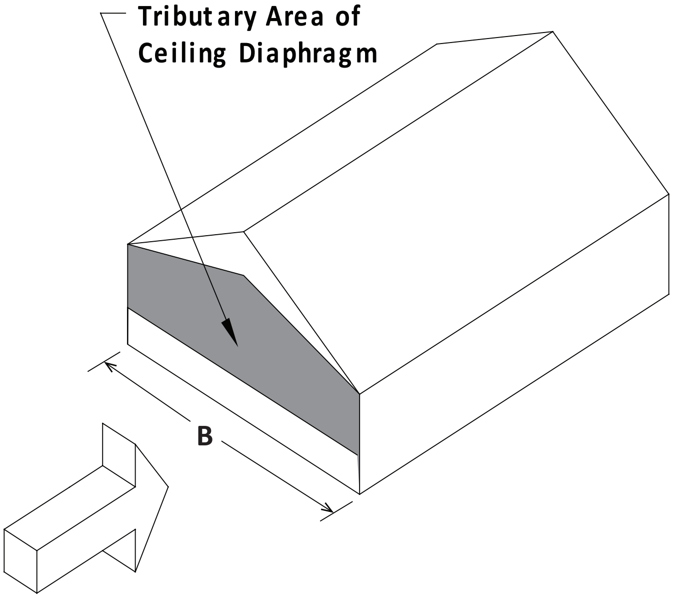


Calculate the wall pressure:

$$p = 15.53 \text{ psf} \quad \text{(See Commentary for Table 2.5B)}$$

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:

	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.

Table 2.6 Lateral Loads from Seismic

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

Description: Calculation of lateral loads from seismic for diaphragms, shear walls, and connections.

Procedure: Lateral loads (lbs) from seismic are based on the assumption of a rectangular building with shear forces determined in accordance with the Simplified procedures of *ASCE 7-10* Section 12.14.8. The vertical distribution of seismic forces is assumed to be proportional to the seismic weight at each level modified by factors of 1.0, 1.1, or 1.2 for one, two, or three-story buildings, respectively.

Background: Table 2.6 describes the method for determination of lateral loads from seismic for calculation of diaphragm unit shear capacity requirements, shear capacity requirements for shear walls, and connections.

Example:

Given – Two-story building above grade plane with rectangular dimensions W and L.

Calculate the effective seismic weight at each level.

Effective seismic weight at the roof/ceiling diaphragm level, W_{RD}:

$$W_{RD} = W_{roof} + W_{(wall\ W)}/2 + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}$$

where:

W_{roof} = weight of the roof

$W_{(wall\ W)}$ = weight of exterior walls in W dimension

$W_{(wall\ L)}$ = weight of exterior walls in L dimension

$W_{partition}$ = weight of partition walls

W_{gable} = weight of gable end walls

Effective seismic weight at the second floor diaphragm level, W_{FD2}:

$$W_{FD2} = W_{floor\ 2} + W_{(wall\ W)} + W_{(wall\ L)} + W_{partition}$$

where:

$W_{floor\ 2}$ = weight of the floor at level 2

Effective seismic weight at the first floor diaphragm level, W_{FD1}:

$$W_{FD1} = W_{floor\ 1} + W_{(wall\ W)} + W_{(wall\ L)} + W_{partition}$$

where:

$W_{floor\ 1}$ = weight of the floor at level 1.

Calculate the shear at each level:

Shear at the roof diaphragm level, V_{RD}:

$$V_{RD} = 1.1 \times W_{RD} \times S_{DS}/R$$

where:

1.1 = vertical force distribution factor for two-story construction in accordance with simplified procedures of *ASCE 7-10* Section 12.14.8. A factor of 1.0 applies for one-story construction and a factor of 1.2 applies for three-story construction.

S_{DS} = design, 5 percent damped, spectral response acceleration parameter at short periods in accordance with *ASCE 7-10*

R = seismic response modification coefficient in accordance with *ASCE 7-10*

Shear at the second floor diaphragm level, V_{FD2}:

$$V_{FD2} = 1.1 \times W_{FD2} \times S_{DS}/R$$

Shear at the first floor diaphragm level, V_{FD1}:

$$V_{FD1} = 1.1 \times W_{FD1} \times S_{DS}/R$$

Upper Table

Footnote 1:

The presentation of general equations for effective seismic weight at each level and shear at each level are based on the assumption of a rectangular building with dimensions W and L. The weight of walls to their mid-height that are located directly above and below the diaphragm level under consideration is assumed to be tributary to the diaphragm at that level in addition to the diaphragm weight itself.

Footnote 2:

In accordance with *ASCE 7-10*, 20% of the ground snow load is included in effective seismic weight where the ground snow load exceeds 30 psf. For cases where ground snow load is 50 psf, effective seismic weight at the roof level includes roof dead load plus 10 psf (i.e. 0.20 x 50 psf = 10 psf). Similarly, for a ground snow load of 70 psf, effective seismic weight at the roof includes roof dead load plus 14 psf (i.e. 0.20 x 70 psf = 14 psf).

Lower Table

Footnote 1:

Diaphragm loads are calculated assuming all effective seismic weight associated exterior walls parallel to direction of loading are not contributing to diaphragm loading. These walls are assumed to resist their own seismic forces and transfer of these forces is assumed to be by direct connections to shear resisting elements below and not through diaphragm action. Exterior walls perpendicular to the direction of loading contribute directly to diaphragm loading. The diaphragm unit shear capacity requirements for load direction parallel to building dimension W is calculated as follows and excludes weight of exterior walls parallel to building dimension W in accordance with footnote 1:

Diaphragm unit shear capacity requirements in the roof/ceiling diaphragm, $v_{roof/ceiling}$:

$$v_{roof/ceiling} = V_{RD}/(2W) = (W_{roof} + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}) \times (1.1 \times S_{DS} / R) / (2W)$$

where:

2W = the length of the diaphragm parallel to the direction of loading. The diaphragm is assumed to have the same dimensions as the rectangular building with dimensions W and L. For loading parallel to building dimension W, the total diaphragm length providing resistance is 2W (i.e. diaphragm length equal to W at each end of the single span diaphragm).

Diaphragm unit shear capacity requirements in second floor diaphragm level, v_{floor}:

$$v_{floor} = V_{FD2}/(2W) = (W_{floor2} + W_{(wall\ L)} + W_{partition} + W_{gable}) \times (1.1 \times S_{DS} / R) / (2W)$$

Diaphragm unit shear capacity requirements in first floor diaphragm level, v_{floor}:

$$v_{floor} = V_{FD1}/(2W) = (W_{floor1} + W_{(wall\ L)} + W_{partition} + W_{gable}) \times (1.1 \times S_{DS} / R) / (2W)$$

Footnote 2:

Seismic loads for shear walls and connections transferring shear forces are based on the accumulated shear at each level. Shear capacity requirements associated with the first floor (FD1) diaphragm level are applicable for the case where the first floor diaphragm is supported directly on foundation walls, such as in a crawl space or basement foundation where the top of the foundation is the seismic base. Total shear at the first floor (FD1) level includes the contribution of first floor weight to base shear connection requirements.

Tables 2.7A&B Floor Joist Spans for 30 psf Live Load (Habitable Attics and Sleeping Areas) and 40 psf Live Load (Living Areas)

(Example shown is for Table 2.7A, Habitable Attics and Sleeping Areas.)

Description: Calculation of maximum permissible spans for lumber floor joists.

Procedure: Perform span calculation based on L/360 deflection limit. Based on the L/360 maximum span, determine the allowable bending stress (f_b).

Background: Tables 2.7A and B are applicable for simple spans and a continuous span on three supports for uniform load distributions.

Procedure for using this table: First establish the configuration (dead load magnitude, joist spacing, joist size). Next, select the E of the joist for the desired span. If the F_b' value for the joist meets or exceeds the tabulated f_b for that row, then the joist meets the requirements of this table. If the F_b' value for the joist is lower than f_b, then F_b' controls the design, move up in the table to the correct f_b - the maximum span is listed in that row.

Example:

Given - 2x10 floor joist, 16" o.c., E = 1.6 million psi, 30 psf live load, 10 psf dead load, $\Delta_{LL} \leq$ L/360

w_{DL} = 10 psf(16 in./12) = 13.33 plf

w_{LL} = 30 psf(16 in./12) = 40.00 plf

w_{TL} = 13.33 + 40 = 53.33 plf

Calculate the deflection-limited span:

$$\Delta = \frac{L}{360} \leq \frac{5 w_{live} L^4}{384 EI}$$

$$L = \sqrt[4]{\frac{384 EI\Delta}{5 w_{live}}}$$

$$= \sqrt[3]{\frac{(384)(1.6\times10^6)(98.3)}{(5)(360)(40/12)}}$$

$$= 216 \text{ in.}$$

$$= 18 \text{ ft } 0 \text{ in.}$$

L = 18 ft - 0 in. (WFCM Table 2.7A)

Calculate the bending stress, f_b:

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

$$= \frac{(53.33/12)(216.3)^2}{8(21.4)}$$

$$= 1,216 \text{ psi}$$

$f_{b(Tabulated)}$ = 1,216 psi (WFCM Table 2.7A)

Table 2.7C Floor Joist Bearing Stresses for Floor Loads

Description: Calculation of compression perpendicular to grain stresses at joist bearing.

Procedure: Compute load and divide by bearing area.

Background: None.

Example:

Given - 16' floor joist span, 16" o.c. joist spacing, 10 psf dead load, and 40 psf live load.

Calculate total load on the floor joist:

$$L = 16 \text{ ft}$$

$$w = (40 \text{ psf} + 10 \text{ psf})(16 \text{ in.}/12\text{in.}/\text{ft}) = 66.67 \text{ plf}$$

Calculate bearing area:

$$A = 1.5 \text{ in.} \times 1.5 \text{ in.} = 2.25 \text{ in.}^2$$

Calculate the induced compression perpendicular to grain (bearing) stress assuming bearing at each end of a single span joist:

$$f_{c\perp} = [wL] / [2A]$$
$$= [66.67 \text{ plf } (16\text{ft})] / [2(2.25 \text{ in.}^2)]$$
$$237 \text{ psi} \qquad \text{(WFCM Table 2.7C)}$$

Footnote 1:

For continuous spans, the bearing stress shall be multiplied by the following factor to account for increased bearing stress at the interior supports:

$$f_{c\perp} \text{ (continuous span)} = 1.25wL / A$$

$$f_{c\perp} \text{ (single span)} = 0.5wL / A$$

$$\text{Adjustment Factor} = \frac{1.25wL/A}{0.5wL/A} = 2.5$$

Tables 2.8A&B Floor Framing Capacity Requirements for 30 psf Live Load (Habitable Attics and Sleeping Areas) and 40 psf Live Load (Living Areas)

(Example shown is for Table 2.8A, Habitable Attics and Sleeping Areas.)

Description: Calculation of joist properties required under a given span and load.

Procedure: Input spans, solve for Apparent Rigidity, Moment, and Bearing.

Background: Same as for Tables 2.7 A&B.

Note for I-joists:

- $EI_{App.}$ = apparent rigidity (including shear deflection and composite action)
- Minimum bearing length and web stiffener requirements must be satisfied

Example:

Given - 16' span, 24" o.c. floor framing spacing, 30 psf live load, 10 psf dead load, $\Delta_{LL} \leq /360$

w_{dead} = 10 psf(24 in./12) = 20 plf

w_{live} = 30 psf(24 in./12) = 60 plf

w_{total} = 20 + 60 = 80 plf

Calculate the apparent rigidity:

$$\Delta = \frac{L}{360} \leq \frac{5 w_{live} L^4}{384 EI}$$

$$EI_{App.} = \frac{5(360)\, w_{live} L^3}{384}$$

$$= \frac{5(60/12)(360)(16(12))^3}{384}$$

$$= 165.9 \text{ x } 10^6 \text{ in.}^2\text{-lbs}$$

$EI_{App.}$ = 165.9 x 10^6 in.^{2}-lbs (WFCM Table 2.8A)

Calculate the moment:

$$M = \frac{w_{total} L^2}{8} = \frac{80(16^2)}{8}$$
$$= 2,560 \text{ ft-lbs}$$

M = 2,560 ft-lbs (WFCM Table 2.8A)

Calculate the bearing reaction:

$$R = \frac{w_{total} L}{2} = \frac{80(16)}{2}$$
$$= 640 \text{ lbs}$$

R = 640 lbs (WFCM Table 2.8A)

Footnote 2:

For a continuous span with load only between two supports $EI_{App.}$ is:

$$EI_{App.} = \frac{wL^4}{109\Delta}$$

For a single span, $EI_{App.}$ is:

$$EI_{App.} = \frac{5wL^4}{384\Delta}$$

Taking the ratio of tabulated continuous span $EI_{App.}$ to the single span $EI_{App.}$:

= 384 / [109(5)]

= 0.71 conservatively use 0.75

= 0.75 (WFCM Table 2.8 A&B)

Footnote 3:

See Commentary to Table 2.7C, footnote 1.

Footnote 4:

At interior supports, shear requirements are ½ of the bearing requirements given in footnote 3.

= 2.5/2

= 1.25

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:

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

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

Footnote 2:

See Commentary Table 2.1 for calculation of footnotes.

2

ENGINEERED DESIGN

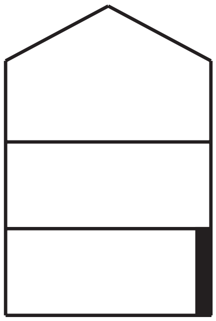
Table 2.9B Exterior Wall Stud Compression Stresses

Description: Direct compression stress in wall studs under live load.

Procedure: Sum gravity loads and calculate stud requirement.

Background: See Commentary for Table 2.11.

Example:



Given - loadbearing wall supporting Roof, Ceiling, & 2 Clear Span Floors, 36' building width, 2' overhangs, 2x6 studs, 12" o.c. stud spacing, 121 plf wall dead load, 20 psf roof dead load, 40 psf floor live load, and 50 psf ground snow load.

Calculate the compression load:

$$P = w_{total}(12\ \text{in.}/12)$$

$$w_{header} = 2{,}665\ \text{plf} \quad \text{(from Table 2.11)}$$

$$w_{wall} = 11\ \text{psf}\ (11\ \text{ft}) = 121\ \text{plf}$$

$$w_{total} = 2{,}665\ \text{plf} + 121\ \text{plf} = 2{,}786\ \text{plf}$$

$$P = 2{,}786\ (12\ \text{in.}/12\text{in.}/\text{ft}) = 2{,}786\ \text{lbs}$$

Calculate compression stress:

$$f_c = P / A$$

$$= 2{,}786 / 8.25$$

$$= 338\ \text{psi} \qquad \text{(WFCM Table 2.9B)}$$

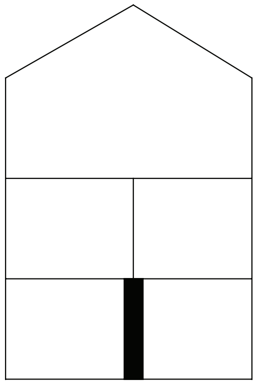
Table 2.9C Interior Loadbearing Wall Stud Compression Stresses from Live Loads

Description: Direct compression stress in wall studs under floor live and dead loads.

Procedure: Sum gravity loads and calculate stud requirements.

Background: See Commentary for Table 2.11.

Example:



Given – interior loadbearing wall supporting 2 Floors, 36' building width, 2x6 studs, 12" o.c. stud spacing, and 121 plf wall dead load.

Calculate the compression load:

$$P = w_{total}\,(12\text{ in.}/12)$$

$$w_{header} = 1{,}921\text{ plf} \quad \text{(from Table 2.11)}$$

$$w_{wall} = 11\text{ psf}\,(11\text{ ft}) = 121\text{ plf}$$

$$w_{total} = 1{,}921\text{ plf} + 121\text{ plf} = 2{,}042\text{ plf}$$

$$P = 2{,}042\,(12\text{ in.}/12\text{in.}/\text{ft}) = 2{,}042\text{ lbs}$$

Calculate compression stress:

$$f_c = P/A = 2{,}042\ /\ 8.25 = 248\text{ psi} \qquad \text{(WFCM Table 2.9C)}$$

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})

Table 2.11 Loadbearing Wall Loads From Snow or Live Loads

(For Wall Studs, Headers, and Girders)

Description: Gravity loads on walls, headers, and girders for 1-3 story building configurations.

Procedure: Sum gravity loads and calculate wall and header/girder requirements.

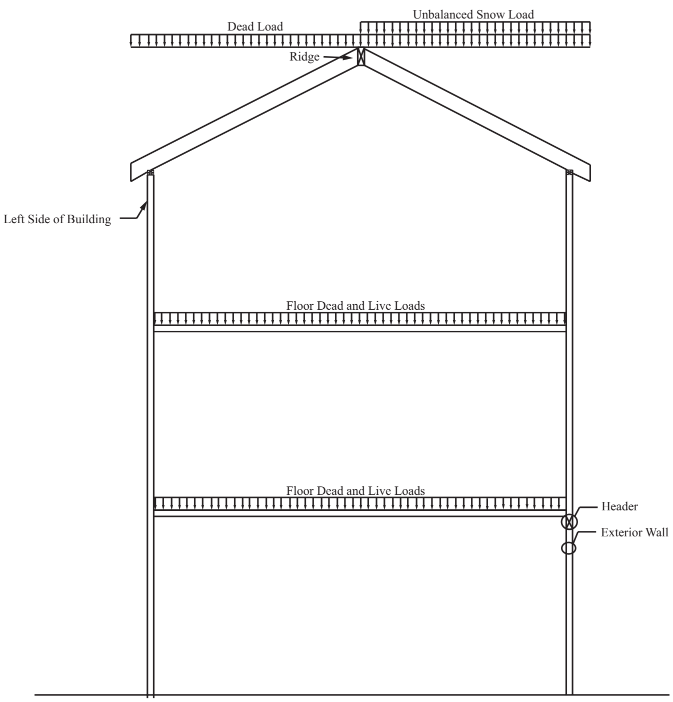
Background: In calculating the unit header/girder beam loads for each building configuration in Table 2.11, the following *ASCE 7-10* load combinations were considered. When designing structural wood members, it is also necessary to consider the effect of load duration as follows:

ASCE 7-10 ASD Load Combinations		Governing Load Duration, C_D to be Applied in Designing Structural Wood Members
#1	Dead + Roof Live	1.25
#2	Dead + Floor Live	1.00
#3	Dead + Snow	1.15
#4	Dead + 0.75 Floor Live + 0.75 Roof Live	1.25
#5	Dead + 0.75 Floor Live + 0.75 Snow	1.15

As a result of the 0.75 factor used in load combinations #4 and #5 (above), several of the multi-story cases in Table 2.11 were controlled by the Dead Load + Floor Live Load case as shown in the example below:

Example:

Given - Loadbearing wall supporting Roof, Ceiling, & 2 Clear Span Floors, 36' building width, 2' overhangs, 2x6 studs, 12" o.c., 20 psf roof dead load, 30 psf ground snow load, 121 plf wall dead load, and a 40 psf floor live load.



LOAD COMBINATIONS:

Per *ASCE 7-10*, the following load combinations were considered for this example (snow load):

1. Dead + Floor Live
2. Dead + Snow
3. Dead + 0.75 Floor Live + 0.75 Snow

LOADS:

Dead Loads:

$w_{roof\ dead}$ = 20 psf [(36 ft/2) + 2ft] = 400 plf

$w_{walldead}$ = 11 psf (11 ft)(2 walls) = 242 plf

$w_{floordead}$ = 10 psf (36 ft/2) (2 floors) = 360 plf

TOTAL = 1,002 plf

Snow Loads:

Balanced Snow Load:

$$q_{snow} = 0.7C_eC_tIp_g = 0.7(1.0)(1.1)(1.0)(30\text{ psf}) = 23.1\text{ psf}$$

$$R_{right}(\text{Snow}) = 23.1\text{ psf }(36\text{ft} + 2\text{ft} + 2\text{ft})/2 = 462\text{ plf}$$

Unbalanced Snow Load:

$$q_{snow} = Ip_g\ (\text{building width} < 40\text{ ft}) = (1.0)(30\text{ psf}) = 30.0\text{ psf}$$

For building widths greater than 40 ft (i.e. 60 ft widths in Table 2.11), a more complex unbalanced snow loading is required per *ASCE 7-10* which places 30 percent of the balanced snow load on the windward side of the roof and 100 percent of the balanced snow load plus a rectangular surcharge snow load on the leeward side of the roof. This surcharge load spans horizontally from the ridge to a distance equal to $8/3h_dS^{(1/2)}$ where h_d is the roof step drift height and S is the slope expressed as the roof run for a rise of one (i.e. for a 6 on 12 slope, S = 12/6 = 2). The magnitude of this surcharge snow load is equal to $h_d\gamma/S^{(1/2)}$ where γ is the unit weight of snow, which is a function of the ground snow load, $\gamma = 0.13\ (p_g) + 14$ (pcf). The slope, S, assumed in calculating the unit header loads for building widths equal to 60 ft in Table 2.11 was taken to conservatively maximize the resulting unbalanced snow load on the header for each building configuration.

Sum moments about the top of the wall opposite the unbalanced roof snow load.

$$\sum M_{left} = 0$$

$$\sum M_{left} = [(30.0\text{ psf})(28)][(36\text{ ft}/2) + 2] - 36R_{Right}$$

R_{right}(Snow) = 467 plf   Unbalanced Case Governs

In accordance with *ASCE 7-10,* balanced and unbalanced snow loads are checked and the larger used in the calculation. Unbalanced snow loads are 1.3 (30.0/23.1) times larger than balanced snow loads, but only act along one half of a dual pitched roof for rafters 20' or less in span (horizontal projected length). Therefore, for forces along the exterior, unbalanced snow loads result in the maximum force to the wall.

Floor Live Loads:

$$w_{live} = 40\text{ psf }(36\text{ ft}/2)(2\text{ floors}) = 1{,}440\text{ plf}$$

Summarizing the loads:

$$w_{dead} = 1{,}002\text{ plf}$$

$$w_{floorlive} = 1{,}440\text{ plf}$$

$$w_{snow} = 467\text{ plf}$$

Evaluating the load combinations:

Dead + Floor Live

$$= 1{,}002 + 1{,}440$$
$$= 2{,}442\text{ plf}$$

Dead + Snow

$$= 1{,}002 + 467$$
$$= 1{,}469\text{ plf}$$

Dead + 0.75 Floor Live + 0.75 Snow

$$= 1{,}002 + 0.75(1{,}440) + 0.75(467)$$
$$= 2{,}432\text{ plf}$$

$$w_{total} = 2{,}442\text{ plf}\quad\text{(WFCM Table 2.11)}$$

Tabulated values in Table 2.11 denoted with a "*" are intended to make the designer aware that these are governed by the Dead Load plus Floor Live Load combination. The designer should therefore apply the appropriate load duration factor, C_D, of 1.0 when using these loads to design a wood header.

Tables 2.12A&B Ceiling Joist Spans for 10 psf Live Load (Uninhabitable Attics Without Storage) and 20 psf Live Load (Uninhabitable Attics With Storage)

(Example shown is for Table 2.12A, Uninhabitable Attics Without Storage)

Description: Calculation of maximum permissible spans for lumber ceiling joist.

Procedure: Perform span calculation for a given set of E and f_b properties.

Background: Based on simple bending calculation for the ceiling joist assuming lateral support of the compression edge.

Example:

Given - 2x6 ceiling joist, 16" o.c. ceiling joist spacing, 10 psf live load, E = 1.4 million psi, 5 psf dead load, $\Delta_{LL} \leq$ L/240.

w_{dead} = 5 psf(16 in./12 in./ft) = 6.67 plf

w_{live} = 10 psf(16 in./12 in./ft) = 13.33 plf

w_{total} = 6.67 + 13.33 = 20 plf

Calculate the deflection-limited span:

$$\Delta = \frac{L}{240} \leq \frac{5w_{live}L^4}{384EI}$$

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{(384)(1.4\times10^6)(20.8)}{(5)(240)(13.33/12)}}$$

$$= 203 \text{ in.}$$

$$= 16 \text{ ft } 11 \text{ in.}$$

L = 16 ft 11 in. (WFCM Table 2.12A)

Calculate the bending stress, f_b:

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

$$= \frac{(20/12)(203)^2}{8(7.56)}$$

$$= 1{,}137 \text{ psi}$$

$f_{b(Tabulated)}$ = 1,137 psi (WFCM Table 2.12A)

Tables 2.13A&B Ceiling Framing Capacity Requirements for 10 psf Live Load (Uninhabitable Attics Without Storage) and 20 psf Live Load (Uninhabitable Attics With Storage)

(Example shown is for Table 2.13A, Uninhabitable Attics Without Storage)

Description: Calculation of properties required for a ceiling joist.

Procedure: Input spans, solve for Apparent Rigidity, Moment, and Bearing.

Background: Based on simple bending calculation for the ceiling joist assuming lateral support of the compression edge.

Example:

Given - 20' ceiling joist span, 16" o.c. ceiling joist spacing, 10 psf live load, 5 psf dead load, $\Delta_{LL} \leq$ L/240.

w_{dead} = 5 psf(16 in./12 in./ft) = 6.67 plf

w_{live} = 10 psf(16 in./12 in./ft) = 13.33 plf

w_{total} = 6.67 + 13.33 = 20 plf

Calculate the apparent rigidity:

$$EI_{App.} = \frac{5 w_{live} L^4}{384\Delta}$$

$$= \frac{5(240) w_{live} L^3}{384}$$

$$= \frac{5(13.33/12)(240)(16(12))^3}{384}$$

$$= 48.0 \ x \ 10^6 \text{in.}^2 - \text{lbs}$$

$EI_{App.}$ = 48,000,000 in.²-lbs (WFCM Table 2.13A)

Calculate the moment:

$$M = \frac{w_{total} L^2}{8} = \frac{20(20^2)}{8}$$

$$= 1,000 \text{ ft-lbs}$$

M = 1,000 ft-lbs (WFCM Table 2.13A)

Calculate the bearing reaction:

$$R = \frac{w_{total} L}{2} = \frac{20(20)}{2}$$

$$= 200 \text{ lbs}$$

R = 200 lbs (WFCM Table 2.13A)

Footnotes 2-4:

See Commentary for Tables 2.8A&B.

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, $\Delta_{LL} \leq L/180$.

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:

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

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

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

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

E = 1.239 x 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:

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

2 ENGINEERED DESIGN

Substituting the tabulated moment into the equation for maximum moment and solving for the ratio of actual span to tabulated span:

$$\frac{L}{L_{tabulated}} = \sqrt{\frac{1}{4\left(\frac{H_C}{H_R}\right)\left(2-\frac{H_C}{H_R}\right)}}$$

For:

$$\frac{H_C}{H_R} = \frac{1}{2}$$

$$\frac{L}{L_{tabulated}} = 0.58 \qquad \text{(WFCM Table 2.14A)}$$

Note: When ceiling joists or rafter ties are not located at the bottom of the attic space, horizontal deflection at the top of the bearing wall may occur. Applied loads are governed by the capacity of the roof rafter. Deflection of the wall is a function of the thrust connector location, vertical wall reaction, and roof load and may be derived using the following calculations:

$$\Delta_{total} = \frac{P'a'^3}{3EI} - \frac{w'_{LL}a'^4}{8EI}$$

$$P' = w_{LL}(L)\cos(\theta)$$

$$a' = L\left(\frac{H_C}{H_R}\right)/\cos(\theta)$$

$$w'_{LL} = w_{LL}(cos^2(\theta))$$

where:

θ = the angle of the roof slope

The total deflection, Δ_{total} can be expanded to:

$$\Delta_{total} = \frac{wL^3}{EI}\left[\frac{576(L)\left(\frac{H_C}{H_R}\right)^3}{cos^2\theta} - \frac{216(L)\left(\frac{H_C}{H_R}\right)^4}{cos^2\theta}\right]$$

The maximum rafter deflection calculated in Table 2.14A-D is given by the following equation:

$$\Delta_{Rafter} = \frac{5w_{LL}L^4}{384\,EI}$$

Assuming rafter deflections are limited to L/180, the maximum rafter deflection may be rearranged into the following form:

$$\frac{w_{LL}L^3}{EI} = \frac{384(12)}{180(5)(1728)}$$

Substituting the rafter deflection into the equation for total deflection, yields the following:

$$\Delta_{total} = \frac{1}{cos^2\theta}L[1.70\left(\frac{H_C}{H_R}\right)^3 - 0.64\left(\frac{H_C}{H_R}\right)^4]$$

Footnote 2:

ASCE 7-10 Section 4.8.2 provides roof live load reduction factors based on roof slope. For a 6:12 roof slope, the adjustment factor is:

Reduced Live Load (L_R)
= $w_{LL}(R_1)(R_2)$
= 20 psf(1.0)(0.9)
= 18 psf

where:

R_1 = 1.0 Conservative assumption based on worst case roof area

R_2 = 1.2-0.05F F is equal to the roof slope and is applicable for roof slopes >4:12 and <12:12

For the reduced live load, the critical span based on flexural capacity is: 19 ft-5in.

Dividing the adjusted span by the tabulated span:

(19.4 ft) / (18.75 ft) = 1.04 (WFCM Table 2.14A)

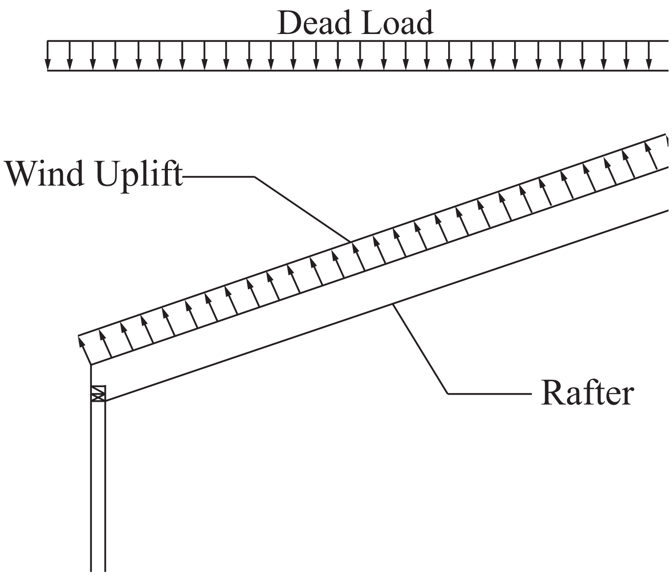
Total deflection is measured normal to the roof surface. To determine horizontal deflection at the top of the wall, the equation becomes:

$$\Delta_{HOR} = \frac{tan\theta}{cos\theta}L[1.70\left(\frac{H_C}{H_R}\right)^3 - 0.64\left(\frac{H_C}{H_R}\right)^4]$$

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 = 21.15psf (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} = 10psf(0.6)(16 in./12)
= 8 plf

Determining net uplift load:

= p - w_{DL}
= 98.0 plf – 8plf
= 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 in.

= 14.12 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

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:

$p = [-39.8 \text{ psf}] / [(\cos(6/12))(\cos(6/12))]$
$= -49.75$ psf

Adjusting for 16 in. rafter spacing:

$p = -49.75(16 \text{ in.}/12)$
$= -66.33$ plf

Accounting for dead load:

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

Determining net uplift load:

$= p - w_{DL}$
$= 66.33$ plf – 8plf
$= 58.33$ 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)

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.

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

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

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

$$= 211.10 \textit{ in.}$$

$$= 17.6 \textit{ ft}$$

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:

$$\text{Max. span} = 18.75 \text{ ft } (1.12)$$
$$= 21.0 \text{ ft}$$

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

Tables 2.14 B-D Rafter Spans for 30, 50, and 70 psf Ground Snow Loads

(Example shown for Table 2.14D, 70 psf Ground Snow Load)

Description: Calculation of maximum permissible spans for lumber rafters.

Procedure: Perform span calculation for a given set of E and f_b properties.

Background: See Table 2.14A Commentary.

Example:

Given - 2x8 rafter, 16" o.c. rafter spacing, f_b = 1,600 psi, 10 psf dead load, $\Delta_{LL} \leq L/180$.

w_{dead} = 10 psf (16 in./12 in./ft)
= 13.33 plf

w_{snow} = Ip_g (Note: Unbalanced snow load per *ASCE 7-10*)
= (1.0)(70 psf)(16 in./12 in./ft)
= 93.3 plf

w_{total} = 13.33 + 93.3 = 106.7 plf

Calculate the moment-limited span:

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

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

$$= \sqrt{\frac{(8)(13.14)(1,600)}{106.7/12}}$$

$$= 137.6 \text{ in.}$$

L = 11 ft 6 in. (WFCM Table 2.14D)

Calculate the modulus of elasticity:

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

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

$$= \frac{(5)(180)\left(\frac{93.3}{12}\right)(137.6)^3}{384(47.63)}$$

$$= 1.0\ x\ 10^6 \text{ psi}$$

E = 1.0 x 10^6psi (WFCM Table 2.14D)

Footnote 1:

See Commentary for Footnote 1 in Table 2.14A.

Table 2.15A Roof Framing Capacity Requirements for 20 psf Roof Live Load

Description: Calculation of rafter properties required for a given span and load.

Procedure: Input spans, solve for Apparent Rigidity, Moment, and Bearing.

Background: Rafter is assumed to be a simple bending member. Loads are based on a horizontal projection.

Example:

Given – 18' rafter span, 16" o.c. rafter spacing, 10 psf dead load, $\Delta_{LL} \leq L/180$.

w_{dead} = 10 psf(16 in./12) = 13.33 plf

w_{live} = 20 psf(16 in./12) = 26.67 plf

w_{total} = 13.33 + 26.67 = 40 plf

Calculate the apparent rigidity:

$$EI_{App.} = \frac{5\,w_{live}L^4}{384\Delta}$$

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

$$= \frac{(5)(180)\left(\frac{26.67}{12}\right)(18(12))^3}{384}$$

$$= 52.5\ x\ 10^6\,\text{in.}^2\text{-lbs}$$

$EI_{App.}$ = 52.5 x 10^6 in.^{2}–lbs (WFCM Table 2.15A)

Calculate the moment:

$$M = \frac{w_{total}L^2}{8} = \frac{40(18^2)}{8}$$

$$= 1{,}620\ \text{ft-lbs}$$

M = 1,620 ft-lbs (WFCM Table 2.15A)

Calculate the bearing reaction:

$$= \frac{w_{total}L}{2} = \frac{40(18)}{2}$$

360 lbs

R = 360 lbs (WFCM Table 2.15A)

Footnotes 2-4:
See Commentary for Tables 2.8A&B for calculations.

Footnote 5 and 6:
See Commentary for Table 2.14A for calculations.

Tables 2.15 B-D Roof Framing Capacity Requirements for 30, 50, and 70 psf Ground Snow Loads

(Example shown for Table 2.15D, 70 psf Ground Snow Load)

Description: Calculation of rafter properties required for a given span and load.

Procedure: Input spans, solve for Apparent Rigidity, Moment, and Bearing.

Background: Rafter is assumed to be a simple bending member. Loads are based on a horizontal projection.

Example:

Given – 18' rafter span, 16" o.c. rafter spacing, 10 psf dead load, $\Delta_{LL} \leq L/180$.

w_{dead} = 10 psf (16 in./12) = 13.33 plf

w_{snow} = Ip_g (Note: Unbalanced snow load per *ASCE 7-10*)

= (1.0)(70 psf)(16 in./12)

= 93.33 plf

w_{total} = 13.33 + 93.33 = 106.67 plf

Calculate the apparent rigidity:

$$EI_{App.} = \frac{5 w_{live} L^4}{384\Delta}$$

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

$$= \frac{(5)(180)\left(\frac{93.33}{12}\right)(18(12))^3}{384}$$

$$= 183.7 \text{ x } 10^6 \text{ in.}^2\text{-lbs}$$

$EI_{App.}$ = 183.7 x 10^6 in.^{2}–lbs (WFCM Table 2.15D)

Calculate the moment:

$$M = \frac{w_{total} L^2}{8} = \frac{106.67(18^2)}{8}$$

$$= 4,320 \text{ ft-lbs}$$

M = 4,320 ft–lbs (WFCM Table 2.15D)

Calculate the bearing reaction:

$$R = \frac{w_{total} L}{2} = \frac{106.67(18)}{2}$$

$$= 960 \text{ lbs}$$

R = 960 lbs (WFCM Table 2.15D)

Footnotes 2-4:

See Commentary for Tables 2.8A&B for calculations.

Table 2.16 Ridge Beam Capacity Requirements for Interior Center Bearing Roof and Ceiling

Description: Calculation of uniform load on the ridge beam.

Procedure: Sum gravity loads, calculate the ridge beam requirements.

Background: Assume tributary span of the ridge beam equals half of the roof span.

Example:

Given - 36' roof span (W), 18' tributary width, 10 psf roof dead load, and 50 psf ground snow load.

$$w_{dead} = 10 \text{ psf} = 13.33 \text{ plf}$$

$$w_{snow} = (0.7)C_eC_tI_sp_g \qquad (\textit{ASCE 7-10})$$

$$= (0.7)(1.0)(1.1)(50 \text{ psf}) = 38.5 \text{ psf}$$

$$w_{total} = 10 + 38.5 = 48.5 \text{ plf}$$

$$\text{Total Load} = (W / 2)(w_{total})$$

$$= (36/2)(48.5) = 873 \text{ plf}$$

(WFCM Table 2.16)

Table 2.17 Hip and Valley Beam Capacity Requirements

Description: Calculation of member properties required under a given span and load.

Procedure: Input spans, solve for Apparent Rigidity, Moment, and Bearing.

Background: Triangular loadings imposed from members framing into hips and valleys.

Example:
Given - 12' x 12' hip or valley area, 20 psf roof dead load, 30 psf ground snow load, $\Delta_{LL} \leq L/180$

Calculate the actual hip or valley beam length and spacing of the jack rafters along the hip or valley beam.

$$L' = (12\text{ ft})/\cos(45) = 17.0\text{ ft}$$

$$S' = (2\text{ ft})/\cos(45) = 2.83\text{ ft}$$

Calculate the length of each jack rafter and the concentrated load from each jack rafter which frames into the hip or valley beam.

a = Location along Hip or Valley

j = Jack Length

LL = Live Load on Hip or Valley

TL = Total Load on Hip or Valley

By superposition, calculate the apparent rigidity, EI_{App}, assuming Δ_{max} occurs at L'/2:

$$\Delta_{max} = \Sigma\Delta_i$$

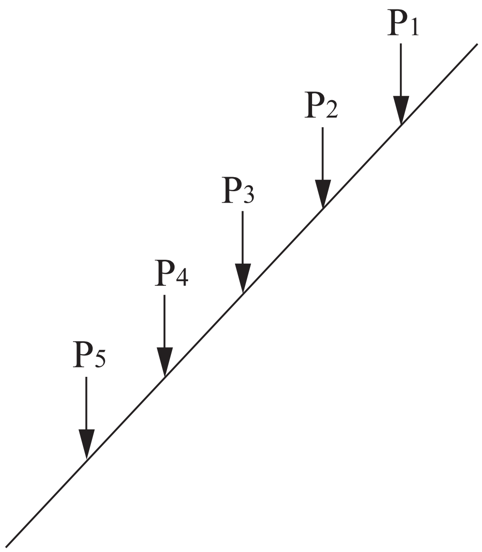
$$\Delta_{max} \leq L'/180$$

$$L'/180 = \Sigma(K_i/EI)$$

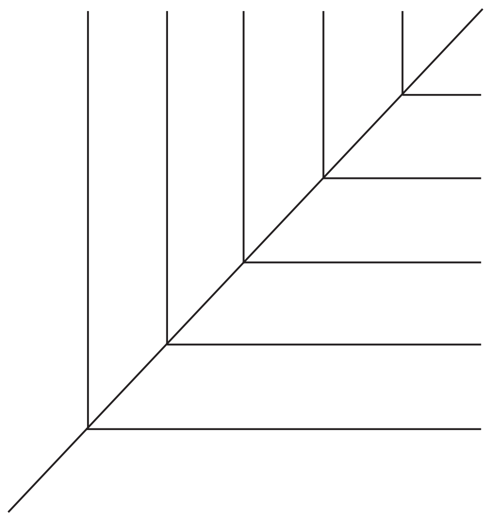
$$(EI)L'/180 = \Sigma K_i$$

$$EI_{Min} = \Sigma(180K_i/L')$$

$$EI_{Min} = \Sigma(EI_i)$$



Elevation View



Plan View

	a = L' - nS'	j = (a)cos(45)	LL = (j/2)S(23.1 psf)	TL = (j/2)S(43.1 psf)
Jack#1	14.2'	10.0'	231.0 lbs	431.0 lbs
Jack#2	11.3'	8.0'	184.8 lbs	344.8 lbs
Jack#3	8.5'	6.0'	138.6 lbs	258.6 lbs
Jack#4	5.7'	4.0'	92.4 lbs	172.4 lbs
Jack#5	2.8'	2.0'	46.2 lbs	86.2 lbs

$EI_{total} = 2\Sigma(EI_i)$ multiplied by 2 to account for loads from jack rafters on each side

where:

$EI_i = (P(L'-a)/48)(180/L')[3L'^2 - 4(L'-a)^2]$ when $a \geq L'/2$

$EI_i = (Pa/48)(180/L')[3L'^2 - 4a^2]$ when $a \leq L'/2$

$EI_1 = (231(17.0-14.2)/48)(180/17.0)[3(17)^2-4(17.0-14.2)^2](144)$
$= 17.1x10^6$ in.^{2}-lbs

$EI_2 = (184.8(17.0-11.3)/48)(180/17.0)[3(17)^2-4(17.0-11.3)^2](144)$
$= 24.6x10^6$ in.^{2}-lbs

$EI_3 = (138.6(8.5)/48)(180/17.0)[3(17)^2-4(8.5)^2](144)$
$= 21.6x10^6$ in.^{2}-lbs

$EI_4 = (92.4(5.7)/48)(180/17.0)[3(17)^2-4(5.7)^2](144)$
$= 12.3x10^6$ in.^{2}-lbs

$EI_5 = (46.2(2.8)/48)(180/17.0)[3(17)^2-4(2.8)^2](144)$
$= 3.4x10^6$ in.^{2}-lbs

$EI_{total} = 2(79{,}000{,}000) = 158.1x10^6$ in.^{2}-lbs

$EI_{App.} = 158.1x10^6$ in.^{2}-lbs (WFCM Table 2.17)

By superposition, calculate the top reaction (also maximum shear):

$R_{top} = 2\Sigma(R_{i\,(top)})$ (multiplied by 2 to account for loads from jack rafters on each side)

where:

$R_i = P(a)/L'$ for reaction at top

R_1 =	(431(14.2)/17.0)	=	360.0 lbs
R_2 =	(345(11.3)/17.0)	=	229.3 lbs
R_3 =	(259(8.5)/17.0)	=	129.5 lbs
R_4 =	(172(5.7)/17.0)	=	57.7 lbs
R_5 =	(86(2.8)/17.0)	=	14.2 lbs
R_{top} =	2(790)	=	1,580 lbs

R = 1,580 lbs (WFCM Table 2.17)

Finally, calculate the moment:
M_{Max} is located where V=0.

By subtracting the concentrated jack rafter loads from the top reaction ($R_{top}/2$), the location of the maximum moment in the hip or valley beam can be obtained.

$V_{J\#1} = R_{top}/2 - TL_{J\#1}$
= 790 lbs – 431 lbs = 359 lbs

$V_{J\#2} = V_{J\#1} - TL_{J\#2}$
= 359 lbs – 345 lbs = 14 lbs

$V_{J\#3} = V_{J\#2} - TL_{J\#3}$
= 14 lbs – 259 lbs = -245 lbs

The maximum moment occurs at jack rafter #3 since the location of zero shear (shear changes from 14 lbs to -245 lbs) occurs at the jack rafter #3 location.

Summing moments to one side of this location yields:

$\Sigma M_{J\#3}$ = 1,580 lbs (8.5 ft) - 2(431 lbs)(5.7 ft) - 2(345 lbs)(2.8 ft)
= 6,583 ft-lbs

M = 6,583 ft-lbs (WFCM Table 2.17)

2

ENGINEERED DESIGN

PRESCRIPTIVE DESIGN

C3.1 General Provisions

C3.1.1 Prescriptive Requirements

Chapter 3 provides a specific set of prescriptive provisions necessary to meet the minimum requirements of this document.

C3.1.2 Equivalent Materials and Systems

The prescriptive provisions provided in this Chapter are a solution to the loading requirements in Chapter 2. They are not intended to preclude the use of any alternative materials and systems.

C3.1.3 Prescriptive Design Limitations

Chapter 3 is developed to provide prescriptive solutions for building design based on accepted engineering practice. Where prescriptive limitations are exceeded, the engineered design provisions of Chapter 2 may still be applicable.

C3.1.3.1 Wind Exposure and Mean Roof Height

Chapter 3 provisions assume a 33 ft mean roof height (MRH) and are tabulated for both wind Exposure Categories B and C.

C3.1.3.2 Floor Systems

C3.1.3.2a Framing Member Spans Framing member spans are limited to 26 feet for floors based on the bending capacity of the double top plate supporting floor framing members. The worst case assumption is that a floor framing member bears directly between two studs creating a concentrated load at mid-span of the top plates. Section 3.1.3.3g requires bandjoists, blocking, or other methods to transfer roof, wall, and/or floor loads from upper stories to alleviate the concern of additional loads being transferred through the floor framing members into the top plate.

Framing member spans are based on L/360 for serviceability, however some product manufacturers recommend more conservative criteria than L/360 for floor live load deflection based on feedback from users related to floor performance in the field.

C3.1.3.2b Framing Member Spacings Floor assemblies in the *WFCM* have been designed using the concept of closely-spaced framing members spaced no more than 24 inches on center, and sheathed with load-distributing elements, such as wood structural panels. Closely spaced floor framing members provide redundancy ensuring reliable performance of repetitive member floor assemblies.

C3.1.3.2c Cantilevers Limiting floor cantilevers to the depth of the joist for lumber joists supporting a loadbearing non-shear wall is based on joist strength considerations.

Limiting floor cantilevers to L/4 for lumber joists supporting a non-loadbearing non-shear wall is based primarily on deflection considerations. See C3.1.3.3d for background on shear wall story offset requirements.

The requirement to have floor framing members line up directly over studs in cantilever conditions supporting a loadbearing wall, avoids the case where a floor framing member bears between two studs creating a concentrated load at mid-span of the top plates. In this case the floor framing member would also be carrying roof, ceiling, wall, and potentially additional floor loads to the top plate which would exceed the bending capacity of the wall double top plates.

C3.1.3.2d Setbacks Limiting setbacks to the depth of the joist for lumber joists supporting a loadbearing wall is based on joist strength considerations. Similar to cantilevers, floor framing members shall line up over studs in setback conditions.

C3.1.3.2e Vertical Floor Offsets Prescriptively limiting vertical floor offsets to the depth of the floor framing member allows the offset to be connected with a shear connector as shown in Figure 2.1i. The floor discontinuity restriction keeps the vertical distance between adjacent diaphragms small enough to be reasonably connected. Larger vertical discontinuities would require special detailing for moment resisting elements at the connection, which is beyond the scope of this document. Floor discontinuities are not limited when the individual floor levels are considered as separate structures. In such cases, each separate floor diaphragm should be designed, detailed, and supported in accordance with all applicable provisions of the *WFCM*.

C3.1.3.2f Diaphragm Aspect Ratio In calculating the minimum and maximum floor diaphragm lengths in Tables 3.16B and C, a diaphragm aspect ratio of 3:1 is used, based on maximum limitations for unblocked horizontal diaphragms in the *Special Design Provisions for Wind and Seismic (SDPWS)* standard.

C3.1.3.2g Diaphragm Openings Diaphragm openings are limited to the lesser of 12 feet or 50% of the building dimension. Where larger openings are used, design of the diaphragm should be in accordance with accepted engineering practice.

C3.1.3.3 Wall Systems

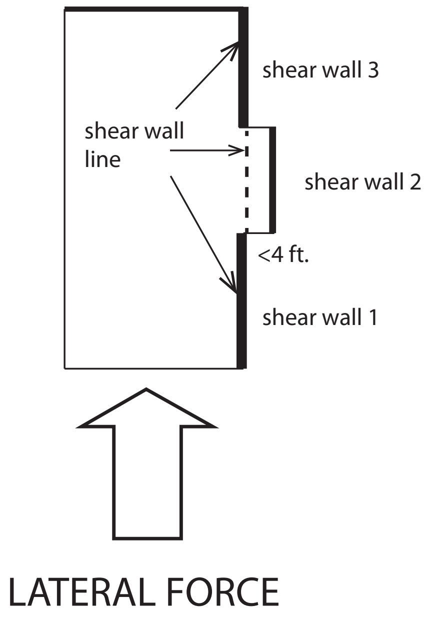
C3.1.3.3a Wall Heights The 10 foot height limit is the maximum prescriptive span permitted in the model building codes for loadbearing walls.

Limiting non-loadbearing walls to 20 feet in height is a practical approach since typical lumber lengths do not exceed this limit.

C3.1.3.3b Wall Stud Spacings Wall assemblies in the *WFCM* have been designed using the concept of closely-spaced framing members, spaced no more than 24 inches on center, and sheathed with load-distributing elements, such as wood structural panels, structural fiberboard, and gypsum wallboard. Closely spaced wall framing members provide redundancy, ensuring reliable performance of repetitive member wall assemblies.

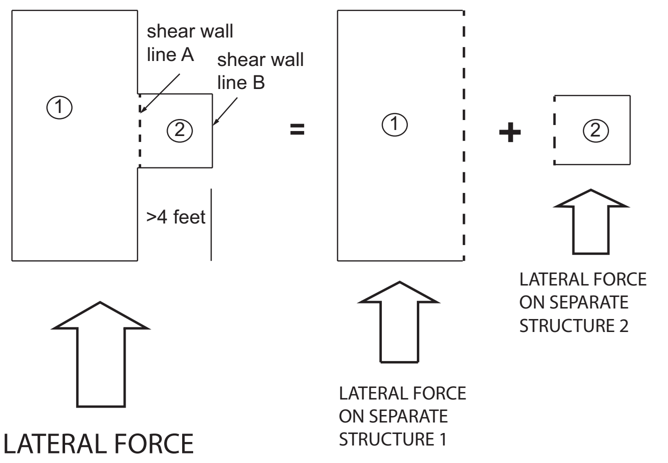
C3.1.3.3c Shear Wall Line Offsets Where shear walls are offset less than 4 feet in plan view, the walls shall be assumed to act along the same shear wall line for purposes of collecting diaphragm lateral forces into the shear walls as shown in Figure C3.1.3.3-1.

Figure C3.1.3.3-1 Shear Walls Offset < 4 feet.



For buildings where shear walls are offset greater than or equal to 4 feet in plan, the *WFCM* requires the building to be designed as separate structures attached in the plane of the offset as shown in Figure C3.1.3.3-2.

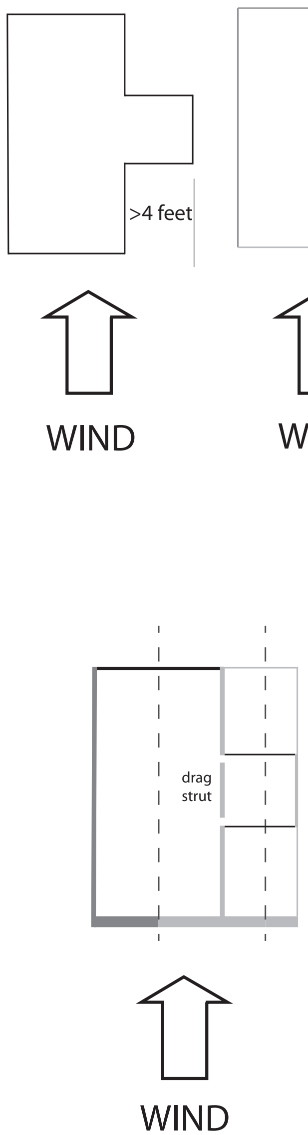
Figure C3.1.3.3-2 Separate Structures Method for Shear Walls Offset ≥ 4 feet.



Shear wall sheathing lengths are therefore determined based on the geometry of each "separate" structure comprising the overall building. Shear wall sheathing length for the shared wall (shown as part of shear wall line A in Figure C3.1.3.3-2) is calculated as the sum of the shear wall sheathing lengths required for each separate structure at that location.

For wind loads, the structure shall be permitted to be considered as a rectangular structure with perimeter dimensions which inscribe the total structure as shown in Figure C3.1.3.3-3 (left). Distribution of shear loads from lateral wind forces is shown in Figure C3.1.3.3-3 (right) proportional to the diaphragm area tributary to each shear wall line.

Figure C3.1.3.3-3 Inscribed Method Permitted for Determining Distribution of Shear Loads for Wind.



C3.1.3.3d Shear Wall Story Offsets Shear wall story offsets are not permitted in the *WFCM* unless they are specifically designed and detailed for the moment which is created from overturning forces at shear wall ends – the magnitude of which will vary with the amount of offset. A specific set of conditions are specified in the exception statement in Section 3.1.3.3d (also refer to details in Figure 3.1c) which if met, permits a maximum offset equal to the depth, d, of the floor framing member.

C3.1.3.3e Shear Wall Aspect Ratio Shear wall aspect ratio limitations vary by shear wall sheathing type and loading and are specified in *SDPWS.*

C3.1.3.3g Load Transfer The requirement for bandjoists, blocking, or other methods to transfer roof, wall, and or floor loads from upper stories is based on considerations for compression perpendicular to grain and bending of top plates if loads from upper levels are carried into these members as concentrated loads through the floor framing members.

C3.1.3.4 Roof Systems

C3.1.3.4a Framing Spans Limiting lumber rafter spans (horizontal projection) to 26 feet is a practical limitation based on availability of those lengths for lumber. Limiting I-joist rafter spans to 26 feet is a carry-over from limits for floor I-joist span limitations (see C3.1.3.2a). Longer I-joists are manufactured.

C3.1.3.4b Framing Spacings Roof assemblies in the *WFCM* have been designed using the concept of closely-spaced framing members, spaced no more than 24 inches on center, and sheathed with load-distributing elements, such as wood structural panels. Closely spaced roof framing members provide redundancy, ensuring reliable performance of repetitive member roof assemblies.

C3.1.3.4c Overhang Lengths The rafter overhang length limitation of L/3 is based on moment capacity of the rafters due to notching for bearing at the top of the wall (birdsmouth notch). For sawn lumber roof rafters that utilize a birdsmouth notch as depicted in Figure C2.5.1.4, code provisions typically limit the depth of notch to 25% the depth of the solid sawn rafter where the depth of the cut is measured perpendicular to the length dimension of the rafter. For engineered wood products, the user should refer to the manufacturer's recommendations.

C3.1.3.4d Slope Roof slope is limited to 12:12 or less consistent with limited scope of tabular requirements that are dependent on roof slope.

C**3.1.3.4e Diaphragm Aspect Ratio** In calculating the minimum and maximum roof diaphragm lengths in Tables 3.16A and C, a diaphragm aspect ratio of 3:1 is used, based on maximum limitations for unblocked horizontal diaphragms in *SDPWS*.

C3.1.4 Interpolation

Most of the tabulated values in Chapter 3 attempt to address the range of possible results using the prescriptive provisions. In order to tabulate a relatively large range of conditions, information was provided such that interpolation could be used to determine cases where the requirements were not specifically given.

C3.2 Connections

Prescriptive connection provisions, as well as connection capacity requirements, are provided in Chapter 3. While most of Chapter 3 provides some prescriptive requirements, it is recognized that proprietary connectors are often used to meet the connection requirements and are provided for the benefit of the user.

C3.2.2.3 Wall Assembly to Foundation
Steel connectors to resist uplift are required to be a minimum 20 gage, ASTM A633 grade steel. A specific decimal thickness reference is not made in the *WFCM* to avoid confusing a design thickness (which is shown in the *NDS*) from being interpreted as being the minimum thickness required. For example a 0.036 inch design thickness often correlates to a minimum thickness of 0.034 inches. In cold-formed steel design, there are adjustments to minimum thickness that are made in order to arrive at a design thickness, with design thickness permitted to be 5% greater than minimum thickness (See section 2.4 of the AISI *North American Specification for Design of Cold-Formed Steel Structural Members*).

C3.2.3.2 Wood Structural Panels Resisting Uplift
Requirements from *SDPWS* Section 4.4 (Wood Structural Panels Designed to Resist Combined Shear and Uplift from Wind) have been incorporated into this section of the *2012WFCM*. These requirements permit wood structural panels with a performance category or minimum thickness of 7/16" and strength axis oriented parallel to studs to be used in combined uplift and shear applications for resistance to wind forces. As a prescriptive alternative to Table 3.4, tabulated values of maximum roof span are provided in Table 3.4B. The uplift force is a function of the rafter span, and therefore a corresponding uplift resistance can be calculated for various combinations of nailing schedules, panel type, and panel thickness in accordance with *SDPWS* Table 4.4.1.

C3.3 Floor Systems

Wood I-joist and wood truss systems are not covered directly in the prescriptive provisions of Chapter 3 since product properties and installation recommendations are manufacturer specific.

C3.4 Wall Systems

This section provides prescriptive requirements for wall design.

C3.4.1.1 Wood Studs Maximum stud lengths tabulated in Tables 3.20A and B are based on wind load only – acting perpendicular to the stud. Maximum stud lengths in Table 3.20C are based on dead and live gravity loads.

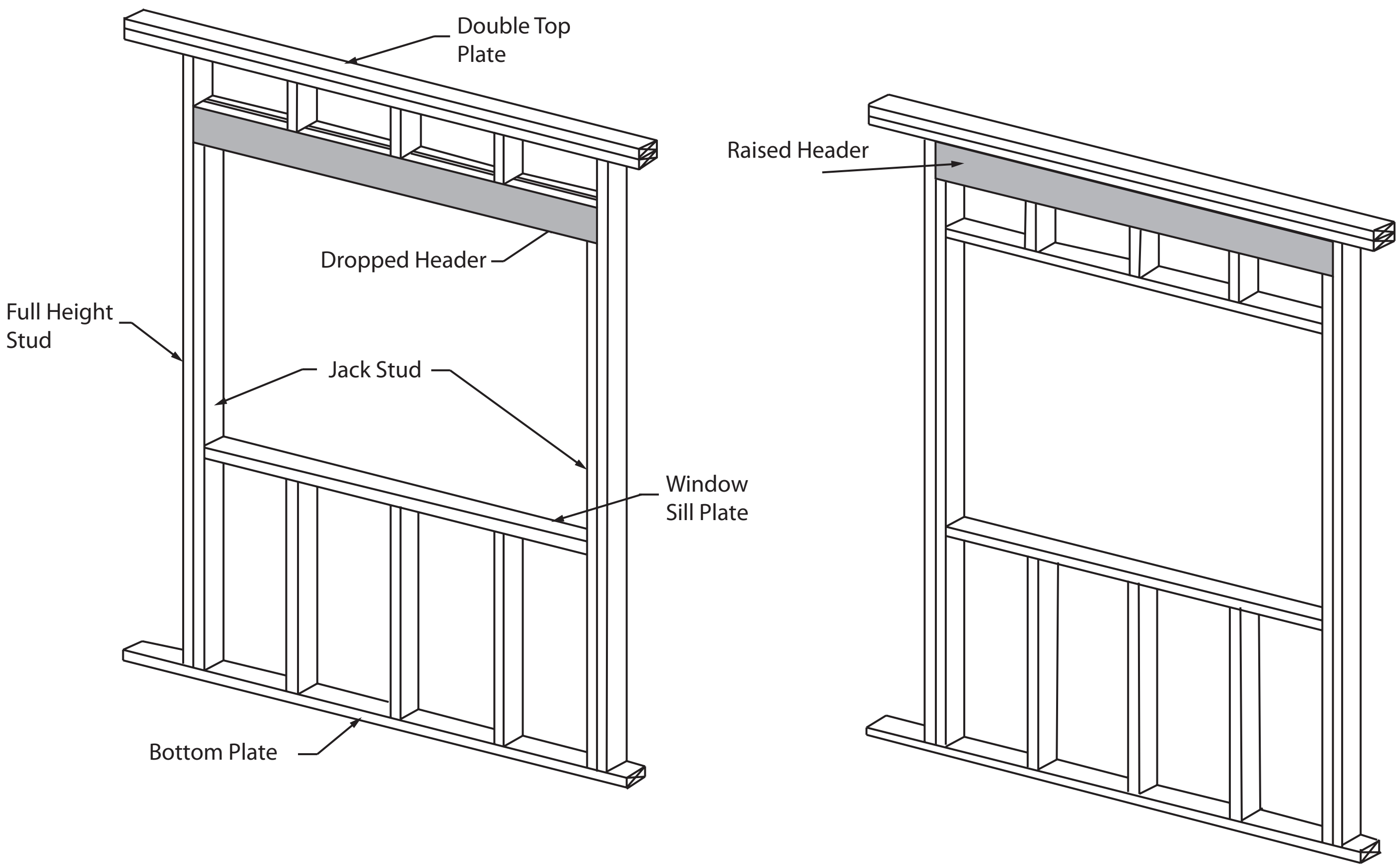
C3.4.1.2 Top Plates Capacity of top plate splices is typically controlled by diaphragm chord forces per Table 3.21. If large shear forces from the upper stories of a structure are transferred through the top plates to shear walls at the bottom story, drag strut forces, as opposed to chord forces, may govern the top plate slice design and should be considered.

C3.4.1.4 Wall Openings

C3.4.1.4.1 Headers Maximum header spans are tabulated for both "raised" headers − headers are assumed to be laterally supported at ends and along their length, and "dropped" headers − headers are assumed to be laterally supported at ends only. Dropped headers address the case where the header is not attached directly to the double-top plate framing as shown in Figure C3.4.1.4.1.

C**3.4.4.2 Exterior Shear Walls** Minimum length of full height sheathing on exterior shear walls is provided for wind design in Table 3.17A and for seismic design in Table 3.17C. Minimum lengths are calculated for Table 3.17A by assuming a top plate-to-ridge height of 10 feet and adjustments for other top plate-to-ridge heights are provided in footnote 4.

Figure C3.4.1.4.1 Dropped and Raised Headers



C3.5 Roof Systems

Wood I-joists and wood trusses are not covered directly in the prescriptive provisions of Chapter 3 since product properties and installation recommendations are manufacturer specific.

List of Tables

3

PRESCRIPTIVE DESIGN

Table 3.1 Nailing Schedule

Description: Framing connection requirements.

Background: The following table provides lateral resistance (nZ′) and withdrawal (nW′p_t) design values for the prescriptive nailing in Table 3.1.

Joint Description	Common Nails			Box Nails			Nail Spacing
	Quantity and size of nails	nZ' lbs	nW'p_t lbs	Quantity and size of nails	nZ' lbs	nW'p_t lbs	
ROOF FRAMING							
Rafter to Top Plate (Toe-nailed)	Table 3.4A			Table 3.4A			per rafter
Ceiling Joist to Top Plate (Toe-nailed)	Table 3.4A			Table 3.4A			per joist
Ceiling Joist to Parallel Rafter (Face-nailed)	Table 3.9A			Table 3.9A			each lap
Ceiling Joist Laps over Partitions (Face-nailed)	Table 3.9A			Table 3.9A			each lap
Collar Tie to Rafter (Face-nailed)	Table 3.6A			Table 3.6A			per tie
Blocking to Rafter (Toe-nailed)	2-8d	198	67	2-10d	208	80	each end
Rim Board to Rafter (End-nailed)	2-16d	257	0	3-16d	283	0	each end
WALL FRAMING							
Top Plate to Top Plate (Face-nailed)	2-16d[1]	384	125	2-16d[1]	281	100	per foot
Top Plates at Intersections (Face-nailed)	4-16d	768	250	5-16d	704	252	joints - each side
Stud to Stud (Face-nailed)	2-16d	384	125	2-16d	281	100	24" o.c.
Header to Header (Face-nailed)	16d	192	62	16d	140	50	16" o.c. along edges
							per stud
Top or Bottom Plate to Stud (End-nailed)	Table 3.5A			Table 3.5A			
							per foot
Bottom Plate to Floor joist, Bandjoist, Endjoist, or Blocking (Face-nailed)	2-16d[1,2]	384	166	2-16d[1,2]	422	201	
FLOOR FRAMING							
Joist to Sill, Top Plate, or Girder (Toe-nailed)	4-8d	396	134	4-10d	416	160	per joist
Bridging to Joist (Toe-nailed)	2-8d	198	67	2-10d	208	80	each end
Blocking to Joist (Toe-nailed)	2-8d	198	67	2-10d	208	80	each end
Blocking to Sill or Top Plate (Toe-nailed)	3-16d	475	177	4-16d	467	198	each block
Ledger Strip to Beam (Face-nailed)	3-16d	576	249	4-16d	563	268	each joist
Joist on Ledger to Beam (Toe-nailed)	3-8d	297	100	3-10d	312	120	per joist
Band Joist to Joist (End-nailed)	3-16d	385	0	4-16d	377	0	per joist
Band Joist to Sill or Top Plate (Toe-nailed)	2-16d[1]	316	118	3-16d	350	148	per foot
ROOF SHEATHING							
Wood Structural Panels	8d	94	67	10d	89	80	(Table 3.10)
Diagonal Board Sheathing							
1"x6" or 1"x8"	2-8d	224	117	2-10d	217	144	per support
1"x10" or wider	3-8d	336	176	3-10d	326	216	per support
CEILING SHEATHING							
Gypsum Wallboard	5d coolers			5d coolers			7" edge/10" field

3 PRESCRIPTIVE DESIGN

Table 3.1 Nailing Schedule (Cont.)

WALL SHEATHING							
Wood Structural Panels	8d	94	67	10d	89	80	(Table 3.11)
Structural Fiberboard Panels							
1/2"	11 ga. galv. roofing nail (0.120"x 1-1/2" long x 7/16" head)			- -			3" edge/6" field
25/32"	11 ga. galv. roofing nail (0.120"x 1-3/4"long x 3/8" head)						3" edge/6" field
Gypsum Wallboard	5d coolers			5d coolers			7" edge/10" field
Hardboard	8d			8d			(Table 3.11)
Particleboard Panels	8d			8d			(see manufacturer)
Diagonal Board Sheathing							
1"x6" or 1"x8"	2-8d	224	117	2-10d	217	144	per support
1"x10" or wider	3-8d-	336	176	3-10d	326	216	per support
FLOOR SHEATHING							
Wood Structural Panels							
1" or less	8d	94		10d	89		6" edge/12" field
greater than 1"	10d	153		16d	137		6" edge/6" field
Diagonal Board Sheathing							
1"x6" or 1"x8"	2-8d	224		2-8d	217		per support
1"x10" or wider	3-8d	336		3-8d	326		per support

nZ' – Lateral design values based on G=0.42, load duration factor = 1.6, and number of nails, n, shown for each connection.
nW'p_t - Withdrawal design values based on G=0.42, load duration factor = 1.6, length of fastener penetration into wood member for withdrawal calculations, p_t, and number of nails, n, shown for each connection.

Table 3.2 Sill or Bottom Plate to Foundation Connection Requirements for Wind

Description: Uplift, lateral, and shear connection requirements for sill or bottom plate to foundation connections.

Procedure: Using uplift, lateral, and shear loads calculated from Chapter 2, determine sill plate or bottom plate to foundation connections required to resist those loads.

Background: Uplift loads from wind are based on Table 2.2A, offset by 60% of the design dead load.

Lateral forces from wind are calculated based on Table 2.1 values assuming ½ the stud wall height as the tributary area.

Shear loads from wind are based on aspect ratio limits and are calculated assuming an unblocked flexible diaphragm.

Example:

Given - 150 mph, Exposure B, wind perpendicular to ridge, 10' top plate-to-ridge height, 36' roof span, 15 psf roof/ceiling dead load, 10' wall height, 121 plf wall weight.

Uplift
From WFCM Table 2.2A:

$$w_{uplift} = 326\text{plf}$$

Reduce load for weight of the walls above:

$$w_{1\,Story} = 326\text{ plf} - 0.6(121)\text{ plf} = 326\text{ plf} - 73\text{plf}$$
$$= 253\text{ plf} \quad \text{(WFCM Table 3.2) specifies } 258\text{ plf}^{1}$$

$$w_{2\,Story} = 326\text{ plf} - 2(73\text{ plf})$$
$$= 180\text{ plf} \quad \text{(WFCM Table 3.2) specifies } 185\text{ plf}^{1}$$

$$w_{3\,Story} = 326\text{ plf} - 3(73\text{ plf})$$
$$= 107\text{ plf} \quad \text{(WFCM Table 3.2) specifies } 112\text{ plf}^{1}$$

[1] Table 3.2 specifies slightly higher uplift loads than those specified in Table 2.2A. The difference results from external pressure coefficients used for the windward overhang. Wind uplift connections loads in Table 3.2 are developed using a C_p of 0.8; wind uplift connections loads in Table 2.2A are developed using the slightly lower C_p of 0.7 per section 28.4.3 of *ASCE 7-10*.

Lateral - Sill Plate
Required lateral capacities are determined in the foundation design per section 1.1.4.

Lateral - Bottom Plate
From Table 2.1:

$$w_{lateral} = 148\text{ plf} \qquad \text{(WFCM Table 3.2)}$$

Shear - Sill Plate
For 150 mph Exposure B tabulated values in Table 3.2 include a roof diaphragm load of 249 plf calculated assuming a 10' top plate-to-ridge height (see Commentary for Footnote 4 Base Case below for additional information). Lateral loads into the floor diaphragm are 251 plf per Table 2.5A.

Unit shear (plf) is dependent on the diaphragm aspect ratio. Thus, the unit shear (v) is equal to:

$$v = [wL] / [2W] \text{(wind loads perpendicular to ridge)}$$
$$v = [wW] / [2L] \qquad \text{(wind loads parallel to ridge)}$$

where:

L = Diaphragm Length
W = Diaphragm Width

Assuming R= L/W for wind perpendicular to ridge and R = W/L for wind parallel to ridge, v is:

For 1, 2, and 3 story structures, v for wind perpendicular to ridge is:

$$v_1\text{ Story} = [249L + 251L] / [2W]$$
$$= [(500)L/2W]$$

(Substituting R = L/W)

$$= [(500)/2]R = 250R\text{ plf} \qquad \text{(WFCM Table 3.2)}$$

$$v_{2\,Story} = [249L + (251L + (251)L] / [2W]$$
$$= [(751)\,L/2W]$$

(Substituting R = L/W)

$$= [(751)/2]R$$
$$= 376R\text{ plf} \qquad \text{(WFCM Table 3.2)}$$

3

PRESCRIPTIVE DESIGN

$$v_{3\ Story} = (249L + 251L + 251L + 251)L)/2W$$
$$= [(1002)L\ /2W]$$

(Substituting R = L/W) = 501R plf (WFCM Table 3.2)

Shear - Bottom Plate
Connector shear load (parallel to the wall), S, is calculated based on the shear capacity of walls constructed with 7/16" wood structural panels, with 8d common nails at 6"o.c. and ½" gypsum wallboard with 5d cooler nails at 7" o.c. at edges.

S = 436 plf (WFCM Table 3.2)

Footnote 1:
See Commentary for Tables 2.1 and 2.2A for calculations.

Footnote 4:
A top plate-to-ridge height of 10 feet is used in the tabular shear capacity requirements. Adjustment factors for other top plate-to-ridge heights of 5 feet, 15 feet, and 20 feet are provided in Footnote 4.

Tabulated shear capacity requirements for connections are based on a 10 ft wall height and a 10 ft top plate-to-ridge height. For other wall heights or top plate-to-ridge heights or combinations thereof, adjustment factors are provided for adjustment of the tabulated shear capacity requirements.

An example calculation of the adjustment factors for the “Roof + 1 Floor” case follows:

Building width and length: 24 ft (R = L/W = 1)

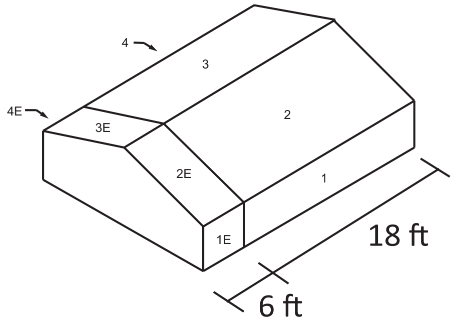
Wind Speed: 150 mph (700-yr. 3-second gust) Exposure Category B

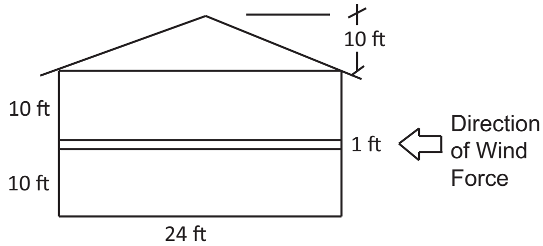
BASE CASE (Used in tabulated shear capacity requirements)

Top Plate-To-Ridge Height = 10 ft

Wall Height = 10 ft

Roof Pitch > 6:12





Roof Diaphragm

MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch > 6:12 (30-45 degrees) are used as this is the most conservative case (see Table C3.2A):

q = 21.15 psf (See Table C1.1)

Calculate the load from roof interior zones (2 & 3):

$$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$$
$$= 18ft(10ft)(21.15\ psf)(0.21+0.43)$$
$$= 2{,}436\ lbs$$

Calculate the load from roof end zones (2E & 3E):

$$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$$
$$= 6ft(10ft)(21.15\ psf)(0.27+0.53) = 1{,}015\ lbs$$

Calculate the load from the wall interior zones (1 & 4) that is tributary to the roof/ceiling diaphragm:

$$= A_{Wall}\ q(GC_{pf})$$
$$= 18ft(10ft/2)(21.15\ psf)(0.56+0.37)$$
$$= 1{,}770\ lbs$$

Table C3.2A Coefficients used for Roof Diaphragms (Roof Pitch > 6:12)

	Interior Zones				End Zones			
Roof Angle, degrees	1	2	3	4	1E	2E	3E	4E
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
26.6 (6:12)	0.55	-0.10	-0.45	-0.39	0.73	-0.19	-0.585	-0.535
30-45	**0.56**	**0.21**	**-0.43**	**-0.37**	**0.69**	**0.27**	**-0.53**	**-0.48**

Table C3.2B Coefficients used for Floor Diaphragms

	Interior Zones				End Zones			
Roof Angle, degrees	1	2	3	4	1E	2E	3E	4E
20	**0.53**	**-0.69**	**-0.48**	**-0.43**	**0.80**	**-1.07**	**-0.69**	**-0.64**
26.6 (6:12)	0.55	-0.10	-0.45	-0.39	0.73	-0.19	-0.585	-0.535
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48

Calculate the load from the wall end zones (1E & 4E) that is tributary to the roof/ceiling diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 6ft(10ft/2)(21.15\ psf)(0.69+0.48)$
$= 742\ lbs$

The total load resisted by the roof/ceiling diaphragm is:

$= 2{,}436 + 1{,}015 + 1{,}770 + 742 = 5{,}963\ lbs$

Note that the roof diaphragm load of 249 plf used earlier to calculate shear in the sill plate is calculated from: 5,963 lbs / 24 ft = 249 plf

Floor Diaphragm
MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch equal to 20 degrees is used to calculate the shear force in the floor diaphragm as this is the most conservative case (see Table C3.2B):

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

Calculate the load from the wall interior zones (1 & 4) that is tributary to the floor diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 18ft(10ft/2+1ft+10ft/2)(21.15\ psf)(0.53+0.43) = 4{,}020\ lbs$

Calculate the load from wall end zones (1E & 4E) that is tributary to the floor diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 6ft(10ft/2+1ft+10ft/2)(21.15\ psf)(0.80+0.64) = 2{,}010\ lbs$

The total load resisted by the floor diaphragm is:

$= 4{,}020 + 2{,}010 = 6{,}030\ lbs$

Required Shear Capacity
The total roof/ceiling diaphragm load and floor diaphragm load is:

$= 5{,}963 + 6{,}030 = 11{,}993\ lbs$

The required shear capacity along each exterior wall is:

$= (11{,}993/2)lbs/24\ ft$
$= 250\ plf$ (WFCM Table 3.2)

Correspondingly, the adjustment factor in Footnote 4 is equal to 1.0 for the base case.

ALTERNATE CASE #1:

Top Plate-To-Ridge Height = 10 ft

Wall Height = 10 ft

Roof Pitch ≤ 6:12

Roof Diaphragm
MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch equal to 6:12 (26.6 degrees) are used as this is the most conservative case for roof pitches less than or equal to 6:12 (see Table C3.2C):

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

Table C3.2C Coefficients used for Roof Diaphragms (Roof Pitch ≤ 6:12)

	Interior Zones				End Zones			
Roof Angle, degrees	1	2	3	4	1E	2E	3E	4E
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
26.6 (6:12)	**0.55**	**-0.10**	**-0.45**	**-0.39**	**0.73**	**-0.19**	**-0.585**	**-0.535**
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48

3

PRESCRIPTIVE DESIGN

Calculate the load from roof interior zones (2 & 3):

$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$
$= 18ft(10ft)(21.15\ psf)(-0.10+0.45)$
$= 1,332\ lbs$

Calculate the load from roof end zones (2E & 3E):

$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$
$= 6ft(10ft)(21.15\ psf)(-0.19+0.585)$
$= 501\ lbs$

Calculate the load from the wall interior zones (1 & 4) that is tributary to the roof/ceiling diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 18ft(10ft/2)(21.15\ psf)(0.55+0.39)$
$= 1,789\ lbs$

Calculate the load from the wall end zones (1E & 4E) that is tributary to the roof/ceiling diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 6ft(10ft/2)(21.15\ psf)(0.73+0.535)$
$= 803\ lbs$

The total load resisted by the roof/ceiling diaphragm is:
$= 1,332 + 501 + 1,789 + 803 = 4,425\ lbs$

Floor Diaphragm
As in the base case, MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch equal to 20 degrees are used to calculate the shear force in the floor diaphragm.

The total load resisted by the floor diaphragm is:
$= 6,030\ lbs$

Required Shear Capacity
The total roof/ceiling diaphragm load and floor diaphragm load is:
$= 4,425 + 6,030 = 10,455\ lbs$

The required shear capacity along each exterior wall is:
$= (10,455/2)lbs/24\ ft = 218\ plf$

Correspondingly, the adjustment factor for this case is calculated as follows:

Adjustment factor = Required shear capacity for Alternate Case #1/Required shear capacity for Base Case

$= 218\ plf/250\ plf$
$= 0.87$ (Footnote 4 for WFCM Table 3.2)

ALTERNATE CASE #2:

Top Plate-To-Ridge Height = 5 ft

Wall Height = 8 ft

Roof Pitch ≤ 6:12

Roof Diaphragm
MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch equal to 6:12 (26.6 degrees) are used as this is the most conservative case for roof pitches less than or equal to 6:12 (see Table C3.2C):

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

Calculate the load from roof interior zones (2 & 3):

$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$
$= 18ft(5ft)(21.15\ psf)(-0.10+0.45) = 666\ lbs$

Calculate the load from roof end zones (2E & 3E):

$= A_{Vertical\ Roof\ Projection}\ q(GC_{pf})$
$= 6ft(5ft)(21.15\ psf)(-0.19+0.585) = 251\ lbs$

Calculate the load from the wall interior zones (1 & 4) that is tributary to the roof/ceiling diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 18ft(8ft/2)(21.15\ psf)(0.55+0.39)$
$= 1,431\ lbs$

Calculate the load from the wall end zones (1E & 4E) that is tributary to the roof/ceiling diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 6ft(8ft/2)(21.15\ psf)(0.73+0.535) = 642\ lbs$

The total load resisted by the roof/ceiling diaphragm is:
$= 666 + 251 + 1,431 + 642 = 2,990\ lbs$

Floor Diaphragm
As in the base case, MWFRS GC_{pf} coefficients for wind loads perpendicular to the ridge for a roof pitch equal to 20 degrees are used to calculate the shear force in the floor diaphragm.
$q = 21.15\ psf$ (See Table C1.1)

Calculate the load from the wall interior zones (1 & 4) that is tributary to the floor diaphragm:

$= A_{Wall}\ q(GC_{pf})$
$= 18ft(8ft/2+1ft+8ft/2)(21.15\ psf)(0.53+0.43)$
$= 3,289\ lbs$

Calculate the load from wall end zones (1E & 4E) that is tributary to the floor diaphragm:

$$= A_{Wall}\, q(GC_{pf})$$
$$= 6ft(8ft/2+1ft+8ft/2)(21.15\ psf)(0.80+0.64) = 1{,}644\ lbs$$

The total load resisted by the floor diaphragm is:

$$= 3{,}289 + 1{,}644 = 4{,}933\ lbs$$

Required Shear Capacity

The total roof/ceiling diaphragm load and floor diaphragm load is:

$$= 2{,}990 + 4{,}933 = 7{,}923\ lbs$$

The required shear capacity along each exterior wall is:

$$= (7{,}923/2)lbs/24\ ft = 165\ plf$$

Correspondingly, the adjustment factor for this case is calculated as follows:

Adjustment factor = Required shear capacity for Alternate Case #2/Required shear capacity for Base Case

$$= 165\ plf/250\ plf$$
$$= 0.66 \qquad \text{(Footnote 4 for WFCM Table 3.2)}$$

Table 3.2A Sill Plate to Foundation Connections Resisting Shear Loads from Wind

Description: Number of anchor bolts for sill or bottom plate to foundation connections resisting shear loads.

Procedure: Using shear loads calculated from Table 3.2, determine the minimum number of anchor bolts required in a shear wall line for both 1/2" and 5/8" anchor bolts.

Background: Anchor bolt lateral capacities are based on *2012 NDS* wood to concrete connection values. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed.

Example:

Given - 150 mph, Exposure B, 36' building width, sill plate (G=0.55), 6" (min) anchor bolt embedment.

Sill Plate

Anchor bolt capacities:

1/2" anchor bolt

$Z_{||}' = Z_{||}(C_D)$
= 680 lbs (1.6) (NDS Table 11E, G=0.55, loaded parallel to grain)
= 1,088 lbs

5/8" anchor bolt

$Z_{||}' = Z_{||}(C_D)$
= 970 lbs (1.6) (NDS Table 11E, G=0.55, loaded parallel to grain)
= 1,552 lbs

Calculate the number of 1/2" and 5/8" anchor bolts required for a foundation supporting a roof and one floor to resist shear based on the building dimension perpendicular to the shear wall line:

From WFCM Table 3.2:

v = 250R plf

1/2" bolts required

= $vW/Z_{||}'$
= 250 plf (36 ft)/1,088 lbs
= 8.7 bolts
= 9 bolts (WFCM Table 3.2A)

5/8" bolts required

= $vW/Z_{||}'$
= 250 plf (36 ft)/1,552 lbs
= 5.8 bolts
= 6 bolts (WFCM Table 3.2A)

Calculate the number of 1/2" and 5/8" anchor bolts required for a foundation supporting a roof and two floors to resist shear based on the building dimension perpendicular to the shear wall line.

From WFCM Table 3.2:

v = 376R plf

1/2" bolts required

= $vW/Z_{||}'$
= 376 plf (36 ft)/1,088 lbs
= 12.4 bolts
= 13 bolts (WFCM Table 3.2A)

5/8" bolts required

= $vW/Z_{||}'$
= 376 plf (36 ft)/1,552 lbs
= 8.7 bolts
= 9 bolts (WFCM Table 3.2A)

Calculate the number of 1/2" and 5/8" anchor bolts required for a foundation supporting a roof and three floors to resist shear based on the building dimension perpendicular to the shear wall line.

From WFCM Table 3.2:

v = 501R plf

1/2" bolts required

= $vW/Z_{||}'$
= 501 plf (36 ft)/1,088 lbs
= 16.5 bolts
= 17 bolts (WFCM Table 3.2A)

5/8" bolts required

= $vW/Z_{||}'$
= 501plf (36 ft)/1,552 lbs
= 11.6 bolts
= 12 bolts (WFCM Table 3.2A)

Footnote 4:
See Commentary for Table 3.2 for calculations.

Table 3.2B Bottom Plate to Foundation Connections Resisting Lateral and Shear Loads From Wind

***Description*:** Anchor bolt spacings for bottom plate to foundation connections resisting lateral and shear loads.

***Procedure*:** Using shear loads calculated from Table 3.2, determine the maximum permitted spacing of anchor bolts in a bottom plate.

Background: Anchor bolt lateral capacities are based on *2012 NDS* wood to concrete connection values. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed.

Example: Bottom Plate

Given - 150 mph, 36' building width,1/2" and 5/8" diameter anchor bolts, bottom plate G=0.55.

Anchor bolt capacities:

1/2" anchor bolt

$Z_{||}' = Z_{||}(C_D)$
= 680 lbs(1.6) (NDS Table 11E, G=0.55, loaded parallel to grain)
= 1,088 lbs

$Z_{\perp}' = Z_{\perp}(C_D)$
= 410 lbs(1.6) (NDS Table 11E, G=0.55, loaded perpendicular to grain)
= 656 lbs

5/8" anchor bolt

$Z_{||}' = Z_{||}(C_D)$
= 970 lbs(1.6) (NDS Table 11E, G=0.55, loaded parallel to grain)
= 1,552 lbs

$Z_{\perp}' = Z_{\perp}(C_D)$
= 580 lbs(1.6) (NDS Table 11E, G=0.55, loaded perpendicular to grain)
= 928 lbs

Shear Forces:

Calculate number of 1/2" and 5/8" bolts per bottom plate to resist shear forces (assume a 16' long bottom plate):

v = 436 plf (WFCM Table 3.2)

1/2" anchor bolts. A bolt is required at each end of the plate, therefore:

Required Bolts
= Shear Force / $Z_{||}'$
= [(436) plf (16) ft – 1,088 lbs] / 1,088 lbs
= 5.41 bolts

Calculate spacing:

A bolt is required 12" from each end of the plate, therefore:

Spacing = (16 ft - 2 ft) / 5.41 bolts
= 2.59 ft
= 31 in. (WFCM Table 3.2B)

5/8" anchor bolts. A bolt is required at each end of the plate, therefore:

Required Bolts
= [(436) plf (16) ft – 1,552 lbs] / 1,552
= 3.49 bolts

Calculate spacing:

A bolt is required 12" from each end of the plate, therefore:

Spacing = (16 ft - 2 ft) / 3.49 bolts
= 4.00 ft
= 48 in. (WFCM Table 3.2B)

Lateral Forces:

Calculate the number of 1/2" and 5/8" bolts required to resist lateral forces.

Lateral Force = 148 plf (WFCM Table 3.2)

Maximum bolt spacing for lateral loads is:

Bolt Spacing = Bolt Capacity / Lateral Force
= $Z_{\perp}'$/Lateral Force

1/2" anchor bolts
= 656 lbs / 148 plf
= 4.43 ft
= 53 in.

5/8" anchor bolts
= 928 lbs / 148 plf
= 6.27 ft
= 75 in.

Shear forces control. Maximum anchor bolt spacing is:

1/2" anchor bolts 31" (WFCM Table 3.2B)
5/8" anchor bolts 48" (WFCM Table 3.2B)

Table 3.2C Sill or Bottom Plate to Foundation Connections Resisting Uplift Loads From Wind

Description: Anchor bolt spacing for sill or bottom plate to foundation connections resisting uplift loads.

Procedure: Using uplift loads calculated from Table 3.2, determine the maximum permitted spacing of anchor bolts in a sill or bottom plate based on bending and compression perpendicular to grain considerations.

Background: Sill plates and wall bottom plates are included in calculations to resist the bending moment and compression perpendicular to grain forces due to uplift. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed. A flat use adjustment factor of C_{fu} = 1.1 is used.

Example:

Given - 150 mph, 36' building width, 15 psf roof/ceiling dead load, 1 story, No. 2 Southern Pine sill or bottom plate.

Sill or Bottom Plate

1 Story:

w_{uplift} = 258 plf (WFCM Table 3.2)

Assume:

$$M = \frac{w_{uplift} L^2}{8}$$

$$L = \sqrt{\frac{8M}{w_{uplift}}}$$

No. 2 Southern Pine sill or bottom plate exposed to weak axis bending:

S_{2x4} = $bd^2/6$ in.^{3}

S_{2x4} = $(3.5)(1.5)^2/6$ in.^{3}

S_{2x4} = 1.3125 in.^{3}

F_b' = $F_b(C_{fu})(C_D)$ psi

F_b' = (1,500 psi)(1.1)(1.6)

F_b' = 2,640 psi

M_{2x4} = F_b' (S_{2x4})

M_{2x4} = 2,640 (1.3125) in.-lbs

M_{2x4} = 3,465 in.-lbs

$$M = \frac{w_{uplift} L^2}{8}$$

$$L = \sqrt{\frac{8M}{w_{uplift}}}$$

$$Spacing = \sqrt{\frac{8(3,465)}{\frac{258}{12}}}$$

$$= 35.9 \text{ in.}$$

Maximum spacing = 35 in. (WFCM Table 3.2C)

The uplift load used in this example comes from WFCM Table 3.2. Footnote 1 indicates that tabulated uplift loads shall be permitted to be multiplied by 0.75 for framing not located within 8 feet of building corners – therefore the values are for the 8' end zones. The value shown in Table 3.2C for the interior zone can be calculated by applying the 0.75 adjustment to the 258 plf uplift load in the spacing equation. The result is 41 in. – consistent with the value in Table 3.2C for Interior Zones.

Check compression perpendicular to grain:

P_{reqd} = wL

= (258 plf)(35/12 ft)

= 753 lbs

$F_{c\perp}'$ = 565 psi for No. 2 Southern Pine 2x4

Try 3"x 3" square washers with 5/8" anchor bolts (assume 1/16" oversize washer hole):

$$A_{3x3washers} = (3)^2 - \frac{\pi}{4}\left(\frac{5}{8} + \frac{1}{16}\right)$$

$$= 8.46 \text{ in.}^2$$

P' = $F_{c\perp}A$ = (565 psi)(8.46 in.^{2})

= 4,780 lbs

P_{reqd} < P'

753 lbs < 4,780 lbs (ok)

Table 3.3 Sill Plate to Foundation Connection Shear Load for Seismic

(Example shown is for 30 psf Ground Snow Load)

Description: Calculation of foundation sill plate connection shear loads for seismic.

Procedure: Calculate the required foundation sill plate connection shear loads for seismic in accordance with the Simplified Procedures of *ASCE 7-10* Section 12.14.8 as described in Commentary to Table 2.6.

Background: Tabulated loads are in accordance with Simplified Procedures of *ASCE 7-10* Section 12.14.8.

Example:

Given – Two-story building above grade plane with rectangular dimensions of 36' x 54' (e.g. L/W=1.5) and Seismic Design Category C.

Dead load assumptions: Roof/ceiling = 15 psf, Floor = 12 psf, Partition = 8 psf, Wall = 110 plf for 10' wall height; Ground Snow Load = 30 psf; Lateral force resisting system: wood structural panel shear walls.

In accordance with Table 2.6 and Commentary to Table 2.6, the effective seismic weight is determined as follows:

At the roof/ceiling level:

$$W_{RD} = W_{roof} + W_{(wall\ W)}/2 + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable} = 55{,}446 \text{ lbs}$$

where:

W_{roof} = weight of the roof which includes consideration of 2' overhangs
= 15 psf [(36 ft + 4 ft) (54 ft + 4 ft)]
= 34,800 lbs

$W_{(wall\ W)}$ = weight of exterior walls in W dimension
= 110 plf (36 ft + 36 ft) = 7,920 lbs

$W_{(wall\ L)}$ = weight of exterior walls in L dimension
= 110 plf (54 ft + 54 ft) = 11,880 lbs

$W_{partition}$ = weight of partition walls
= 8 psf (36 ft) (54 ft) = 15,552 lbs

W_{gable} = weight of the gables is based on an assumption that the gable has the same unit weight as the wall construction, is located on the larger building dimension, L, and has a wall to ridge height of 5'
= 110 plf (2) (1/2) (54 ft) / 2 = 2,970 lbs

At the floor diaphragm 2 level:

$$W_{FD2} = W_{floor\ 2} + W_{(wall\ W)} + W_{(wall\ L)} + W_{partition} = 58{,}680 \text{ lbs}$$

where:

W_{floor2} = 12 psf (36 ft) (54 ft) = 23,328 lbs

At the floor diaphragm 1 level:

$$W_{FD1} = W_{floor\ 1} + W_{(wall\ W)}/2 + W_{(wall\ L)} + W_{partition}/2 = 46{,}944 \text{ lbs}$$

where:

W_{floor1} = 12 psf (36 ft) (54 ft) = 23,328 lbs

In calculation of W_{FD1}, contribution to effective seismic weight tributary to FD1 is from mid-height weight of exterior walls and partition walls directly above FD1 in addition to the floor diaphragm weight itself. The contribution of the foundation wall to effective seismic weight at the floor diaphragm 1 level for loading perpendicular to the L dimension is assumed to equal the mid-height weight of the L dimension wall (i.e. $W_{(wall\ L)}/2$).

Total shear at the 1st Floor (FD1) level is determined as:

$$\begin{aligned} V &= (W_{RD} + W_{FD2} + W_{FD1})(1.1)(S_{DS}/R)(0.7) \\ &= (55{,}446 \text{ lbs} + 58{,}680 \text{ lbs} + 46{,}944 \text{ lbs})(1.1)(S_{DS}/R)(0.7) \\ &= 161{,}070 \text{ lbs}\ (1.1)(S_{DS}/R)(0.7) \\ &= 9{,}540 \text{ lbs} \end{aligned}$$

where:

1.1 = vertical force distribution factor for two-story construction in accordance with simplified procedures of *ASCE 7-10* Section 12.14.8

S_{DS} = 0.5 for upper bound of Seismic Design Category C

R = 6.5 for wood frame wood structural panel shear wall systems

0.7 = ASD load factor for seismic loads

Unit shear force in the shorter building dimension W is:

$$\begin{aligned} v &= 9{,}540 \text{ lbs} / 36 \text{ ft} / 2 \\ &= 133 \text{ plf} \end{aligned}$$

(WFCM Table 3.3, GSL = 30)

Footnote 1

Tabulated unit shear forces are applicable for the minimum building dimension W. An appropriate unit shear value for the L dimension can be determined as follows:

$$v_{(L\ dimension)} = \text{(Unit shear for minimum building dimension, W) / (building aspect ratio, L/W)}$$

In lieu of a direct calculation based on building aspect ratio, footnote 1 allows for determination of a unit shear load for the longer L dimension based on unit shear forces tabulated for L/W=1.0.

Footnote 2

Tabulated unit shear loads are based on use of the reference system which is wood frame walls sheathed with wood structural panels. Where other sheathing materials are used, increased seismic loads are applicable in proportion to the ratio of the applicable seismic R values. The 3.25 force increase factor for sheathing materials other than wood structural panels is determined as follows:

Reference R for wood structural panels: 6.5
Alternative R for other sheathing materials: 2
(R for wood structural panels) / (R for other sheathing materials) = 6.5/2 = 3.25

Footnote 3

Footnote 3 allows for adjustment of unit shear loads for other common load cases that may involve larger roof weight, floor weight, or wall weight than was used for tabulated requirement reference conditions. Adjustment of tabulated values for other than the reference condition weights (denoted by the column of factors equal to 1.0) is by use of factors that account for the increase in forces for different weight materials. The largest applicable increase factor for a given building dimension W is tabulated in lieu of tabulating adjustment factors that vary by building aspect ratio.

Assuming all weights remained unchanged in the prior example except that floor weight was increased from 12 psf to 20 psf, an increase in shear load would result and could be calculated directly as follows:

$$W_{floor\ 1} = W_{floor\ 2} = 20\text{ psf (36 ft) (54 ft)} = 38{,}880\text{ lbs}$$

$$= (55{,}446\text{ lbs} + 74{,}232\text{ lbs} + 62{,}546\text{ lbs})(1.1)(S_{DS}/R)(0.7)$$

$$= (192{,}174\text{ lbs})(1.1)(S_{DS}/R)(0.7) = 11{,}382\text{ lbs}$$

Ratio: $V_{(20\text{ psf floor})}/V_{(12\text{ psf floor})}$

$$= 11{,}382\text{ lbs} / 9{,}540\text{ lbs}$$

$$= 1.20 \qquad \text{(WFCM Table 3.3 Footnote 3)}$$

Footnote 4

Sill plate to foundation connection shear loads are tabulated for ground snow loads of 30 psf, 50 psf, and 70 psf. Effective seismic weight includes 20% of ground snow load where ground snow load exceeds 30 psf. For 50 psf and 70 psf tables, effective seismic weight at the roof is increased by 10 psf and 14 psf to account for snow.

Values of S_{DS} are associated with the upper boundaries of SDC A, B, C, D_0, D_1, and D_2 used to tabulate requirements.

Footnote 5

For buildings where 32' and 36' are the minimum building dimension, W, the corresponding length associated with L/W = 2.5 and L/W = 3 exceed the maximum 80 ft building dimension limit. For these cases, the tabulated requirements are based on a maximum building dimension L equal to 80 ft – not the length derived by maintaining L/W = 2.5 or L/W = 3.

Tables 3.3A1-A6 Anchor Bolts – Foundation Sill Plate Connection Resisting Shear Loads from Seismic (prescriptive alternative to Table 3.3)

(Example shown is for 30 psf Ground Snow Load)

Description: Required number of anchor bolts in foundation sill plate connection.

Procedure: Using shear loads calculated in Table 3.3, determine the number of anchor bolts required to resist the load.

Background: Tabulated requirements for number of anchor bolts are provided for 1/2" and 5/8" diameter anchor bolts with lateral design values determined in accordance with the *NDS*.

Example:

Given – Two-story building above grade plane with rectangular dimensions of 36' x 54' (e.g. L/W = 1.5), Seismic Design Category C.

Dead Load Assumptions: Roof/ceiling = 15 psf, Floor = 12 psf, Partition = 8 psf, Wall = 110 plf; Ground Snow Load = 30 psf; Lateral force resisting system: wood structural panel shear walls.

Shear load from Table 3.3 (see WFCM Commentary to Table 3.3): 133 plf

From NDS Table 11E for anchor bolts with G = 0.55, t_s = 1.5", and C_D = 1.6:

> 1/2" diameter anchor bolt lateral ASD design value, Z' = 1,088 lbs
>
> 5/8" diameter anchor bolt lateral ASD design value, Z' = 1,552 lbs

Determine number of 1/2" diameter anchor bolts:

> (133 plf x 36')/(1,088 lbs per bolt) = 4.4 bolts. Round up to 5 bolts. (WFCM Table 3.3A1)

Determine number of 5/8" diameter anchor bolts:

> (133 plf x 36')/(1,552 lbs per bolt) = 3.1 bolts. Round up to 4 bolts. (WFCM Table 3.3A4)

Footnote 1

Tabulated requirements for number of anchor bolts are based on assumptions used in determining shear loads calculated in Table 3.3 (see WFCM Commentary to Table 3.3). Tabulated requirements for number of anchor bolts are based on a rectangular building with dimensions W and L.

Footnote 2

In addition to number of anchor bolts required by this table, requirements for anchor bolts include minimum distance from each end of each plate, maximum spacing of 6' on center, and minimum embedment in accordance with WFCM Section 3.2.1.7.

Footnote 3

Sill plates are assumed to be 2x nominal with G=0.55, which establishes the reference lateral design value parallel to grain for 1/2" and 5/8" diameter anchor bolts. The number of anchor bolts based on use of sill plates with G = 0.5 can be determined by dividing the required shear capacity from Table 3.3 (see WFCM Commentary to Table 3.3) by the bolt lateral ASD design value to arrive at the number of bolts required.

From NDS Table 11E for anchor bolts with G = 0.50, t_s = 1.5", and C_D = 1.6:

> 1/2" diameter anchor bolt lateral ASD design value, Z' = 1,040 lbs
>
> 5/8" diameter anchor bolt lateral ASD design value, Z' = 1,488 lbs

Footnote 4

See WFCM Commentary to Table 3.3 Footnote 2.

Footnote 5

See WFCM Commentary to Table 3.3 Footnote 3.

Footnote 6

See WFCM Commentary to Table 3.3 Footnote 4.

Footnote 7

See WFCM Commentary to Table 3.3 Footnote 5.

3

PRESCRIPTIVE DESIGN

Table 3.3B Bottom Plate to Foundation Connections (Anchor Bolts) Resisting Shear from Seismic

Description: Required number of anchor bolts in bottom plate to foundation connections.

Procedure: Using unit shear capacity associated with reference shear wall construction in the WFCM as the shear load, determine the number of anchor bolts required to resist the load.

Background: Tabulated requirements for anchor bolt spacing for 1/2" and 5/8" diameter anchor bolts are based on shear loads associated with the reference shear wall construction and resistance values for the anchor bolts determined in accordance with the *NDS*.

Example:

Given – Two-story building above grade plane with rectangular dimensions of 36' x 54' (e.g. L/W = 1.5), Seismic Design Category C, and ground snow load, GSL, equal to 30 psf.

From WFCM Table 3.17D for WFCM reference shear wall construction (i.e. 3/8", 7/16", and 15/32" Wood Structural Panels (Blocked), maximum stud spacing 16" on center, 8d common nails - 6" edge spacing):

ASD unit shear capacity of the shear wall = 239 plf

From NDS Table 11E for anchor bolts with G = 0.55, t_s = 1.5", and C_D = 1.6:

1/2" diameter anchor bolt lateral ASD design value, Z' = 1,088 lbs

5/8" diameter anchor bolt lateral ASD design value, Z' = 1,552 lbs

Determine number of 1/2" diameter anchor bolts per bottom plate to resist shear forces (assume bottom plate length = 16 ft):

A bolt is required within 12" from each end of the plate, therefore:

Bolts required = [(239 plf) (16 ft) - 1088 lbs]/(1,088 lbs per bolt) = 2.5 bolts

Calculate spacing:

A bolt is required within 12" from each end of the plate, therefore:

Bolt spacing = [16 ft – 2 ft]/2.5 bolts = 5.57 ft
= 67 in. (WFCM Table 3.3B)

Determine number of 5/8" diameter anchor bolts per bottom plate to resist shear forces (assume bottom plate length = 16 ft):

A bolt is required at each end of the plate, therefore:

Bolts required = [(239 plf) (16 ft) – 1,552 lbs]/(1,552 lbs per bolt) = 1.5 bolts

Calculate spacing:

A bolt is required within 12" from each end of the plate, therefore:

Bolt spacing = [16 ft – 2 ft]/1.5 bolts
= 9.56 ft = 115 in.
Maximum spacing = 72 in. (WFCM Table 3.3B)

Footnote 1

Tabulated anchor bolt spacings are based on unit shears associated with the reference wood structural panel shear wall construction (see WFCM Section 3.4.4.2). For other wall sheathing types, adjustment of anchor bolt spacings is in accordance with adjustment factors in Table 3.17D. In addition to anchor bolt spacing required by this table, requirements for anchor bolts including minimum distance from each end of each plate, maximum spacing of 6' on center, and minimum embedment shall be in accordance with Section 3.2.1.7.

Footnote 2

Sill plates are assumed to be 2x nominal with G = 0.55 which establishes the reference lateral design value parallel to grain for 1/2" and 5/8" diameter anchor bolts. Anchor bolt spacings for other bottom plate conditions, including bottom plate thickness and species, can be accomplished by utilizing the same calculation methodology except with the appropriate bolt lateral design value, Z', determined in accordance with the *NDS*. See Commentary to Tables 3.3A1-A6, Footnote 3 for an example.

Table 3.4 Roof Rafter/Truss Framing to Wall Connection Requirements for Wind Loads

***Description*:** Required uplift, lateral, and shear capacities for roof rafter or truss connections.

***Procedure*:** Using loads calculated in Chapter 2, calculate required capacities for given spacing and span of rafters or trusses.

Background: Uplift loads are based on a worst case 20 degree roof slope with wind parallel to the ridge and include the effect of overhangs. Shear connection requirements are governed by the roof diaphragm capacity.

Example:

Given - 150 mph, 24' roof span, 10' wall height, 16" o.c. rafter/truss spacing, 15 psf roof/ceiling dead load.

Uplift:

From WFCM Table 2.2A:

w_{Uplift} = 236 plf

Required connection uplift capacity:

P_{Uplift} = 236 plf (16 in./12 in./ft)
= 315 lbs (WFCM Table 3.4)

Lateral:

Rafter/truss framing to wall connections resist out-of-plane wind forces on the wall below.

From WFCM Table 2.1:

$w_{Lateral}$ = 148 plf

Required connection lateral capacity:

$P_{Lateral}$ = 148 plf (16 in./12 in./ft)
= 197 lbs (WFCM Table 3.4)

Shear:

Shear loads in Table 3.4 are tabulated for a 10' top plate to ridge height. For the 24' roof span, the 10' top plate to ridge height corresponds to a 10:12 roof pitch. Adjustment factors for other top plate to ridge heights and other wall heights are provided in Footnote 4.

From WFCM Table 2.5B:

w_{roof} = 155 plf (based on 24' roof span 10:12 roof pitch)

S = [155W] / [2L]
= [155 / 2]R
= 77.5R plf

where:

L = Building Length

W = Building Width

R = L/W for wind perpendicular to ridge, or

= W/L for wind parallel to ridge

Required connection shear capacity:

S = 77.5R plf (16 in./12)
= 103R lbs (WFCM Table 3.4)

Footnote 1:

See Commentary for Tables 2.1 and 2.2A for calculations.

Footnote 4:

See Commentary for Footnote 4 in Table 3.2 for a specific example regarding how these adjustment factors are calculated.

3

PRESCRIPTIVE DESIGN

Table 3.4A Rafter and/or Ceiling Joist to Top Plate Lateral and Shear Connection Requirements

Description: Required number of 8d common nails or 10d box nails (toenailed) in each rafter or ceiling joist to top plate connection.

Procedure: Using loads calculated in Table 3.4, calculate the number of nails required to resist the load.

Background: Lateral capacity of 8d common nails and 10d box nails (toenailed) are based on *2012 NDS* nail values as tabulated in AWC's *Design Aid No. 2 - Toenail Connections* which includes adjustments for toenailing, C_{tn} = 0.83. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed. Adjustments for nail penetration, p, are also required when $6D \leq p < 10D$. Tabulated connection requirements are the greater of the lateral and shear connection requirements. Spruce-Pine-Fir with G = 0.42 is assumed for framing members.

Example:

Given - 150 mph, Exposure B, 36' roof span, 10' wall height, 16" o.c. rafter/joist spacing.

Required connection lateral capacity:

From WFCM Table 3.4:

w = 197 lbs

From *Design Aid No. 2*:

Lateral capacity of 8d common nail (D = 0.131″):

= 62 lbs (1.6)

= 99 lbs/nail

Calculate required number of nails based on 8d common:

n = 197 lbs / (99 lbs/nail)
= 1.99 nails
= 2 nails

Required connection shear capacity:

From WFCM Table 3.4:

S = 102R lbs (Connector shear load parallel to the wall)

Shear capacity of 8d common nail (D = 0.131″):

= 62 lbs (1.6)

= 99 lbs/nail

From WFCM Table 3.16A1:

Minimum roof diaphragm length:

L = 13 ft

Calculate required number of nails based on 8d common:

n = [102(36 ft/13 ft)] / (99 lbs/nail)
= 2.8 nails
= 3 nails

Check maximum number of nails based on unblocked diaphragm shear capacity:

Diaphragm capacity = 225 plf (16 in./12 in./ft) = 300 lbs (SDPWS Case 3 Capacity for unblocked diaphragm)

n = 300 lbs/99 lbs/nail
= 3.0 nails

Required number of 8d common nails in each rafter to top plate connection,

n = 3 (WFCM Table 3.4A)

Footnote 4:

See Commentary for Footnote 4 in Table 3.2 for a specific example regarding how these adjustment factors are calculated.

Table 3.4B Shear Walls Resisting Uplift and Shear

Description: Maximum roof spans based on wood structural panel shear wall nailing, top and bottom of panel nailing requirements, and wind speed.

Procedure: Using uplift loads calculated in Table 3.4, determine the amount of additional nailing required to resist uplift for shear wall sheathing and shear wall nailing configurations.

Background: Wood structural panels (WSP) with a minimum thickness of 7/16" and strength axis oriented parallel to studs are permitted to be used in combined uplift and shear applications for resistance to wind forces.

Example:

Given - 150 mph, Exposure B, roof spans of 20' and 24', 7/16" OSB, shear wall nailing of 8d common nails @ 6″ o.c. panel edge spacing.

Uplift:

From WFCM Table 3.4:

Required connection uplift capacity for 20' roof span:

P = 206 lbs for 12" rafter/truss spacing or 206 plf. (WFCM Table 3.4)

From *Special Design Provision for Wind and Seismic (SDPWS)* Table 4.4.1, the allowable design uplift capacity for 7/16" WSP with 8d common nails @ 6" o.c. panel edge spacing are as follows:

Using 3" single row or 6" double row alternate nail spacing at top and bottom plate edges:

= 216 plf 216 plf > 206 plf (ok)

Required connection uplift capacity for 24' roof span:

P = 236 lbs for 12" rafter/truss spacing or 236 plf. (WFCM Table 3.4)

Using 3" single row or 6" double row alternate nail spacing at top and bottom plate edges:

= 216 plf 216 plf < 236 plf (no good)

Maximum roof spans in Table 3.4B are tabulated in increments of 4 feet – which is the same span increment used in Table 3.4.

Therefore, the maximum roof span for the 3" single row or 6" double row alternate nail spacing option is limited to a 20' maximum roof span as shown in Table 3.4B for 7/16" OSB with shear wall nailing of 8d common nails @ 6" o.c. panel edge spacing.

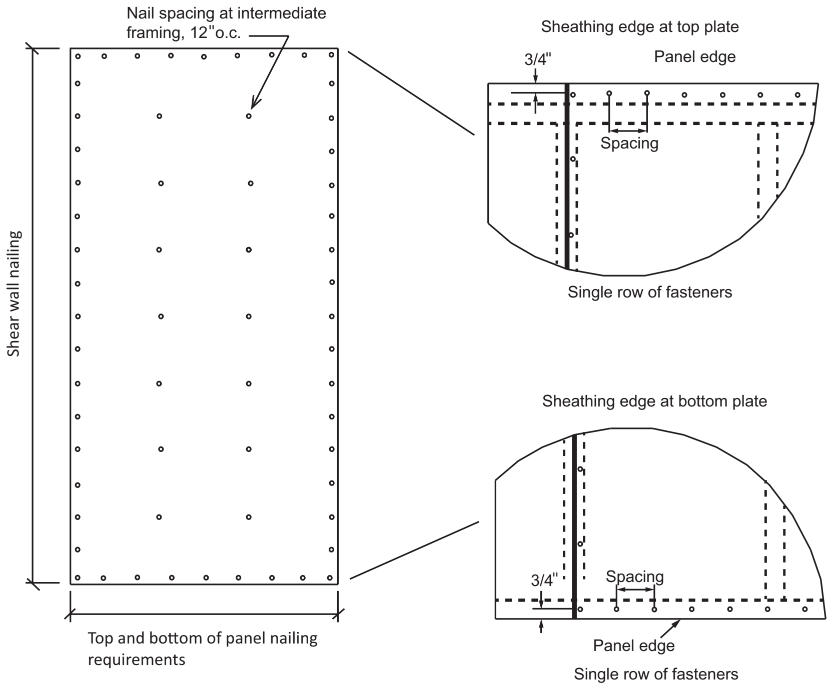
Footnote 2:

For a framing specific gravity of 0.49 or greater, the uplift capacity of sheathing-to-framing nailing is increased by the ratio of the nail value for G = 0.49 framing to the nail value for G = 0.42 per *SDPWS 2008* Table 4.4.2 Footnote 2.

Footnote 3:

For plywood with a species of plies having a specific gravity less than 0.49, the uplift capacity of the sheathing-to-framing nailing is reduced by the ratio of the nail value for wood structural panel sheathing with an effective G = 0.42 to the nail value for wood structural panel sheathing with an effective G = 0.49 per *SDPWS 2008* Table 4.4.2 Footnote 3.

Footnote 4:
From *SDPWS*, an example of a single row of nails with minimum plate thickness, spacing and panel edge distance follows:



Footnote 5:
From SDPWS, an example of a double row of nails with minimum plate thickness, spacing, and panel edge distance follows:

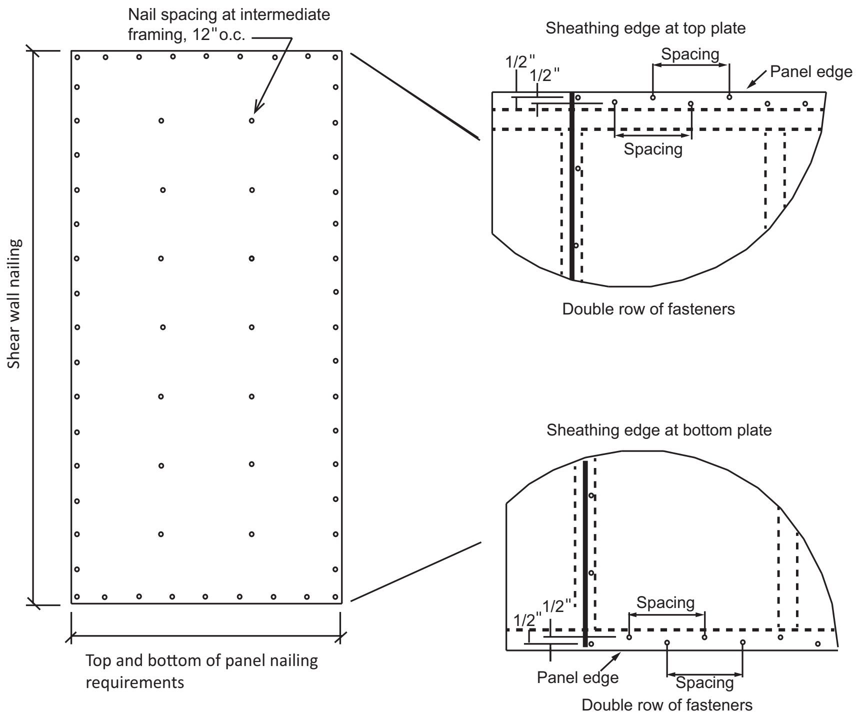


Table 3.4C Rake Overhang Outlooker Uplift Connection Requirements

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 interior truss to calculate the uplift connection load.

Background: Connection requirements are tabulated to be used in conjunction with uplift connector capacities. See Commentary for Table 2.2C for calculations.

Table 3.5 Top and Bottom Plate to Stud Lateral Connection Requirements for Wind Loads

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

Procedure: Compute wind load on studs using C&C pressure coefficients assuming that half of the load is transferred to the top plate and half to bottom plate.

Background: Tabulated loads are intended for use in determining stud to plate connection requirements. Prescriptive requirements are given in Table 3.5A. See Commentary for Table 2.1 for calculations.

Table 3.5A Top and Bottom Plate to Stud Lateral Connections for Wind Loads

(Prescriptive Alternative to Table 3.5)

Description: Required number of 16d common nails or 40d box nails in top and bottom plate to stud connection.

Procedure: Using lateral loads tabulated in Table 3.5, determine the number of nails required to resist the load.

Background: Lateral capacity of 16d common nails and 40d box nails (end nailed) are based on tabulated values in the *2012 NDS*. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed. An end grain adjustment factor, C_{eg} = 0.67 for lateral loads is also required. Spruce-Pine-Fir with G = 0.42 is assumed for framing members.

Example:

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

Unit framing load at top/bottom plate:

From WFCM Table 3.5:

$$w_{Lateral} = 148 \text{ plf}$$

Required connection lateral capacity:

$$P = 148 \text{ plf } (16 \text{ in.}/(12 \text{ in.}/\text{ft}))$$
$$= 197 \text{ lbs}$$

From *2012 NDS* Table 11N:

Reference lateral design value of 16d common nail:

$$Z = (120 \text{ lbs}/\text{nail})$$

$$Z' = ZC_DC_{eg}$$

$$Z' = 120(1.6)(0.67) \text{ lbs}/\text{nail}$$

$$Z' = 128 \text{ lbs}/\text{nail}$$

From *2012 NDS* Table 11N:

Reference Lateral Design Value of 40d box nail:

$$Z = (120 \text{ lbs}/\text{nail})$$

$$Z' = ZC_DC_{eg}$$

$$Z' = 120(1.6)(0.67) \text{ lbs}/\text{nail}$$

$$Z' = 128 \text{ lbs}/\text{nail}$$

Calculate required number of nails:

$$n = (197 \text{ lbs}) / (128 \text{ lbs}/\text{nail})$$
$$= 1.5 \text{ nails}$$
$$= 2 \text{ nails} \qquad \text{(WFCM Table 3.5A)}$$

Footnote 2:

See Commentary for Table 2.1 for calculations.

Table 3.6 Ridge Connection Requirements for Wind

***Description*:** Required capacity of each ridge tension connection.

Procedure: Using loads calculated in Chapter 2, calculate the required capacities for given spacing and span of rafters or trusses. Compute the horizontal load on the roof ridge, sum moments to compute maximum horizontal tension force using MWFRS coefficients.

Background: Tabulated loads are intended for use in determining ridge connection requirements. Prescriptive requirements are provided in WFCM Table A-3.6.

See Commentary for Table 2.2B for calculations.

Table 3.7 Header Connection Requirements for Wind

Description: Required capacity of connections at each end of a header.

Procedure: Using loads calculated in Chapter 2, calculate required connection capacities to resist the load at each end of a header for a given span of rafters or trusses and header span.

Background: Loads at the ends of headers come from uplift and lateral wind forces on the stud wall where the header is located. Lateral load calculations assume that half the load on the 10' wall is carried by the header. Uplift loads are based on worst case 20 degree roof slope with wind perpendicular to the ridge and include the effect of overhangs. Connections at each end of the header must resist loads in both directions.

Example:

Given - 150 mph, Exposure B, 36' roof span, 12' header span, 15 psf roof/ceiling dead load, 10' wall height.

Unit uplift load:

From WFCM Table 2.2A:

$$w_{Uplift} = 326 \text{ plf}$$

Calculate the required header connection uplift capacity:

$$\begin{aligned} P_{Uplift} &= [326(12 \text{ ft})] / 2 \\ &= 1{,}956 \text{ lbs} \\ &= 1{,}981 \text{ lbs} \qquad \text{(WFCM Table 3.7)} \end{aligned}$$

Table 3.7 of the 2012 WFCM was developed using slightly higher uplift loads than those specified in Table 2.2A. The difference results from external pressure coefficients used for the windward overhang. Wind uplift connection loads in Table 3.7 are developed using a C_p of 0.8; wind uplift connections loads in Table 2.2A are developed using the slightly lower C_p of 0.7 per section 28.4.3 of *ASCE 7-10*. For 150 mph in Exposure B, Table 3.7 specifies uplift forces which are 4.3 plf higher than Table 2.2A.

Unit lateral load:

From WFCM Table 2.1:

$$w_{Lateral} = 148 \text{ plf (rounded from 147.56 plf)}$$

Calculate the required header connection lateral capacity:

$$\begin{aligned} P_{Lateral} &= [147.56(12\text{ft})] / 2 \\ &= 885 \text{ lbs} \qquad \text{(WFCM Table 3.7)} \end{aligned}$$

Footnote 1:

See Commentary for Tables 2.1 and 2.2A for calculations.

Footnote 3:

Dead load resisting uplift from walls above headers is:

$$\begin{aligned} &= w_{wall}(L/2)(0.6) \\ &= (121 \text{ plf})(L/2)(0.6) \\ &= 36L \qquad \text{(WFCM Table 3.7)} \end{aligned}$$

Therefore, uplift loads may be reduced by 36 plf times the header span.

Table 3.8 Window Sill Plate Connection Requirements for Wind

***Description*:** Required capacity of connection at each end of window sill plate resisting lateral wind loads.

***Procedure*:** Using loads calculated in Chapter 2, determine required connection capacities to resist the load at each end of a window sill plate.

Background: Loads at the ends of window sill plates come from lateral wind forces transferred from the stud walls and windows into the sill plate. Calculations assume that half the load on a 10 foot wall is carried by the window sill plate.

Example:
Given - 150 mph, 6' window sill span, 10' wall height.

Lateral:

From WFCM Table 2.1:

$w_{Lateral}$ = 148 plf (rounded up from 147.56 plf)

Required sill connection lateral capacity:

$P_{Lateral}$ = 147.56(6 ft) / 2
= 443 lbs (WFCM Table 3.8)

Footnote 1:
See Commentary for Table 2.1 for calculations.

Table 3.9 Rafter/Ceiling Joist Heel Joint Connection Requirements

Description: "Tie force" required at heel joint connection of rafter and ceiling joist.

Procedure: Under live plus dead load, sum moments about one heel joint.

Background: Calculations sum forces at the heel. Note that the connection of rafter to ceiling joist is generally a single shear connection. When forces are very high, the eccentricity of the load at this connection can be substantial and should be considered in design. Overhangs are ignored in the connection calculation as they reduce the amount of thrust. For calculations see Commentary for Table 2.3.

Table 3.9A Rafter/Ceiling Joist Heel Joint Connection Requirements

Description: Required number of 16d common nails or 40d box nails per heel joint splice.

Procedure: Under live plus dead load, sum moments about one heel joint.

Background: Lateral design values for 16d common nails and 40d box nails are based on values calculated in accordance with the *2012 NDS*. A load duration adjustment factor of C_D = 1.15 (snow) is assumed. Spruce-Pine-Fir with G = 0.42 is assumed for framing members.

Example:
Given - 4:12 roof pitch, 36' roof span, 16" o.c. rafter spacing, 10 psf roof dead load, 30 psf ground snow load.

From WFCM Table 2.3:

Unit thrust connection load:

$$w = 894 \text{ plf}$$

Calculate required connection capacity:

$$P = 894 \text{ plf } (16 \text{ in.}/12)$$
$$= 1{,}192 \text{ lbs}$$

From *2012 NDS*:

Reference Lateral Design Value for 16d common nail and 40d box nail:

$$Z = (120 \text{ lbs/nail})$$

$$Z' = ZC_D$$

$$Z' = 120(1.15) \text{ lbs/nail}$$

$$Z' = 138 \text{ lbs/nail}$$

Calculate required number of nails:

$$n = (1{,}192 \text{ lbs}) / (138 \text{ lbs/nail})$$
$$= 8.6 \text{ nails}$$
$$= 9 \text{ nails} \qquad \text{(WFCM Table 3.9A)}$$

Footnote 6:
See Commentary for Table 2.3 for calculations.

Table 3.10 Roof Sheathing Attachment Requirements for Wind Loads

***Description*:** Maximum panel field nail spacing for 8d common nails or 10d box nails loaded in withdrawal.

***Procedure*:** Using suction loads calculated from Chapter 2, determine the required number of nails in roof sheathing to resist those loads.

Background: Withdrawal capacities of common and box nails are based on tabulated values. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed. Spruce-Pine-Fir with G = 0.42 is assumed for framing members. For information on assumed tributary areas see Commentary for Table 2.4.

Example:

Given - 150 mph, Exposure B, perimeter edge zone, 16" o.c. rafter/truss spacing, 7/16" Wood Structural Panel Sheathing.

Required nail spacing to resist sheathing uplift:
From WFCM Table 2.4:

p = 78.3 psf
P = 78.3(16 in./12)
= 104.4 lbs/nail when spaced 12" o.c.

From *2012 NDS*:

Withdrawal capacity of 8d common nail (D = 0.131"; L = 2.5"):

W = 21 lbs/in. (G=0.42)
W' = W(C_D) = 21 lbs/in.(1.6) = 33.6 lbs/in.
p_t = 2.5" – 7/16" = 2.063"
W'p_t = 33.6 lbs/in.(2.063 in.) = 69 lbs/nail

Withdrawal capacity of 10d box nail (D = 0.128"; L = 3.0"):

W = 20 lbs/in. (G=0.42)
W' = W(C_D) = 20 lbs/in.(1.6) = 32 lbs/in.
p_t = 3.0" – 7/16" = 2.563"
W'p = 32 lbs/in.(2.563 in.) = 82 lbs/nail

Calculate panel field nail spacing based on 8d common nail:

spacing = 69 lbs / (104.4 lbs/ft)
= 0.66 ft
= 7.9"

Since calculated nail spacing in panel field is greater than 6" o.c. but less than 12" (max. allowed), 6" is specified.

= 6" o.c. (WFCM Table 3.10)

Calculate the nail spacing required at the gable endwall. At gable endwalls, applied loads to nails shall consider the following load cases:

- C&C uplift
- C&C lateral (out-of-plane)
- Combined C&C uplift + C&C lateral

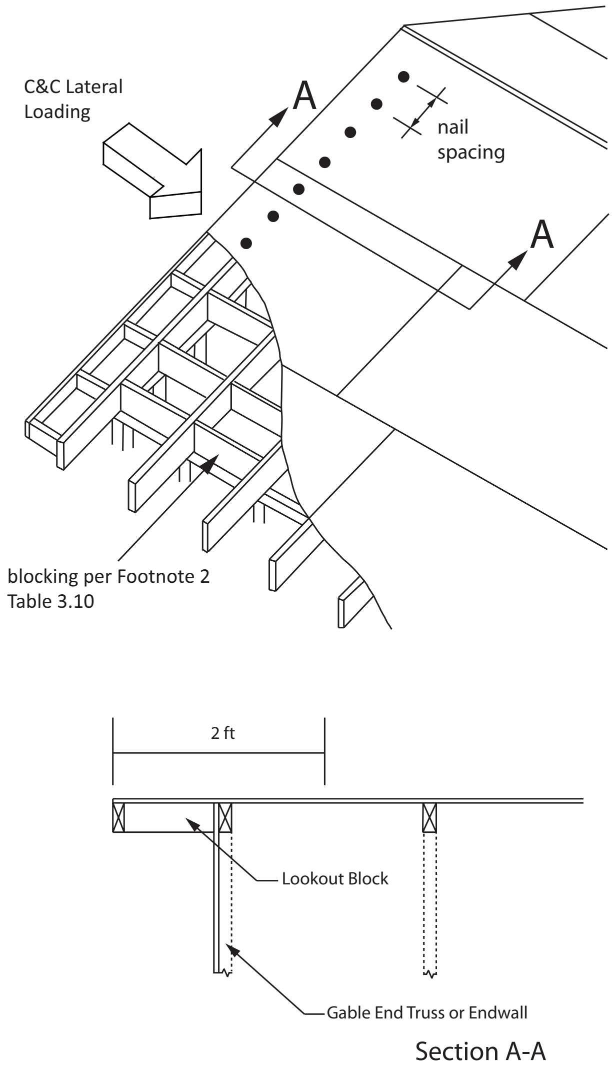


Figure C3.10 Gable endwall nail spacing requirements

Nail spacing based on C&C uplift force (assuming 2 foot tributary width as shown in Figure C3.10):

From WFCM Table 2.4 (Zone 3 overhang):

C&C Uplift Force
= (78.3 psf)(2 ft)
= 157 plf

Withdrawal capacity of 8d common nail:
= 69 lbs/nail

Required nail spacing
= (12 (in./ft) /157 lbs/ft) (69 lbs/nail)
= 5.27 in.

Nail spacing based on C&C lateral force
From WFCM Table 2.1 (20 ft gable end stud height):

C&C Lateral Force
= 250 plf

For wind speeds greater than 130 mph (700 yr return period, 3-second gust), blocking is required which transfers the lateral force to two additional roof joists or rafters. Therefore the C&C lateral force, 250 plf, is divided by 3 in calculating the required nail spacing as follows:

From *2012 NDS*:

Lateral capacity of 8d common nail (D = 0.131"; L = 2.5"):
Z = 67 lbs/nail
$Z' = Z\,(C_D)$
$Z' = (67)(1.6)$
Z' = 107 lbs/nail

Lateral capacity of 10d box nail (D = 0.128"; L = 3.0"):
Z = 65 lbs/nail
$Z' = Z\,(C_D)$
$Z' = (65)(1.6)$
Z' = 104 lbs/nail

Required nail spacing = 12 (in./ft) /[(250/3) lbs/ft)/(104 lbs/nail)]
= 14.98 in.

The minimum required nail spacing is controlled by the C&C uplift force and is equal to 5.27 in.

Where the required nail spacing by calculation exceeds 4" o.c. and is less than 6" o.c., as is the case in this example, a 4" o.c. nail spacing is used.

Check 4" nail spacing based on combined C&C uplift and C&C lateral loading:

The applied C&C lateral force divided by the lateral nail capacity plus the C&C uplift force divided by the nail withdrawal capacity shall not exceed 1.

At 4" o.c., the applied forces are:

Lateral Force
= [250 plf)(4 in./12)]/3
= 28 lbs

Uplift Force
= 157 plf)(4 in./12)
= 52 lbs

Check nail capacity for 10d box:
= (28 lbs)/(104 lbs/nail) + (52 lbs)/(82 lbs/nail)
= 0.27 + 0.63
= 0.9

Check nail capacity for 8d common:
= (28 lbs)/(107 lbs/nail) + (52 lbs)/(69 lbs/nail)
= 0.26 + 0.75
= 1.0

Nail spacing at 4" o.c. is adequate (WFCM Table 3.10)

3
PRESCRIPTIVE DESIGN

Table 3.11 Wall Sheathing and Cladding Attachment Requirements for Wind Loads

***Description*:** Maximum panel field nail spacing for 8d common nails or 10d box nails loaded in withdrawal.

***Procedure*:** Using suction loads for the edge zone calculated from Chapter 2, determine the required number of nails in wall sheathing to resist those loads.

Background: Withdrawal capacities of common and box nails are based on tabulated values. A load duration adjustment factor of C_D = 1.60 (wind/earthquake) is assumed. Spruce-Pine-Fir with G = 0.42 is assumed for framing members. For information on assumed tributary areas see Table 2.4 commentary.

Example:

Given - 150 mph, Exposure B, perimeter edge zone, 16" o.c. stud spacing, 7/16" wood structural panel sheathing.

Calculate the required wall sheathing attachment:

From WFCM Table 2.4:

Calculate suction load on 4' Edge Zone (Zone 5):

$$p = 33.4 \text{ psf}$$

$$p = 33.4 \text{ psf } (16 \text{ in.}/12 \text{ in./ft})$$

$$= 44.5 \text{ plf}$$

From *2012 NDS*:

Withdrawal capacity of 8d common nail (D = 0.131"; L = 2.5"):

$W'p_t$ = 69 lbs/nail (See Commentary on Table 3.10 for calculations)

Withdrawal capacity of 10d box nail (D = 0.128"; L = 3.0"):

$W'p_t$ = 82 lbs/nail (See Commentary on Table 3.10 for calculations)

Calculate panel field nail spacing:

$$\text{spacing} = 69 \text{ lbs} / 44.5 \text{ plf}$$

$$= 1.55 \text{ ft}$$

$$= 18.6 \text{ in.}$$

Use maximum allowed nail spacing in panel field

= 12" o.c. (WFCM Table 3.11)

Table 3.12A Roof Sheathing Requirements for Wind Loads

Description: Minimum sheathing thicknesses for different rafter/truss spacings and wind loads.

Procedure: Using suction loads calculated from Chapter 2, determine the minimum sheathing thickness to resist these loads.

Background: Roof sheathing capacities are tabulated based on sheathing thickness and rafter/truss spacing.

Examples:
Given - 150 mph, Exposure B.

Wood Structural Panel:

Assume a 24" o.c. rafter/truss spacing.

From WFCM Table 2.4 (Zone 3 Overhang):

Roof sheathing suction load:
= 78.3 psf

Taking capacities of wood structural panels from *2008 SDPWS* Table 3.2.2 and applying the (1/1.6) ASD reduction factor per *SDPWS* Section 3.2.3:

Capacity of 3/8" Wood Panel Sheathing
= 105 psf /(1.6)
= 65.6 psf < 78.3 psf

Capacity of 7/16" Wood Panel Sheathing
= 135 psf /(1.6)
= 84.4 psf > 78.3 psf

Use 7/16" Wood Structural Panel (WFCM Table 3.12A)

Board Sheathing:
Assume a 16" o.c. rafter/truss spacing.

Maximum spans and allowable total uniform loads for lumber board sheathing are provided in Table C3.12A. Common board grades of lumber are not assigned design values; however, they have historically been used for floor and roof sheathing in the building code. For use in the WFCM, design capacities for lumber sheathing were estimated from the maximum live load permitted for the maximum span in the building code. For 1" nominal boards (3/4" actual thickness), a 40 psf floor live load is permitted when supports are spaced at 24 inches on center. For 3/4"nominal boards (5/8" actual thickness), a 40 psf load is permitted when supports are spaced at 16 inches on center. For lumber roof sheathing, longer spans are permitted to resist roof live loads and snow loads which have shorter load durations; therefore, the 40 psf roof live load was set as a maximum for 1" nominal boards spaced at 24 inches on center and capacities for 3/4" nominal boards and closer support spacings were then estimated based on this value. The 40% increase is the ratio of the load duration factor for wind (C_D = 1.6) divided by the load duration factor for snow (C_D = 1.15).

Capacity of 5/8" Board Sheathing
= 60 (1.4)
= 84 > 78.3 psf

Use 5/8" Board Sheathing (WFCM Table 3.12A)

Table C3.12A Lumber sheathing spans and allowable total uniform loads

Lumber Grade	Minimum Thickness (in.)	Maximum Rafter/ Truss Spacing (in.)	Rafter/Truss Spacing (in.) 12	16	19.2	24
			Allowable Total Uniform Loads (psf)			
#4 Common or Utility	5/8	16	130	60	40	NP
#4 Common or Utility	3/4	24	170	110	70	40

Note: For wind design, tabulated allowable total uniform loads are permitted to be increased 40%.
NP: Not Permitted

3 PRESCRIPTIVE DESIGN

Table 3.12B Maximum Roof Sheathing Spans for Roof Live and Snow Loads

Description: Minimum sheathing thicknesses for different rafter/truss spacings and roof live loads.

Procedure: Based on roof live load or ground snow load, determine minimum sheathing thickness and maximum rafter/truss spacings.

Background: Roof sheathing capacities are tabulated based on sheathing thickness and rafter/truss spacing.

Example:

Given - 10 psf roof dead load, 30 psf ground snow load, 24" o.c. rafter/truss spacing

p_{dead} = 10 psf

p_{snow} = Ip_g (Note: Unbalanced snow load per *ASCE 7-10*)

= (1.0)(30 psf)

= 30 psf

p_{total} = 40 psf

Capacity of 3/8" sheathing with 24" spans, and the long dimension perpendicular to supports is the minimum from WFCM Supplement Tables S-2A and S-2B.

From WFCM Supplement Table S-2A for bending and shear:

42(1.15) = 48.3 psf (per Footnote 1)

From WFCM Supplement Table S-2B for deflection: 49 psf

Therefore, the value from Table S-2A for bending and shear controls:

48.3 psf > 40 psf

3/8" sheathing installed with long dimension perpendicular to supports is adequate for 24" o.c. rafter/truss spacing.

(WFCM Table 3.12B)

Table 3.13A Wall Sheathing Requirements for Wind Loads

Description: Minimum sheathing thicknesses for different stud spacings and wind loads.

Procedure: Using suction loads calculated from Chapter 2, determine the minimum sheathing thickness to resist these loads.

Background: Wall sheathing capacities are tabulated based on sheathing thickness and stud spacing.

Example:

Given - 150 mph, Exposure B, 16" o.c. stud spacing, sheathing strength axis parallel to supports.

Wall sheathing suction load:

From WFCM Table 2.4 (Zone 5):

$$p = 33.4 \text{ psf}$$

Taking capacities of wood structural panels from the *2008 SDPWS* Table 3.2.1 and applying the (1/1.6) ASD reduction factor per *SDPWS* Section 3.2.1:

Capacity of 7/16" sheathing

$$= 60 \text{ psf}/(1.6)$$

$$= 37.5 \text{ psf}$$

$$37.5 \text{ psf} > 33.4 \text{ psf}$$

7/16" wood structural panels installed with strength axis parallel to supports is adequate to resist 150 mph Exposure B wind pressures. (WFCM Table 3.13A)

Table 3.13B Wall Cladding Requirements for Wind Loads

Description: Minimum cladding (siding) thicknesses for different stud spacings and wind loads.

Procedure: Using suction loads from Table 2.4, determine the minimum siding thickness to resist these loads.

Background: Siding capacities are tabulated based on siding thickness and stud spacing.

Example:

Given - 150 mph, Exposure B, 16" o.c. stud spacing, sheathing strength axis parallel to supports.

Wall cladding suction load:

From WFCM Table 2.4 (Zone 5):

$$p = 33.4 \text{ psf}$$

Taking capacities of wood structural panels from the *2008 SDPWS* Table 3.2.1 and applying the (1/1.6) ASD reduction factor per *SDPWS* Section 3.2.1 (wood structural panel siding is assumed to have the same capacity as wood structural panel sheathing):

Capacity of 7/16" siding

$$= 60 \text{ psf}/(1.6)$$

$$= 37.5 \text{ psf} > 33.4 \text{ psf}$$

7/16" wood structural panel sheathing installed with strength axis parallel to supports is adequate to resist 150 mph Exposure B wind pressures. (WFCM Table 3.13B)

Table 3.14 Maximum Floor Sheathing Spans for Live Loads

Description: Minimum sheathing thicknesses and maximum floor joist spacings for occupancy live loads.

Procedure: Based on 40 psf occupancy live load, determine minimum sheathing thickness and maximum floor joist spacings.

Background: Floor sheathing capacities are tabulated based on sheathing thickness and floor joist spacing.

Example:

Given - 10 psf floor dead load, 40 psf floor live load, 16" o.c. floor joist spacing, long dimension of the sheathing is perpendicular to supports.

$$p_{dead} = 10 \text{ psf}$$
$$p_{live} = 40 \text{ psf}$$
$$p_{total} = 50 \text{ psf}$$

From WFCM Supplement Table S-1:

Capacity of 15/32" Wood Structural Panel sheathing:

$$= 110 \text{ psf} > 50 \text{ psf}$$

Use:

Span rating = 32/16

Minimum thickness = 15/32" (WFCM Table 3.14)

Table 3.15 Minimum Attic Floor/Ceiling Length when Bracing Gable Endwall for Wind Loads

Description: Minimum length of sheathing required from gable end on attic floor or ceiling.

Procedure: Using loads calculated from Chapter 2, determine the minimum sheathing length required to brace the gable end wall against those loads.

Background: Capacities for horizontal unblocked diaphragm assemblies are tabulated based on sheathing thickness, nail size, and panel edge nailing.

Example:
Given - 150 mph, Exposure B, 6:12 roof pitch, 24' roof span, G = 0.42, Unblocked diaphragm.

Determine the length of sheathing required using wood structural panels:

Lateral diaphragm load from wind parallel to ridge:

From WFCM Table 2.5C:

$$w = 101 \text{ plf}$$

Calculate load into the floor / ceiling diaphragm:

$$V = 101 \text{ plf } (24 \text{ ft}) = 2{,}424 \text{ lbs}$$

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

Capacity of structural sheathing:

$$v = 505/(2.0)$$ (15/32 nominal panel thickness, 2" nominal width framing, and Cases 2,3,4,5, 6)

$$= 252.5 \text{ plf}$$

Adjusting for specific gravity of framing per *2008 SDPWS* Table 4.2C Footnote 2:

$$v = 252.5 \text{ plf } [1 - (0.5 - G)] = 252.5\,[0.92] \text{ plf} = 232 \text{ plf}$$

Required structural sheathing attic floor or ceiling diaphragm length:

$$L = [2{,}424 \text{ lbs } / 2] / 232 \text{ plf} = 5.2 \text{ ft}$$

Given a maximum diaphragm aspect ratio of 3:1 for unblocked diaphragms per *SDPWS* Table 4.2.4, for a building end wall width (roof span) equal to 24', the minimum length of attic floor/ceiling diaphragm is 8'.

(WFCM Table 3.15)

Determine the required attic floor/ceiling diaphragm length using Gypsum Wallboard:

From WFCM Supplement Table S-3:

ASD Unit Shear Capacity of gypsum wallboard:

$$v = 70 \text{ plf}$$

Required attic floor/ceiling diaphragm length using gypsum wallboard:

$$L = [2{,}424 \text{ lbs } / 2] / 70 \text{ plf}$$

$$= 18 \text{ ft (rounded up from 17.3 ft)}$$ (WFCM Table 3.15)

Footnote 1:
See Commentary for Table 2.5C Footnote 3 for calculations.

Footnote 4:
From Supplement Table S-3 Footnote 1:

For ceiling framing at 16" o.c. or less, the tabulated capacity for gypsum wallboard is 90 plf.

Dividing the tabulated capacity by the increased capacity:

$$= 70 \text{ plf } / 90 \text{ plf} = 0.78$$ (WFCM Table 3.15)

Table 3.16A1 Roof Diaphragm Limits for Wind

(Applicable to All Roof Slopes with and without Roof Irregularities)

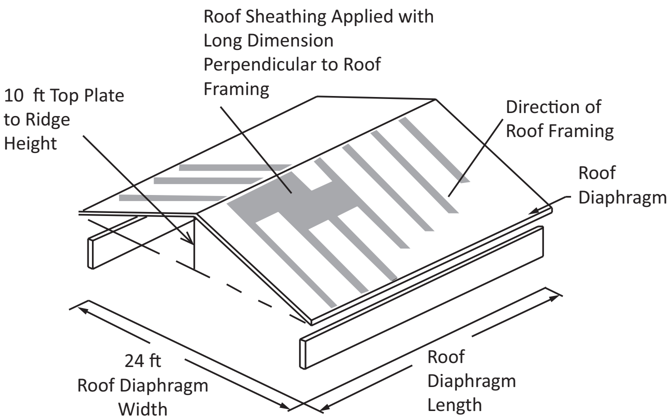
Description: Minimum and maximum roof diaphragm lengths for given roof diaphragm widths and wind speeds based on roof diaphragm limitations.

Procedure: Using the procedures from Chapter 2 for calculating wind loads, determine the minimum and maximum roof diaphragm lengths permitted based on roof diaphragm capacity. To account for possible irregularities in roof shape caused by roof features such as dormers or complex roof framing layouts, wind loads are calculated assuming conservative pressure coefficients regardless of the actual roof pitch associated with the top plate-to-ridge height and roof diaphragm width dimensions.

Background: Capacities for horizontal diaphragm assemblies (roof and floor) are based on sheathing thickness, nail size, panel edge nailing, and supported panel edges.

Example:

Given – 150 mph, Exposure B, 1 story slab-on-grade, 10' maximum top plate to ridge height, 24' roof diaphragm width.



The minimum diaphragm aspect ratio limit is equal to 0.33 (1:3) and the maximum diaphragm aspect ratio limit is equal to 3.0 (3:1).

Calculation of Minimum Roof Diaphragm Length

Calculate minimum roof diaphragm length, L_{Min}, based on the maximum value obtained from the following two calculations:

a) Calculate the roof diaphragm length required based on wind load acting parallel to the ridge (Note: pressure coefficients for wind acting parallel to ridge are constant and do not vary with roof pitch):

Roof diaphragm load for a top plate to ridge height of 10 feet (i.e. 10:12 roof pitch for a roof diaphragm width of 24 feet):

W_{RD} = 155 plf (Table 2.5B)

Calculate load into shear wall:

From roof diaphragm:

V = [155 plf (24 ft)] / 2
= 1,860 lbs

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

v_{RD} = 450 plf / (2.0) (3/8 nominal panel thickness, 2" nominal width framing, and Case 3)

v_{RD} = 225 plf

Minimum roof diaphragm length:

L_{Min} = 1,860 lbs / 225 plf
= 9 ft (rounded up from 8.3 ft)

b) Calculate the minimum roof diaphragm length needed based on aspect ratio limits:

L_{Min} = 24 ft / 3
= 8 ft

The minimum roof diaphragm length is:

L_{Min} = 9 ft (WFCM Table 3.16A1)

3 PRESCRIPTIVE DESIGN

Calculation of Maximum Roof Diaphragm Length

Calculate the maximum roof diaphragm length , L_{Max}, based on the minimum value obtained from the following four calculations:

a) Calculate roof diaphragm length needed based on the wind load acting perpendicular to the ridge:

Roof diaphragm load for a top plate to ridge height of 10 feet utilizing conservative pressure coefficients associated with a 12:12 roof pitch are calculated as follows:

The lateral load on the roof diaphragm will take load from half the wall below and load directly applied to the diaphragm.

Calculate wind forces in the roof diaphragm:

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

where:

p = pressure on the roof/walls

q = 21.15 psf (See Table C1.1)

Calculated the average pressure on the wall for a 12:12 roof pitch:

	Interior Zone		End Zone	
	Windward	Leeward	Windward	Leeward
GC_{pf}	0.56	-0.37	0.69	-0.48
GC_{pi}	0.18	0.18	0.18	0.18
p (psf)	8.0	-11.6	10.8	-14.0

$$\begin{aligned} p_{wall} &= [19.6(L\text{-}X) + 24.8(X)] / L \\ &= [19.6 \text{ psf } (18 \text{ ft}) + 24.8 \text{ psf } (6 \text{ ft})] / 24 \\ &= 20.9 \text{ psf} \end{aligned}$$

where:

L = Building Length (parallel to ridge)

X = End Zone Length

Calculate the average pressure on the roof for a 12:12 roof pitch:

	Interior Zone		End Zone	
	Windward	Leeward	Windward	Leeward
GC_{pf}	0.21	-0.43	0.27	0.53
GC_{pi}	0.18	0.18	0.18	0.18
p (psf)	0.6	-12.9	1.9	-15.0

$$\begin{aligned} p_{roof} &= [13.5(L\text{-}X) + 16.9(X)] / L \\ &= [13.5(18 \text{ ft}) + 16.9(6 \text{ ft})] / 24 \\ &= 14.4 \text{ psf} \end{aligned}$$

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 diaphragm.

$$\begin{aligned} W_{RD} &= 20.9(5 \text{ ft}) + 14.4(10 \text{ ft}) \\ &= 249 \text{ plf} \end{aligned}$$

Calculate load into the shear wall:

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

v_{RD} = 600 plf / 2.0 (3/8 nominal panel thickness, 2" nominal width framing, and Case 1)

v_{RD} = 300 plf

Maximum sidewall length:

$$\begin{aligned} L_{Max} &= [300 (24 \text{ ft})] / [249 \text{ plf } / 2] \\ &= 57 \text{ ft (rounded down from 57.8 ft)} \end{aligned}$$

b) Maximum roof diaphragm length shall not exceed 80' (WFCM limitation):

L_{Max} = 80 ft

c) Calculate maximum roof diaphragm length based on aspect ratio limits:

$$\begin{aligned} L_{Max} &= 24(3) \\ &= 72 \text{ ft} \end{aligned}$$

The maximum roof diaphragm length is:

L_{Max} = 57 ft (WFCM Table 3.16A1)

Table 3.16A2 Roof Diaphragm Limits for Wind

(Applicable to Simple Gable and Hip Roofs with Roof Slopes < 6:12 without Roof Irregularities)

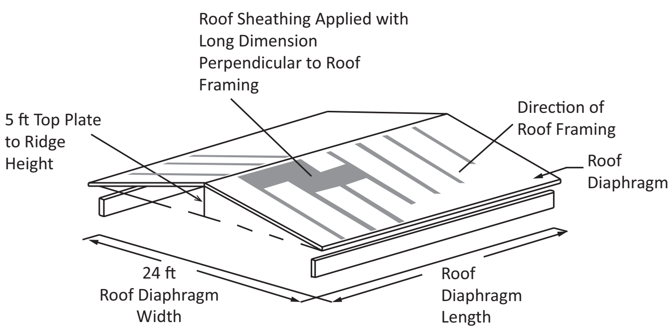
Description: Minimum and maximum roof diaphragm lengths for given roof diaphragm widths and wind speeds based on roof diaphragm limitations.

Procedure: Using procedures from Chapter 2 for calculating wind loads, determine minimum and maximum roof diaphragm lengths permitted based on roof diaphragm capacity. Wind loads are calculated assuming conservative pressure coefficients regardless of the actual roof pitch associated with the top plate to ridge height and roof diaphragm width dimensions.

Background: Capacities for horizontal unblocked diaphragm assemblies are tabulated based on minimum nominal panel thickness of 3/8, 2" nominal width framing, and supported panel edges.

Example:

Given – 150 mph, Exposure B, 1 story slab-on-grade, 5' maximum top plate to ridge height, 24' roof diaphragm width.



The minimum diaphragm aspect ratio limit is equal to 0.33 (1:3) and the maximum diaphragm aspect ratio limit is equal to 3.0 (3:1).

Calculation of Minimum Roof Diaphragm Length

Calculate minimum roof diaphragm length, L_{Min}, based on maximum value obtained from the following two calculations:

a) Calculate the roof diaphragm length required based on wind load acting parallel to the ridge (Note: pressure coefficients for wind acting parallel to ridge are constant and do not vary with roof pitch):

Roof diaphragm load for a top plate to ridge height of 5 feet (i.e. 5:12 roof pitch for a roof diaphragm width of 24 feet):

$$W_{RD} = 116 \text{ plf} \qquad \text{(Table 2.5B)}$$

Calculate load into shear wall:

From roof diaphragm:

$$V = [116 \text{ plf}(24 \text{ ft})] / 2$$
$$= 1{,}392 \text{ lbs}$$

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

$$v_{RD} = 450 \text{ plf} / 2.0 \qquad \text{(Case 3)}$$

$$v_{RD} = 225 \text{ plf}$$

Minimum roof diaphragm length:

$$L_{Min} = 1{,}392 \text{ lbs} / 225 \text{ plf}$$
$$= 6.18 \text{ ft}$$

b) Calculate the minimum roof diaphragm length needed based on aspect ratio limits:

$$L_{Min} = 24 \text{ ft} / 3$$
$$= 8 \text{ ft}$$

The minimum roof diaphragm length is:

$$L_{Min} = 8 \text{ ft} \qquad \text{(WFCM Table 3.16A2)}$$

3 PRESCRIPTIVE DESIGN

Calculation of Maximum Roof Diaphragm Length

Calculate the maximum roof diaphragm length, L_{Max}, based on the minimum value obtained from the following three calculations:

a) Calculate roof diaphragm length based on the wind load acting perpendicular to the ridge:

Roof diaphragm load for a top plate to ridge height of 5 feet utilizing conservative pressure coefficients associated with a 6:12 roof pitch:

W_{RD} = 146 plf (see Commentary on Table 3.16A1)

Calculate load into the shear wall:

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

v_{RD} = 600 plf / 2.0 (Case 1)

v_{RD} = 300 plf

Maximum sidewall length:

L_{Max} = [300(24 ft)] / [146 plf / 2]
= 98.6 ft

b) Maximum roof diaphragm length shall not exceed 80 ft (WFCM limitation):

L_{Max} = 80 ft

c) Calculate maximum roof diaphragm length based on aspect ratio limits:

L_{Max} = 24(3)
= 72 ft

The maximum roof diaphragm length is:

L_{Max} = 72 ft (WFCM Table 3.16A2)

Table 3.16B Floor Diaphragm Limits for Wind

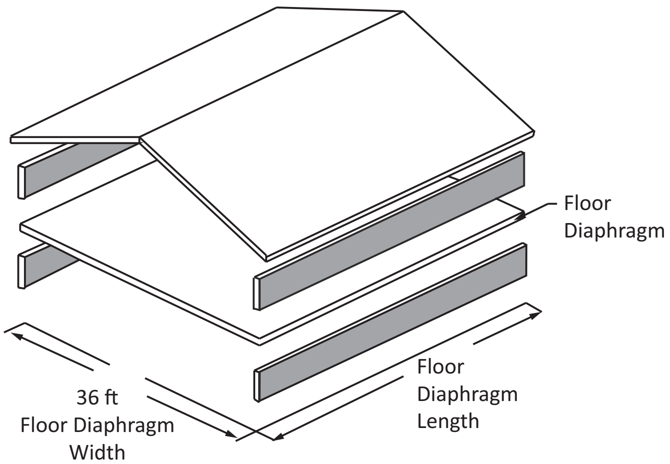
Description: Minimum and maximum floor diaphragm lengths for given floor diaphragm widths and wind speeds based on floor diaphragm limitations.

Procedure: Using the procedures from Chapter 2 for calculating wind loads, determine the minimum and maximum floor diaphragm lengths permitted based on floor diaphragm capacity.

Background: Capacities for horizontal unblocked diaphragm assemblies are tabulated based on a nominal panel thickness of 15/32", 8d common nails, 6" nail spacing at diaphragm boundaries, and supported panel edges.

Example:

Given – 150 mph, Exposure B, 11' tributary wall height (2(10'/2) stud wall height + 1' floor joist depth), and 36' floor diaphragm width.



The minimum diaphragm aspect ratio limit is equal to 0.33 (1:3) and the maximum diaphragm aspect ratio limit is equal to 3.0 (3:1).

Calculation of Minimum Floor Diaphragm Length

Calculate minimum floor diaphragm length, L_{Min}, based on maximum value obtained from the following two calculations:

a) Calculate the floor diaphragm length required based on wind load acting parallel to the ridge:

Floor diaphragm load:

W_{FD} = 171 plf (Table 2.5B)

Calculate load into shear wall:

From floor diaphragm:

V = [171 plf (36 ft)] / 2
= 3,078 lbs

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3 and specific gravity adjustment factor [1 – (0.5 – G)] for G = 0.42 (Footnote 2 Table 4.2C):

v_{FD} = 505 plf [1 – (0.5 – 0.42)] / 2.0 (Case 3)

v_{FD} = 232 plf

Minimum floor diaphragm length:

L_{Min} = 3,078 lbs / 232 plf
= 14 ft (rounded up from 13.3 ft)

b) Calculate the minimum floor diaphragm length needed based on aspect ratio limits:

L_{Min} = 24 ft / 3
= 8 ft

The minimum floor diaphragm length is:

L_{Min} = 14 ft (WFCM Table 3.16B)

3

PRESCRIPTIVE DESIGN

Calculation of Maximum Floor Diaphragm Length

Calculate the maximum floor diaphragm length, L_{Max}, based on the minimum value obtained from the following three calculations:

a) Calculate floor diaphragm length based on the wind load acting perpendicular to the ridge:

Floor diaphragm load:

W_{FD} = 251 plf (Table 2.5A)

Calculate load into shear wall:

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3 and specific gravity adjustment factor [1 – (0.5 – G)] for G = 0.42 (Footnote 2 Table 4.2C):

v_{FD} = 670 plf [(1 – (0.5 – 0.42)] / 2.0 (Case 1)

v_{FD} = 308 plf

Maximum sidewall length:

L_{Max} = [308 (36 ft)] / [251 plf / 2]

= 88.4 ft

b) Maximum floor diaphragm length shall not exceed 80 ft (WFCM limitation):

L_{Max} = 80 ft

c) Calculate maximum floor diaphragm length based on aspect ratio limits:

L_{Max} = 36 (3)

= 108 ft

The maximum roof diaphragm length is:

L_{Max} = 80 ft (WFCM Table 3.16B)

Tables 3.16C1-C3 Diaphragm Limits for Seismic

Description: Maximum diaphragm dimension, L, for seismic.

Procedure: Using shear loads at each level, calculated in accordance with Table 2.6 and Commentary to Table 2.6, determine the maximum diaphragm dimension, L, for seismic such that the allowable unit shear capacity of the reference diaphragm construction is not exceeded.

Background: Tabulated requirements for maximum diaphragm dimension, L, are provided for the reference unblocked diaphragm construction in the WFCM. The aspect ratio of the diaphragm is limited to 3:1 and a maximum dimension, L, not to exceed 80 ft.

Dead Load Assumptions: Roof/Ceiling = 15 psf, Floor = 12 psf, Partition = 8 psf, Wall = 110 plf; Ground Snow Load = 30 psf; Lateral force resisting system: wood structural panel shear walls.

Example:

Given – Two-story building above grade plane with rectangular dimensions of 36' x 80' and Seismic Design Category C.

Roof/Ceiling

For roof diaphragm construction of 3/8" wood structural panel sheathing (unblocked), 8d common nails - 6" edge spacing:

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

v_{RD} = 430 plf / 2.0 (Case 1)

v_{RD} = 215 plf

The effective seismic weight resisted by the roof/ceiling diaphragm assuming loading perpendicular to the L dimension:

$$W_{RD} = W_{roof} + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}$$
$$= 75{,}120 \text{ lbs}$$

where:

W_{roof} = weight of the roof which includes consideration of 2′ overhangs
= 15 psf [(36 ft + 4 ft) x (80 ft + 4 ft)]
= 50,400 lbs

$W_{(wall\ L)}$ = weight of exterior walls in the L dimension
110 plf (80 ft + 80 ft) = 17,600 lbs

$W_{partition}$ = weight of partition walls
= 8 psf (36 ft) (80 ft) = 23,040 lbs

W_{gable} = weight of the gable end wall
= 110 plf (2) (1/2) (80 ft) / 2 = 4,400 lbs

V = (75,120 lbs) (1.1) (S_{DS}/R) (0.7) = 4,450 lbs

v = 4,450lbs / (2W) = 62 plf

Factor for 2 story: 62/0.92 = 67 plf

67 plf < 215 plf OK

Maximum diaphragm dimension, L, tabulated = 80 ft (WFCM Table 3.16C1)

Diaphragm load ratio = 67/215 = 0.31 (WFCM Table 3.16C1)

Floor

For floor diaphragm construction of 15/32" wood structural panel sheathing (unblocked), G = 0.42 framing, 8d common nails - 6" edge spacing:

Using *2008 SDPWS* Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms (unblocked) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.2.3:

v_{FD} = 480 plf / 2.0 (Case 1)

v_{FD} = 240 plf

Specific Gravity Adjustment Factor = [1 - (0.5 - G)] = 0.92 (SDPWS Table 4.2C Footnote 2)

v_{FD} = 240 plf x 0.92 = 221 plf

3

PRESCRIPTIVE DESIGN

The effective seismic weight resisted by the floor diaphragm assuming loading perpendicular to the L dimension:

$$W_{FD} = W_{floor} + +W_{(wall\ W)}/2 + W_{(wall\ L)} + W_{partition}$$
$$= 79{,}160 \text{ lbs}$$

where:

W_{floor} = weight of the roof which includes consideration of 2′ overhangs
= 12 psf (36 ft) (80 ft) = 34,560 lbs

$W_{(wall\ W)}$ = weight of exterior walls in W dimension
= 110 plf (36 ft + 36 ft) = 7,920 lbs

$W_{(wall\ L)}$ = weight of exterior walls in L dimension
110 plf (80 ft + 80 ft) = 17,600 lbs

$W_{partition}$ = weight of partition walls
= 8 psf (36 ft) (80 ft) = 23,040 lbs

V = (79,160 lbs) (1.1) (S_{DS}/R) (0.7) = 4,689 lbs

v = 4,689 lbs / (2W) = 65 plf

Factor for 2-story: 65/0.92 = 71 plf

71 plf < 221 plf OK

Maximum diaphragm dimension, L, tabulated = 80 ft
(WFCM Table 3.16C1)

Diaphragm load ratio = 71/221 = 0.32
(WFCM Table 3.16C1)

Footnote 1
The tabulated requirements are based on reference construction of roof and floor diaphragms with long dimension of panels perpendicular to framing members and joints staggered (i.e. Case 1, unblocked diaphragm per *SDPWS*).

Footnote 2
Tabulated maximum length requirements can be adjusted to determine the maximum length for other cases. The reference case is for 3-story construction, floor weights = 12 psf, roof/ceiling = 15 psf, partition = 8 psf, and wall=110 plf. The maximum aspect ratio of unblocked diaphragms is limited to 3:1 and L is limited to 80 ft.

Footnote 3
The diaphragm load ratio is used to account for other than reference conditions associated with the tabulated load ratio.

The number of stories factor accounts for vertical distribution factors associated with 1, 2, and 3 story buildings in accordance with *ASCE 7-10*. The reference case for Table 3.16C is a three-story building, hence, the adjustment for three-story is 1.0. Reduced shears are associated with two-story and one-story buildings. The two-story factor of 0.92 results from the ratio of 1.1/1.2 and the one-story factor of 0.83 results from the ratio of 1.0/1.2 (see WFCM Commentary to Table 2.6).

Tabulated requirements are based on use of the reference vertical system which is wood frame walls sheathed with wood structural panels. Where other sheathing materials are used, increased seismic loads are applicable in proportion to the ratio of the applicable seismic R values (See Commentary to Table 3.3).

The diaphragm shear adjustment factor allows for adjustment of unit shear loads for other common load cases that may involve larger roof weight, floor weight, or wall weight than used for the reference conditions used in the tabulated requirements. Adjustment of tabulated values for other than the reference condition weights (denoted by the column of factors equal to 1.0) is by use of factors that account for the increase in forces for different weight materials. The largest applicable increase factor for a given building dimension W is tabulated rather than providing adjustment factors that vary by building aspect ratio.

Assuming all weights remained unchanged in the prior example except that roof weight is increased from 15 psf to 25 psf, an increase in shear load would result and could be calculated directly as follows:

$$W_{RD} = W_{roof} + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}$$
$$= 108{,}720 \text{ lbs}$$

where:

W_{roof} = weight of the roof which includes consideration of 2′ overhangs
= 25 psf [(36 ft + 4 ft) (80 ft + 4 ft)]
= 84,000 lbs

V = (108,720) (1.1) (S_{DS}/R) (0.7) = 6,440 lbs

v = 6,440 lbs / (2W) = 90 plf

Factor for 2-story: 90/0.92 = 98 plf

98 plf < 215 plf OK

Ratio: $V_{(25\ psf\ roof)}/V_{(15\ psf\ roof}$

= 98 plf / 67 plf
= 1.46 (WFCM Table 3.3 Footnote 3)

Footnote 4

Requirements are tabulated for ground snow load conditions of 30 psf, 50 psf, and 70 psf. Effective seismic weight includes 20% of ground snow load where ground snow load exceeds 30 psf. For 50 psf and 70 psf tables, effective seismic weight at the roof is increased by 10 psf and 14 psf to account for snow.

Values of S_{DS} are associated with the upper boundaries of SDC A, B, C, D_0, D_1, and D_2 used to tabulate requirements.

Table 3.17A Segmented Shear Wall Sheathing Requirements for Wind

Description: Minimum full-height sheathing length for wind.

Procedure: Using shear loads calculated in accordance with Chapter 2, and assuming a 10′ top plate to ridge height, determine the minimum required length of shear walls for wind resistance.

Background: Tabulated requirements for minimum full height sheathing length are provided for resistance to wind forces and are based on the reference shear wall construction in the WFCM.

Example:

Given - 150 mph, Exposure B, 2-story building, 10' wall height, 10' top plate to ridge height, and 24' building length. Reference shear wall construction is 3/8" wood structural panel exterior wall sheathing applied long dimension perpendicular to studs with all panel edges blocked or backed by framing, G = 0.42 framing, maximum studs spacing 16" o.c., 8d common nails – 6" panel edge nail spacing, and ½" gypsum wallboard interior wall sheathing, 5d cooler nails – 7" panel edge nail spacing.

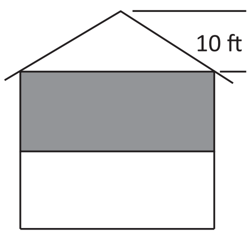
ASD unit shear capacity of the shear wall for resisting wind loads = 436 plf (WFCM Table 3.17D)

Diaphragm loads (see Commentary for Footnote 4 in Table 3.2):

$$w_{RD} = 246 \text{ plf (roof diaphragm)}$$
$$w_{FD} = 230 \text{ plf (floor diaphragm)}$$

Calculate minimum length of full height sheathing:

Top floor:



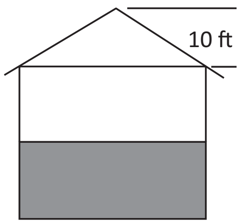
Load into shear wall:

$$V = [246 \text{ plf} (24 \text{ ft})] / 2 = 2{,}952 \text{ lbs}$$

Minimum length of full height sheathing:

$$L_{Min} = V/v_S = 2{,}952 \text{ lbs} / 436 \text{ plf} = 6.8 \text{ ft} \quad \text{(WFCM Table 3.17A)}$$

Bottom floor:



Load into shear wall:

$$V = [(246 \text{ plf} + 230 \text{ plf})(24 \text{ ft})] / 2 = 5{,}712 \text{ lbs}$$

Minimum length of full height sheathing:

$$L_{Min} = V/v_S = 5{,}712 \text{ lbs} / 436 \text{ plf} = 13.1 \text{ ft} \quad \text{(WFCM Table 3.17A)}$$

Footnote 4

See Commentary for Footnote 4 in Table 3.2.

Note: There is no Table 3.17B in the 2012 WFCM. Since tabulated lengths in Table 3.17A assume exterior shear walls perpendicular to either building dimension, L or W, the wind perpendicular to ridge case from ASCE 7-10 is used (worst case) since the ridge may be located along either the L or W dimension. This approach is also intended to facilitate better use with the inscribed method which may have the ridge oriented in multiple directions (i.e. L-shape or U-shape). In the 2001 WFCM, two separate tables (Table 3.17A and B) were used assuming exterior shear walls for a rectangular-shaped building with a gable-end roof configuration – one for wind perpendicular to ridge and one for wind parallel to ridge. Tables 3.17C1-C3 and Tables 3.17D-F were not renumbered in order to maintain continuity for users between the 2001 and 2012 WFCM.

Tables 3.17C1-C3 Segmented Shear Wall Sheathing Requirements

Description: Minimum full-height sheathing length for seismic.

Procedure: Using shear loads at each level, calculated in accordance with Table 2.6 and Commentary to Table 2.6, determine the minimum required length of shear walls for seismic resistance.

Background: Tabulated requirements for minimum full height sheathing length are provided for resistance to seismic shear forces and are based on the reference shear wall construction in the WFCM.

Dead Load Assumptions: Roof/Ceiling = 15 psf, Floor = 12 psf, Partition = 8 psf, Wall = 110 plf; Ground Snow Load = 30 psf; Lateral force resisting system: wood structural panel shear walls.

Example:

Given – Two-story building above grade plane with rectangular dimensions of 36' x 54' (e.g. L / W=1.5) and Seismic Design Category C.

Reference shear wall construction is 3/8" wood structural panel sheathing applied with the long dimension perpendicular to studs with all panel edges blocked or backed by framing, G = 0.42 framing, maximum stud spacing 16" on center, 8d common nails - 6" edge spacing:

Using *2008 SDPWS* Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls (see Footnote 2) and applying the (1/2.0) ASD reduction factor per *SDPWS* Section 4.3.3:

v_{RD} = 520 plf / 2.0
v_{RD} = 260 plf

Specific Gravity Adjustment Factor = [1 - (0.5 - G)] = 0.92 (SDPWS Table 4.3A Footnote 3)

v_{FD} = 260 plf x 0.92 = 239 plf

From Commentary for Table 3.3, the effective seismic weight resisted by the shear wall below the roof/ceiling assuming loading perpendicular to the L dimension:

$W_{RD} = W_{roof} + W_{(wall\ W)}/2 + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}$
= 55,446 lbs

where:

W_{roof} = weight of the roof which includes consideration of 2' overhangs
= 15 psf [(36 ft + 4 ft) (54 ft + 4 ft)]
= 34,800 lbs

$W_{(wall\ W)}$ = weight of exterior walls in W dimension
= 110 plf (36 ft + 36 ft) = 7,920 lbs

$W_{(wall\ L)}$ = weight of exterior walls in L dimension
= 110 plf (54 ft + 54 ft) = 11,880 lbs

$W_{partition}$ = weight of partition walls
= 8 psf (36 ft) (54 ft) = 15,552 lbs

W_{gable} = weight of the gable
= (0.5) (110 plf) (54 ft) = 2,970 lbs

Note: Gable weight is estimated as (0.5) x (wall weight) x (greater of building dimension W or L) and contributes to total weight at the roof level.

V = (55,446 lbs) (S_{DS}/R) (1.1) (0.7) = 3,284 lbs

Shear wall length: 3,284 lbs / 2 / 239 plf = 6.9 ft

Factor for 2 story:

6.9 / 0.92 = 7.5 ft (WFCM Table 3.17C1)

Walls below roof/ceiling and 1 floor:

$W_{FD2} = W_{RD} + W_{floor\ 2} + W_{(wall\ W)} + W_{(wall\ L)} + W_{partition}$

= 114,126 lbs

where:

W_{floor2} = 12 psf (36 ft) (54 ft) = 23,328 lbs

$W_{(wall\ W)}$ = weight of exterior walls in W dimension
= 110 plf (36 ft + 36 ft) = 7,920 lbs

$W_{(wall\ L)}$ = weight of exterior walls in L dimension
= 110 plf (54 ft + 54 ft) = 11,880 lbs

$W_{partition}$ = weight of partition walls
= 8 psf (36 ft) (54 ft) = 15,552 lbs

V = (114,126 lbs) (1.1) (S_{DS}/R) (0.7) = 6,760 lbs

Shear wall length:

$= 6{,}760 \text{ lbs} / 2 / 239 \text{ plf} = 14.1 \text{ ft}$

Factor for 2-story:

$= 14.1 / 0.92 = 15.4 \text{ ft}$

Footnote 1

The tabulated requirements are applicable for each wall of a rectangular building having dimension W and L.

Footnote 2

The number of stories adjustment accounts for vertical distribution factors associated with 1, 2, and 3 story buildings in accordance with *ASCE 7-10*. The reference case for Table 3.16C is three-story construction, hence, no further story adjustment is needed for three-story buildings (i.e. adjustment factor is 1.0). Reduced shears are associated with two-story and one-story buildings. The two-story factor of 0.92 results from the ratio of 1.1/1.2 and the one-story factor of 0.83 results from the ratio of 1.0/1.2 (see WFCM Commentary to Table 2.6).

Footnote 3

Tabulated maximum length requirements can be adjusted to determine the maximum length for other cases. The reference case is for three-story construction, floor weights = 12 psf, roof/ceiling = 15 psf, partition = 8 psf, and wall = 110 plf. The maximum aspect ratio of unblocked diaphragms is limited to 3:1 and L should not exceed 80 ft.

Tabulate requirements are based on use of the reference vertical system which is wood frame walls sheathed with wood structural panels. Where other sheathing materials are used, increased seismic loads are applicable in proportion to the ratio of the applicable seismic R values (See Commentary to Table 3.3).The diaphragm load ratio is used to account for other than reference conditions associated with the tabulated load ratio.

Footnote 4

The diaphragm shear adjustment factor allows for adjustment of unit shear loads for other common load cases that may involve larger roof weight, floor weight, or wall weight other than the reference conditions used in the tabulated requirements. Adjustment of tabulated values for other than the reference condition weights (denoted by the column of factors equal to 1.0) is by use of factors that account for the increase in forces for different weight materials. The largest applicable increase factor for a given building dimension W is tabulated rather than providing adjustment factors that vary by building aspect ratio.

Assuming all weights remained unchanged in the prior example except that roof weight is increased from 15 psf to 25 psf, an increase in shear load would result and could be calculated directly as follows:

$$W_{RD} = W_{roof} + W_{(wall\ W)}/2 + W_{(wall\ L)}/2 + W_{partition}/2 + W_{gable}$$
$$= 78{,}646 \text{ lbs}$$

where:

W_{roof} = weight of the roof which includes consideration of 2′ overhangs

$= 25 \text{ psf} [(36 \text{ ft} + 4 \text{ ft})(54 \text{ ft} + 4 \text{ ft})]$

$= 58{,}000 \text{ lbs}$

Ratio:

$V_{(25\ psf\ roof)}/V_{(15\ psf\ roof)} = 78{,}646/55{,}446$

$= 1.42$ (WFCM Table 3.3 Footnote 3)

Footnote 5

Requirements are tabulated for ground snow loads conditions of 30 psf, 50 psf, and 70 psf. Effective seismic weight includes 20% of ground snow load where ground snow load exceeds 30 psf. For 50 psf and 70 psf tables, effective seismic weight at the roof is increased by 10 psf and 14 psf to account for snow.

Values of S_{DS} are associated with the upper boundaries of SDC A, B, C, D_0, D_1, and D_2 used to tabulate requirements.

Footnote 6

For buildings where 32' and 36' are the minimum building dimension, W, the corresponding length associated with L / W = 2.5 and L / W = 3 exceed the maximum 80 ft building dimension limit in the WFCM. For these cases, the tabulated requirements are based on a maximum building dimension L equal to 80 ft – not the length derived by maintaining L / W = 2.5 or L / W = 3.

Table 3.17D Shear Wall Assembly Sheathing Type Adjustment Factors

***Description*:** Determine the required adjustment for various wall assemblies relative to the default 7/16" wood structural panels used for calculations in Chapter 3.

***Procedure*:** Calculate shear strength for the reference wall, which has studs 16" o.c., blocked 7/16" wood structural panels (WSP) nailed at 6" edge & 12" field on the exterior, and unblocked ½" gypsum nailed at 7" edge and 10" field relative to a tabulated shear wall assembly. Calculate the ratio of other wall assemblies in comparison to the reference wall.

Background: Shear wall assembly capacities are per the 2008 *Special Design Provisions for Wind and Seismic* (SDPWS) and reduced to ASD unit shear capacities by dividing by 2.0. The wood structural panel assembly capacity is also reduced by the specific gravity adjustment factor per a footnote in the SDPWS table. Exterior and interior shear wall assembly resistances are combined differently for wind and seismic forces.

Specific Gravity Adjustment Factor

= [1- (0.5 - G)]

= [1 - (0.5 - 0.42)]

= 0.92 for G = 0.42 (See NDS Table 11.3.3A)

Wind Capacity:

Capacity of 7/16" WSP assembly from SDPWS Table 4.3A (see Footnotes 1, 2, and 3)

= (730 plf)(0.92) / (2.0)

= 336 plf

Capacity of ½" gypsum assembly from SDPWS Table 4.3C

= (200 plf) / (2.0)

= 100 plf

Total Capacity = summation of two capacities

= 336 plf + 100 plf

= 436 plf (WFCM Table 3.17D)

Seismic Capacity:

Capacity of 7/16" WSP assembly from SDPWS Table 4.3A (see Footnotes 1, 2, and 3)

= (520 plf)(0.92) / (2.0)

= 239 plf with R = 6.5

Capacity of ½" gypsum assembly from SDPWS Table 4.3C = (200 plf) / (2.0) = 100 plf with R = 2.0

Total Capacity is the same as the assembly that has the greater capacity multiplied by its R value.

(100 plf)(2.0) = 200 < (239 plf)(6.5) = 1554
Total Capacity = 239 plf (WFCM Table 3.17D)

Example calculations for length adjustment for other sheathing materials or nail spacing:

Example 1.

From SDPWS and following similar calculation as above:

For studs 16" o.c. and a blocked 7/16" WSP on the exterior nailed at 4" (Edge) & 12" (Field) and unblocked ½" gypsum on the interior nailed at 7" (Edge) and 10" (Field)

Shear Capacity for Wind

= (1065 plf)(0.92) / (2.0) + (200 plf) / (2.0)

= 590 plf

Shear Capacity for Seismic

= (760 plf)(0.92) / (2.0)

= 350 plf

Length Adjustment Factors:

Wind = 436 plf / 590 plf

= 0.74 (WFCM Table 3.17D)

Seismic = 239 plf / 350 plf

= 0.68 (WFCM Table 3.17D)

Example 2.
From SDPWS and following similar calculation as above:

For studs 16" o.c. and blocked 25/32" structural fiberboard on the exterior nailed at 3" (Edge) and unblocked ½" gypsum on the interior nailed at 7" (Edge) and 10" (Field)

Shear Capacity for Wind

= (645 plf)(0.92) / (2.0) + (200 plf) / (2.0)

= 397 plf

Shear Capacity for Seismic

= (460 plf)(0.92) / (2.0)

= 212 plf with R = 2.0

Length Adjustment Factors:

Wind = 436 plf / 397 plf

= 1.10 (WFCM Table 3.17D)

Since the R factor has also changed, it needs to be included

Seismic = (239 plf)(6.5) / (212 plf)(2.0)

= 3.66 (WFCM Table 3.17D)

Table 3.17E Perforated Shear Wall – Full Height Sheathing Length Adjustments

Description: Segmented shear wall, full height sheathing lengths are to be multiplied by the tabulated perforated shear wall length adjustment factors to determine the required sheathing lengths of perforated shear walls.

Procedure: Calculate the adjustment for determining the minimum length of sheathing for shear walls with hold-downs at each end of the wall, with or without openings.

Background: Procedures for determining the following full height sheathing length adjustments are given in the following papers: *Perforated Shearwall Design Approach* by Douglas and Sugiyama, ASAE, Winter 1994, *Perforated Shearwall Design* by Stone, Line, and Weeks, *Perforated Shearwall Design Method* by Line, Douglas, 1996.

$$\beta = \frac{L_{full-height}}{L_{wall}}$$

The Sugiyama Equation is:

$$L_{full-height} = L_{wall}\frac{1-\left(\frac{H_{wall}}{3H_{opening}}\right)\left(\frac{V_{wall}}{V_{req}}\right)}{1-\left(\frac{H_{wall}}{3H_{opening}}\right)}$$

The perforated shear wall length adjustment factor tabulated in Table 3.17E is equal to the ratio of betas:

Perforated Shear Wall Length Adjustment Factor $= \frac{\beta_2}{\beta_1}$

Where:

$\beta_1 = \beta$ of a Segmented Wall

$\beta_2 = \beta$ of a Perforated Wall

Derivation from the Sugiyama Equation yields:

Perforated Shear Wall Length Adjustment Factor

$$= \frac{1}{\frac{(1-\beta_1)}{3\left(\frac{H_{opening}}{H_{wall}}\right)}+\beta_1}$$

Example:

The Perforated Shear Wall Length Adjustment Factor on a segmented shear wall with 70% full height sheathing and an opening height of 2/3 the height of the wall is equal to:

Perforated Shear Wall Length Adjustment Factor

$$= \frac{1}{\frac{(1-0.70)}{3\left[\frac{\left(\frac{2H}{3}\right)}{H}\right]}+0.70}$$

$= 1/0.85 = 1.18$

Perforated Shear Wall Length Adjustment Factor = 1.18
(WFCM Table 3.17E)

Table 3.17F Segmented and Perforated Shear Wall Hold-down Capacity Requirements

Description: Calculation of maximum hold-down connection capacities.

Procedure: Multiply the ASD unit shear capacity for the reference shear wall construction times the wall height to determine the maximum hold-down capacity.

Background: Hold-downs are utilized to prevent uplift resulting from overturning moments developed in shear walls.

Example:

Given -10' wall height and the ASD unit shear capacity for the reference shear wall construction from WFCM Table 3.17D (where Sheathing Type Adjustment Factor = 1.0) as follows:

Hold-down Capacity (Seismic):

ASD unit shear capacity = 239 plf

Hold-down Capacity = (239 plf)10 ft

= 2,390 lbs (WFCM Table 3.17F)

Hold-down Capacity (Wind):

ASD unit shear capacity = 436 plf

Hold-down Capacity = (436 plf)10 ft

= 4,360 lbs (WFCM Table 3.17F)

Tables 3.18A & B Floor Joist Spans for Common Lumber Species

(Example shown is for Table 3.18A, residential sleeping areas)

Description: Calculation of maximum permissible span for lumber floor joist.

Procedure: Perform moment and deflection calculations and take the minimum span.

Background: Loads typical for residential sleeping and non-sleeping areas.

Example:

Given - 2x10, No. 2 Douglas Fir-Larch, 16" o.c., $F_b = 900$ psi, E = 1.6 million psi, $C_F = 1.1$, $C_r = 1.15$, $C_D = 1.0$, 30 psf live load, 10 psf dead load, $\Delta_{LL} \leq L/360$.

$$w_{dead} = 10\text{ psf}(16\text{ in.}/12) = 13.33\text{ plf}$$

$$w_{live} = 30\text{ psf}(16\text{ in.}/12) = 40\text{ plf}$$

$$w_{total} = 13.33 + 40 = 53.33\text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F$$
$$= 900(1.0)(1.15)(1.1) = 1{,}138\text{ psi}$$

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

$$L = \sqrt{\frac{8S\left(F_b'\right)}{w_{total}}}$$

$$= \sqrt{\frac{8(21.4)1{,}138}{\frac{53.33}{12}}}$$

$$= 209.3\text{ in.}$$

$$= 17\text{ ft }5\text{ in.}$$

Calculate the deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{(384)(1.6\times10^6)(98.3)}{(5)(360)(40/12)}}$$

$$= 216\text{ in.}$$

$$= 18\text{ ft }0\text{ in.}$$

Maximum Floor Joist Span =17 ft 5 in.
(WFCM Table 3.18A)

3

PRESCRIPTIVE DESIGN

Table 3.19 Metal Plate Connected Wood Floor Truss Spans

Description: Metal plate connected wood floor trusses shall be designed by the manufacturer. Table 3.19 is provided to show representative maximum floor spans.

Tables 3.20A1-A6 Maximum Exterior Loadbearing and Non-Loadbearing Stud Lengths for Common Lumber Species Resisting Interior Zone Wind Loads

(Fully Sheathed with a Minimum Sheathing Material)
(Example shown is for Table 3.20A1)

Description: Calculation of maximum permissible height for loadbearing and non-loadbearing lumber wall studs.

Procedure: Calculate stud requirements for exterior wall studs sheathed with minimum sheathing based on interior zone wind loads. Non-loadbearing wall studs shall be limited by lateral wind loads. Perform moment and deflection calculations and take the minimum span.

Background: This analysis requires iteration to determine load based on the tributary area and effective wind area of the wall stud. Choose a stud length, determine wind load based on the effective wind area using component and cladding (C&C) loads, then calculate moment and deflection to determine if length chosen is over- or under-stressed.

Example:

Given - 150 mph Exposure B, 2x6, No. 2 Hem-Fir, 24" o.c. stud spacing, F_b = 850 psi, E = 1.3 million psi, C_F = 1.3, C_r = 1.15, C_D = 1.6, $\Delta_{LL} \leq H/180$ (where "H" in this case refers to the stud height or stud length).

The program used to create this table iterates to determine the load based on the tributary area of the wall stud. For simplification of calculations, the actual stud length is used here for each calculation check (bending and deflection), rather than performing an iterative set of calculations for each.

Maximum Stud Length Based on Bending Strength

Calculate effective wind area of wall stud (assume H = 14 ft - 8 in.):

$$= h^2/3$$
$$= (14.667\ ft)^2 / 3$$
$$= 71.7\ ft^2$$

Therefore, the lateral load (plf) on the stud is calculated as follows:

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

$$q_h = 21.15 \text{ psf (see Table C1.1)}$$

$$GC_p = -0.8 - 0.3([\log(71.7/500)]/[\log(10/500)])$$
$$= -0.9489$$

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

$$p = 21.15 \text{ psf} (-0.9489 - 0.18)$$
$$= |-23.88 \text{ psf}|$$

$$w = 23.88 \text{ psf (2 ft)}$$
$$= 47.8 \text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F$$
$$= 850(1.6)(1.15)(1.3) = 2{,}033 \text{ psi}$$

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

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

$$= \sqrt{\frac{(8)(7.56)(2{,}033)}{47.8/12}}$$

$$= 176\ in.$$

$$= 14\ ft\ 8\ in.$$

Maximum Stud Length Based on Deflection

Calculate effective wind area of the wall stud (assume H = 13 ft - 4 in.):

$$= h^2/3$$
$$= (13.333\ ft)^2 / 3$$
$$= 59.3\ ft^2$$

PRESCRIPTIVE DESIGN 3

Therefore, the load on the stud is:

GC_p = - 0.8 - 0.3([log(59.3/500)]/[log(10/500)])
= - 0.9635

p = 21.15 psf (- 0.9635 - 0.18)
= |- 24.19 psf|

w = 0.7 (24.19) psf (2 ft) (ASD load for deflection)
= 16.93 psf (2 ft)
= 33.9 plf

Note that per *International Building Code* (IBC) Table 1604.3, the wind load is permitted to be taken as 0.42 times the component and cladding (C&C) load. This equates to 0.70 times the ASD C&C load as the ASD wind load already includes a 0.60 ASD adjustment.

$$\Delta \leq \frac{w_{live} L}{384\, EI}$$

$$\sqrt{\frac{384EI}{\quad_{live}}}$$

$$\sqrt{\frac{384\left(1.3\ \ 10\ \right)(20.8)}{5\left(180\right)\left(\frac{33.9}{12}\right)}}$$

$$160 \text{ in.}$$

$$= 13 \text{ ft } 4 \text{ in.}$$

Maximum Allowable Stud Length = 13 ft - 4 in.
(WFCM Table 3.20A1)

Note a:
Tables do not account for increased wind loads at the corners of the structure. Minimum allowed spans are 7' - 9".

Thus the largest wind coefficient is based on an area equal to:

$$(7.75 \text{ ft})^2/3 = 20 \text{ ft}^2$$

Using the ratio of Zone 5 to Zone 4 windspeeds:

End Zone

$(GC_p - GC_{pi})$ = - 0.8 - 0.6[log(20.0/500)]/[log(10/500)]) - 0.18
= -1.47

Interior Zone

$(GC_p - GC_{cpi})$ = - 0.8 - 0.3([log(20.0/500)]/[log(10/500)]) - 0.18
= - 1.23

Interior/End
= (- 1.23) / (- 1.47)
= 0.83

Given the load applied to a stud is a function of wind pressure and stud spacing, stud spacing shall be multiplied by 0.80 to counteract the additional load.

Where 3/8" or thicker wood structural panel sheathing is applied at the exterior corners, the increase in the repetitive member factor (C_r) is adequate to counteract the additional wind load pressure for both bending strength and stiffness. The increase in stiffness associated with the increase in bending strength is higher than the C_r assigned to bending strength.

Tables 3.20B1-B6 Maximum Exterior Loadbearing and Non-Loadbearing Stud Lengths for Common Lumber Species Resisting Interior Zone Wind Loads

(Fully Sheathed with a Minimum 3/8" Wood Structural Panel Sheathing Material)
(Example shown is for Table 3.20B1)

Description: Calculation of maximum permissible height for loadbearing and non-loadbearing lumber wall studs.

Procedure: Calculate stud requirements for exterior wall studs sheathed with 3/8" Structural Sheathing based on interior zone wind loads. Non-loadbearing wall studs shall be limited by lateral wind loads. Perform moment and deflection calculations and take the minimum span.

Background: See Commentary on Tables 3.20A1-A6.

Example:

Given - 150 mph Exposure B, 2x6, No. 2 Hem-Fir, 24" o.c. stud spacing, F_b = 850 psi, E = 1.3 million psi, C_F = 1.3, C_r = 1.35 (for both bending and stiffness), C_D = 1.6, $\Delta_{LL} \leq H/180$ (where "H" in this case refers to the stud height or stud length).

The program used to create this table iterates to determine the load based on the tributary area of the wall stud. For simplification of calculations, the actual stud length is used here for each calculation check (bending and deflection), rather than performing an iterative set of calculations for each.

Maximum Stud Length Based on Bending Strength

Calculate effective wind area of wall stud (assume H = 16 ft - 0 in.):

$$= h^2/3$$
$$= (16.0\text{ ft})^2 / 3$$
$$= 85.3\text{ ft}^2$$

Therefore, the lateral load (plf) on the stud is calculated as follows:

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

$$q_h = 21.15\text{ psf (see Table C1.1)}$$

$$GC_p = -0.8 - 0.3([\log(85.3/500)]/[\log(10/500)])$$
$$= -0.9356$$
$$GC_{pi} = +/- 0.18$$

$$p = 21.15\text{ psf }(-0.9356 - 0.18)$$
$$= |-23.56\text{ psf}|$$

$$w = 23.56\text{ psf (2 ft)}$$
$$= 47.2\text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F$$
$$= 850(1.6)(1.35)(1.3) = 2{,}387\text{ psi}$$

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

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

$$= \sqrt{\frac{(8)(7.56)(2{,}387)}{47.2/12}}$$

$$= 192\text{ in.}$$

$$= 16\text{ ft } 0\text{ in.}$$

Maximum Stud Length Based on Deflection

Calculate effective wind area of the wall stud (assume H = 14 ft - 10 in.):

$$= h^2/3$$
$$= (14.833\text{ ft})^2 / 3$$
$$= 73.3\text{ ft}^2$$

Therefore, the load on the stud is:

$$GC_p = -0.8 - 0.3([\log(73.3/500)]/[\log(10/500)])$$
$$= -0.9472$$

$$p = 21.15\text{ psf }(-0.9472 - 0.18)$$
$$= |-23.84\text{ psf}|$$

$$w = 0.7\,(23.84)\text{ psf (2 ft)} \quad \text{(ASD load for deflection)}$$
$$= 16.69\text{ psf (2 ft)}$$
$$= 33.4\text{ plf}$$

Note that per *International Building Code* (IBC) Table 1604.3, the wind load is permitted to be taken as 0.42 times the component and cladding (C&C) load. This equates to 0.70 times the ASD C&C load as the ASD load already includes a 0.60 ASD adjustment.

$$\Delta \leq \frac{5w_{live}L^4}{384EI(C_r)}$$

$$L = \sqrt[4]{\frac{384EI(C_r)\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384(1.3x10^6)(1.35)(20.8)}{5(180)(\frac{33.4}{12})}}$$

$$= 178 \text{ in.}$$

$$= 14 \text{ ft } 10 \text{ in.}$$

Maximum Allowable Stud Length = 14 ft -10 in.
(WFCM Table 3.20B1)

Note a:
Tables do not account for increased wind loads at the corners of the structure. Minimum allowed spans are 7 ft – 9 in.

Thus the largest wind coefficient is based on an area equal to:

$(7.75 \text{ ft})^2/3 = 20\text{ft}^2$

Using the ratio of Zone 5 to Zone 4 windspeeds:

End Zone

$(GC_{pf} - GC_{pi})$ = - 0.8 - 0.6[log(20.0/500)]/[log(10/500)]) - 0.18
= - 1.47

Interior Zone

$(GC_{pf} - GC_{cpi})$ = - 0.8 - 0.3([log(20.0/500)]/[log(10/500)]) - 0.18
= - 1.23

Interior/End
= (- 1.23) / (- 1.47)
= 0.83

Given the load applied to a stud is a function of wind pressure and stud spacing, stud spacing shall be multiplied by 0.80 to counteract the additional load.

Table 3.20C Size, Height, and Spacing Limits for Wood Studs

Description: Currently, there is no standardized method for accurately designing loadbearing studs used in wall assemblies. Historically, when required, these studs have been designed as single bare studs, but this method is very conservative since it does not account for significant load-sharing and composite action resulting when studs are sheathed, especially with wood structural panels. Recently, provisions have been enhanced for these effects for wind design, but further work is being conducted on a broader, more accurate analysis method that can be used for gravity design of studs in repetitive member, sheathed wall assemblies. Until those methods are available, the wood stud provisions used in WFCM Table 3.20C are consistent with the prescriptive wood stud provisions from the *International Residential Code (IRC)* and with the limitations of unbraced length and supported gravity loads provided in WFCM Chapter 3.

Table 3.21 Top Plate Requirements for Wind

Description: Calculation of number of 16d common nails in each side of a splice.

Procedure: Calculate chord forces for unblocked diaphragms and design the top plate splice to resist tension forces.

Background: Top plate splice requirements are designed to resist the axial chord force as determined by the building dimension and ASD unit shear capacity of the unblocked diaphragm.

Example:

Given - 36' diaphragm width perpendicular to wind load.

Calculate the ASD unblocked diaphragm capacity by multiplying the nominal unit shear capacity for a wood-frame diaphragm with Sheathing, 8d common nails, 15/32″ nominal panel thickness, nominal 2x supported edges and boundaries, and Case 1 panel layup (Table 4.2C *2008 SDPWS*) by the (1/2.0) ASD reduction factor (*SDPWS* Section 4.2.3) and by the specific gravity adjustment factor [1 - (0.5 – G)] for G = 0.42 (Footnote 2 Table 4.2C *2008 SDPWS*):

= (670 plf /2.0)[1 – (0.5 - 0.42)]
= 308 plf

Calculate chord force:

= 308 plf (36 ft)/4
= 2,772 lbs

From 2012 NDS:

Reference Lateral Design Value for 16d common nail in a 1.5" side member:

Z = 120 lbs/nail

Load Duration Factor for wind, C_D = 1.6

$Z' = Z\,C_D$
= 120 (1.6) lbs/nail
= 192 lbs/nail

Calculate required number of 16d common nails on each side of the splice:

= 2,772 lbs/(192 lbs/nail)
= 14 nails (WFCM Table 3.21)

Tables 3.22A1-E1 Laterally Unsupported (Dropped) Header Spans for Exterior Loadbearing Walls

(Example shown is for Table 3.22A1 Supporting a Roof & Ceiling)

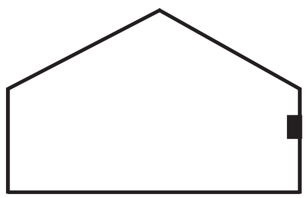
Description: Calculation of maximum permissible span for laterally unsupported (dropped) header resisting gravity loads.

Procedure: Using loads calculated from Chapter 2, perform moment, shear, and deflection calculations and take the minimum span.

Background: Roof system dead load including roof and ceiling is 20 psf. Bending design values are increased by the size factor, C_F, load duration factor, C_D, and repetitive member factor, C_r, and are reduced by the beam stability factor, C_L. Shear design values are increased by the load duration factor, C_D, only. Other adjustment factors are unity.

For 2-ply headers, a repetitive member factor of 1.1 is used based on ASTM D6555 *Standard Guide for Evaluating System Effects in Repetitive-Member Wood Assemblies,* which includes provisions for this increase for solid sawn beams or headers with 2 members in direct contact.

Example:



Given - 36' building width, 2-2x12's, No. 2 Hem-Fir, F_b = 850 psi, F_v = 150 psi, E = 1.3 million psi, E_{min} = 470,000 psi, 20 psf roof live load, 20 psf roof/ceiling assembly dead load, C_F = 1.0, C_r = 1.1, C_D = 1.25, and $\Delta_{LL} \leq L/360$.

Total load on header:

From WFCM Table 2.11:

$$w_{total} = 800 \text{ plf}$$
$$w_{live} = w_{total} - w_{dead} = 800 - [(36\text{ ft}/2) + 2](20) = 400 \text{ plf}$$

Calculate the moment-limited span:

Note: For laterally unsupported beams, determination of the beam stability factor, C_L, involves an iterative calculation where an unsupported span, ℓ_u, is assumed, the effective span, ℓ_e, is calculated, and the resulting beam stability factor is determined. The iteration continues until the resulting beam stability factor is consistent with the actual unsupported span.

For this example, the assumed unsupported span corresponds to the actual unsupported span that results from the iterative calculations.

$$F_b' = F_b C_D C_r C_F C_L = 850(1.25)(1.1)(1.0)C_L = 1{,}169\ C_L \text{ psi}$$

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

$$L = \sqrt{\frac{8S(F_b')}{w_{total}}} = \sqrt{\frac{(8)(2)(31.64)(1{,}169)C_L}{800/12}} = 94\sqrt{C_L}$$

$$C_L = \frac{1+\left(\frac{F_{bE}}{F_b^*}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{F_{bE}}{F_b^*}\right)}{1.9}\right]^2 - \frac{\frac{F_{bE}}{F_b^*}}{0.95}}$$

$$F_{bE} = \frac{1.20E'_{min}}{R_B^2}$$

$$R_B = \sqrt{\frac{\ell_e d}{b^2}}$$

F_b^* = reference bending design value multiplied by all applicable adjustment factors except C_{fu}, C_V, and C_L.

$F_b^* = 850(1.1)(1.0)(1.25) = 1{,}169$ psi

$\ell_e = 2.06\ell_u$ (for a single span beam with uniformly distributed load where $\frac{\ell_u}{d} < 7$)

Therefore, assuming a laterally unsupported span equal to 73" produces the following results:

$\frac{\ell_u}{d} = \frac{73}{11.25} = 6.49 < 7$ therefore, the effective length, ℓ_e, is calculated as follows:

$$\ell_e = 2.06(73) = 150"$$

$$R_B = \sqrt{\frac{150(11.25)}{1.5^2}} = 27.4$$

$$F_{bE} = \frac{1.20(470{,}000)}{27.4^2} = 750$$

$$C_L = \frac{1+\left(\frac{750}{1{,}169}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{750}{1{,}169}\right)}{1.9}\right]^2 - \frac{\frac{750}{1{,}169}}{0.95}} = 0.60$$

Therefore, the moment-limited span is equal to:

$$= 94\sqrt{C_L} = 94\sqrt{0.60} = 73"\text{or 6 ft 1in.}$$

Calculate the shear-limited span:

$F_v' = F_v C_D$

$= 150\,(1.25) = 187.5$ psi

$$V = \frac{wL}{2}$$

$$F_v' \geq \frac{3V}{2A}$$

$$= \frac{3wL/2}{2bd}$$

$$L = \frac{4bdF_v'}{3w} + 2d$$

$$L = \frac{4(2)(1.5)(11.25)(187.5)}{3(800)} + 2\left(\frac{11.25}{12}\right)$$

$= 12$ ft 5 in.

Calculate the live load deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384\left(1.3x10^6\right)(2(178))}{5(360)\left(\frac{400}{12}\right)}}$$

$= 144$ in.

$= 12$ ft 0 in.

$L_{limit} = 6$ ft - 1 in. (WFCM Table 3.22A1)

Footnote 1:

Bending capacity for Hem-Fir #3 is 500 psi compared to 850 psi for Hem-Fir #2.

Dividing the bending capacity of Hem Fir #2 by Hem Fir #3 and taking the square root, the reduction in span is:

$$\sqrt{500/850} = 0.76$$

Dividing the modulus of elasticity of Hem Fir #2 by Hem Fir #3 and taking the third root, the reduction in span is:

$$\sqrt[3]{1{,}200{,}000/1{,}300{,}000} = 0.97$$

Conservatively, spans shall be multiplied by 0.75 for grade #3. The 0.75 adjustment for No. 3 grade is intended to be applied to raised (supported) headers but can be shown to be conservative when applied to dropped (unsupported) headers.

Tables 3.22A2-E2 Laterally Supported (Raised) Header Spans for Exterior Loadbearing Walls

(Example shown is for Table 3.22A2 Supporting a Roof & Ceiling)

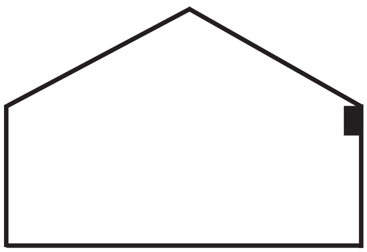
Description: Calculation of maximum permissible span for laterally supported (raised) header resisting gravity loads.

Procedure: Using loads calculated from Chapter 2, perform moment, shear, and deflection calculations and take the minimum span.

Background: Roof system dead load including roof and ceiling is 20 psf. Bending design values are increased by the size factor, C_F, load duration factor, C_D, and repetitive member factor, C_r. Shear design values are increased by the load duration factor, C_D, only. Other adjustment factors are unity.

For 2-ply headers, a repetitive member factor of 1.1 is used based on ASTM D6555 *Standard Guide for Evaluating System Effects in Repetitive-Member Wood Assemblies,* which includes provisions for this increase for solid sawn beams or headers with 2 members in direct contact.

Example:



Given - 36' building width, 2-2x12's, No. 2 Hem-Fir, F_b = 850 psi, F_v = 150 psi, E = 1.3 million psi, 20 psf roof live load, 20 psf roof/ceiling assembly dead load, C_F = 1.0, C_r = 1.1, C_D = 1.25, C_L=1.0, and $\Delta_{LL} \leq L/360$.

Total load on header:

From WFCM Table 2.11:

$$w_{total} = 800 \text{ plf}$$

$$\begin{aligned} w_{live} &= w_{total} - w_{dead} \\ &= 800 - [(36 \text{ ft}/2) + 2](20) \\ &= 400 \text{ plf} \end{aligned}$$

Calculate the moment-limited span:

$$\begin{aligned} F_b' &= F_b C_D C_r C_F C_L \\ &= 850(1.25)(1.1)(1.0)(1.0) = 1{,}169 \text{ psi} \end{aligned}$$

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

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

$$= \sqrt{\frac{(8)(2)(31.64)(1{,}169)}{800/12}}$$

$$= 94 \text{ in.}$$

$$= 7 \text{ ft } 10 \text{ in.}$$

Calculate the shear-limited span:

$$\begin{aligned} F_v' &= F_v C_D \\ &= 150\,(1.25) = 187.5 \text{ psi} \end{aligned}$$

$$V = \frac{wL}{2}$$

$$F_v' \geq \frac{3V}{2A}$$

$$= \frac{3wL/2}{2bd}$$

$$L = \frac{4bdF_v'}{3w} + 2d$$

$$L = \frac{4(2)(1.5)(11.25)(187.5)}{3(800)} + 2(\frac{11.25}{12})$$

$$= 12 \text{ ft } 5 \text{ in.}$$

Calculate the live load deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384\left(1.3x10^6\right)(2(178))}{5\left(360\right)(\frac{400}{12})}}$$

$$= \sqrt[3]{\frac{384\left(1.3x10^6\right)(2(178))}{5\left(360\right)(\frac{400}{12})}}$$

= 144 in.

= 12 ft

L_{limit} = 7 ft - 10 in. (WFCM Table 3.22A2)

Footnote 1:
See Commentary for Tables 3.22A1-E1 for calculations.

Table 3.22F Jack Stud Requirements for Headers in Exterior Loadbearing Walls

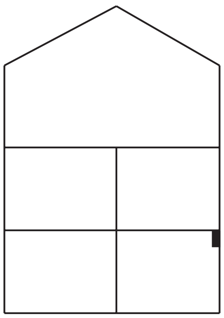
Description: Calculation of minimum number of jack studs to support different header spans.

Procedure: Using loads calculated from Chapter 2, and the adjusted compression perpendicular to grain design value, calculate the required bearing area for a given header span and header width.

Background: Assuming the header has full bearing across its width, the bearing length can be determined based on loads from Chapter 2 and the adjusted compression perpendicular to grain design value.

Example:

Given - 36' building width, 3-2x12's, No. 2 Hem-Fir, $F_{c\perp}$ = 405 psi, 20 psf roof live load, 20 psf roof system dead load, 8' header supporting roof, ceiling, and two center-bearing floors.



Total load on header:

From WFCM Table 2.11:

$$w = 1{,}662 \text{ plf}$$

Reaction at end of header:

$$R = 1{,}662 \text{ plf } (8 \text{ ft}/2)$$
$$= 6{,}648 \text{ lbs}$$

Calculate the required number of jack studs:

$$F_{c\perp}' = F_{c\perp}$$
$$= 405 \text{ psi}$$

$$F_{c\perp}' \geq R/A$$
$$405 = 6{,}648 / [3(1.5)(n)(1.5)]$$
$$n = 6{,}648 / [405(3)(1.5)(1.5)]$$
$$= 2.4$$
$$n = 3 \qquad \text{(WFCM Table 3.22F)}$$

Footnote 1:

Using the calculations from Table 2.11 Commentary, as the roof span decreases, wall and overhang loads remain constant while floor and rafter loads decrease. The jack stud requirements may be reduced based on the following adjustment to account for the decrease in roof span.

The governing load case for this building configuration can be shown to be as follows:

Dead Load + 0.75(Floor Live Load) + 0.75(Roof Live Load or Snow Load)

Sum loads:

Dead Loads

$$w_{overhang} = (20 \text{ psf})(2 \text{ ft})$$
$$= 40 \text{ plf}$$

$$w_{roof} = (20 \text{ plf})(W/2)$$
$$= 10(W)$$

$$w_{wall} = 2(121)$$
$$= 242 \text{ plf}$$

$$w_{floor} = (10)(W/4)(2)$$
$$= 5(W)$$

Live Loads

$$w_{overhang} = (20 \text{ psf})(2 \text{ ft})$$
$$= 40 \text{ plf}$$

$$w_{roof} = (20 \text{ plf})(W/2)$$
$$= 10(W)$$

$$w_{floor} = (40)(W/4)(2)$$
$$= 20(W)$$

Summing the total load with W and using the governing load combination as shown above yields the following:

$$w_{total} = (282 + 15W) + 0.75(20W) + 0.75(40 + 10W)$$
$$= 312 + 37.5W$$

$$w_{total} = 1{,}662 \text{ plf} \qquad \text{(from WFCM Table 2.11)}$$

3

PRESCRIPTIVE DESIGN

Summing the total load with W symbolically noted and dividing by the total load at a 36' roof span yields:

$$= (37.5W + 312) / 1{,}662$$

$$= (W + 8.32) / 44.32$$

Substituting W = 24 ft yields the following:

$$= (24 + 8.32) / 44.32$$

$$= 0.73$$

For simplicity, the adjustment was conservatively taken as 0.75 and the footnote calculation revised as follows:

$$(W + 12)/48 = (24 + 12)/48 = 36/48 = 0.75$$

(WFCM Table 3.22F Footnote 1)

Table 3.23A Laterally Unsupported (Dropped) Header Spans for Exterior Loadbearing Walls Resisting Wind Loads

Description: Calculation of maximum permissible span for exterior loadbearing headers resisting lateral wind loads.

Procedure: Using wind loads calculated in WFCM Table 2.1, perform moment calculations to determine the maximum span.

Background: Given there are no code specified deflection limits for headers resisting lateral wind loads, only bending capacity is evaluated. Bending design values are increased by the size factor, C_F, wind load duration factor, C_D, and flat use factor, C_{fu}. Other adjustment factors are unity.

Example:

Given - 150 mph, Exposure B, Wall Height = 10', 2-2x12's, No. 2 Hem-Fir, F_b = 850 psi, C_F = 1.0, C_D = 1.6, C_{fu} = 1.2.

Lateral wind load on dropped wall header:

From WFCM Table 2.1:

$$w = 148 \text{ plf}$$

Calculate the moment-limited span:

$$\begin{aligned} F_b' &= F_b C_D C_{fu} C_F \\ &= 850(1.6)(1.2)(1.0) \\ &= 1{,}632 \text{ psi} \end{aligned}$$

$$F'_b \geq \frac{wl^2}{8S}$$

$$l = \sqrt{\frac{8SF'_b}{w}}$$

$$l = \sqrt{\frac{8(2)(4.22)1{,}632}{148/12}}$$

Maximum header span = 7 ft 11 in. (WFCM Table 3.23A)

Footnote 1:

The adjustment factor in Table 2.1 for interior zone wind loads is 0.92. Tabulated spans in Table 3.23A may be adjusted by taking the inverse root of 0.92 given the bending capacity controls. Thus, header spans may be divided by:

$$\sqrt{0.92} = 0.96$$

3

PRESCRIPTIVE DESIGN

Table 3.23B Laterally Unsupported (Dropped) Header Spans for Exterior Non-Loadbearing Walls and Window Sill Plate Spans Resisting Wind Loads

Description: Calculation of maximum permissible span for exterior non-loadbearing headers and window sill plates resisting lateral wind loads.

Procedure: Using wind loads calculated in WFCM Table 2.1, perform moment calculations to determine the maximum span.

Background: Given there are no code specified deflection limits for headers resisting lateral wind loads only bending capacity is evaluated. Bending design values are increased by the wind load duration factor, C_D, and size factor, C_F. Other adjustment factors are unity.

Example:

Given - 150 mph, Exposure B, Wall Height = 10', 1-2x4 (flat), No. 2 Hem-Fir, F_b = 850 psi, C_D = 1.6, C_F = 1.5

Lateral wind load on 2x4 window sill plate:

From WFCM Table 2.1:

w = 148 plf

Calculate the moment-limited span:

$$F_b' = F_b C_D C_F$$
$$= 850(1.6)(1.5)$$
$$= 2{,}040 \text{ psi}$$

$$F'_b \geq \frac{wl^2}{8S}$$

$$l = \sqrt{\frac{8SF'_b}{w}}$$

$$l = \sqrt{\frac{8\left(3.0625\right)2{,}040}{148/12}}$$

Maximum sill plate span = 5 ft - 4 in. (WFCM Table 3.23B)

Footnote 1:

See Commentary for Table 3.23A for calculations.

Footnote 2:

Tabulated spans in Table 3.23B may be adjusted by taking the inverse root of the reduction in tributary loads (H/10) given the bending capacity is critical. Thus, header spans may be divided by:

$(H/10)^{1/2}$ (WFCM Table 3.23B)

where:

H = nominal wall height.

Table 3.23C Full Height Stud Requirements for Headers or Window Sill Plates in Exterior Walls Resisting Wind Loads

Description: Calculation of the number of full-height studs required at each end of a header and/or window sill plate, in addition to the number of jack studs required.

Procedure: Using wind loads from Chapter 2, determine the induced moment in full height studs from loads carried by a header and/or window sill plate. Since the stud design from Tables 3.20A1-A6 and 3.20B1-B6 are based on the same set of assumed wind loads, the number of full height studs located at the end of each header and window sill plate can be determined using the moment induced by the header and/or sill plate loads and the moment induced in a single stud design.

Background: Tables 3.23 C and D require full height studs to be located at the end of each header and assume the wall studs adjacent to the opening are adequate to resist wind loads. The number of full height studs (NFH) required are based on the ratio of the applied moment induced in the stud (from the header and/or sill plate) to the wind induced moment in a single wall stud. Table 3.23C conservatively assumes that the reaction from the dropped header or upper window sill plate (assuming a raised header is used) into the adjacent full-height wall stud is located at mid span of the wall stud in order to induce the most conservative bending moment. Full height stud requirements in Table 3.23D are calculated for the actual location of the header and window sill plate.

Tributary areas:
Dropped header or upper window sill plate (where raised header is used):

$$A_a = (a/2 + c/2)L$$

where:

$$c/2 = (h - a - b) / 2$$

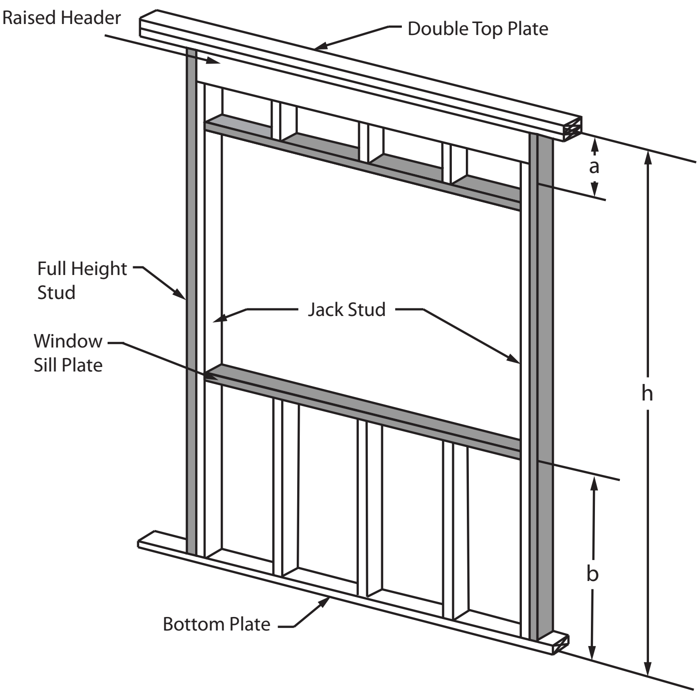
$$A_a = (a/2 + h/2 - a/2 - b/2)L$$
$$= (h/2 - b/2)L$$

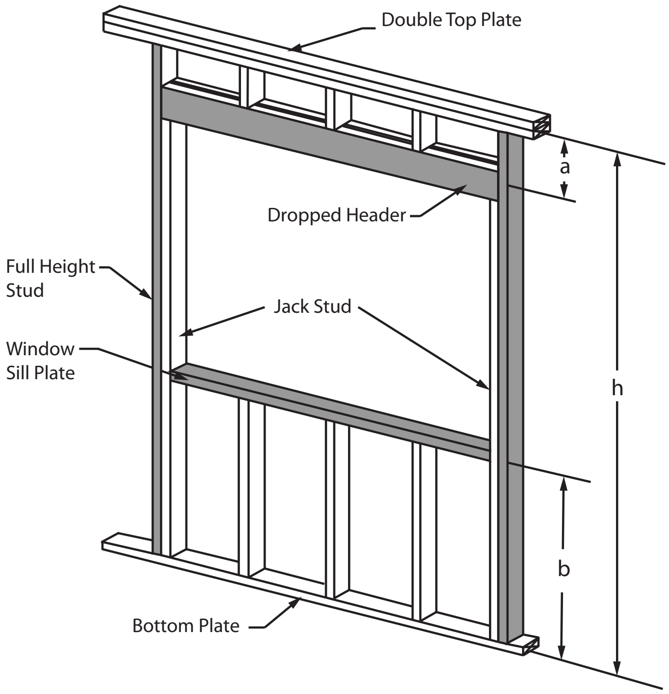
Window sill plate:

$$A_b = (h/2 - a/2)L$$

Point Load from dropped header or upper window sill plate into full height studs:

$$P_a = wL/2 = pA_a/2 = [pL(h/2 - b/2)] / 2$$





Point Load from lower window sill plate into full height studs:

$$P_b = wL/2 = pA_b/2 = [pL(h/2 - a/2)] / 2$$

Reaction at top of full height studs:

$$R_1 = [P_a(h - a) + P_b b] / h$$

Calculate the induced moment in full height studs from the dropped header (or upper window sill plate) and lower window sill plate loads:

$$M_{STUD(H)} = R_1 a$$

$$= \frac{P_a a(h-a) + P_b ab}{h}$$

$$= \frac{\left[\frac{pL}{2}\left(\frac{h}{2}-\frac{b}{2}\right)a(h-a) + \frac{pL}{2}\left(\frac{h}{2}-\frac{a}{2}\right)ab\right]}{h}$$

$$= \frac{a}{h}\frac{pL}{2}\left[\left(\frac{h}{2}-\frac{b}{2}\right)(h-a) + \left(\frac{h}{2}-\frac{a}{2}\right)b\right]$$

$$= \frac{a}{h}\frac{pL}{2}\left[\frac{h^2}{2}-\frac{hb}{2}-\frac{ha}{2}+\frac{ba}{2}+\frac{bh}{2}-\frac{ab}{2}\right]$$

$$= \frac{pLa}{2h}\left[\frac{h}{2}(h-a)\right]$$

$$= \frac{pLh^2}{4}\left[\frac{a}{h}\left(1-\frac{a}{h}\right)\right]$$

$$LET\ x = \frac{a}{h}$$

$$M_{STUD(H)} = \frac{pLh^2}{4}\left[x(1-x)\right]$$

From Single Stud Design:

$$M_{STUD(S)} = \frac{psh^2}{8}$$

Required Number of Full Height Studs (NFH) is equal to the induced moment from the dropped header (or upper window sill plate) and lower window sill plate loads divided by the induced moment for single stud design.

$$NFH = \frac{M_{STUD(H)}}{M_{STUD(S)}}$$

$$= \frac{\frac{pLh^2}{4}\left[x(1-x)\right]}{\frac{psh^2}{8}}$$

$$= \frac{2L}{s}\left[x(1-x)\right]$$

The largest value of NFH is obtained assuming that a = 1/2h or x = 1/2, therefore NFH is further simplified to:

$$NFH = 0.5L/s$$

This equation is used to generate the number of full height studs required in Table 3.23C.

Example:

Given - 12' header span, 16" o.c. stud spacing, header located at ½ stud height (table assumptions).

Calculate number of full height studs (NFH):

$$NFH = 0.5L/s$$

where:

L = header span = 12 ft

s = stud spacing = 16 in.

NFH = 0.5(12 ft)(12 in/ft)/16 in.

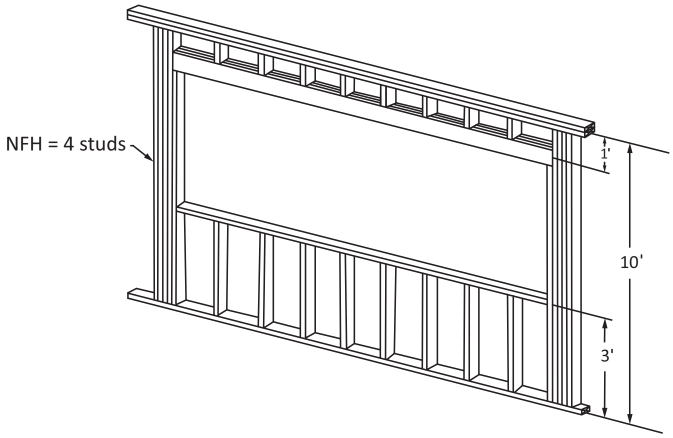
= 4.5 = 5 studs (WFCM Table 3.23C)

Table 3.23D Reduced Full Height Stud Requirements for Headers or Window Sill Plates in Exterior Walls Resisting Wind Loads

Description: Calculation of the number of full-height studs required at each end of a header or window sill plate, in addition to the number of jack studs required, taking into account the distance from the top plate down to the dropped header (a), or distance from the bottom plate up to the window sill plate (b).

Procedure: See Commentary for Table 3.23C.

Background: See Commentary for Table 3.23C.



Example:

Given - 12' header span, 16" stud spacing, 10' wall height, header located 1' down from the top plate, window sill plate located 3' up from the bottom plate.

Calculate the number of full height studs:

From WFCM Table 3.23C Commentary:

$$NFH = 2L/s\,[x(1 - x)]$$

where:

L = header span = 12 ft

s = stud spacing = 16 in.

a = 1 ft

b = 3 ft

h = 10 ft

x = greater of a or b divided by the wall height, h

= 3 ft /10 ft

= 0.3

$$NFH = 2\,[(12\text{ ft})(12\text{ in./ft}) / (16\text{ in.})]\,[0.3(1 - 0.3)]$$

$$= 3.8 = 4\text{ studs} \qquad \text{(WFCM Table 3.23D)}$$

Tables 3.24A1 & B1 Laterally Unsupported (Dropped) Header Spans for Interior Loadbearing Walls

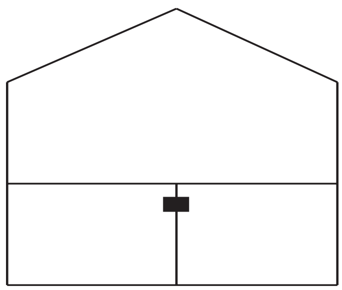
(Example shown is for Table 3.24A1)

Description: Calculation of maximum permissible span for laterally unsupported (dropped) header resisting gravity loads.

Procedure: Using loads calculated from Chapter 2, perform moment, shear, and deflection calculations and take the minimum span.

Background: Bending design values are increased by the size factor, C_F, and repetitive member factor, C_r, and are reduced by the beam stability factor, C_L. Other adjustment factors are unity, including the load duration factor, C_D, since interior loadbearing walls are assumed to only support floor loads in these tables.

Example:



Given - 12' building width, 3-2x8's, No. 2 Hem-Fir, F_b = 850 psi, F_v = 150 psi, E = 1.3 million psi, E_{min} = 470,000 psi, 40 psf floor live load, 10 psf floor dead load, C_F = 1.2, C_r = 1.15, C_D = 1.0, Δ_{LL} = L/360.

Total load on header:

From WFCM Table 2.11:

$$w_{total} = 300 \text{ plf}$$

$$w_{live} = (12 \text{ ft}/2)(40) = 240 \text{ plf}$$

Calculate the moment-limited span:

Note: For laterally unsupported beams, the determination of the beam stability factor, C_L, involves an iterative calculation where an unsupported span, l_u, is assumed, the effective span, l_e, is calculated, and the resulting beam stability factor is determined. The iteration continues until the resulting beam stability factor is consistent with the actual unsupported span.

For this example, the assumed unsupported span corresponds to the actual unsupported span that results from the iterative calculations.

$$F_b' = F_b C_D C_r C_F C_L = 850(1.0)(1.15)(1.2)C_L = 1{,}173C_L \text{ psi}$$

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

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

$$L = \sqrt{\frac{8S(F_b')}{w_{total}}} = \sqrt{\frac{(8)(3)(13.14)(1{,}173)C_L}{300/12}} = 122\sqrt{C_L} \text{ in.}$$

$$C_L = \frac{1+\left(\frac{F_{bE}}{F_b^*}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{F_{bE}}{F_b^*}\right)}{1.9}\right]^2 - \frac{\frac{F_{bE}}{F_b^*}}{0.95}}$$

$$F_{bE} = \frac{1.20E'_{min}}{R_B^2}$$

$$R_B = \sqrt{\frac{\ell_e d}{b^2}}$$

F_b^* = reference bending design value multiplied by all applicable adjustment factors except C_{fu}, C_V, and C_L.

F_b^* = 850(1.15)(1.0)(1.2) = 1,173 psi

ℓ_e = 1.63 ℓ_u + 3d (for a single span beam with a uniformly distributed load where $\frac{\ell_u}{d} \geq 7$)

Therefore, assuming a laterally unsupported span equal to 102″ produces the following results:

$$\frac{\ell_u}{d} = \frac{102}{7.25} = 14 > 7$$

Therefore, the effective length, ℓ_e, is calculated as follows:

$$\ell_e = 1.63 l_u + 3d = 188"$$

$$R_B = \sqrt{\frac{188(7.25)}{1.5^2}} = 24.61$$

$$F_{bE} = \frac{1.20(470,000)}{24.61^2} = 931 \text{ psi}$$

$$C_L = \frac{1+\left(\frac{931}{1,173}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{931}{1,173}\right)}{1.9}\right]^2 - \frac{\frac{931}{1,173}}{0.95}} = 0.71$$

Therefore, the moment-limited span is equal to:

$$= 122\sqrt{C_L} = 122\sqrt{0.71} = 102" \text{ or } 8 \text{ ft } 6 \text{ in.}$$

Calculate the shear-limited span:

$F_v' = F_v C_D$
$= 150(1.0)$
$= 150$ psi

$$V = \frac{wL}{2}$$

$$F_v' \geq \frac{3V}{2A}$$

$$= \frac{3wL/2}{2bd}$$

$$L = \frac{4bdF_v'}{4w} + 2d$$

$$L = \frac{4(3)(1.5)(7.25)(150)}{3(300)} + 2(\frac{7.25}{12})$$

$$= 22 \text{ ft } 11 \text{ in.}$$

Calculate the live load deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384(1.3x10^6)(3)(47.63))}{5(360)(\frac{240}{12})}}$$

$$= 126 \text{ in.}$$

$$= 10 \text{ ft } 6 \text{ in.}$$

L_{limit} = 8 ft 6 in. (WFCM Table 3.24A1 Dropped)

Footnote 1:
See Commentary for Table 3.22A1 for calculations.

Footnote 3:
Based on WFCM Table 2.11 Footnote 1, bearing loads shall be increased by 1.25 when headers support continuous two span floor joists. Therefore, tabulated spans shall be reduced by the inverse root of 1.25.

$\sqrt{\frac{1}{1.25}}$ = 0.89 (WFCM Table 3.24A)

Tables 3.24A2 & B2 Laterally Supported (Raised) Header Spans for Interior Loadbearing Walls

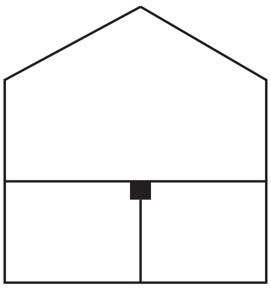
(Example shown is for Table 3.24A2)

Description: Calculation of maximum permissible span for laterally supported (raised) header resisting gravity loads.

Procedure: Using loads calculated from Chapter 2, perform moment, shear, and deflection calculations and take the minimum span.

Background: Bending design values are increased by the size factor, C_F, and repetitive member factor, C_r. Other adjustment factors are unity, including the load duration factor, C_D, since interior loadbearing walls are assumed to only support floor loads in these tables.

Example:



Given - 12' building width, 3-2x8's, No. 2 Hem-Fir, F_b = 850 psi, F_v = 150 psi, E = 1.3 million psi, 40 psf floor live load, 10 psf floor dead load, C_F = 1.2, C_r = 1.15, C_L=1.0, and Δ_{LL} = L/360.

Total load on header:

From WFCM Table 2.11:

$$w_{total} = 300 \text{ plf}$$
$$w_{live} = (12 \text{ ft}/2)(40) = 240 \text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F C_L = 850(1.0)(1.15)(1.2)(1.0) = 1{,}173 \text{ psi}$$

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

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

$$= \sqrt{\frac{(8)(3)(13.14)(1{,}173)}{300/12}}$$

$$= 122 \text{ in.}$$

$$= 10 \text{ ft } 2 \text{ in.}$$

Calculate the shear-limited span:

$$F_v' = F_v C_D = 150(1.0) = 150 \text{ psi}$$

$$\frac{wL}{}$$

$$\geq \frac{}{}$$

$$\frac{3wL/2}{bd}$$

$$L = \frac{bdF}{} + \ d$$

$$= \frac{4(2)(1.5)(7.25)(150)}{3(300)} + 2\left(\frac{7.25}{12}\right)$$

$$= 15 \text{ ft } 8 \text{ in.}$$

Calculate the live load deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384\left(1.3x10^6\right)(3)(47.63))}{5\left(360\right)(\frac{240}{12})}}$$

$$= 126 \text{ in.}$$

$$= 10 \text{ ft } 6 \text{ in.}$$

L_{limit} = 10 ft 2 in. (WFCM Table 3.24A2 Raised)

Footnote 1:
See Commentary for Table 3.22A1-E1 for calculations.

Footnote 3:
See Commentary for Tables 3.24A1 and B1 for Dropped Interior Beam for calculations.

3

PRESCRIPTIVE DESIGN

Table 3.24C Jack Stud Requirements for Headers in Interior Loadbearing Walls

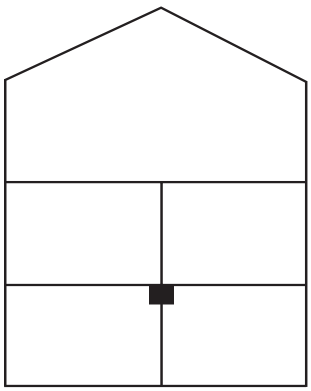
Description: Calculation of minimum number of jack studs to support different header spans.

Procedure: Using loads calculated from Chapter 2, and the adjusted compression perpendicular to grain design value, calculate the required bearing area for a given header span and header width.

Background: Assuming the header has full bearing across its width, the bearing length can be determined based on loads from Chapter 2 and the adjusted compression perpendicular to grain design value.

Example:

Given - 36' building width, 3-2x12's, No. 2 Hem-Fir, $F_{c\perp}$ = 405 psi, 40 psf floor live load, 10 psf floor dead load, 8' header supporting two center bearing floors.



Total load on header:

From WFCM Table 2.11:

$$w = 1{,}921 \text{ plf}$$

Reaction at end of header:

$$R = 1{,}921 \text{ plf } (8 \text{ ft}/2)$$
$$= 7{,}684 \text{ lbs}$$

Calculate the required number of jack studs:

$$F_{c\perp}' = F_{c\perp}$$
$$= 405 \text{ psi}$$

$$F_{c\perp}' \geq R/A$$
$$405 = 7{,}684 / [3(1.5)(n)(1.5)]$$
$$n = 7{,}596 / [405(3)(1.5)(1.5)]$$
$$= 2.8$$
$$n = 3 \qquad \text{(WFCM Table 3.24C)}$$

Tables 3.25A & B Ceiling Joist Spans for Common Lumber Species

(Example shown is for Table 3.25A, Uninhabitable Attics Without Storage)

Description: Calculation of maximum permissible span for lumber ceiling joist.

Procedure: Using typical ceiling loads for uninhabitable attics, perform moment and deflection calculations and take the minimum span.

Background: Based on simple bending and deflection calculations for ceiling joists. The allowable bending design value is based on a fully supported member, properly sheathed and nailed on one edge of the joist. Bending design values are increased by the repetitive member factor, C_r, and size factor, C_F. Other adjustment factors are unity.

Example:

Given - 2x6, No. 2 Douglas Fir-Larch, 16" o.c., F_b = 900 psi, E = 1.6 million psi, C_r = 1.15, C_F = 1.3, C_D = 1.0, 10 psf live load, 5 psf dead load, and $\Delta_{LL} \leq L/240$.

$$w_{dead} = 5\text{ psf}(16\text{ in.}/12\text{ in./ft}) = 6.67\text{ plf}$$
$$w_{live} = 10\text{ psf}(16\text{ in.}/12\text{ in./ft}) = 13.33\text{ plf}$$
$$w_{total} = 6.67 + 13.33 = 20\text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F = 900(1.0)(1.15)(1.3) = 1{,}345\text{ psi}$$

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

$$L = \sqrt{\frac{8S\left(F_b'\right)}{w_{total}}} = \sqrt{\frac{8(7.56)(1{,}345)}{\frac{20}{12}}}$$

$$= 221\text{ in.}$$
$$= 18\text{ ft }5\text{ in.}$$

Calculate the deflection-limited span:

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

$$L = \sqrt[4]{\frac{384 EI\Delta}{5 w_{live}}}$$

$$= 212\text{ in.}$$
$$= 17\text{ ft }8\text{ in.}$$

L_{limit} = 17 ft - 8 in. (WFCM Table 3.25A)

Footnote 1:

NDS bracing provisions require that the compression edge of bending members be fully braced where d/b exceeds 4. Alternatively, NDS 3.3.3 allows reductions in the adjusted allowable bending strength, F_b'. This reduction has not been taken in these tabulated spans.

Tables 3.26A & B Rafter Spans for Common Lumber Species for Live Load

(Example shown is for Table 3.26A, Rafter Spans for Common Lumber Species for Ceilings Not Attached to Rafters)

Description: Calculation of maximum permissible span for a rafter resisting vertical gravity loads. Deflection is limited to L/180 if the ceiling is not attached to rafters but L/240 if it is attached.

Procedure: Using construction live load, perform moment and deflection calculations and take the minimum span.

Background: Based on simple bending and deflection calculations, assuming rafter support at each end of the rafter. Span is assumed to be equal to the horizontal projection of the rafter. This assumption, while only approximately correct, provides for significant simplification of the tabulations. Generally, a deflection limitation of L/240 applies to rafters with a gypsum wallboard ceiling attached to the underside (e.g., cathedral ceilings).

The assumptions built into Tables 3.26A-H are simplistic. A more sophisticated approach would take the following offsetting effects into account:

- additional compression load in the rafter when a ridge board replaces the ridge beam
- additional length of the sloped rafter relative to the horizontal projection
- additional load capacity provided by the roof sheathing

Bending design values are increased by the repetitive member factor, C_r, load duration factor, C_D, and size factor, C_F. Other adjustment factors are unity.

Example:

Given - 2x8, No. 2 Spruce-Pine-Fir, 16" o.c., $F_b = 875$ psi, E = 1.4 million psi, $C_D = 1.25$, $C_F = 1.2$, $C_r = 1.15$, 20 psf live load, 10 psf dead load, and $\Delta_{LL} \leq$ L/180.

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

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F = 875(1.25)(1.15)(1.2) = 1{,}509\text{ psi}$$

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

$$L = \sqrt{\frac{8S\left(F_b'\right)}{w_{total}}} = \sqrt{\frac{8(13.14)(1{,}509)}{\frac{40}{12}}}$$

$$= 218\text{ in.}$$
$$= 18\text{ ft }2\text{ in.}$$

Calculate the deflection-limited span:

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

$$L = \sqrt[4]{\frac{384 EI\Delta}{5 w_{live}}}$$

$$= \sqrt[3]{\frac{(384)\left(1.4\times10^6\right)(47.63)}{(5)(180)(26.67/12)}}$$

$$= 234\text{ in.}$$
$$= 19\text{ ft }6\text{ in.}$$

L_{limit} = 18 ft 2 in. (WFCM Table 3.26A)

Footnotes 1-3:

See Commentary for Table 2.14A for calculations.

Tables 3.26 C–H Rafter Spans for Common Lumber Species for Ground Snow Loads

(Example shown is for Table 3.26G: 70 psf Ground Snow Load, 10 psf dead load, and $\Delta_{LL} \leq L/180$)

Description: Calculation of maximum permissible span for a rafter resisting vertical gravity loads. Deflection is limited to L/180 if the ceiling is not attached to rafters but L/240 if it is attached.

Procedure: Using ground snow load, perform moment and deflection calculations and take minimum span.

Background: See Commentary for Tables 3.26A and B for background information.

Example:

Given - 2x8, No. 2 Spruce-Pine-Fir, 16" o.c., F_b = 875 psi, E = 1.4 million psi, C_D = 1.15, C_F = 1.2, C_r = 1.15, 10 psf dead load, 70 psf ground snow load, $\Delta_{LL} \leq L/180$.

w_{dead} = 10 psf(16 in. / 12 in./ft)
= 13.33 plf

w_{snow} = Ip_g (Note: Unbalanced snow load per ASCE 7-10)
= (1.0)(70 psf)(16 in. / 12 in./ft)
= 93.3 plf

w_{total} = 13.33 + 93.33
= 106.7 plf

Calculate the moment-limited span:

F_b' = $F_bC_DC_rC_F$
= 875(1.15)(1.15)(1.2)
= 1,389 psi

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

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

$$= \sqrt{\frac{(8)(13.14)(1{,}389)}{106.7/12}}$$

= 128 in.

= 10 ft 8 in.

Calculate the deflection-limited span:

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

$$L = \sqrt[4]{\frac{384EI\Delta}{5w_{live}}}$$

$$= \sqrt[3]{\frac{384\left(1.4x10^6\right)(47.64)}{5\left(180\right)(\frac{106.7}{12})}}$$

= 147 in.

= 12 ft 3 in.

L_{limit} = 10 ft 8 in. (WFCM Table 3.26G)

Footnote 1:

See Commentary for Table 2.14A for calculations.

Table 3.27 Representative Metal Plate Connected Wood Roof Truss Spans

Description: Metal plate connected wood roof trusses shall be designed by the manufacturer. Table 3.27 is provided to show representative maximum roof spans.

Table 3.28 Hip and Valley Beam Sizes

Description: Calculation of lumber hip and valley beam sizes for a given span based on hip or valley area supporting vertical gravity loads.

Procedure: Determine minimum section required based on moment and deflection calculations.

Background: Hip or valley rafters are subject to concentrated loads from jack rafters framing into them. Determining the location of maximum moment is key to determining appropriate rafter sizes.

Bending design values are increased by the size factor, C_F, load duration factor, C_D, and repetitive member factor, C_r. Other adjustment factors are unity.

Example:

Given - 12' x 12' hip or valley area, No. 2 Hem-Fir, F_b = 850 psi, E = 1.3 million psi, C_F = 1.1, C_D = 1.15, C_r = 1.15, 30 psf ground snow load, 20 psf roof dead load, $\Delta_{LL} \leq$ L/180.

From Table 2.17

Required moment capacity = 6,583 ft-lbs

Required apparent rigidity = 158,100,000 in.^{2}-lbs

Try 3-2x10's

$$F_b' \geq M / S$$

where:

$$F_b' = F_b C_D C_r C_F = 850 \text{ psi } (1.15)(1.15)(1.1) = 1{,}236 \text{ psi}$$

$$M = 6{,}584 \text{ ft-lbs} = 6{,}584 \text{ ft-lbs } (12 \text{ in./ft}) = 79{,}008 \text{ in.-lbs}$$

Solving for the required section modulus

$$S = M / F_b = 79{,}008 \text{ in.-lbs } / 1{,}236 \text{ psi} = 63.9 \text{ in.}^3$$

3-2x10's have a section modulus of 64.2 in.^{3}

Use 3-2x10's (WFCM Table 3.28)

Check Deflections:

$$EI_{3\text{-}2x10} = 1.3x10^6 \text{ psi}(296.7 \text{ in.}^4) = 385x10^6 \text{ in.}^2\text{-lbs}$$

$$EI_{required} = 158x10^6 \text{ in.}^2\text{-lbs}$$

Required apparent rigidity capacity is met with 3-2x10's.

3

PRESCRIPTIVE DESIGN

Table 3.29 Ridge Beam Spans

Description: Calculation of maximum permissible span for ridge beam supporting vertical gravity load.

Procedure: Using ground snow load, perform moment and deflection calculations and take the minimum span.

Background: Bending design values are increased by the size factor, C_F, load duration factor, C_D, and repetitive member factor, C_r. Other adjustment factors are unity.

For 2-ply headers, a repetitive member factor of 1.1 is used based on ASTM D6555 *Standard Guide for Evaluating System Effects in Repetitive-Member Wood Assemblies,* which includes provisions for this increase for solid sawn beams or headers with 2 members in direct contact.

Example:

Given – 36' roof span, 2-2x12's, No. 2 Hem-Fir, F_b = 850 psi, E = 1.3 million psi, C_F = 1.0, C_r = 1.1, C_D = 1.15, 20 psf dead load, 30 psf ground snow load, $\Delta_{LL} \leq$ L/240.

From WFCM Table 2.16:

$$w_{total} = 776 \text{ plf}$$

$$w_{snow} = w_{total} - (36 \text{ ft}/2)(20)$$
$$= 776 - 360$$
$$= 416 \text{ plf}$$

Calculate the moment-limited span:

$$F_b' = F_b C_D C_r C_F$$
$$= 850(1.15)(1.1)(1.0)$$
$$= 1{,}075 \text{ psi}$$

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

$$L = \sqrt{\frac{8S\left(F_b'\right)}{w_{total}}}$$

$$= \sqrt{\frac{8(63.28)(1{,}075)}{\frac{776}{12}}}$$

$$= 91.7 \text{ in.}$$

$$= 7 \text{ ft } 8 \text{ in.}$$

Calculate the live load deflection-limited span:

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

$$L = \sqrt[4]{\frac{384 EI\Delta}{5 w_{live}}}$$

$$= \sqrt[3]{\frac{(384)\left(1.3\times10^6\right)(356)}{(5)(240)(416/12)}}$$

$$= 162 \text{ in.}$$

$$= 13 \text{ ft } 6 \text{ in.}$$

L_{limit} = 7 ft 8 in. (WFCM Table 3.29)

Table A-3.4 Uplift Strap Connection Requirements

Description: Required number of 8d common nails or 10d box nails in each end of 1-1/4" x 20 gage strap, for roof-to-wall, wall-to-wall, and wall-to-foundation connections.

Procedure: Using uplift loads calculated in Table 3.4, determine the number of nails required to resist the load.

Background: Lateral capacity of 8d common nails and 10d box nails are calculated based on the *2012 NDS*. A load duration adjustment factor of C_D = 1.6 (wind/earthquake) is assumed. Spruce-Pine-Fir with G = 0.42 is assumed for framing members.

Example:

Given - 150 mph, Exposure B, 24' roof span, 16" o.c. framing spacing, 15 psf roof/ceiling assembly dead load (10 psf roof dead load, 5psf ceiling dead load).

Uplift:

From WFCM Table 3.4:

Required connection uplift capacity:

$$U = 315 \text{ lbs}$$

From 2012 NDS Table 11P:

Lateral design value of 8d common nail:

$$Z = 81 \text{ lbs/nail}$$
$$Z' = Z\,(C_D)$$
$$Z' = (81)(1.6)$$
$$Z' = 130 \text{ lbs/nail}$$

Lateral design value of 10d box nail:

$$Z = 77 \text{ lbs/nail}$$
$$Z' = (77)(1.6)$$
$$Z' = 123 \text{ lbs/nail}$$

Calculate required number of nails based on 10d box:

$$n = 315 \text{ lbs} / 123 \text{ lbs/nail}$$
$$= 2.6 \text{ nails}$$
$$= 3 \text{ nails} \qquad \text{(WFCM Table A-3.4)}$$

Table A-3.6 Ridge Tension Strap Connection Requirements for Wind

Description: Based on the required capacity of each ridge tension connection, calculate the required number of 8d common nails or 10d box nails in each end of 1-1/4" x 20 gage strap connecting roof rafters at the ridge.

Procedure: Strap requirements are based on the nail connection capacities and the tensile strap capacity.

Background: Ridge straps restrain the ridge from separating under suction wind loads. Lateral capacity of 8d common nails and 10d box nails are calculated based on the *2012 NDS*. A load duration adjustment factor of C_D = 1.6 (wind/earthquake) is assumed. Spruce-Pine-Fir with G = 0.42 is assumed for framing members.

Example:

Given - 150 mph, Exposure B, 6:12 roof pitch, 24' roof span, 12" ridge strap spacing, 15 psf roof/ceiling dead load (10 psf roof assembly dead load, 5 psf ceiling assembly dead load).

Unit connection load:

From WFCM Table 3.6

w_{Ridge} = 239 plf

Calculate required connection capacity:

= 239 plf (12 in./strap)/(12 in./ft)
= 239 lbs/strap

Number of 8d common nails or 10d box nails in each end of strap:

From 2012 NDS Table 11P:

Lateral design value of 8d common nail:

Z = 81 lbs/nail
Z' = Z (C_D)
Z' = 81(1.6) lbs/nail
= 130 lbs/nail

Lateral design value of 10d box nail:

Z = 77 lbs/nail
Z' = 77(1.6) lbs/nail
= 123 lbs/nail

Calculate required number of nails based on 10d box:

n = 239 lbs / 123 lbs/nail
= 1.9 nails
= 2 nails (WFCM Table A-3.6)

Footnote 1:

See Commentary for WFCM Table 2.2B for calculations.

Footnote 5:

Collar ties may be used in lieu of ridge straps provided the collar tie connection has adequate capacity to resist roof uplift.

Lateral capacity of collar tie to rafter connection:

From *2012 NDS* Table 11N:

Lateral design value of 10d common nail for 1-1/2″ main and side members:

Z = 100 lbs/nail
Z' = Z (C_D)
Z' = 100(1.6) lbs/nail
= 160 lbs/nail

Lateral design value of 12d box nail for 1-1/2″ main and side members:

Z = 79 lbs/nail
Z' = 79(1.6) lbs/nail
Z' = 126 lbs/nail

Comparison of 8d common and 10d box nails with a steel strap to 10d common and 12d box nails with a 1½" wood side member indicates the 8d common and 10d box nails may be replaced with the same number of 10d common and 12d box nails in a collar tie, respectively.

COMMENTARY REFERENCES

R

AMERICAN WOOD COUNCIL

References

1. ANSI/AF&PA SDPWS-2008, Special Design Provisions for Wind and Seismic (SDPWS), American Wood Council, Leesburg, VA, 2008.

2. ANSI/AWC NDS-2012, National Design Specification (NDS) for Wood Construction, American Wood Council, Leesburg, VA, 2012.

3. ASCE/SEI Standard 7-10 Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA, 2010.

4. Design Aid No. 2, Toenail Connections, American Wood Council, Leesburg, VA, 2007.

5. Douglas, B.K. and H. Sugiyama, Perforated Shearwall Design Approach, ASAE, 1994.

6. Douglas, B.K. and B. R. Weeks, Considerations in Wind Design of Wood Structures, Leesburg, VA, American Wood Council, 1998.

7. International Building Code, International Code Council, Washington, DC, 2012.

8. International Residential Code, International Code Council, Washington, DC, 2012.

9. Line, P. and B.K. Douglas, Perforated Shearwall Design Method, American Wood Council, Leesburg, VA, 1996.

10. North American Specification for the Design of Cold Formed Steel Structural Members, AISI, Washington, DC, 2007.

11. Stone, J., P. Line, and B.R. Weeks, Perforated Shearwall Design, American Wood Council, Leesburg, VA, 2000.

American Wood Council

AWC Mission Statement

To increase the use of wood by assuring the broad regulatory acceptance of wood products, developing design tools and guidelines for wood construction, and influencing the development of public policies affecting the use and manufacture of wood products.

American Wood Council
222 Catoctin Circle, SE, Suite 201
Leesburg, VA 20175
www.awc.org
info@awc.org

ISBN 978-1-940383-00-2

9 781940 383002

08-13