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DESIGN OF LOADBEARING TALL WOOD STUDS FOR WIND AND GRAVITY LOADS (DES230)

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Description

Proper design of wood structures to resist high wind loads requires the correct use of wind load provisions and member design properties. A thorough understanding of the interaction between wind loads and material properties is important in the design process. Adjustments from reference wind conditions to extreme-value peak gusts require designers to make similar adjustments to design properties to ensure equivalent and economic designs. Wind load provisions have been developed for design of major structural elements using Main Wind-Force Resisting System (MWFRS) loads and secondary cladding elements using Component & Cladding (C&C) loads. Elements and subassemblies which receive loads both directly and as part of the main wind force resisting system, such as wall studs, must be checked independently for MWFRS loads and C&C loads. A load bearing stud wall design example based on the allowable stress design methods outlined in AWC's *2015 National Design Specification® (NDS®) for Wood* Construction and *2015 Wood Frame Construction Manual* along with *ASCE 7-10 Minimum Design Loads for Buildings and Other Structures* will demonstrate standard design checks for limit states of strength and deflection.

Learning Objectives

Upon completion of this webinar, participants will:

1. Understand how to analyze wall framing as part of the MWFRS per ASCE 7-10
2. Understand why wall framing is analyzed using out of plane C&C wind pressures independent of gravity loads
3. Be familiar with various ASCE 7-10 ASD load combinations used for bearing walls
4. Be knowledgeable of standards including the 2015 NDS, 2015 WFCM, and ASCE 7-10 used for design of tall walls

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- Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

Poll Question

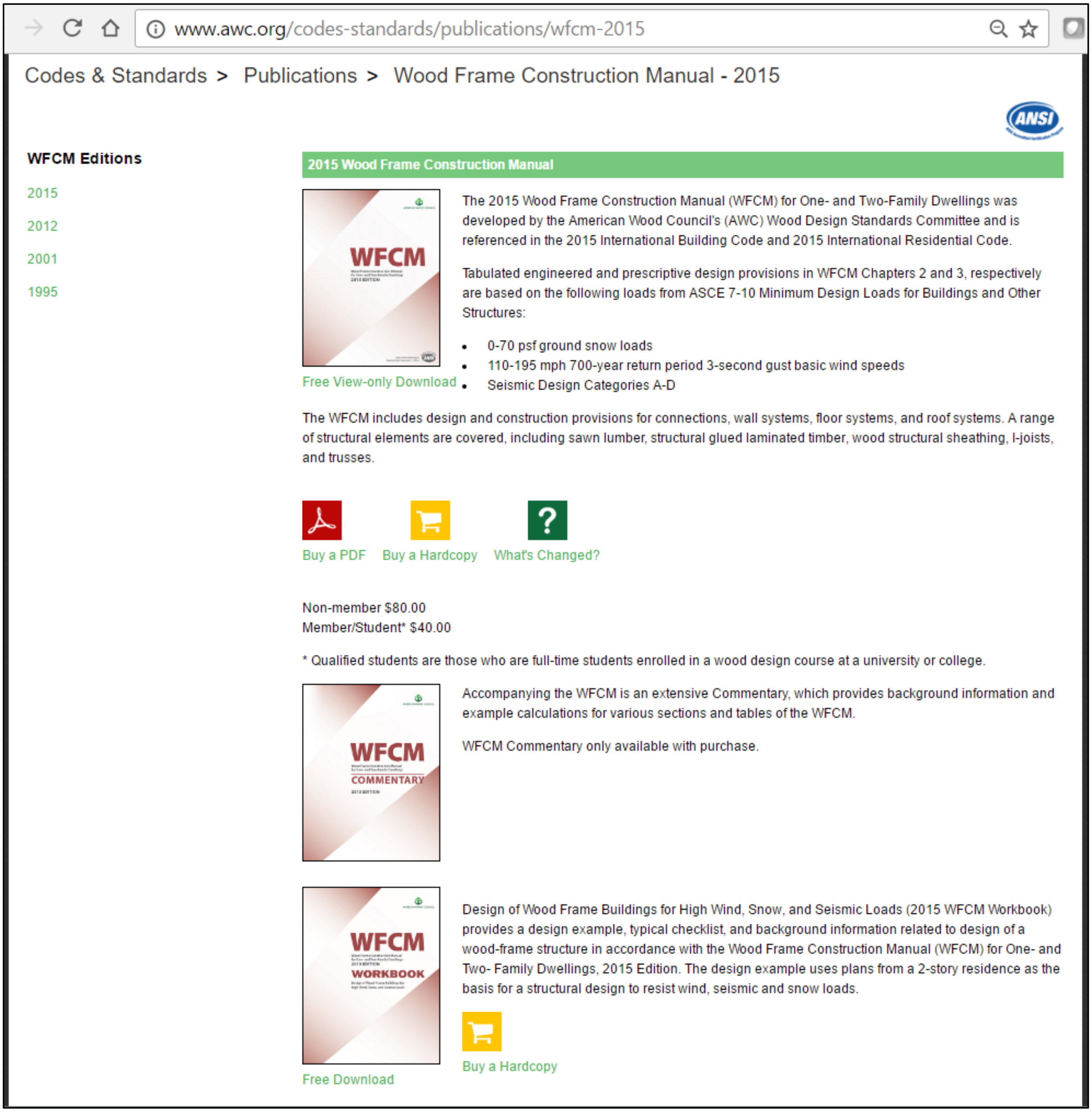
What is your profession?

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

Course Outline

- **Background on Design Loads – Wind and Gravity**
- **Engineered Provisions – Calculating Design Loads**
- **Prescriptive Provisions – Wood Frame Solutions**
- **Design Example**

Reference Material



www.awc.org/codes-standards/publications/wfcm-2015

Codes & Standards > Publications > Wood Frame Construction Manual - 2015

WFCM Editions

2015

2012

2001

1995

2015 Wood Frame Construction Manual

The 2015 Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings was developed by the American Wood Council's (AWC) Wood Design Standards Committee and is referenced in the 2015 International Building Code and 2015 International Residential Code.

Tabulated engineered and prescriptive design provisions in WFCM Chapters 2 and 3, respectively are based on the following loads from ASCE 7-10 Minimum Design Loads for Buildings and Other Structures:

- 0-70 psf ground snow loads
- 110-195 mph 700-year return period 3-second gust basic wind speeds
- Seismic Design Categories A-D

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The WFCM includes design and construction provisions for connections, wall systems, floor systems, and roof systems. A range of structural elements are covered, including sawn lumber, structural glued laminated timber, wood structural sheathing, I-joists, and trusses.

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* Qualified students are those who are full-time students enrolled in a wood design course at a university or college.

Accompanying the WFCM is an extensive Commentary, which provides background information and example calculations for various sections and tables of the WFCM.

WFCM Commentary only available with purchase.

Design of Wood Frame Buildings for High Wind, Snow, and Seismic Loads (2015 WFCM Workbook) provides a design example, typical checklist, and background information related to design of a wood-frame structure in accordance with the Wood Frame Construction Manual (WFCM) for One- and Two- Family Dwellings, 2015 Edition. The design example uses plans from a 2-story residence as the basis for a structural design to resist wind, seismic and snow loads.

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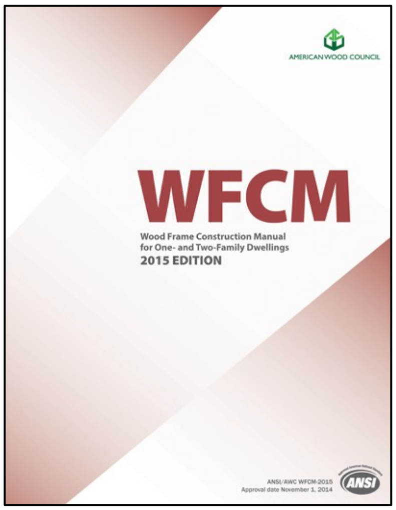
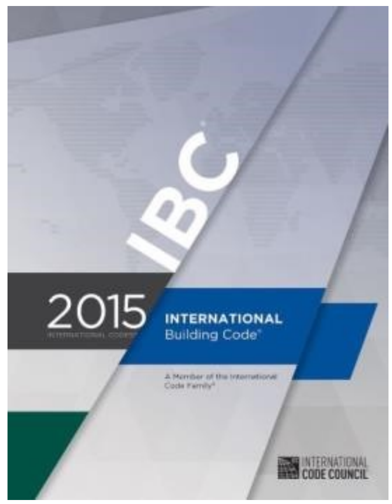
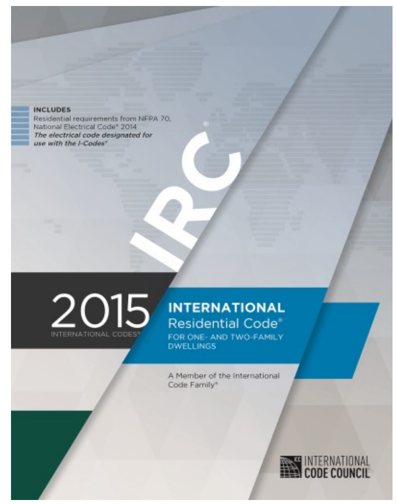
Reference Codes and Standards

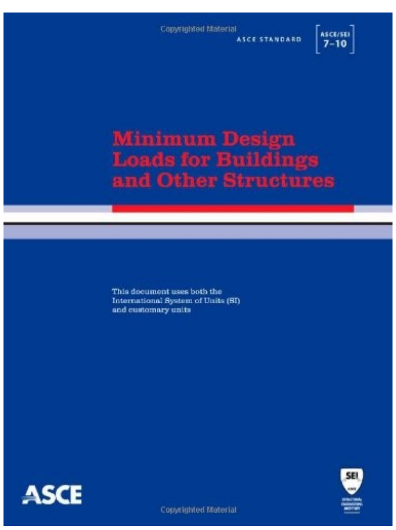
- **2001 WFCM → 2003, 2006, 2009 IRC/IBC**
- **2012 WFCM → 2012 IRC/IBC**
- **2015 WFCM → 2015 IRC/IBC**







- **ASCE 7-10 → 2015 WFCM**



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WFCM

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

COMMENTARY

2015 EDITION

WFCM Commentary

C1.1 Scope

C1.1.1 General

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

C1.1.2 Design Loads

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

Dead Loads

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

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

Live Loads

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

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

Wind Loads

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

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

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

Basic Design Equations:

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

MWFRS – Envelope Procedure:

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

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.

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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:

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

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

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shear walls. Uplift forces arise solely from uplift on the roof, and are transferred directly into the walls supporting the roof framing.

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

Snow Loads

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

Seismic Loads

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

C1.1.2.1 Torsion

Design for torsion is outside the scope of this document.

C1.1.2.2 Sliding Snow

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

C1.1.3 Applicability

C1.1.3.1 Building Dimensions

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

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

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

C1.1.3.2 Floor, Wall, and Roof Systems

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

C1.1.4 Foundation Provisions

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

C1.1.5 Protection of Openings

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

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

2015 EDITION

ANSI/AWC WFCM-2015
Approval date October 10, 2014

Approved American National Standard

ANSI

2015 WFCM

Table 1 Applicability Limitations

Attribute	Limitation	Reference Section	Figures
BUILDING DIMENSIONS			
Mean Roof Height (MRH)	33'	1.1.3.1a	1.2
Number of Stories	3	1.1.3.1a	-
Building Length and Width	80'	1.1.3.1b	-

LOAD ASSUMPTIONS (See Chapter 2 or Chapter 3 tables for load assumptions applicable to the specific tabulated requirement)	
Load Type	**Load Assumption**
Partition Dead Load	0-8 psf of floor area
Wall Assembly Dead Load	11-18 psf
Floor Dead Load	10-20 psf
Roof/Ceiling Assembly Dead Load	0-25 psf
Floor Live Load	30-40 psf
Roof Live Load	20 psf
Ceiling Live Load	10-20 psf
Ground Snow Load	0-70 psf
Wind Load	110-195 mph wind speed (700-yr. return period, 3-second gust) Exposure B, C, and D
Seismic Load	Seismic Design Category (SDC) SDC A, B, C, D_0, D_1, and D_2

ENGINEERED DESIGN

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Table 2 Engineered Design Limitations

	Attribute	Limitation	Reference Section	Figures
BUILDING DIMENSIONS				
Building	**Mean Roof Height (MRH)**	33'	2.1.3.1	1.2
	Number of Stories	3	1.1.3.1a	-
	Building Length and Width	80'	1.1.3.1b	-
FLOOR SYSTEMS				
Lumber Joists	**Joist Span**	26'	2.1.3.2a	-
	Joist Spacing	24" o.c.	2.1.3.2b	-
	Cantilevers - Supporting loadbearing	d	2.1.3.2c	2.1a
	Setbacks - Loadbearing walls[1]	d	2.1.3.2d	2.1d
Wood I-Joists	**I-Joist Span**	26'	2.1.3.2a	-
	I-Joist Spacing	24" o.c.	2.1.3.2b	-
	Cantilevers	(see manufacturer)	2.3.2.6	2.4e, 2.9a, 2.9b
	Setbacks	(see manufacturer)	2.3.2.5	2.4d
Wood Floor Trusses	**Truss Span**	26'	2.1.3.2a	-
	Truss Spacing	24" o.c.	2.1.3.2b	-
	Cantilevers	(see truss plans)	2.3.3.6	2.13a, 2.13b
	Setbacks	(see truss plans)	2.3.3.5	-
Floor Diaphragms	**Vertical Floor Offset**[1]	d_f	2.1.3.2e	2.1i
	Floor Diaphragm Aspect Ratio[1]	4:1	2.1.3.2f	2.1j
	Floor Diaphragm Openings	Lesser of 12' or 50% of Building Dimension	2.1.3.2g	2.1k
WALL SYSTEMS				
Wall Studs	**Loadbearing Wall Height**	20'	2.1.3.3a	-
	Non-Loadbearing Wall Height	20'	2.1.3.3a	-
	Wall Stud Spacing	24" o.c.	2.1.3.3b	-
Shear Walls	**Shear Wall Line Offset**[1]	4'	2.1.3.3c	2.1ℓ
	Shear Wall Story Offset[1]	No offset unless per Exception	2.1.3.3d	
	Shear Wall Segment Aspect Ratio	(see *SDPWS*)	2.1.3.3e	
ROOF SYSTEMS				
Lumber Rafters	**Rafter Span (Horizontal Projection)**[2]	26'	2.1.3.4a	-
	Rafter Spacing	24" o.c.	2.1.3.4b	-
	Eave Overhang Length[1]	Lesser of 2' or rafter span/3	2.5.1.1.2	2.1f
	Roof Slope	Flat - 12:12	2.1.3.4d	-
Wood I-Joist Roof System	**I-Joist Span**	26'	2.1.3.4a	-
	I-Joist Spacing	24" o.c.	2.1.3.4b	-
	Eave Overhang Length	(see manufacturer)	2.5.2.1.2	-
	Roof Slope	Flat - 12:12	2.1.3.4d	-
Wood Roof Trusses	**Truss Span**	60'	2.1.3.4a	-
	Truss Spacing	24" o.c.	2.1.3.4b	-
	Eave Overhang Length	(see truss plans)	2.5.3.1	-
	Roof Slope	Flat - 12:12	2.1.3.4d	-
Rakes	**Overhang Length**[1]	Lesser of 2' or purlin span/3	2.1.3.4c	2.1g
Roof Diaphragms	**Roof Diaphragm Aspect Ratio**[1]	4:1	2.1.3.4e	2.1j

[1] See exceptions.

[2] For roof snow loads, tabulated spans are limited to 20 ft.

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2.3.3.3 End Restraint

Restraint against twisting shall be provided at the end of each truss by fastening to a full-height rim, band joist, header, or other member or by using blocking panels between truss ends. Framing details (see Figure 2.15a) for end restraint shall be provided in a manner consistent with SBCA/TPI's *Building Component Safety Information (BCSI) – Guide to Good Practice for Handling, Installing, Restraining, & Bracing of Metal Plate Connected Wood Trusses,* or *ANSI/TPI 1*, or 2.3.3.1.

2.3.3.4 Chord and Web Bracing

Chord and web bracing shall be provided in a manner consistent with the guidelines provided in *BCSI, ANSI/TPI 1*, or in accordance with 2.3.3.1, and the bracing requirements specified in the construction design documents (see Figure 2.14).

2.3.3.5 Single or Continuous Floor Trusses Supporting Walls

Floor trusses shall be designed for any intermediate loads and supports as shown on the construction documents and/or plans.

2.3.3.6 Cantilevered Trusses

Cantilevered floor trusses shall be designed for all anticipated loading conditions (see Figures 2.13a-b).

2.3.3.7 Floor Openings

Framing around floor openings shall be designed to transfer loads to adjacent framing members that are designed to support the additional concentrated loads. Fasteners, connections, and stiffeners shall be designed for the loading conditions.

2.3.4 Floor Sheathing

2.3.4.1 Sheathing Spans

Floors shall be fully sheathed with sheathing capable of resisting and transferring the applied gravity loads to the floor framing members. Sheathing shall be continuous over two or more spans.

2.3.4.2 Shear Capacity

Floor sheathing and fasteners shall be capable of resisting the total shear loads calculated using Tables 2.5A and 2.5B for wind perpendicular and parallel to ridge respectively, or using Table 2.6 for seismic motion.

2.3.4.2.1 Diaphragm Chords Diaphragm chords shall be continuous for the full length of the diaphragm. Diaphragm members and chord splices shall be capable of resisting the chord forces, calculated by the following equation:

$$T = \frac{vL}{4} \qquad (2.3\text{-}1)$$

where:

T = Chord force, lbs

v = Required unit shear capacity of the floor diaphragm, plf

L = Floor diaphragm dimension perpendicular to the lateral load, ft

2.3.4.3 Sheathing Edge Support

Edges of floor sheathing shall have approved tongue-and-groove joints or shall be supported with blocking, unless 1/4-inch minimum thickness underlayment or 1-1/2 inches of approved cellular or lightweight concrete is installed, or unless the finish floor is of 3/4-inch wood strip.

2.3.5 Floor Diaphragm Bracing

At panel edges perpendicular to floor framing members, framing and connections shall be provided to transfer the lateral wind loads from the exterior wall to the floor diaphragm assembly in accordance with the requirements of Table 2.1 (see Figure 2.3).

2.4 Wall Systems

2.4.1 Exterior Walls

2.4.1.1 Wood Studs

Exterior wall studs shall be in accordance with the requirements of Table 2.9A or Table 2.10 for the wind loads specified. Exterior loadbearing studs shall be in accordance with the requirements of Table 2.9B or Table 2.11 for the gravity loads specified. Exterior loadbearing studs shall be designed to resist the uplift loads specified in Table 2.2A, independent of the requirements of Tables 2.9A, 2.9B, 2.10, and 2.11. Exterior non-loadbearing studs shall be designed to resist the rake overhang uplift loads specified in Table 2.2C.

2.4.1.1.1 Notching and Boring Notches in either edge of studs shall not be located in the middle one-third of the stud length. Notches in the outer thirds of the stud length shall not exceed 25% of the actual stud depth. Bored holes shall not exceed 40% of the actual stud depth and the edge of the hole shall not be closer than 5/8-inch to the edge of the stud. Notches and holes shall not occur in the same cross-section (see Figure 3.3b).

EXCEPTION: Bored holes shall not exceed 60% of the actual stud depth when studs are doubled.

2.4.1.1.2 Stud Continuity Studs shall be continuous between horizontal supports, including but not limited to: girders, floor diaphragm assemblies, ceiling diaphragm assemblies, and roof diaphragm assemblies. Where attic floor diaphragm or ceiling diaphragm assemblies are used to brace gable endwalls, the sheathing and fasteners shall be capable of resisting the minimum shear requirements of Table 2.5C.

2.4.1.1.3 Corners Corner framing shall be capable of transferring axial tension and compression loads from the shear walls and the structure above, connecting adjoining walls, and providing adequate backing for the attachment of sheathing and cladding materials.

2.4.1.2 Top Plates

Exterior stud walls shall be capped with a single or double top plate with bearing capacity in accordance with Table 2.9B, and bending capacity in accordance with Table 2.11. Top plates shall be tied at joints, corners, and intersecting walls to resist and transfer lateral loads to the roof or floor diaphragm in accordance with the requirements of Table 2.1. Double top plates shall be lap spliced and overlap at corners and intersections with other exterior and interior loadbearing walls.

2.4.1.3 Bottom Plate

Wall studs shall bear on a bottom plate with bearing capacity in accordance with Table 2.9B. The bottom plate shall not be less than 2 inch nominal thickness and not less than the width of the wall studs. Studs shall have full bearing on the bottom plate. Bottom plates shall be connected to transfer lateral loads to the floor diaphragm or foundation in accordance with the requirements of Table 2.1. Bottom plates that are connected directly to the foundation shall have full bearing on the foundation.

2.4.1.4 Wall Openings

Headers shall be provided over all exterior wall openings. Headers shall be supported by wall studs, jack studs, hangers, or framing anchors.

2.4.1.4.1 Headers Headers shall be in accordance with the lateral capacity requirements of Table 2.1 and the gravity capacity requirements of Table 2.11.

2.4.1.4.2 Studs Supporting Header Beams Wall and jack studs shall be in accordance with the same requirements as exterior wall studs selected in 2.4.1.1. Wall and jack studs shall be designed for additional lateral and uplift loads from headers and window sill plates in accordance with Table 2.1 and Table 2.2A.

2.4.1.4.3 Window Sill Plates Window sill plates shall be in accordance with the lateral capacity requirements of Table 2.1.

2.4.2 Interior Loadbearing Partitions

2.4.2.1 Wood Studs

Interior loadbearing studs shall be in accordance with the requirements of Table 2.9C or Table 2.11 for gravity loads.

2.4.2.1.1 Notching and Boring Notches in either edge of studs shall not be located in the middle one-third of the stud length. Notches in the outer thirds of the stud length shall not exceed 25% of the actual stud depth. Bored holes in interior loadbearing studs shall not exceed 40% of the actual stud depth and shall not be closer than 5/8-inch to the edge. Notches and holes shall not occur in the same cross-section (see Figure 3.3b).

EXCEPTION: Bored holes shall not exceed 60% of the actual stud depth when studs are doubled.

2.4.2.1.2 Stud Continuity Studs shall be continuous between horizontal supports, including but not limited to: girders, floor diaphragm assemblies, ceiling diaphragm assemblies, and roof diaphragm assemblies.

2.4.2.2 Top Plates

Interior loadbearing partition walls shall be capped with a single or double top plate with bearing capacity in accordance with Table 2.9C, and bending capacity in accordance with Table 2.11. Top plates shall be tied at joints, corners, and intersecting walls. Double top plates shall be lap spliced and overlap at corners and at intersections with other exterior and interior loadbearing walls.

2.4.2.3 Bottom Plate

Wall studs shall bear on a bottom plate with bearing capacity in accordance with Table 2.9C. The bottom plate shall not be less than 2 inch nominal thickness and not less than the width of the wall studs. Studs shall have full bearing on the bottom plate.

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List of Tables

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Table 2.1 Lateral Framing Connection Loads from Wind

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

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

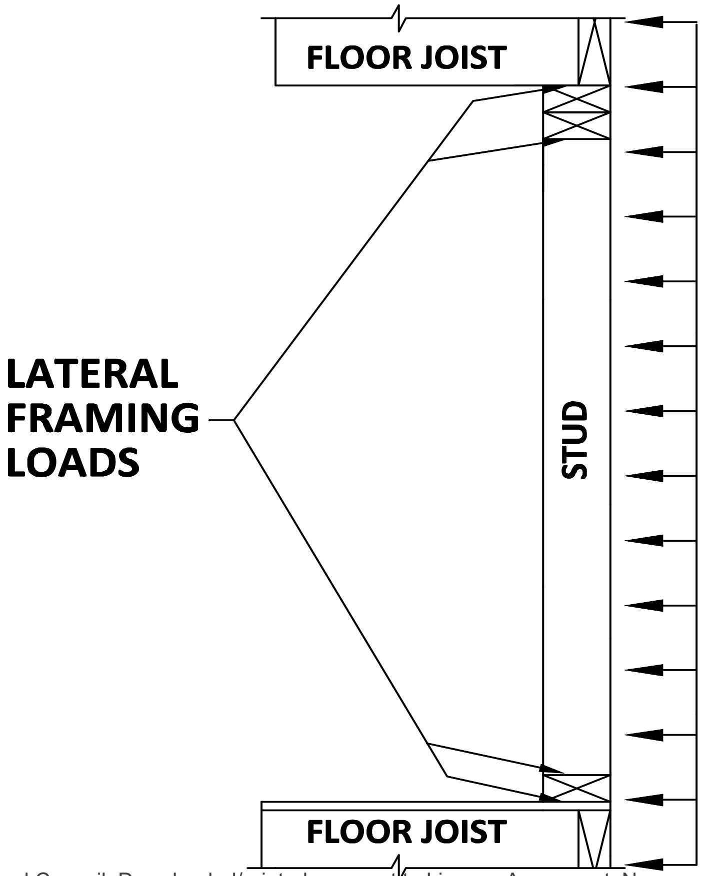
1 Tabulated framing loads shall be permitted to be multiplied by 0.92 for framing not located within 3 feet of corners for buildings less than 30 feet in width (W), or within W/10 of corners for buildings greater than 30 feet in width.

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

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

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

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



WFCM Commentary

Table 2.1 Lateral Framing Connection Loads from Wind

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

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

Background: Components and cladding (C&C) coefficients result in higher wind loads relative to main wind force resisting system (MWFRS) coefficients. When determining C&C pressure coefficients (GC_p), the effective wind area equals the tributary area of the framing member. For long and narrow tributary areas, the area width may be increased to one-third the framing member span to account for actual load distributions. This results in lower average wind pressures. The increase in width applies only to calculation of wind force coefficients.

Example:

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

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

where:

p_{max} = pressure on the wall

q = 21.15 psf (See Table C1.1)

GC_p = external pressure coefficients for C&C

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

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

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

GC_p = -1.4 for $A \leq 10$ ft²

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

GC_p = -0.8 for $A > 500$ ft²

therefore:

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

GC_p = -1.22

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

GC_{pi} = +/- 0.18

therefore:

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

= -29.61 psf (Negative pressure denotes suction)

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

= -29.61(10/2)

= |-148 plf| (WFCM Table 2.1)

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

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

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

Footnote 1:

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

End Zone

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

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

= -1.00

Interior Zone

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

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

= -0.9

The ratio of Zone 4 to Zone 5 loads is:

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

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

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

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

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

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

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

WFCM Commentary

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 psi (WFCM Table 2.9A)

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

Considerations in Wind Design of Wood Structures

http://awc.org/pdf/codes-standards/publications/archives/AWC-Considerations-0310.pdf

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Table 2.10 Exterior Wall Induced Moments from Wind Loads

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

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

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

WFCM Commentary

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})

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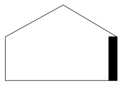
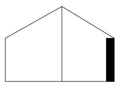
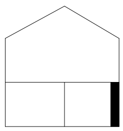
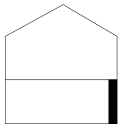
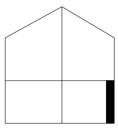
Table 2.9B Exterior Wall Stud Compression Stresses

Dead Load Assumptions: Roof Assembly DL = 20 psf, Wall Assembly DL = 121 plf, Floor Assembly DL = 10 psf, Floor LL = 40 psf

Ground Snow Load or Roof Live Load			20 psf RLL				30 psf GSL				50 psf GSL				70 psf GSL			
			Building Width (ft)															
Loadbearing Wall Supporting	Stud Spacing	Stud Size	12	24	36	60	12	24	36	60	12	24	36	60	12	24	36	60
			Induced f_c (psi)[1]															
Roof & Clear Span Ceiling	12 in.	2x4	84	130	175	267	92	140	188	286	117	182	247	380	142	224	307	473
		2x6	53	83	112	170	58	89	120	182	74	116	157	242	91	143	195	301
		2x8	41	63	85	129	44	67	91	138	56	88	119	183	69	108	148	229
	16 in.	2x4	112	173	234	356	122	186	251	381	156	243	330	506	190	299	409	631
		2x6	71	110	149	226	78	119	160	242	99	154	210	322	121	190	260	402
		2x8	54	83	113	172	59	90	121	184	75	117	159	244	92	144	197	305
	24 in.	2x4	168	259	351	534	183	279	376	572	234	364	495	-	285	448	613	-
		2x6	107	165	223	340	117	178	239	364	149	232	315	483	181	285	390	603
		2x8	81	125	169	258	88	135	182	276	113	176	239	367	138	216	296	457
Center Bearing Roof & Ceiling	12 in.	2x4	61	84	107	153	73	100	128	172	93	131	171	230	113	162	213	291
		2x6	39	53	68	97	46	64	82	109	59	84	109	147	72	103	136	185
		2x8	30	41	52	74	35	48	62	83	45	63	82	111	55	78	103	141
	16 in.	2x4	82	112	142	203	97	134	171	229	124	175	228	307	151	217	284	388
		2x6	52	71	91	129	62	85	109	146	79	111	145	196	96	138	181	247
		2x8	39	54	69	98	47	64	83	111	60	85	110	148	73	105	137	187
	24 in.	2x4	122	168	214	305	145	200	257	344	186	263	342	461	226	325	426	582
		2x6	78	107	136	194	92	128	163	219	118	167	217	293	144	207	271	371
		2x8	59	81	103	147	70	97	124	166	90	127	165	223	109	157	206	281
Roof, Ceiling, & 1 Center Bearing Floor	12 in.	2x4	122	185	248	374	128	193	258	388	147	224	302	458	171	259	347	529
		2x6	78	118	158	238	81	123	164	247	94	143	192	292	109	165	221	336
		2x8	59	89	120	180	62	93	124	187	71	108	146	221	83	125	167	255
	16 in.	2x4	163	247	331	498	171	257	343	517	196	299	403	611	228	345	462	-
		2x6	104	157	210	317	109	163	219	329	125	190	256	389	145	219	294	449
		2x8	79	119	160	241	82	124	166	250	95	144	194	295	110	166	223	340
	24 in.	2x4	245	370	496	-	256	385	515	-	294	449	604	-	342	517	-	-
		2x6	156	236	316	476	163	245	328	494	187	285	384	583	218	329	441	-
		2x8	118	179	239	361	124	186	249	375	142	217	292	442	165	250	335	510
Roof, Ceiling, & 1 Clear Span Floor	12 in.	2x4	145	231	317	488	151	238	326	502	170	270	371	573	189	302	415	643
		2x6	92	147	201	311	96	152	208	320	108	172	236	364	120	192	264	409
		2x8	70	111	153	236	73	115	157	242	82	130	179	276	91	146	200	310
	16 in.	2x4	194	308	422	651	201	318	435	-	227	360	494	-	252	402	553	-
		2x6	123	196	269	414	128	202	277	426	144	229	314	486	160	256	352	546
		2x8	93	149	204	314	97	153	210	323	109	174	239	369	122	194	267	414
	24 in.	2x4	290	462	633	-	302	477	652	-	340	540	-	-	378	603	-	-
		2x6	185	294	403	621	192	303	415	639	216	344	472	-	240	384	528	-
		2x8	140	223	306	471	146	230	315	485	164	261	358	553	182	291	401	621
Center Bearing Roof, Ceiling, & 1 Floor	12 in.	2x4	102	145	188	274	111	157	204	288	126	181	236	332	142	204	268	378
		2x6	65	92	120	174	71	100	130	183	80	115	150	211	90	130	170	240
		2x8	49	70	91	132	54	76	99	139	61	87	114	160	69	98	129	182
	16 in.	2x4	136	194	251	365	148	210	272	384	168	241	315	443	189	272	357	503
		2x6	87	123	160	232	94	133	173	245	107	153	200	282	120	173	227	320
		2x8	66	93	121	176	71	101	131	186	81	116	152	214	91	131	172	243
	24 in.	2x4	205	290	376	547	222	315	408	576	252	361	472	-	284	408	535	-
		2x6	130	185	239	348	141	200	260	367	160	230	300	423	181	260	341	481
		2x8	99	140	182	264	107	152	197	278	122	174	228	321	137	197	258	365

1 Tabulated compression stresses (f_c) shall be less than or equal to the allowable compression perpendicular to grain design value ($F_{c\perp}'$) for top and bottom plates, and less than or equal to the allowable compression parallel to grain design value ($F_{c\parallel}'$) for studs.

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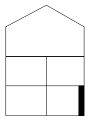
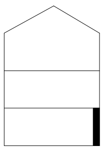
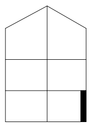
Table 2.9B Exterior Wall Stud Compression Stresses (Cont.)

Dead Load Assumptions: Roof Assembly DL = 20 psf, Wall Assembly DL = 121 plf, Floor Assembly DL = 10 psf, Floor LL = 40 psf

Ground Snow Load or Roof Live Load			20 psf RLL				30 psf GSL				50 psf GSL				70 psf GSL			
			Building Width (ft)															
Loadbearing Wall Supporting	Stud Spacing	Stud Size	12	24	36	60	12	24	36	60	12	24	36	60	12	24	36	60
			Induced f_c (psi)[1]															
Roof, Ceiling, & 2 Center Bearing Floors	12 in.	2x4	168	254	340	511	174	261	349	525	193	293	394	596	212	325	438	-
		2x6	107	162	216	325	111	166	222	334	123	187	250	379	135	207	279	424
		2x8	81	123	164	247	84	126	169	254	93	141	190	288	102	157	211	322
	16 in.	2x4	224	339	453	-	232	349	466	-	257	391	525	-	283	433	584	-
		2x6	143	215	288	434	148	222	296	446	164	249	334	505	180	276	372	565
		2x8	108	163	219	329	112	168	225	338	124	189	253	383	136	209	282	429
	24 in.	2x4	336	508	-	-	348	523	-	-	386	586	-	-	424	649	-	-
		2x6	214	323	432	650	221	333	444	-	246	373	501	-	270	413	557	-
		2x8	162	245	328	493	168	252	337	507	186	283	380	575	205	314	423	643
Roof, Ceiling, & 2 Clear Span Floors	12 in.	2x4	214*	351*	488*	-	220	353	488*	-	239	385	531	-	258	416	575	-
		2x6	136*	223*	311*	485*	140	225	311*	485*	152	245	338	524	164	265	366	569
		2x8	103*	169*	236*	368*	106	170	236*	368*	115	186	256	398	124	201	278	432
	16 in.	2x4	285*	468*	651*	-	293	470	651*	-	318	513	-	-	344	555	-	-
		2x6	181*	298*	414*	647*	186	299	414*	647*	203	326	450	-	219	353	488	-
		2x8	138*	226*	314*	491*	141	227	314*	491*	154	247	342	531	166	268	370	576
	24 in.	2x4	428*	-	-	-	439	-	-	-	477	-	-	-	515	-	-	-
		2x6	272*	447*	621*	-	280	449	621*	-	304	489	-	-	328	530	-	-
		2x8	207*	339*	471*	-	212	341	471*	-	230	371	512	-	249	402	555	-
Center Bearing Roof, Ceiling, & 2 Floors	12 in.	2x4	148	214	282	420	157	226	296	426	172	249	328	469	187	273	359	515
		2x6	94	136	180	267	100	144	188	271	109	159	208	299	119	174	229	328
		2x8	72	103	136	203	76	109	143	205	83	120	158	227	90	132	173	249
	16 in.	2x4	198	285	377	559	209	301	394	567	229	333	437	626	250	364	479	-
		2x6	126	181	240	356	133	192	251	361	146	212	278	398	159	231	305	437
		2x8	95	138	182	270	101	146	190	274	111	161	211	302	121	176	231	331
	24 in.	2x4	296	428	565	-	314	452	592	-	344	499	655	-	374	545	-	-
		2x6	189	272	360	534	200	288	376	542	219	317	417	598	238	347	457	655
		2x8	143	207	273	405	151	218	286	411	166	241	316	453	181	263	347	497

1 Tabulated compression stresses (f_c) shall be less than or equal to the allowable compression perpendicular to grain design value ($F_{c\perp}'$) for top and bottom plates, and less than or equal to the allowable compression parallel to grain design value ($F_{c||}'$) for studs.

* Tabulated compression stresses are based on the maximum load combination: Dead Load + Floor Live Load (i.e. D + L). Reduced unit loads are permitted for load combinations that include Roof Live Load (RLL) and Ground Snow Load (GSL).

WFCM Commentary

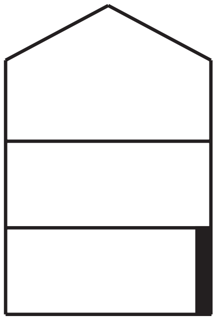
Table 2.9B Exterior Wall Stud Compression Stresses

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 in./12)

w_{header} = 2,665 plf (from Table 2.11)

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

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

P = 2,786 (12 in./12in./ft) = 2,786 lbs

Calculate compression stress:

f_c = P / A

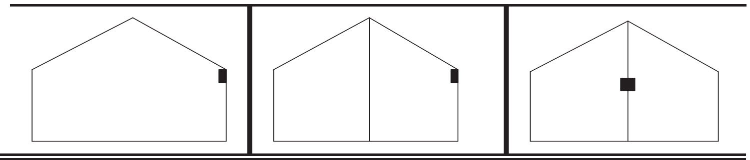
= 2,786 / 8.25

= 338 psi (WFCM Table 2.9B)

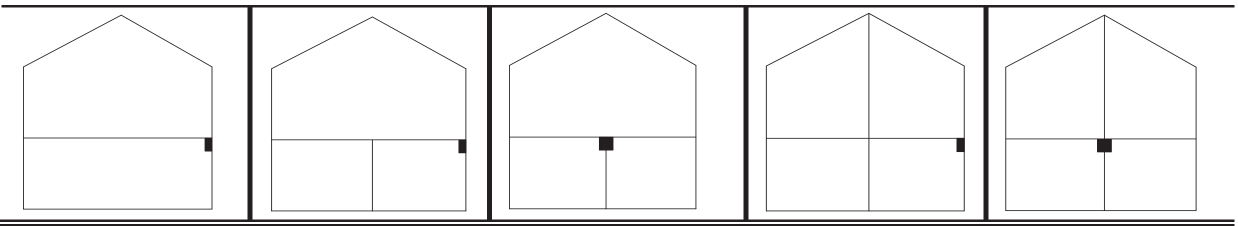
2015 WFCM

Table 2.11 Loadbearing Wall Loads from Snow or Live Loads

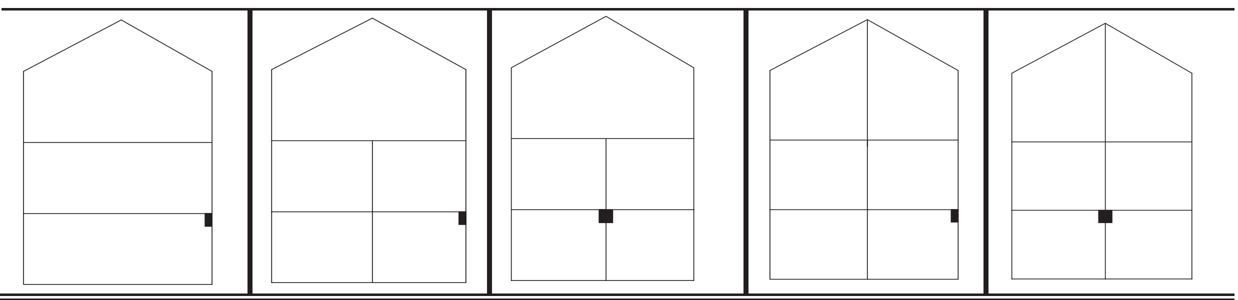
(For Wall Studs, Headers, and Girders)
Dead Load Assumptions: Roof Assembly DL = 20 psf, Wall Assembly DL = 121 plf, Floor Assembly DL = 10 psf, Floor LL = 40 psf



Ground Snow Load or Roof Live Load (psf)	RLL	GSL			RLL	GSL			RLL	GSL		
	20	30	50	70	20	30	50	70	20	30	50	70
Roof Span (ft)	Unit Header/Girder Beam Loads (plf)[1]											
12	320	360	493	627	200	260	367	473	240	259	351	443
24	560	613	834	1056	320	405	568	732	480	517	702	887
36	800	867	1178	1489	440	553	776	998	720	776	1053	1330
60	1280	1379	1872	2365	680	782	1089	1407	1200	1293	1755	2217



Ground Snow Load or Roof Live Load (psf)	RLL	GSL			RLL	GSL			RLL	GSL			RLL	GSL			RLL	GSL		
	20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
Roof Span (ft)	Unit Header/Girder Beam Loads (plf)[1]																			
12	641	671	771	871	521	551	651	778	300*	300*	300*	300*	416	461	541	624	571	585	654	724
24	1091	1130	1297	1463	851	890	1057	1237	600*	600*	600*	600*	641	705	827	950	1021	1049	1188	1326
36	1541	1591	1824	2058	1181	1231	1464	1700	900*	900*	900*	900*	866	951	1118	1284	1471	1513	1721	1929
60	2441	2515	2885	3255	1841	1915	2285	2655	1500*	1500*	1500*	1500*	1316	1392	1623	1862	2371	2441	2787	3134



Ground Snow Load or Roof Live Load (psf)	RLL	GSL			RLL	GSL			RLL	GSL			RLL	GSL			RLL	GSL		
	20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
Roof Span (ft)	Unit Header/Girder Beam Loads (plf)[1]																			
12	1002*	1032	1132	1232	762	792	892	992	721*	721*	721*	721*	657	702	782	862	962*	962*	1015	1085
24	1722*	1731	1898	2064	1212	1251	1418	1584	1321*	1321*	1321*	1321*	1002	1066	1188	1311	1682*	1682*	1789	1927
36	2442*	2442*	2665	2899	1662	1712	1945	2179	1921*	1921*	1921*	1921*	1362	1432	1599	1765	2402*	2402*	2562	2770
60	3882*	3882*	4206	4576	2562	2636	3006	3376	3121*	3121*	3121*	3121*	2082	2113	2344	2583	3842*	3842*	4108	4455

1 Tabulated loads assume simply-supported single span floor joists. For continuous two span floor joists, loads on interior loadbearing walls, headers, and girders shall be multiplied by 1.25.

* Tabulated unit header/girder beam loads (plf) are based on the maximum load combination: Dead Load + Floor Live Load (i.e. D + L). Reduced unit loads are permitted for load combinations that include Roof Live Load (RLL) and Ground Snow Load (GSL).

WFCM Commentary

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.

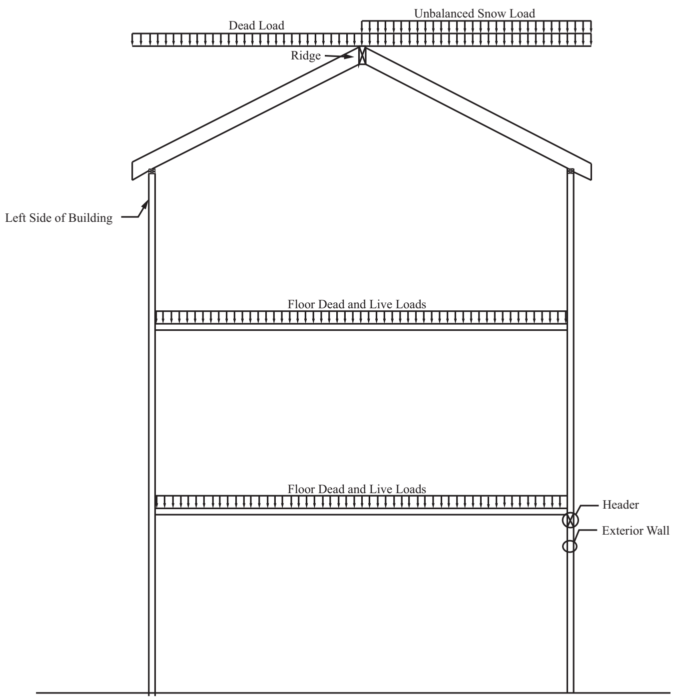
Background: In calculating the unit header/girder beam loads for each building configuration in Table 2.11, the following *ASCE 7-10* load combinations were considered. When designing structural wood members, it is also necessary to consider the effect of load duration as follows:

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:

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

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

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

$$\text{TOTAL} = 1{,}002\text{ plf}$$

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:

$$q_{snow} = Ip_g\,(\text{building width} < 40\text{ ft})$$

$$= (1.0)(30\text{ psf}) = 30.0\text{ psf}$$

WFCM Commentary

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.

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

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

$R_{right}(\text{Snow}) = 467$ 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.

http://awc.org/pdf/codes-standards/publications/archives/AWC-ASCE7-05-SnowProvisions-0607.pdf

Poll Question

True or False:

Studs should be checked for either MWFRS or C&C wind loads, but not both.

PRESCRIPTIVE DESIGN

2015 WFCM

Table 3 Prescriptive Design Limitations

	Attribute	Limitation	Reference Section	Figures
BUILDING DIMENSIONS				
Building	Mean Roof Height (MRH)	33'	2.1.3.1	1.2
	Number of Stories	3	1.1.3.1a	-
	Building Length and Width	80'	1.1.3.1b	-
FLOOR SYSTEMS				
Lumber Joists	Joist Span	26'	3.1.3.2a	-
	Joist Spacing	24" o.c.	3.1.3.2b	-
	Cantilevers - Supporting loadbearing walls[1]	d	3.1.3.2c	2.1a
	Setbacks - Loadbearing walls[1]	d	3.1.3.2d	2.1d
Floor Diaphragm	Vertical Floor Offset	d_f	3.1.3.2e	2.1i
	Floor Diaphragm Aspect Ratio	Tables 3.16B and 3.16C	3.1.3.2f	-
	Floor Diaphragm Openings	Lesser of 12' or 50% of Building Dimension	3.1.3.2g	2.1k
WALL SYSTEMS				
Wall Studs	Loadbearing Wall Height	10'	3.1.3.3a	-
	Non-Loadbearing Wall Height	20'	3.1.3.3a	-
	Wall Stud Spacing	24" o.c.	3.1.3.3b	-
Shear Walls	Shear Wall Line Offset[1]	4'	3.1.3.3c	2.1ℓ, 3.1b
	Shear Wall Story Offset[1]	No offset unless per Exception	3.1.3.3d	
	Shear Wall Segment Aspect Ratio	Table 3.17D	3.1.3.3e	
ROOF SYSTEMS				
Lumber Rafters	Rafter Span (Horizontal Projection)[2]	26'	3.1.3.4a	-
	Rafter Spacing	24" o.c.	3.1.3.4b	-
	Eave Overhang Length[1]	Lesser of 2' or rafter span/3	3.1.3.4c	2.1f
	Rake Overhang Length[1]	Lesser of 2' or purlin span/2	3.1.3.4c	2.1g
	Roof Slope	Flat - 12:12	3.1.3.4d	-
Roof Diaphragms	Roof Diaphragm Aspect Ratio[1]	Tables 3.16A and 3.16C	3.1.3.4e	-

[1] See exceptions.

[2] For roof snow loads, tabulated spans are limited to 20 ft.

2015 WFCM

3.3.4 Floor Sheathing

3.3.4.1 Sheathing Spans

Floor sheathing spans shall not exceed the provisions of Table 3.14.

3.3.4.2 Sheathing Edge Support

Edges of floor sheathing shall have approved tongue-and-groove joints or shall be supported with blocking, unless ¼ inch minimum thickness underlayment or 1½ inches of approved cellular or lightweight concrete is installed, or unless the finish floor is of ¾ inch wood strip.

3.3.5 Floor Diaphragm Bracing

For 700-year return period, 3-second gust wind speeds greater than 130 mph, blocking and connections shall be provided at panel edges perpendicular to floor framing members in the first two bays of framing and shall be spaced at a maximum of 4 feet on center. Nailing requirements are given in Table 3.1 (see Figure 3.7b).

3.4 Wall Systems

3.4.1 Exterior Walls

3.4.1.1 Wood Studs

Wall studs shall be in accordance with the maximum spans for common species and grades of walls studs specified in Tables 3.20A-B and spaced in accordance with Table 3.20C. Exterior loadbearing studs shall be limited to a height of 10 feet or less between horizontal supports as specified in Table 3.20C. Exterior non-loadbearing studs shall be limited to a height of 14 feet or less for 2x4 studs and 20 feet or less for 2x6 and 2x8 studs in accordance with Table 3.20C.

3.4.1.1.1 Notching and Boring Notches in either edge of studs shall not be located in the middle one-third of the stud length. Notches in the outer thirds of the stud length shall not exceed 25% of the actual stud depth. Bored holes shall not exceed 40% of the actual stud depth and the edge of the hole shall not be closer than 5/8 inch to the edge of the stud (see Figure 3.3b). Notches and holes shall not occur in the same cross-section.

> **EXCEPTION:** Bored holes shall not exceed 60% of the actual stud depth when studs are doubled.

3.4.1.1.2 Stud Continuity Studs shall be continuous between horizontal supports, including but not limited to: girders, floor diaphragm assemblies, ceiling diaphragm assemblies, and roof diaphragm assemblies. When attic floor diaphragm or ceiling diaphragm assemblies are used to brace gable endwalls, the sheathing and fasteners shall be as specified in Table 3.15. The framing and connections shall be capable of transferring the loads into the ceiling or attic floor diaphragm (see Figures 3.7a-b).

3.4.1.1.3 Corners A minimum of three studs shall be provided at each corner of an exterior wall (see Figures 3.8a-b).

> **EXCEPTION:** Reduced stud requirements shall be permitted provided shear walls are not continuous to corners. Framing must be capable of transferring axial tension and compression loads from above and providing adequate backing for the attachment of sheathing and cladding materials.

3.4.1.2 Top Plates

Double top plates shall be provided at the top of all exterior stud walls. The double plates shall overlap at corners and at intersections with other exterior or interior loadbearing walls (see Figure 3.8d). Double top plates shall be lap spliced with end joints offset in accordance with the minimum requirements given in Table 3.21.

3.4.1.3 Bottom Plates

Bottom plates shall not be less than 2 inch nominal thickness and not less than the width of the wall studs. Studs shall have full bearing on the bottom plate.

3.4.1.4 Wall Openings

Headers shall be provided over all exterior wall openings. Headers shall be supported by wall studs, jack studs, hangers, or framing anchors (see Figures 3.9a-b).

3.4.1.4.1 Headers Maximum spans for common species of lumber headers and structural glued laminated timber beams used in exterior loadbearing walls shall not exceed the lesser of the applicable spans given in Tables 3.22A-E and Table 3.23A. Maximum spans for common species of lumber headers used in exterior non-loadbearing walls shall not exceed spans given in Table 3.23B.

3.4.1.4.2 Full Height Studs Full height studs shall meet the same requirements as exterior wall studs selected in 3.4.1.1 (see Figures 3.9a-b). The minimum number of full height studs at each end of the header shall not be less

AMERICAN WOOD COUNCIL

WFCM

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

2015 EDITION

WORKBOOK

Design of Wood Frame Buildings for High Wind, Snow, and Seismic Loads

BUILDING DESCRIPTION

WFCM Workbook

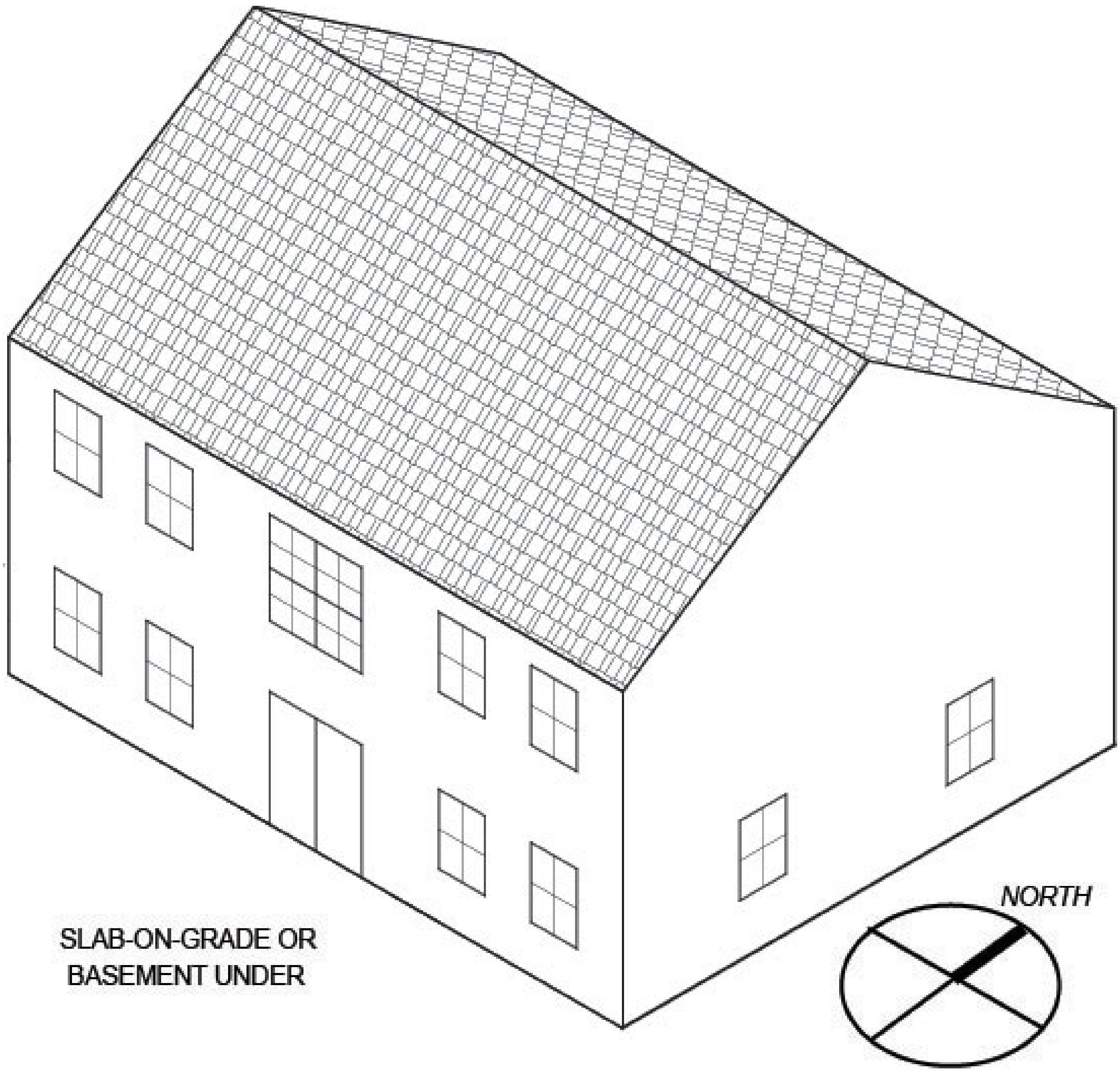
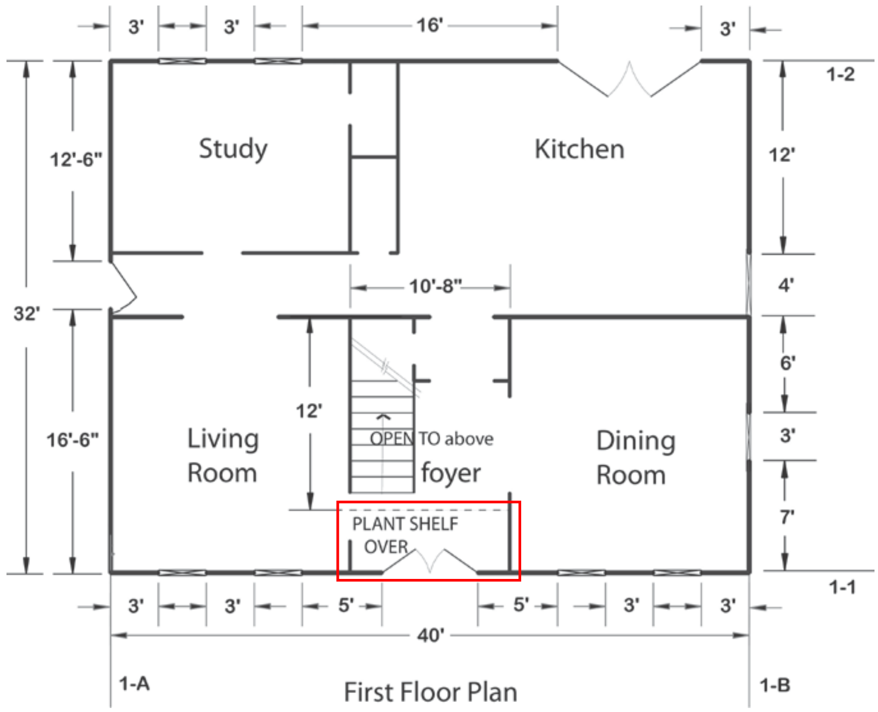


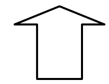
Figure 1: Isometric view (roof overhangs not shown).

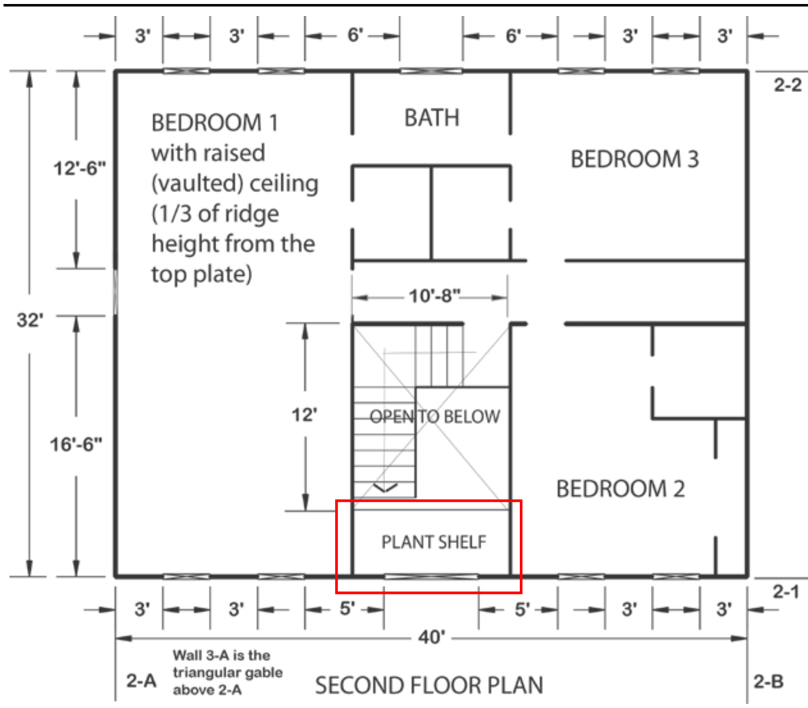
BUILDING DESCRIPTION

WFCM Workbook

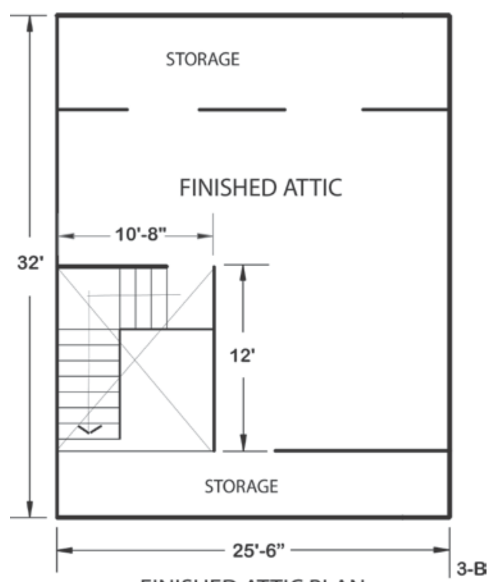


First Floor Plan





SECOND FLOOR PLAN



FINISHED ATTIC PLAN

Wall Heights	= 9'	Windows
Finished Grade to Foundation Top	= 1'	Typical 3'x4'-6"
Floor Assembly Height	= 1'	Foyer 6'x4'-6"
Roof Pitch	= 7:12	Kitchen 4'x4'-6"
House Mean Roof Height	= 24.7'	Bath 4'x6'
Roof Overhangs	= 2'	Doors
Building Length (L)	= 40'	Typical 3'x7'-6"
Building Width (W)	= 32'	Foyer 6'x7'-6"
Top plate to ridge height	= 9.3'	Kitchen 9'x7'-6"

WFCM Workbook

LOADS ON THE BUILDING

Structural systems in the *WFCM 2015Edition* have been sized using dead, live, snow, seismic and wind loads in accordance with *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures*.

Lateral Loads:

Wind:

3-second gust wind speed in Exposure Category B (700 yr. return) = 160 mph

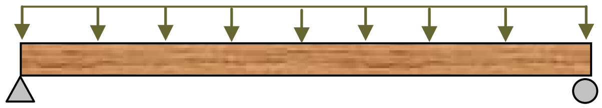
Seismic:

Simplified Procedure (ASCE 7-10 Section 12.14)

Seismic Design Category (SDC) – (ASCE 7-10 Section 11.4.2 and IRC Subcategory) = D_1

Vertical force distribution factor (F) - (ASCE 7-10 Section 12.14.8.1) = 1.2

Gravity Loads*:



Roof:

Roof Dead Load	=	10 psf
Ground Snow Load, P_g	=	30 psf
Roof Live Load	=	20 psf

Ceiling:

Roof Ceiling Load	=	10 psf

*Assumptions vary for wind and seismic dead loads

Floors:

First Floor Live Load	=	40 psf
Second Floor Live Load	=	30 psf
Attic Floor Live Load	=	30 psf
Floor Dead Load	=	10 psf

Walls:

Wall Dead Load	=	11 psf

Deflection limits per *2015 IRC*

Roof Rafters with flexible Ceiling Attached	L/Δ =	240
Roof Rafters with no Ceiling Attached	L/Δ =	180
Raised Ceiling Joists with flexible finish	L/Δ =	240
Floor Joists	L/Δ =	360
Exterior Studs (gypsum interior)	H/Δ =	180

Note: See comparable deflection limits in *2015 IBC* section 2308 for joists and rafters.

TABLE R301.7
ALLOWABLE DEFLECTION OF STRUCTURAL MEMBERS[b, c]

STRUCTURAL MEMBER	ALLOWABLE DEFLECTION
Rafters having slopes greater than 3:12 with finished ceiling not attached to rafters	*L*/180
Interior walls and partitions	*H*/180
Floors	*L*/360
Ceilings with brittle finishes (including plaster and stucco)	*L*/360
Ceilings with flexible finishes (including gypsum board)	*L*/240
All other structural members	*L*/240
Exterior walls—wind loads[a] with plaster or stucco finish	*H*/360
Exterior walls—wind loads[a] with other brittle finishes	*H*/240
Exterior walls—wind loads[a] with flexible finishes	*H*/120[d]
Lintels supporting masonry veneer walls[e]	*L*/600

Note: *L* = span length, *H* = span height.

d. Deflection for *exterior walls* with interior gypsum board finish shall be limited to an allowable deflection of *H*/180.

2015 International Residential Code for One- and Two-Family Dwellings, International Code Council, Inc., Washington, DC. www.iccsafe.org

Wall Framing

WFCM Workbook

Non-Loadbearing (3-A and 2-A)

There are 2 options for designing gable end studs: 1) balloon framing from the second floor to the rafters with a maximum stud length of 18.3 ft, or 2) stud length of 12.1 ft to the raised ceiling and gable studs of 6.2 ft above with the raised ceiling diaphragm used for bracing.

Choose Studs from Table 3.20A or 3.20B and Table 3.20C

Three second gust wind speed (700 yr) and Exposure category:................ 160 mph Exp. B
Exterior Studs (ext. wood siding and int. gypsum bd.) Deflection:............ H/180 in.
Sheathing Type (wood structural panel or minimum sheathing): .... WSP

Option 1: Wall Height:.. 18.3 (max) ft

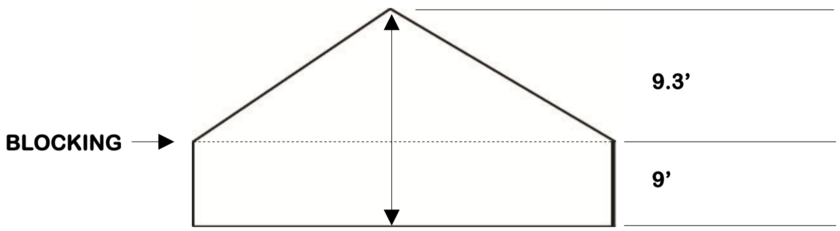
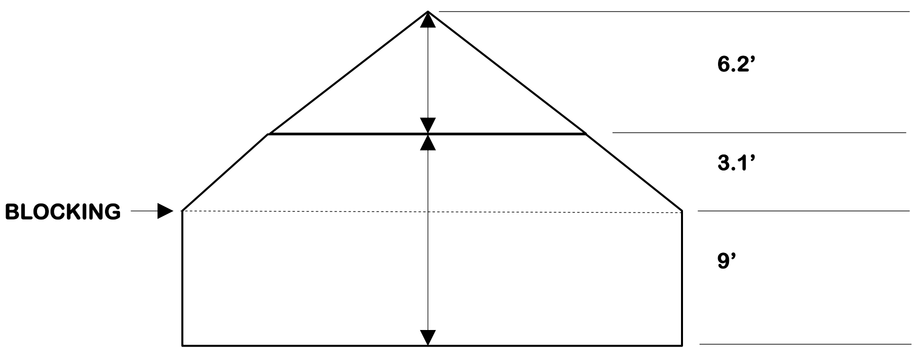


Table W4.5 Selection of Species, Grade, Size, and Spacing for Non-loadbearing Studs (Tables 3.20B1 and 3.20C)

Specie	Douglas Fir-Larch	Hem-Fir	Southern Pine	Spruce-Pine-Fir
Spacing	12" *	12" *	12" *	12" *
Grade	No. 2	No. 2	No. 2	No. 2
Size	2x6	2x6	2x6	2x6
Maximum Length (Wind)	**19'-5"** OK	**18'-0"** NG**	**18'-6"** OK	**18'-6"** OK
Maximum Length (D+L)	20'-0" OK	20'-0" OK	20'-0" OK	20'-0" OK

* Stud spacing can be increased to 16" o.c. at a distance of roughly 4-5' on either side of the ridge where stud heights drop to levels that allow greater spacing. Stud spacing of 16" o.c. at the corners also works based on Table 3.20B1 Footnote “a” since allowable stud heights at 24" o.c. are greater than 9'.
** Double studs at the ridge location.

Option 2: Wall Height:.. 12.1 (max) ft



Option 2 solution is shown in Table W5.3. Choose Option 2 to keep stud sizes at 2x6 for consistency with other framing. No.3/Stud grade 2x6 can be used for framing above the ceiling diaphragm level (3-A) based on calculations from Table W4.6.

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Table 3.20B1 (Cont.) Maximum Exterior Loadbearing[1] and Non-Loadbearing Stud Lengths for Common Lumber Species Resisting Interior Zone Wind Loads - Stud Deflection Limit = H/180

(Fully Sheathed with a Minimum of 3/8" Wood Structural Panels)[a]

Exposure B
H/180
3/8" WSP

700-yr. Wind Speed 3-second gust (mph)			150			160			170			180			195		
Stud Spacing (in.)	Species	Grade	2x4	2x6	2x8	2x4	2x6	2x8	2x4	2x6	2x8	2x4	2x6	2x8	2x4	2x6	2x8
			Maximum Allowable Stud Length (ft-in.)[1]														
12	DFL	SS	13 - 11	20-0†	20-0†	13 - 3	20-0†	20-0†	12 - 9	19 - 9	20-0†	12 - 3	18 - 11	20-0†	11 - 7	17 - 11	20-0†
	DFL	No.1	13 - 4	20-0†	20-0†	12 - 9	19 - 9	20-0†	12 - 3	19 - 0	20-0†	11 - 9	18 - 3	20-0†	11 - 2	17 - 3	20-0†
	DFL	No.2	13 - 1	20-0†	20-0†	12 - 6	19 - 5	20-0†	12 - 0	18 - 7	20-0†	11 - 6	17 - 10	20-0†	10 - 11	16 - 10	20-0†
	DFL	No.3/Stud	12 - 6	17 - 11	20-0†	11 - 9	16 - 8	20-0†	11 - 0	15 - 8	19 - 4	10 - 4	14 - 8	18 - 2	9 - 6	13 - 6	16 - 8
	DFL	Standard	10 - 8	-	-	9 - 11	-	-	9 - 4	-	-	8 - 9	-	-	8 - 0	-	-
	HF	SS	13 - 1	20-0†	20-0†	12 - 6	19 - 5	20-0†	12 - 0	18 - 7	20-0†	11 - 6	17 - 10	20-0†	10 - 11	16 - 10	20-0†
	HF	No.1	12 - 10	19 - 10	20-0†	12 - 3	18 - 11	20-0†	11 - 9	18 - 2	20-0†	11 - 3	17 - 5	20-0†	10 - 8	16 - 6	20-0†
	HF	No.2	12 - 2	18 - 10	20-0†	11 - 8	18 - 0	20-0†	11 - 2	17 - 3	20-0†	10 - 9	16 - 7	20-0†	10 - 2	15 - 8	20-0†
	HF	No.3/Stud	11 - 10	17 - 5	20-0†	11 - 4	16 - 3	20-0†	10 - 9	15 - 3	18 - 10	10 - 1	14 - 4	17 - 9	9 - 3	13 - 2	16 - 3
	HF	Standard	10 - 5	-	-	9 - 8	-	-	9 - 1	-	-	8 - 7	-	-	7 - 10	-	-
	SP	SS	13 - 8	20-0†	20-0†	13 - 1	20-0†	20-0†	12 - 6	19 - 4	20-0†	12 - 0	18 - 7	20-0†	11 - 4	17 - 7	20-0†
	SP	No.1	13 - 1	20-0†	20-0†	12 - 6	19 - 5	20-0†	12 - 0	18 - 7	20-0†	11 - 6	17 - 10	20-0†	10 - 11	16 - 10	20-0†
	SP	No.2	12 - 6	19 - 4	20-0†	11 - 11	18 - 6	20-0†	11 - 5	17 - 9	20-0†	11 - 0	17 - 0	20-0†	10 - 5	16 - 1	20-0†
	SP	No.3	11 - 4	16 - 4	20-0†	10 - 7	15 - 3	18 - 9	9 - 11	14 - 3	17 - 6	9 - 4	13 - 5	16 - 6	8 - 7	12 - 4	15 - 2
	SP	Stud	11 - 4	16 - 4	20-0†	10 - 7	15 - 3	18 - 9	9 - 11	14 - 3	17 - 6	9 - 4	13 - 5	16 - 6	8 - 7	12 - 4	15 - 2
	SP	Standard	9 - 7	-	-	9 - 0	-	-	8 - 5	-	-	7 - 11	-	-	-	-	-
	SPF	SS	12 - 10	19 - 10	20-0†	12 - 3	18 - 11	20-0†	11 - 9	18 - 2	20-0†	11 - 3	17 - 5	20-0†	10 - 8	16 - 6	20-0†
	SPF	No.1	12 - 6	19 - 4	20-0†	11 - 11	18 - 6	20-0†	11 - 5	17 - 9	20-0†	11 - 0	17 - 0	20-0†	10 - 5	16 - 1	20-0†
	SPF	No.2	12 - 6	19 - 4	20-0†	11 - 11	18 - 6	20-0†	11 - 5	17 - 9	20-0†	11 - 0	17 - 0	20-0†	10 - 5	16 - 1	20-0†
	SPF	No.3/Stud	11 - 10	17 - 5	20-0†	11 - 4	16 - 3	20-0†	10 - 9	15 - 3	18 - 10	10 - 1	14 - 4	17 - 9	9 - 3	13 - 2	16 - 3
	SPF	Standard	10 - 5	-	-	9 - 8	-	-	9 - 1	-	-	8 - 7	-	-	7 - 10	-	-
16	DFL	SS	12 - 7	19 - 6	20-0†	12 - 0	18 - 7	20-0†	11 - 6	17 - 10	20-0†	11 - 1	17 - 2	20-0†	10 - 6	16 - 2	20-0†
	DFL	No.1	12 - 1	18 - 9	20-0†	11 - 7	17 - 11	20-0†	11 - 1	17 - 2	20-0†	10 - 8	16 - 6	20-0†	10 - 1	15 - 7	20-0†
	DFL	No.2	11 - 10	18 - 4	20-0†	11 - 4	17 - 6	20-0†	10 - 10	16 - 10	20-0†	10 - 5	16 - 2	20-0†	9 - 10	15 - 3	19 - 1
	DFL	No.3/Stud	10 - 9	15 - 4	19 - 0	10 - 1	14 - 4	17 - 8	9 - 5	13 - 5	16 - 7	8 - 11	12 - 7	15 - 7	8 - 2	11 - 7	14 - 4
	DFL	Standard	9 - 1	-	-	8 - 6	-	-	8 - 0	-	-	-	-	-	-	-	-
	HF	SS	11 - 10	18 - 4	20-0†	11 - 4	17 - 6	20-0†	10 - 10	16 - 10	20-0†	10 - 5	16 - 2	20-0†	9 - 10	15 - 3	19 - 10
	HF	No.1	11 - 7	17 - 11	20-0†	11 - 1	17 - 2	20-0†	10 - 7	16 - 5	20-0†	10 - 2	15 - 9	20-0†	9 - 8	14 - 11	19 - 5
	HF	No.2	11 - 0	17 - 1	20-0†	10 - 6	16 - 4	20-0†	10 - 1	15 - 7	20-0†	9 - 8	15 - 0	19 - 6	9 - 2	14 - 2	18 - 5
	HF	No.3/Stud	10 - 6	14 - 11	18 - 6	9 - 10	13 - 11	17 - 3	9 - 2	13 - 1	16 - 2	8 - 8	12 - 3	15 - 2	7 - 11	11 - 3	13 - 11
	HF	Standard	8 - 11	-	-	8 - 4	-	-	7 - 9	-	-	-	-	-	-	-	-
	SP	SS	12 - 4	19 - 1	20-0†	11 - 10	18 - 3	20-0†	11 - 4	17 - 6	20-0†	10 - 10	16 - 10	20-0†	10 - 3	15 - 11	20-0†
	SP	No.1	11 - 10	18 - 4	20-0†	11 - 4	17 - 6	20-0†	10 - 10	16 - 10	20-0†	10 - 5	16 - 2	20-0†	9 - 10	15 - 3	19 - 10
	SP	No.2	11 - 4	17 - 6	20-0†	10 - 10	16 - 9	20-0†	10 - 4	16 - 0	20-0†	10 - 0	15 - 5	19 - 2	9 - 5	14 - 2	17 - 7
	SP	No.3	9 - 9	14 - 0	17 - 2	9 - 1	13 - 1	16 - 0	8 - 6	12 - 3	15 - 0	8 - 0	11 - 6	14 - 2	-	10 - 7	13 - 0
	SP	Stud	9 - 9	14 - 0	17 - 2	9 - 1	13 - 1	16 - 0	8 - 6	12 - 3	15 - 0	8 - 0	11 - 6	14 - 2	-	10 - 7	13 - 0
	SP	Standard	8 - 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	SPF	SS	11 - 7	17 - 11	20-0†	11 - 1	17 - 2	20-0†	10 - 7	16 - 5	20-0†	10 - 2	15 - 9	20-0†	9 - 8	14 - 11	19 - 5
	SPF	No.1	11 - 4	17 - 6	20-0†	10 - 10	16 - 9	20-0†	10 - 4	16 - 0	20-0†	10 - 0	15 - 5	20-0†	9 - 5	14 - 7	18 - 10
	SPF	No.2	11 - 4	17 - 6	20-0†	10 - 10	16 - 9	20-0†	10 - 4	16 - 0	20-0†	10 - 0	15 - 5	20-0†	9 - 5	14 - 7	18 - 10
	SPF	No.3/Stud	10 - 6	14 - 11	18 - 6	9 - 10	13 - 11	17 - 3	9 - 2	13 - 1	16 - 2	8 - 8	12 - 3	15 - 2	7 - 11	11 - 3	13 - 11
	SPF	Standard	8 - 11	-	-	8 - 4	-	-	7 - 9	-	-	-	-	-	-	-	-
24	DFL	SS	10 - 11	16 - 11	20-0†	10 - 5	16 - 2	20-0†	10 - 0	15 - 6	20-0†	9 - 7	14 - 10	19 - 4	9 - 1	14 - 1	18 - 3
	DFL	No.1	10 - 6	16 - 3	20-0†	10 - 0	15 - 6	20-0†	9 - 8	14 - 11	18 - 10	9 - 3	14 - 4	17 - 9	8 - 9	13 - 2	16 - 3
	DFL	No.2	10 - 3	15 - 11	20-0†	9 - 10	15 - 2	19 - 0	9 - 5	14 - 5	17 - 10	9 - 1	13 - 6	16 - 9	8 - 7	12 - 5	15 - 4
	DFL	No.3/Stud	8 - 8	12 - 4	15 - 3	8 - 1	11 - 6	14 - 3	-	10 - 9	13 - 4	-	10 - 2	12 - 6	-	9 - 4	11 - 6
	DFL	Standard	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	HF	SS	10 - 3	15 - 11	20-0†	9 - 10	15 - 2	19 - 9	9 - 5	14 - 7	19 - 0	9 - 1	14 - 0	18 - 3	8 - 7	13 - 3	17 - 3
	HF	No.1	10 - 1	15 - 7	20-0†	9 - 7	14 - 10	19 - 4	9 - 3	14 - 3	18 - 6	8 - 10	13 - 8	17 - 6	8 - 5	13 - 0	16 - 0
	HF	No.2	9 - 7	14 - 10	19 - 3	9 - 2	14 - 2	18 - 5	8 - 9	13 - 7	17 - 3	8 - 5	13 - 0	16 - 3	8 - 0	12 - 1	14 - 11
	HF	No.3/Stud	8 - 6	12 - 0	14 - 10	7 - 11	11 - 3	13 - 10	-	10 - 6	13 - 0	-	9 - 11	12 - 3	-	9 - 1	11 - 3
	HF	Standard	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	SP	SS	10 - 9	16 - 7	20-0†	10 - 3	15 - 10	20-0†	9 - 10	15 - 2	19 - 9	9 - 5	14 - 7	19 - 0	8 - 11	13 - 10	17 - 11
	SP	No.1	10 - 3	15 - 11	20-0†	9 - 10	15 - 2	19 - 9	9 - 5	14 - 7	19 - 0	9 - 1	14 - 0	18 - 1	8 - 7	13 - 3	16 - 7
	SP	No.2	9 - 10	15 - 2	18 - 9	9 - 5	14 - 1	17 - 6	9 - 0	13 - 3	16 - 5	8 - 7	12 - 5	15 - 5	7 - 10	11 - 5	14 - 2
	SP	No.3	7 - 10	11 - 3	13 - 10	-	10 - 6	12 - 11	-	9 - 10	12 - 1	-	9 - 3	11 - 4	-	8 - 6	10 - 5
	SP	Stud	7 - 10	11 - 3	13 - 10	-	10 - 6	12 - 11	-	9 - 10	12 - 1	-	9 - 3	11 - 4	-	8 - 6	10 - 5
	SP	Standard	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	SPF	SS	10 - 1	15 - 7	20-0†	9 - 7	14 - 10	19 - 4	9 - 3	14 - 3	18 - 6	8 - 10	13 - 8	17 - 10	8 - 5	12 - 11	16 - 10
	SPF	No.1	9 - 10	15 - 2	19 - 9	9 - 5	14 - 6	18 - 9	9 - 0	13 - 11	17 - 6	8 - 8	13 - 4	16 - 6	8 - 2	12 - 3	15 - 2
	SPF	No.2	9 - 10	15 - 2	19 - 9	9 - 5	14 - 6	18 - 9	9 - 0	13 - 11	17 - 6	8 - 8	13 - 4	16 - 6	8 - 2	12 - 3	15 - 2
	SPF	No.3/Stud	8 - 6	12 - 0	14 - 10	7 - 11	11 - 3	13 - 10	-	10 - 6	13 - 0	-	9 - 11	12 - 3	-	9 - 1	11 - 3
	SPF	Standard	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

See Table 3.20B footnotes.

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Table 3.20B Footnotes

† Allowable stud length exceeds 20 feet.
†† Maximum stud length for 2x4's is limited to 14 feet per Table 3.20C.
\- Lumber grade not available or allowable stud length is less than 7 ft - 9 in. (for 8 ft wall height).
a Maximum stud lengths in Table 3.20B are based on interior zone loads and assume that all studs are covered on the inside with a minimum of 1/2 inch gypsum wallboard, attached in accordance with minimum building code requirements and sheathed on the exterior side with a minimum of 3/8 inch wood structural panel sheathing with all panel joints occurring over studs or blocking and attached using a minimum 8d common nails spaced a maximum of 6" on center at panel edges and 12" on center at intermediate framing members. To address additional end zone loading requirements, end zone stud spacings shall be multiplied by 0.80. The additional bending capacity provided by the reduced stud spacing is assumed to be sufficient to resist the additional end zone loading requirements.
1 Exterior studs shall be limited to a height between horizontal supports per Table 3.20C. DFL = Douglas Fir-Larch, HF = Hem-Fir, SP = Southern Pine, SPF = Spruce-Pine-Fir.

Table 3.20C Size, Height, and Spacing Limits for Wood Studs[1, 2]

	2x3	2x4	2x5	2x6	2x8
Loadbearing Studs Supporting	**Maximum Unsupported Stud Length (ft)**				
	-	10	10	10	10
	Maximum Stud Spacing (in. o.c.)				
Roof & Ceiling Only	-	24	24	24	24
1 Floor Only	-	24	24	24	24
Roof, Ceiling, & 1 Floor Only	-	16	16	24	24
2 Floors Only	-	16	16	24	24
Roof, Ceiling, & 2 Floors	-	-	-	16	24
Non-loadbearing Studs	**Maximum Unsupported Stud Length (ft)**				
	10	14	16	20	20
	Maximum Stud Spacing (in. o.c.)				
	16	24	24	24	24

1 Maximum stud lengths in Tables 3.20A and B are based on wind loads. For dead and live loads, stud lengths shall be limited to the requirements in this table.
2 Habitable attics shall be considered an additional floor for purposes of determining gravity and seismic loads in accordance with Section 3.1.3.1.

WFCM Workbook

Wall Framing

Wall Studs (WFCM 3.4.1.1)

Loadbearing (2-1 and 2-2)

Choose Studs from Table 3.20A or 3.20B and Table 3.20C

Three second gust wind speed (700 yr) and Exposure category: .... 160 mph Exp. B
Exterior Studs (ext. wood siding and int. gypsum bd.) Deflection:............ H/180 in.
Wall Height: .... 9 ft
Studs supporting (Roof, Ceiling, Floors): .... R+C+1F
Sheathing Type (3/8" wood structural panel or minimum sheathing): .... WSP

To show that this is an iterative approach and that other factors may drive selection of stud size, the first attempt will use 2x4 stud grade material. Start with Table 3.20B1 because shear walls will require WSP sheathing.

Table W5.1 Selection of Species, Grade, Size, and Spacing for Loadbearing Studs (Developed from WFCM Tables 3.20B1 and 3.20C)

Specie	**Douglas Fir-Larch**	**Hem-Fir**	**Southern Pine**	**Spruce-Pine-Fir**
Spacing	16"	16"	16"	16"
Grade	No. 3/Stud	No. 3/Stud	No. 3 or Stud	No. 3/Stud
Size	2x4	2x4	2x4	2x4
Maximum Length (Wind) [2]	10-1"	**9'-10"**	**9'-1"**	**9'-10"**
Maximum Length (Dead and Live Loads) [1]	**10'-0"**	10'-0"	10'-0"	10'-0"

1. Studs support roof, ceiling, and attic floor, therefore from Table 3.20C spacing is 16" o.c. The remainder of wall which supports only roof and ceiling could increase spacing to 24" o.c., however due to standard construction practice, the spacing remains at 16" o.c.
2. Footnote “a” requires the stud spacing to be multiplied by 0.80 for framing within 4 ft of the corners to address additional end zone loading requirements. Options:
   a. Space studs at 12" o.c. within 4 ft of the corners.
   b. Design for minimum sheathing materials per Table 3.20A1 and apply Footnote “a”.
   c. Design for a higher grade or 2x6 studs at 24" o.c. and then space them at 16" o.c.

   Since wall W2-A will require 2x6 studs, choose Option (c). Calculations per Table W5.2.

Table W5.2 Selection of Species, Grade, Size, and Spacing for Loadbearing Studs (Developed from WFCM Tables 3.20B1 and 3.20C)

Specie	**Douglas Fir-Larch**	**Hem-Fir**	**Southern Pine**	**Spruce-Pine-Fir**
Spacing	24" *	24" *	24" *	24" *
Grade	No. 3/Stud	No. 3/Stud	No. 3 or Stud	No. 3/Stud
Size	2x6	2x6	2x6	2x6
Maximum Length (Wind) [1]	11'-6" OK	11'-3" OK	10'-6" OK	11'-3" OK
Maximum Length (Dead and Live Loads)	**10'-0"** OK	**10'-0"** OK	**10'-0"** OK	**10'-0"** OK

* Decrease all stud spacing to 16" o.c. to satisfy Table 3.20B Footnote “a” criteria.

Wall Framing

Wall Studs (WFCM 3.4.1.1)

Loadbearing (1-1 and 1-2)

Choose Studs from Table 3.20A or 3.20B and Footnotes

Three second gust wind speed (700 yr.) and Exposure category: .... 160 mph Exp. B
Exterior Studs (ext. wood siding and int. gypsum bd.) Deflection:............ H/180 in.
Wall Height:.. 9 ft
Studs supporting (Roof, Ceiling, Floors):.. R+C+2F
Sheathing Type (wood structural panel or minimum sheathing): .... WSP

Based on second floor wall designs, start with 2x6 studs @ 24" o.c.

Table W6.1 Selection of Species, Grade, Size, and Spacing for Loadbearing Studs (Developed from WFCM Tables 3.20B1 and 3.20C)

Specie	Douglas Fir-Larch	Hem-Fir	Southern Pine	Spruce-Pine-Fir
Spacing	24" *	24" *	24" *	24" *
Grade	No. 3/Stud	No. 3/Stud	No. 3 or Stud	No. 3/Stud
Size	2x6	2x6	2x6	2x6
Maximum Length (Wind) [1]	11'-6" OK	11'-3" OK	10'-6" OK	11'-3" OK
Maximum Length (Dead and Live Loads)	**10'-0"** OK	**10'-0"** OK	**10'-0"** OK	**10'-0"** OK

* Decrease all stud spacing to 16" o.c. to satisfy Table 3.20B Footnote “a” criteria.

Non-Loadbearing (1-A and 1-B)

Choose Studs from Table 3.20A or 3.20B and Table 3.20C

Three second gust wind speed (700 yr) and Exposure category: .... 160 mph Exp. B
Exterior Studs (ext. wood siding and int. gypsum bd.) Deflection:............ H/180 in.
Wall Height:.. 9 ft
Sheathing Type (wood structural panel or minimum sheathing): .... WSP

Plan for Footnote “a” stud spacing adjustment factor of 0.8 by starting with 24" stud spacing. Even though 2x4 studs might work, start with 2x6 studs based on all other walls being framed with 2x6.

Table W6.2 Selection of Species, Grade, Size, and Spacing for Non-loadbearing Studs (Developed from WFCM Tables 3.20B1 and 3.20C)

Specie	**Douglas Fir-Larch**	**Hem-Fir**	**Southern Pine**	**Spruce-Pine-Fir**
Spacing	24" *	24" *	24" *	24" *
Grade	No.3/Stud	No.3/Stud	No. 3 or Stud	No.3/Stud
Size	2x6	2x6	2x6	2x6
Maximum Length (Wind)	**11'-6" OK**	**11'-3" OK**	**10'-6" OK**	**11'-3" OK**
Maximum Length (Dead and Live Loads)	20'-0" OK	20'-0" OK	20'-0" OK	20'-0" OK

* Decrease all stud spacing to 16" o.c. per Table 3.20B Footnote “a”.

Poll Question

True or False:

Out of plane bending does not need to be checked in stud designs if wood structural panel sheathing is applied to the studs.

Example

Full Height (19 ft) Loadbearing Wall Wood Stud Design Example per 2015 WFCM Workbook Assumptions

Background: The 2 story home considered in the 2015 WFCM Workbook has a Foyer with a vaulted ceiling. The south bearing wall of the Foyer must support gravity loads from the roof and attic above, must resist reactions from uplift wind forces on the roof and must resist out-of-plane wind pressures. The home is located in an area with a basic wind speed of 160 mph - Exposure B.

The Foyer originally had a 4 foot wide plant shelf at the second floor level. The plant shelf provided lateral support for the wall framing and limited stud length to one story. The resulting configuration was within the limitations of the prescriptive provisions of the 2015 WFCM and the wall framing could be determined from Chapter 3 of the 2015 WFCM.

Removing the plant shelf requires the wall to be balloon framed and will increase the stud lengths where they are no longer within the limitations of the prescriptive provisions of the WFCM.

Goal: Determine requirements for studs that are balloon framed from the first floor to the roof.

Approach: Analyze wall framing as part of the Main Wind Force Resisting System (MWFRS) exposed to in-plane and out-of-plane load combinations specified by ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*. Analyze wall framing as Components and Cladding (C&C) exposed to out of plane C&C wind pressures only. Design wall framing per the 2015 National Design Specification® (NDS®) for Wood Construction.

In this example, the following loads are assumed:

<u>Roof Loads</u>		<u>Attic/Ceiling</u>	
Dead Load	10 psf	Dead Load	15 psf
Live Load	20 psf	Attic Live Load	30 psf
Ground Snow Load	30 psf		

Rain & Earthquake effects not considered in the analyses

Additionally, the following design assumptions apply:

Stud spacing = 16" o.c.
Exterior Sheathing = Wood Structural Panels
Interior Sheathing = 1/2" Gypsum Wallboard

The analysis involves an iterative approach. Initial values are selected for the member properties (depth, number of members and their specie and grade); initial analyses are completed and stresses and deflections determined and compared to allowable values. The member properties are then varied and analyses repeated until stress and deflection criteria are satisfied.

Example

Reference and Adjusted Design Values (values were revised during iterations - final values are for No. 2 SP)

$F_b := 925 \cdot psi$ $\quad E := 1400000 \cdot psi$ $\quad Emin := 510000 \cdot psi$ (Table 4B)

$F_c := 1350 \cdot psi$ $\quad C_M := 1.0$ $\quad C_t := 1.0$ $\quad C_F := 1.0$ (Table 4.3.1)

$C_{fu} := 1.0$ $\quad C_i := 1.0$ $\quad C_r := 1.25$

Cr used to represent Wall Stud Repetitive Member Factors (SDPWS 3.1.1.1)

$C_V := 1.0$ $\quad C_T := 1.0$ $\quad c := 0.8$

factor "c" in column stability factor C_P equation for sawn lumber. (3.7.1)

$E'min := Emin \cdot C_M \cdot C_t \cdot C_i \cdot C_T$ $\quad E'min = 510000\,psi$

Member Properties

$n := 1$ $\quad b := 1.5 \cdot in$ $\quad d := 7.25 \cdot in$

n is the number of full length studs

$A_g := n \cdot b \cdot d$ $\quad S_x := \frac{n \cdot b \cdot d^2}{6}$ $\quad S_y := \frac{n \cdot d \cdot b^2}{6}$ $\quad I_x := \frac{n \cdot b \cdot d^3}{12}$ $\quad I_y := \frac{n \cdot d \cdot b^3}{12}$

$A_g = 10.9 \cdot in^2$ $\quad S_x = 13.1 \cdot in^3$ $\quad S_y = 2.7 \cdot in^3$ $\quad I_x = 47.6 \cdot in^4$ $\quad I_y = 2 \cdot in^4$

For weak axis bending, composite action is not considered. The moment of intertia (I_y)equation contains the term $n(b)^3$ not $(nb)^3$.

Home Dimensions

$L := 19 \cdot ft$ length of balloon-framed studs $\quad W := 32 \cdot ft$ building width $\quad W_{ovhg} := 2 \cdot ft$ width of roof overhang

Determine Distributed Loads Supported by the South Wall

Dead and Live Loads

$w_{DLAttic} := 15 \cdot \frac{lbf}{ft^2} \cdot \frac{1}{2} \cdot 16 \cdot ft$ $\quad w_{DLRoof} := 10 \cdot \frac{lbf}{ft^2} \cdot \frac{1}{2} \cdot 32 \cdot ft$

$w_{DLAttic} = 120 \cdot plf$ $\quad w_{DLRoof} = 160 \cdot plf$

$w_{LLAttic} := 30 \cdot \frac{lbf}{ft^2} \cdot \frac{1}{2} \cdot 16 \cdot ft$ $\quad w_{LLRoof} := 20 \cdot \frac{lbf}{ft^2} \cdot \frac{1}{2} \cdot 32 \cdot ft$

$w_{LLAttic} = 240 \cdot plf$ $\quad w_{LLRoof} = 320 \cdot plf$

$w_{totalDead} := w_{DLAttic} + w_{DLRoof}$

$w_{totalDead} = 280 \cdot plf$

Rain Load

$R := 0 \cdot plf$

Earthquake Load

$E_l := 0 \cdot plf$

Rain and earthquake loads are included in ASCE 7-10 load combinations). The subscript for the earthquake load is used to differentiate the earthquake load from the modulus of elasticity

Example

Snow Load

$p_g := 30 \cdot \frac{lbf}{ft^2}$ $C_e := 1.0$ $C_{ts} := 1.0$ $I_s := 1.0$

subscript "s" added to C_t to distinguish the temperature factor for snow load calculations from C_t for stress calculations

$p_f := 0.7 \cdot C_e \cdot C_{ts} \cdot I_s \cdot p_g$

flat roof snow load

$p_f = 21 \cdot psf$

$C_s := 1.0$

Slope factor per ASCE 7-10 Figure 7-2

$p_{sBal} := C_s \cdot p_f$ $p_{sBal} = 21 \cdot psf$

Balanced snow load (applied to the entire roof)

$p_{sUnBal} := I_s \cdot p_g$ $p_{sUnBal} = 30 \cdot psf$

Unbalanced snow load (applied to leeward side of roof with no snow on windward side roof)

$p_{sBal} \cdot \frac{1}{2} \cdot 32 \cdot ft = 336 \cdot plf$

load on south wall from a balanced snow condition

$p_{sUnBal} \cdot \frac{3}{4} \cdot 16 \cdot ft = 360 \cdot plf$

load on south wall from an unbalanced snow condition

$$w_{snow} := \max\left(\begin{pmatrix} p_{sBal} \cdot \frac{1}{2} \cdot 32 \cdot ft \\ p_{sUnBal} \cdot \frac{3}{4} \cdot 16 \cdot ft \end{pmatrix}\right)$$

controlling snow condition

$w_{snow} = 360 \cdot plf$

Unbalanced snow load controls

Calculate MWFRS Wind Loads

MWFRS Wind Pressures are calculated using the Envelope Procedure contain in Chapter 28 of ASCE 7-10. The wind pressure equation 28.4-1 is:

$$p = q_h[(GC_{pf})\text{-}(GC_{pi})]$$

Where:

qh is the velocity pressure
GCpf is the external pressure coefficient for the surface being analyzed and
GCpi is the internal pressure coefficient

Determine Velocity Pressure q_h

$V := 160$

$K_z := 0.70$

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

$K_{zt} := 1.0$ ASCE 7-10 Section 26.8.2

$q_h := (0.60) \cdot 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot \frac{lbf}{ft^2}$

$q_h = 23.4 \cdot psf$

Note: The 160 Exp B velocity pressures q_h in the WFCM is 24.06 psf and is based on a 33 ft MRH where the velocity pressure coefficient K_z for Exp B is 0.72.

The velocity pressure coefficient for the 25 ft MRH in this example is 0.70 per ASCE 7-10 Table 28.3.1 and produces a slightly lower velocity pressure of 23.4 psf.

The 0.60 factor in the velocity pressure equation incorporates ASCE 7-10 load factors for allowable stress design (ASD) load combinations

Example

Determine MWFRS Roof Pressure Coefficients (GC_{pf})

ASCE 7-10 Figure 28.4-1 shows the external pressure coefficient for interior and end zones for two load cases. Load Case A is for wind perpendicular to the ridge; Load Case B is for wind parallel to the ridge.

	Zone 2 (windward)	Zone 3 (leeward)	Roof Overhang	Internal
Load Case A	$GC_{pfAWW} := 0.21$	$GC_{pfALW} := -0.43$	$C_{pOH} := -0.70$	$GC_{pi} := 0.18$
Load Case B	$GC_{pfBWW} := -0.69$	$GC_{pfBLW} := -0.37$	(ASCE Section 28.4.3)	(ASCE Section 26.11-1)

$G := 0.85$ Gust factor (ASCE 7 Section 26.9.1)

Determine MWFRS Wind Pressures on Roofs for Load Cases A and B

Load Case A

windward roof overhang

$-1 \cdot q_h \cdot \left(\left(-GC_{pfAWW} - G \cdot C_{pOH}\right)\right) = -9 \cdot psf$

windward roof

$-1 \cdot q_h \cdot \left[\left(\left(-GC_{pfAWW} + GC_{pi}\right)\right)\right] = 0.7 \cdot psf$

leeward roof

$-1 \cdot q_h \cdot \left(-GC_{pfALW} + GC_{pi}\right) = -14.3 \cdot psf$

Load Case B

windward roof overhang

$-1 \cdot q_h \cdot \left(\left(-GC_{pfBWW} - G \cdot C_{pOH}\right)\right) = -30.1 \cdot psf$

windward roof

$-1 \cdot q_h \cdot \left[\left(\left(-GC_{pfBWW} + GC_{pi}\right)\right)\right] = -20.4 \cdot psf$

leeward roof

$-1 \cdot q_h \cdot \left(-GC_{pfBLW} + GC_{pi}\right) = -12.9 \cdot psf$

Determine Wind Load Reactions

Reactions at the top of the bearing wall are determined by summing overturning moments about the top of leeward wall for both load cases and determining the controlling reaction to use in the design. Horizontal projections are used in the analysis.

Note: The component of the overturning moment that results from wind pressures on the leeward roof overhang was not considered because: (1) it has a short (1 ft) moment arm and (2) the uplift pressures on the overhang occur downwind of the leeward wall and reduce the net overturning moment reaction slightly. This approach provides slightly conservative results.

Load Case A

$$R_{windwardA} := \frac{-1}{W} \cdot q_h \cdot \left[W_{ovhg} \cdot \left(W + \frac{W_{ovhg}}{2}\right) \cdot \left(-GC_{pfAWW} - G \cdot C_{pOH}\right) \cdots \atop + \left[\frac{W}{2} \cdot \frac{3 \cdot W}{4} \cdot \left(-GC_{pfAWW} + GC_{pi}\right) + \frac{W}{2} \cdot \frac{1 \cdot W}{4} \cdot \left(-GC_{pfALW} + GC_{pi}\right)\right] \right]$$

$R_{windwardA} = -67 \cdot plf$

Load Case B

$$R_{windwardB} := \frac{-1}{W} \cdot q_h \cdot \left[W_{ovhg} \cdot \left(W + \frac{W_{ovhg}}{2}\right) \cdot \left(-GC_{pfBWW} - G \cdot C_{pOH}\right) \cdots \atop + \left[\frac{W}{2} \cdot \frac{3 \cdot W}{4} \cdot \left(-GC_{pfBWW} + GC_{pi}\right) + \frac{W}{2} \cdot \frac{1 \cdot W}{4} \cdot \left(-GC_{pfBLW} + GC_{pi}\right)\right] \right]$$

$R_{windwardB} = -358 \cdot plf$

Example

Determine Controlling Reaction

$$R_{windward} := \min\left(\begin{pmatrix} R_{windwardA} \\ R_{windwardB} \end{pmatrix}\right)$$

$R_{windward} = -358 \cdot plf$ Load Case B controls

Note: The uplift reaction on the windward wall can be determined from WFCM Table 2.2A by interpolating the uplift connection loads between the 24 and 36 foot roof spans for the 0 psf roof/ceiling dead load and multiplying the uplift by 0.75 to account for the wall framing not being located in an exterior zone (footnote 1). That approach produces an uplift reaction of 391 plf which is approximately 10% higher than the results of these calculations. The higher reactions result primarily because uplift values in Table 2.2A are based on the worst case (20°) roof slope. The velocity pressure being calculated at 33 ft instead of 25 ft also contributes to slightly higher values.

Determine Out-Of-Plane MWFRS Wind Pressures on Wall

External and internal pressure coefficients (GC_{pf} and GC_{pi}) are from ASCE 7-10 Figure 28.4-1 and Table 26.11-1 (resp.). By observation, Load Case A of Figure 28.4-1(wind perpendicular to ridge) produces the highest external wall pressure coefficient GC_{pf} for an interior wall zone. The highest pressure coefficient is negative internal pressures (from a leeward wall) produce the highest out-of-plane MWFRS wind pressure.

$GC_{pfwall} := 0.56 \quad -GC_{pi} = -0.18$

$$p_{mwfrs} := q_h \cdot \left[GC_{pfwall} - \left(-GC_{pi}\right)\right]$$

$p_{mwfrs} = 17.3 \cdot psf$ Out-of-plane MWFRS wind pressure on the wall

Determine the Distributed Loads Supported by the Bearing Wall for ASCE 7-10 ASD Load Combinations

ASCE 7-10 (section 2.4.1) includes the following ASD load combinations.
Respective NDS Load duration factors are shown in [brackets] next to load combination

1. D [C_D = 0.9]
2. D + L [C_D = 1.0]
3. D + (L_r or S or R)
3a. D + (L_r) [C_D = 1.25]
3b. D + (S) [C_D = 1.15]
4. D + 0.75L + 0.75(L_r or S or R)
4a. D + 0.75L + 0.75(L_r) [C_D = 1.25]
4b. D + 0.75L + 0.75(S) [C_D = 1.15]
5. D + (0.6W or 0.7E) [C_D = 1.6]
6. D + 0.75L + 0.75(0.6W or 0.7E) + 0.75(L_r or S or R)
6a1. D + 0.75L + 0.75(0.6W) + 0.75(L_r) [C_D = 1.6]
6a2. D + 0.75L + 0.75(0.6W) + 0.75(S) [C_D = 1.6]
6b. D + 0.75L + 0.75(0.7E) + 0.75S [C_D = 1.6]
7. 0.6D + 0.6W [C_D = 1.6]
8. 0.6D + 0.7E [C_D = 1.6]

where
D = dead load
L = live load
L_r = roof live load
W = wind load (note the 0.6 load factor has already been included in the velocity pressure q_h)
S = snow load
R = rain load
E = earthquake load

Example

Load combination are included in the following array (note that rain and earthquake load are neglected (E_1= R = 0)

$$P_{dist} := \begin{bmatrix} w_{totalDead} \\ w_{totalDead} + \left(w_{LLAttic}\right) \\ w_{totalDead} + w_{LLRoof} \\ w_{totalDead} + w_{snow} \\ w_{totalDead} + 0.75\cdot\left(w_{LLAttic}\right) + 0.75\cdot w_{LLRoof} \\ w_{totalDead} + 0.75\cdot\left(w_{LLAttic}\right) + 0.75\cdot w_{snow} \\ w_{totalDead} + R_{windward} \\ w_{totalDead} + 0.75\cdot w_{LLAttic} + 0.75\cdot\left(R_{windward}\right) + 0.75\cdot \max\left(\begin{pmatrix} w_{LLRoof} \\ w_{snow} \end{pmatrix}\right) \\ w_{totalDead} + 0.75\cdot w_{LLAttic} + 0.75\cdot \max\left(\begin{pmatrix} w_{LLRoof} \\ w_{snow} \end{pmatrix}\right) \\ 0.6\cdot w_{totalDead} + R_{windward} \end{bmatrix}$$

$P_{dist} =$

	1
1	280
2	520
3	600
4	640
5	700
6	730
7	-78
8	462
9	730
10	-190

$\cdot plf$

Load Combinations 1, 2, 3 and 4 model gravity only loads (dead load, live load and/or snow load). Load Combinations 5, 6a and 7 include MWFRS wind loads. Load Combinations are keyed to the array as follows:

Load Combo	Pdist
1	1
2	2
3a	3
3b	4
4a	5
4b	6
5	7
6a	8
6b	9
7	10

Since all combinations that include wind will use a 1.6 load duration factor, load combination 6a will be used as the controlling load for combinations 5-7. Since load combinations 1-4 each have different load duration factors, those combinations will be analyzed.

Example

Analyze Framing for Load Combinations 1-4 and 6a
(compute actual and allowable stresses and deflections. Iterate material properties to develop design)

The bearing walls must resist distributed loads from the attic floor and roof and out-of-plane MWFRS wind loads proportional to the width of their tributary areas. The analyses are conducted for 16 inch stud spacing. The number of studs on either side of the framed openings in the south wall shall be determined from the 2015 WFCM Table 3.23C. Reductions allowed by 2015 WCM Section 3.4.1.4.2 and Table 3.23D are acceptable.

Load Combination 1: D

Determing compressive force in framing for load combination 1

$P_1 := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_1}\right)$ $P_1 = 373 \cdot lbf$

Compressive force in the framing on each side of the wall openings for Load Combination 1.

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 1

$C_{D1} := 0.9$

dead load duration factor C_D for Load Combination 1. NDS Appendix B Section B.2

$F_{c1*} := F_c \cdot C_{D1} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c1*} = 1215\,psi$

F_{c1}* is reference compressive design value for Load Combination 1 adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$

Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

$d_1 := d \quad l_1 := L \qquad l_e := K_e \cdot l_1 \cdot \left(12 \cdot \frac{in}{ft}\right)$

$d_2 := b \quad l_2 := \frac{7}{12} \cdot ft$

stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$l_e = 228 \cdot in$

Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support (NDS A.11.3)

controlling effective length (strong axis)

$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2}$ $F_{cE} = 424\,psi$

critical bucking design value for compression member

Example

Determine Column Stability Factor C_p for Load Combination 1

$$C_{P1} := \frac{1+\left(\frac{F_{cE}}{F_{c1*}}\right)}{2\cdot c} - \sqrt{\left[\frac{1+\left(\frac{F_{cE}}{F_{c1*}}\right)}{2\cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c1*}}\right)}{c}}$$

$C_{P1} = 0.319$ Column Stability Factor for Load Combination 1 (NDS 3.7-1)

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$Fc'_1 := F_{c1*} \cdot C_{P1}$ $Fc'_1 = 388 \cdot psi$ Adjusted compressive design value

$f_{c1} := \frac{P_1}{A_g}$ $f_{c1} = 34 \cdot psi$ Actual compressive stress

$\frac{f_{c1}}{Fc'_1} = 0.09$ Ok. Actual compressive stress f_{c1} is less than the adjusted compressive design value F'_{c1}. Ratio of actual stress to adjusted compressive design value < 1.

Load Combination 2: D + L

Determing compressive force in framing for load combination 2

$P_2 := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_2}\right)$ $P_2 = 693 \cdot lbf$ Compressive force in the framing on each side of the wall openings for Load Combination 2.

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 2

$C_{D2} := 1.0$ floor live load duration factor C_D for Load Combination 2. NDS Appendix B Section B.2

$F_{c2*} := F_c \cdot C_{D2} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c2*} = 1350\,psi$ F_{c2}* is reference compressive design value for Load Combination 2 adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$ Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

$d_1 := d$ $l_1 := L$ $l_e := K_e \cdot l_1 \cdot \left(12 \cdot \frac{in}{ft}\right)$ stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$d_2 := b$ $l_2 := \frac{7}{12} \cdot ft$ Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support (NDS A.11.3)

$l_e = 228 \cdot in$ controlling effective length (strong axis)

Example

$$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2} \qquad F_{cE} = 424\,psi$$

critical bucking design value for compression member

Determine Column Stability Factor C_p for Load Combination 2

$$C_{P2} := \frac{1 + \left(\frac{F_{cE}}{F_{c2*}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c2*}}\right)}{2 \cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c2*}}\right)}{c}}$$

$C_{P2} = 0.29$ — Column Stability Factor for Load Combination 2 (NDS 3.7-1)

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$Fc'_2 := F_{c2*} \cdot C_{P2}$ — $Fc'_2 = 392 \cdot psi$ — Adjusted compressive design value

$f_{c2} := \frac{P_2}{A_g}$ — $f_{c2} = 64 \cdot psi$ — Actual compressive stress

$\frac{f_{c2}}{Fc'_2} = 0.16$

Ok. Actual compressive stress f_{c2} is less than the adjusted compressive design value F'_{c2}. Ratio of actual stress to adjusted compressive design value < 1.

Load Combination 3a: D + L_r

Determing compressive force in framing for load combination 3a

$P_{3a} := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_3}\right)$ — $P_{3a} = 800 \cdot lbf$ — Compressive force in the framing on each side of the wall openings for Load Combination 3.

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 3a

$C_{D3a} := 1.25$ — roof live load duration factor C_D for Load Combination 3a. NDS Appendix B Section B.2

$F_{c3a*} := F_c \cdot C_{D3a} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c3a*} = 1688\,psi$

F_{c3a}* is reference compressive design value for Load Combination 3a adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$ — Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

Example

$d_1 := d \quad l_1 := L \qquad l_e := K_e \cdot l_1 \cdot \left(12 \cdot \frac{in}{ft}\right)$

stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$d_2 := b \quad l_2 := \frac{7}{12} \cdot ft$

$l_e = 228 \cdot in$

Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support (NDS A.11.3)

controlling effective length (strong axis)

$$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2} \qquad F_{cE} = 424\,psi$$

critical bucking design value for compression member

Determine Column Stability Factor C_p for Load Combination 3a

$$C_{P3a} := \frac{1 + \left(\frac{F_{cE}}{F_{c3a*}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c3a*}}\right)}{2 \cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c3a*}}\right)}{c}}$$

$C_{P3a} = 0.237$

Column Stability Factor for Load Combination 3a (NDS 3.7-1)

$Fc'_{3a} := F_{c3a*} \cdot C_{P3a} \qquad Fc'_{3a} = 399 \cdot psi$

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$f_{c3a} := \frac{P_{3a}}{A_g} \qquad f_{c3a} = 74 \cdot psi$

Actual compressive stress

$\frac{f_{c3a}}{Fc'_{3a}} = 0.18$

Ok. Actual compressive stress f_{c3a} is less than the adjusted compressive design value F'_{c3a}. Ratio of actual stress to adjusted compressive design value < 1.

Example

Load Combination 3b: D + S

Determing compressive force in framing for load combination 3b

$P_{3b} := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_4}\right)$ $\quad P_{3b} = 853 \cdot lbf$

Compressive force in the framing on each side of the wall openings for Load Combination 3b.

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 3b

$C_{D3b} := 1.15$

snow load duration factor C_D for Load Combination 3b.
NDS Appendix B Section B.2

$F_{c3b*} := F_c \cdot C_{D3b} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c3b*} = 1552\,psi$

F_{c3b}* is reference compressive design value for Load Combination 3b adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$

Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

$d_1 := d \quad l_1 := L \qquad l_e := K_e \cdot l_1 \cdot \left(12 \cdot \frac{in}{ft}\right)$

stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$d_2 := b \quad l_2 := \frac{7}{12} \cdot ft$

$l_e = 228 \cdot in$

Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support (NDS A.11.3)

controlling effective length (strong axis)

$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2} \qquad F_{cE} = 424\,psi$

critical bucking design value for compression member

Example

Determine Column Stability Factor C_p for Load Combination 3b

$$C_{P3b} := \frac{1 + \left(\frac{F_{cE}}{F_{c3b*}}\right)}{2\cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c3b*}}\right)}{2\cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c3b*}}\right)}{c}}$$

$C_{P3b} = 0.255$ Column Stability Factor for Load Combination 3b (NDS 3.7-1)

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$Fc'_{3b} := F_{c3b*}\cdot C_{P3b}$ $\quad Fc'_{3b} = 397\cdot psi$ Adjusted compressive design value

$f_{c3b} := \frac{P_{3b}}{A_g}$ $\quad f_{c3b} = 78\cdot psi$ Actual compressive stress

$\frac{f_{c3b}}{Fc'_{3b}} = 0.2$

Ok. Actual compressive stress f_{c3b} is less than the adjusted compressive design value F'_{c3b}. Ratio of actual stress to adjusted compressive design value < 1.

Load Combination 4a: D + 0.75 L + 0.75 L_r

Determing compressive force in framing for load combination 4a

$P_{4a} := \frac{(16)}{12}\cdot ft\cdot\left(P_{dist_5}\right)$ $\quad P_{4a} = 933\cdot lbf$

Compressive force in the framing on each side of the wall
Adjusted compressive design value

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 4a

$C_{D4a} := 1.25$ roof live load duration factor C_D for Load Combination 4a.
NDS Appendix B Section B.2

$F_{c4a*} := F_c\cdot C_{D4a}\cdot C_M\cdot C_t\cdot C_F\cdot C_i$

$F_{c4a*} = 1688\,psi$

F_{c4a}* is reference compressive design value for Load Combination 4a adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$ Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

$d_1 := d \quad l_1 := L \quad l_e := K_e\cdot l_1\cdot\left(12\cdot\frac{in}{ft}\right)$

$d_2 := b \quad l_2 := \frac{7}{12}\cdot ft$

stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$l_e = 228\cdot in$

Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support (NDS A.11.3)

controlling effective length (strong axis)

Example

$$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2} \qquad F_{cE} = 424\ \text{psi}$$

critical bucking design value for compression member

Determine Column Stability Factor C_p for Load Combination 4a

$$C_{P4a} := \frac{1 + \left(\frac{F_{cE}}{F_{c4a*}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c4a*}}\right)}{2 \cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c4a*}}\right)}{c}}$$

$C_{P4a} = 0.237$ Column Stability Factor for Load Combination 4a (NDS 3.7-1)

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$Fc'_{4a} := F_{c4a*} \cdot C_{P4a}$ $\qquad Fc'_{4a} = 399 \cdot \text{psi}$ Adjusted compressive design value

$f_{c4a} := \frac{P_{4a}}{A_g}$ $\qquad f_{c4a} = 86 \cdot \text{psi}$ Actual compressive stress

$\frac{f_{c4a}}{Fc'_{4a}} = 0.22$

Ok. Actual compressive stress f_{c4a} is less than the adjusted compressive design value F'_{c4a}. Ratio of actual stress to adjusted compressive design value < 1.

Example

Load Combination 4b: D + 0.75 L + 0.75 S

Determing compressive force in framing for load combination 4b

$P_{4b} := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_6}\right)$ $P_{4b} = 973 \cdot lbf$

Compressive force in the framing on each side of the wall openings for Load Combination 4b.

Calculate Reference Compressive Design & Adjusted Compressive Design Values for Load Combination 4b

$C_{D4b} := 1.15$

snow load duration factor C_D for Load Combination 4b.
NDS Appendix B Section B.2

$F_{c4b*} := F_c \cdot C_{D4b} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c4b*} = 1552\,psi$

F_{c4b}* is reference compressive design value for Load Combination 4b adjusted with all adjustment factors except the column stability factor C_P

$K_e := 1.0$

Buckling length coefficient K_e for strong axis bending (pinned/pinned) column (Appendix G Table G1)

$d_1 := d$ $l_1 := L$ $l_e := K_e \cdot l_1 \cdot \left(12 \cdot \frac{in}{ft}\right)$

$d_2 := b$ $l_2 := \frac{7}{12} \cdot ft$

stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

$l_e = 228 \cdot in$

Effective lengths in each axis. Assume gypsum wallboard is connected to the studs at 7 inches o.c. and provides lateral support

controlling effective length (strong axis)

$$F_{cE} := \frac{0.822 \cdot E'min}{\left(\frac{l_e}{d_1}\right)^2}$$ $F_{cE} = 424\,psi$

critical bucking design value for compression member

Determine Column Stability Factor C_p for Load Combination 4b

$$C_{P4b} := \frac{1 + \left(\frac{F_{cE}}{F_{c4b*}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c4b*}}\right)}{2 \cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c4b*}}\right)}{c}}$$

$C_{P4b} = 0.255$

Column Stability Factor for Load Combination 4b (NDS 3.7-1)

Compare Actual Compressive Stress to Adjusted Compressive Design Value

$Fc'_{4b} := F_{c4b*} \cdot C_{P4b}$ $Fc'_{4b} = 397 \cdot psi$

Adjusted compressive design value

Example

$f_{c4b} := \frac{P_{4b}}{A_g}$ $\qquad f_{c4b} = 90 \cdot psi$

Actual compressive stress

$\frac{f_{c4b}}{Fc'_{4b}} = 0.23$

Ok. Actual compressive stress f_{c4b} is less than the adjusted compressive design value F'_{c4b}. Ratio of actual stress to adjusted compressive design value < 1.

Load Combination	f_c/F'_c
1	0.09
2	0.16
3a	0.18
3b	0.20
4a	0.22
4b	0.23

Table comparing ratio of applied stress to allowable stresses for gravity load controlled combinations

Load Case 6a: D + 0.75L + 0.75 W

Determing compressive load in framing for load combination 6a

$P_6 := \frac{(16)}{12} \cdot ft \cdot \left(P_{dist_8}\right)$ $\qquad P_6 = 616 \cdot lbf$

Determine Reference and Adjusted Compression Design Values for Load Combination 6a

$C_{D6} := 1.6$

Wind load duration factor C_D controls for Load Combination 6a - NDS Appendix B Section B.2

$F_{c6*} := F_c \cdot C_{D6} \cdot C_M \cdot C_t \cdot C_F \cdot C_i$

$F_{c6*} = 2160\,psi$

Fc* is reference compressive design value adjusted with all adjustment factors except the column stability factor $C_{P,}$. The following calculations determine the column stabilty factor C_P:

Determine Column Stability Factor C_p for Load Combination 6a

$$C_{P6} := \frac{1 + \left(\frac{F_{cE}}{F_{c6*}}\right)}{2 \cdot c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{c6*}}\right)}{2 \cdot c}\right]^2 - \frac{\left(\frac{F_{cE}}{F_{c6*}}\right)}{c}}$$

$C_{P6} = 0.188$

Column Stability Factor for Load Combination 6a (NDS 3.7-1)

Determine Adjusted Compressive Design Value for Load Combination 6a

$Fc'_6 := F_{c6*} \cdot C_{P6}$ $\qquad Fc'_6 = 405 \cdot psi$

Adjusted compressive design value for Load Combination 6a

Compare Actual Compressive Stress with Adjusted Compressive Design Value

Example

$$f_{c6} := \frac{P_6}{A_g}$$

$f_{c6} = 57 \cdot psi$

$Fc'_6 = 405 \cdot psi$

Ok - Actual compressive stress f_{c6} does not exceed adjusted compressive design value F'_{c6}. Ratio of actual compressive stress to adjusted compressive design value < 1.

$$\frac{f_{c6}}{Fc'_6} = 0.14$$

Example

Determine Bending Stress from Out-of-Plane MWFRS Wind Pressures

Moment

$w_{wind} := 0.75 \cdot \frac{(16)}{12} \cdot ft \cdot p_{mwfrs}$

Load Combination 6a includes 75% of the MWFRS Wind Load

$w_{wind} = 17.31 \cdot plf$

$M_{mwfrs} := \frac{w_{wind} \cdot L^2}{8} \cdot 12 \cdot \frac{in}{ft}$

Bending moment from out-of-plane MWFRS wind loads

$M_{mwfrs} = 9375 \cdot in \cdot lbf$

Determine Reference and Adjusted Bending Design Values for Load Combination 6a

$C_L := 1.0$

Depth to breadth (d/b) ratio 2<d/b<4 End restraints for the beam-column satisfy NDS 4.4.1.2 (b) and sheathing/gypsum wall board nailing provides lateral support for the compression edges NDS 4.4.1.2 (c)

$F'_{b6} := F_b \cdot C_{D6} \cdot C_M \cdot C_L \cdot C_t \cdot C_F \cdot C_i \cdot C_r$

F'_{b6} is adjusted bending design value for Load Combination 6a

$F'_{b6} = 1850\,psi$

Compare Actual Bending Stress with Adjusted Bending Design Values for Load Combination 6a

$f_{b6} := \frac{M_{mwfrs}}{S_x}$ $\quad f_{b6} = 713 \cdot psi$

$F'_{b6} = 1850\,psi$

Bending stress resulting from out-of-plane MWFRS wind loads

$\frac{f_{b6}}{F'_{b6}} = 0.39$

Ok. Actual bending stress f_{b6} is less than adjusted bending design value F'_{b6}. Ratio of actual bending stress to adjusted bending design value < 1

Check Combined Uniaxial Bending and Axial Compression

$$\left(\frac{f_{c6}}{Fc'_6}\right)^2 + \frac{f_{b6}}{F'_{b6} \cdot \left[1 - \left(\frac{f_{c6}}{F_{cE}}\right)\right]} = 0.46$$

< 1.0 ok (NDS 3.9-3)

$f_{c6} = 57\,psi$ $\quad F_{cE} = 424\,psi$

Ok - Actual compressive stress f_{c6} does not exceed adjusted compressive design value in plane of lateral support for edgewise bending F_{cE}.

Example

Check Adequacy of Framing to Resist Components and Cladding (C&C) loads

Calculate C&C Pressures on Wall

$C_{DCC} := 1.6$ — C_D for C&C loading

Determine External C&C Pressure Coefficient

$EWA := \frac{L^2}{3} \cdot \frac{1}{ft^2}$ $\quad EWA = 120$

Note: "EWA" is used for Effective Wind Area since "A" is a previously defined variable. Per ASCE 7-10 Chapter 26, EWA need not be less than $(L)^2/3$

$GC_p(EWA) := -0.8 - 0.3 \cdot \left(\frac{\log\left(\frac{EWA}{500}\right)}{\log\left(\frac{10}{500}\right)} \right)$

The south wall is in Zone 4. ASCE 7-10 Figure 30.4-1. The equation for GC_p for Zone 4 for 10 ft^2 < EWA < 500 ft^2

GC_p for 10 ft^2 < EWA < 500 ft^2 ASCE 7-10 Fig 30.4-1

$GC_p(EWA) = -0.909$ — external pressure coefficient for full height studs in Foyer wall

$p_{CC}(EWA) := q_h \cdot \left[GC_p(EWA) - (GC_{pi}) \right]$ — equation for C&C pressures for framing in the Foyer wall

$q_h = 23.4 \cdot psf$

$\left[GC_p(EWA) - (GC_{pi}) \right] = -1.089$

By observation negative external pressure coefficients (GC_p) are greater than positive external pressure coefficients. So negative external pressures and positive internal pressures (windward) create the greatest C&C pressures

$p_{CC}(EWA) = -25.48 \cdot psf$ — C & C pressures for full height framing in the south wall

Apply C&C Pressures to Wall Framing and Check Bending and Deflection

Bending

$w_{CC} := \frac{-(16)}{12} \cdot ft \cdot p_{CC}(EWA)$ $\quad w_{CC} = 34 \cdot plf$

Almost double the MWFRS pressure of 17.3 plf

$M_{CC} := \frac{(w_{CC}) \cdot L^2 \cdot 12 \cdot \frac{in}{ft}}{8}$

$M_{CC} = 18399 \cdot in \cdot lbf$

Determine Reference and Adjusted Bending Design Values for C&C loading

$F'_{bCC} := F_b \cdot C_{DCC} \cdot C_M \cdot C_L \cdot C_t \cdot (1.0) \cdot C_F \cdot C_i \cdot C_r$

$F'_{bCC} = 1850\, psi$

Compare Actual Bending Stress with Adjusted Bending Design Values for C&C loading

$f_{bCC} := \frac{M_{CC}}{S_x}$ $\quad f_{bCC} = 1400\, psi$

Ok. Actual bending stresses that result from C&C pressures f_{bCC} do not exceed adjusted bending design value F'_{bCC}

$\frac{f_{bCC}}{F'_{bCC}} = 0.76$ $\quad F'_{bCC} = 1850\, psi$

Combined MWFRS and gravity loading interaction ratio = 0.46

The fb/F'b ratio is greater for C&C loading than for MWFRS loading. C&C controls for strength calculations.

Example

Determine Deflection for C&C loading

$$\Delta_{CC} := \frac{5}{384 \cdot C_r \cdot E \cdot I_x} \cdot \frac{0.70 \cdot w_{CC}}{12} \cdot \frac{ft}{in} \cdot \left(L \cdot 12 \cdot \frac{in}{ft} \right)^4$$

IBC Table 1604 footnote (f) allows the wind load used in deflection calculations to be 0.42 times the C&C load. A factor of 0.70 is applied since a 0.60 factor has already been incorporated into the velocity pressure q_h.

$$\Delta_{CC} = 0.84 \cdot in$$

$$\frac{L \cdot 12 \cdot \frac{in}{ft}}{\Delta_{CC}} = 273$$

OK - Span to deflection ratio is greater than L / 180

Results - Framing the south wall of the Foyer using No.2 SP 2 X 8 studs on 16 inch centers is adequate to resist ASCE 7-10 loads.

Note: A 2x6 stud was analyzed and found to be sufficient in compression and bending stress capacity, however the deflections were in excess of the L / 180 deflection criteria allowed for some finishes.

Poll Question

Which load duration factor, C_D, applies to the following load combination:
$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

a. 0.9
b. 1.0
c. 1.15
d. 1.25
e. 1.6

This concludes the American Institute of Architects Continuing Education Systems Course

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