WoodWorks™
WOOD PRODUCTS COUNCIL

Introduction to Wood: Structural Design – Gravity & Lateral

Special Thanks to WEI

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Course Description

This presentation will provide an introductory review of allowable uses and best design practices associated with wood-frame construction. Wood science including wood's cell structure and moisture interaction properties will be discussed. Wood species and grades will be covered along with an overview of their structural properties and durability. Structural wood design for vertical (gravity) loads including bending, shear, deflection, vibration, tension, compression, and connections will be introduced. Common wood-framed lateral force-resisting systems will be discussed as will the components included in wood shear walls. Architectural considerations associated with wood framing will be examined, including fire protection and sprinklers, construction types, acoustics, and building envelopes. Design and detailing best practices for wood-frame buildings will be explained in an effort to highlight the items that play an important role in the construction process and ultimate building performance.

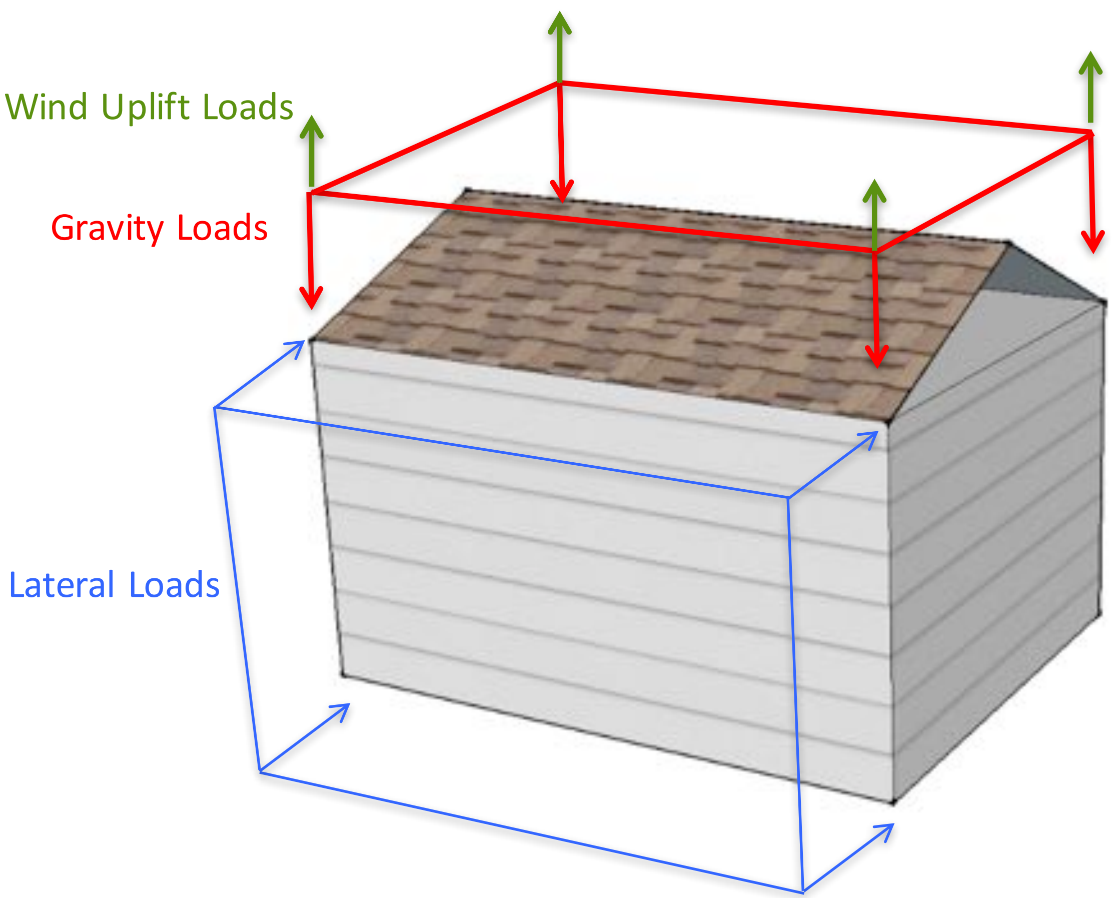
Learning Objectives

1. Review wood's role and allowable uses under current building codes.
2. Discuss design considerations specific to wood framing in non-residential and multi-family buildings.
3. In the context of wood as a biological material, identify best practices for its use in non-residential and multi-family buildings.
4. Explore the variety of available wood building products and discuss how to efficiently utilize each.

Structural Wood Design: Gravity Loads

For structural building design, two main loading directions exist: gravity (vertical) and lateral (horizontal)

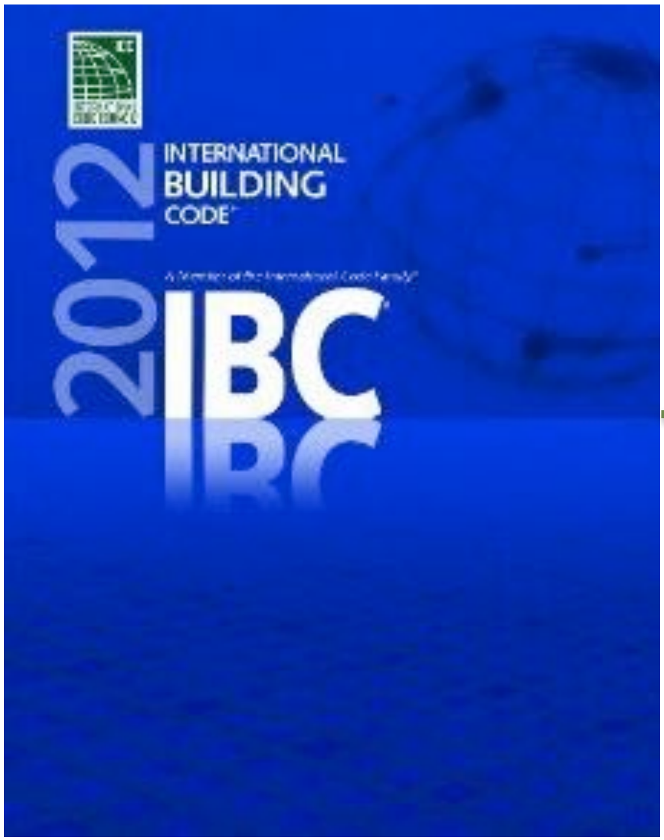
- This presentation will focus on structural wood design for gravity loads
- Gravity loads include dead, live, snow, and rain



Outline

- Design Basis & Notation
- Allowable Stresses
- Wood Member Design
- Connections: Design & Options

Gravity Load Demand

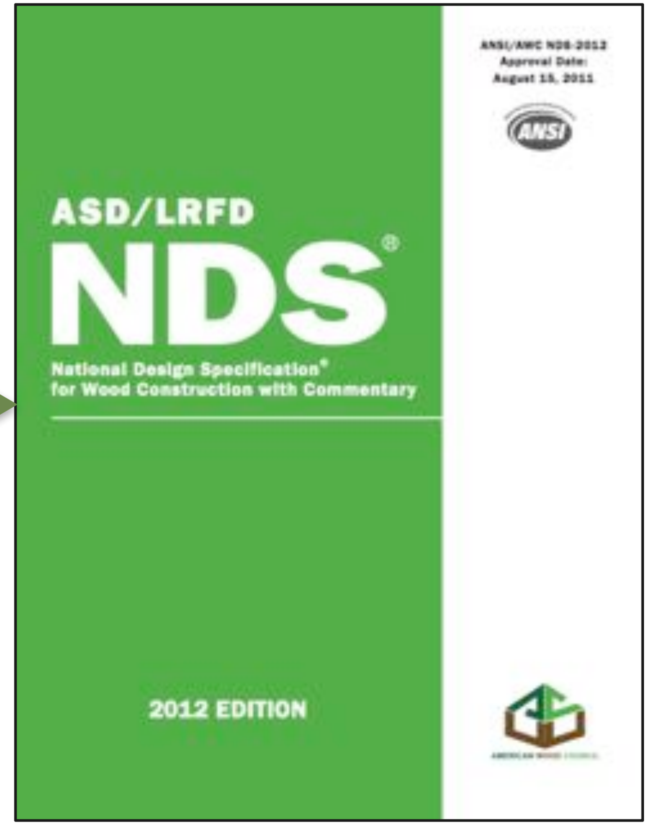


IBC: Base Code – References ASCE 7 for determination of gravity loads

ASCE 7: Referenced Standard. Provides information required to determine gravity loads on a structure

Structural Wood Design: Codes

IBC: References National Design Specification (NDS) for design of wood construction



National Design Specification (NDS): Provides design procedures and reference design values used in the structural design of wood framing members and connections

Structural Wood Design: ASD vs. LRFD

ASD

- Allowable Stress Design
- Traditional for wood design
- Based on allowable strengths and nominal (unfactored) loads

LRFD

- Load and Resistance Factor Design
- NDS 2005 – 1st time was included
- Based on nominal strengths and factored loads

Dual format has been in NDS since the 2005 edition

Structural Wood Design: Nomenclature

Wood Design Nomenclature	
Demand	The external load or stimuli applied to a structure
Capacity	The amount of resistance that a member, connection, or system is capable of resisting before a limit state is reached
Limit State	A defined point in a system or structural response (i.e. deflection limit state, bending limit state)
ASD	Allowable Stress Design. Utilizes unfactored service load estimates (demands) and compares those with scaled down capacities adjusted by safety factors (checks are typically made at the stress level).
LRFD	Load Resistance Factor Design. Uses scaled up (factored) demands (based on probabilities) to compare with scaled down more realistic capacities (based on probabilities and protection of non-ductile (catastrophic) failure mechanisms).

Structural Wood Design: ASD Notation

Demand – indicated by the LOWER CASE letter "f" representing a stress

f_t

Member stress

Stress type

Member stresses are determined from loads and members sizes

Capacity – indicated by the UPPER CASE letter "F" representing a reference allowable stress

F_t

Reference Design Value

Stress type

Reference design values are found in the NDS Supplement

Structural Wood Design: LRFD Notation

Load (Demand) - indicated an UPPER CASE letter representing a type of FORCE

$$T_u$$

Member FORCE → T

Indicates force due to factored loads → u

Member FORCES are determined from loads and members sizes

Resistance (Capacity) - indicated by an UPPER CASE letter representing a nominal reference design value (FORCE)

Reference design values are found in the NDS Supplement and adjusted for Nominal Design Values

$$T_n = (F'_t \times K_F) \times A = F'_{tn} \times A$$

Nominal Resistance → T

Indicates nominal → n

Adjusted reference design value → F'_t

Format conversion factor → K_F

Cross sectional area → A

Nominal adjusted reference value → F'_{tn}

Wood Design: Demand vs. Capacity

Wood member design (ASD or LRFD) compares demand to capacity. An adequate design is one where capacity is equal to or greater than demand.

ASD

$$f_t \leq F'_t$$

LRFD

$$T_u \leq \varnothing T_n$$

Most wood member design is a 3 step process:

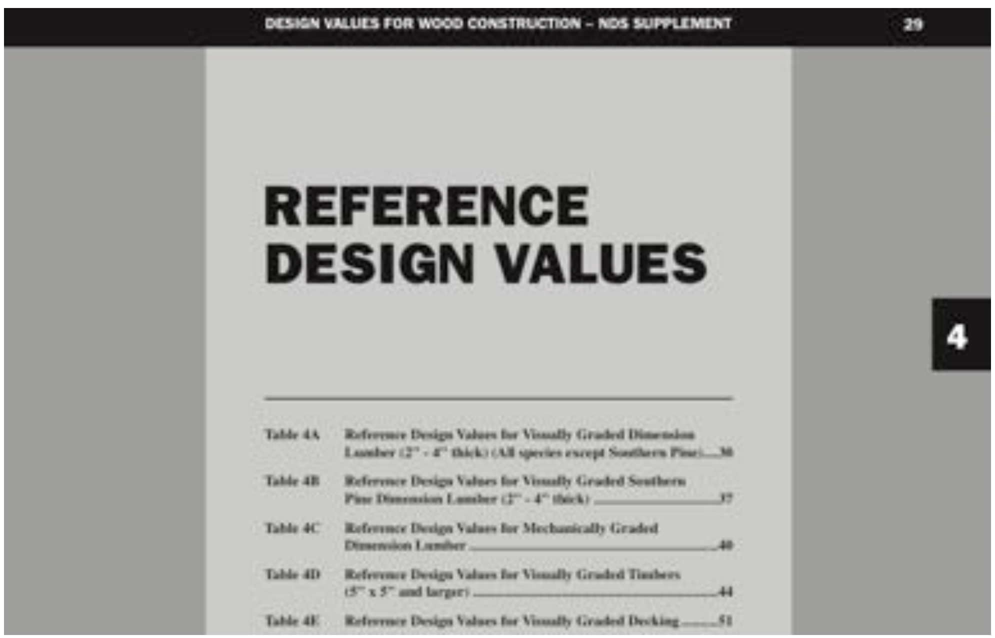
1. Determine loads & resulting forces
2. Determine member cross-sectional properties & resulting stresses
3. Compare actual to allowable stresses (or forces)

Note that 3 design step process can be re-arranged as needed:

- If allowable stresses and actual loads are known, can determine required member size
- If actual loads and member size are known, can determine required allowable stresses

Wood Design: Reference Design Values

Reference Design Values: The quantifiable mechanical properties that are associated with each identifiable commercial grade of wood


DESIGN VALUES FOR WOOD CONSTRUCTION – NDS SUPPLEMENT 29

REFERENCE DESIGN VALUES

4

Table 4A Reference Design Values for Visually Graded Dimension Lumber (2" - 4" thick) (All species except Southern Pine)....36
Table 4B Reference Design Values for Visually Graded Southern Pine Dimension Lumber (2" - 4" thick)....37
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Table 4D Reference Design Values for Visually Graded Timbers (5" x 5" and larger)....44
Table 4E Reference Design Values for Visually Graded Decking....51

Reference Design Values	
F_b	Bending stress
F_v	Shear stress
F_t	Tension stress
F_c	Compression stress
$F_{c\perp}$	Compression stress perpendicular to grain
E/E_{min}	Modulus of elasticity

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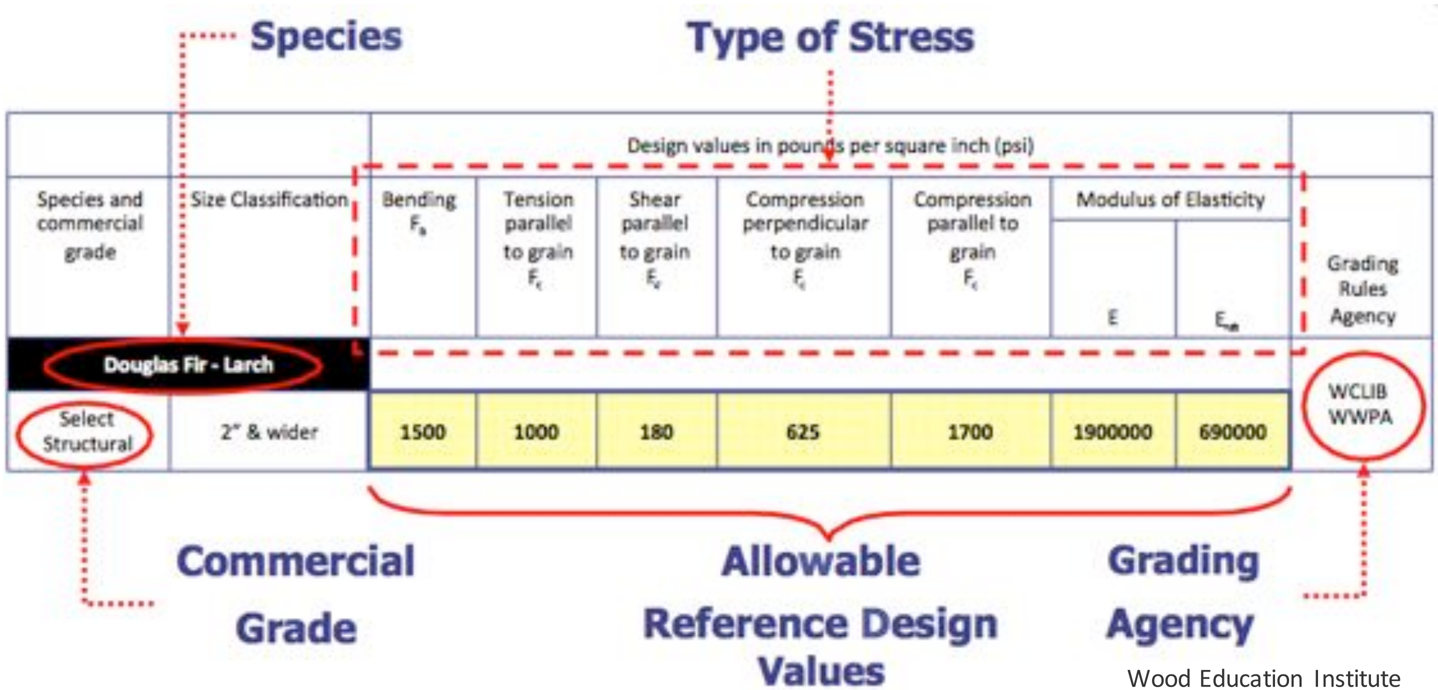
Wood Design: Reference Design Values

Reference Design Values in NDS are given based on four main variables:

- Grading Method
- Species Group
- Commercial Grade
- Size Classification



Wood Design: Adjustment Factors

Adjustment Factors: adjust **reference design values (ASD)** or **nominal design values (LRFD)** to **adjusted design values**

- Account for the unique properties and behavior of wood under a variety of conditions

ASD

$F'_t = F_t$ x (Adjustment Factors)

LRFD

$F'_{tn} = F_{tn}$ x (Adjustment Factors)

- Most adjustment factors apply to bc design methodologies
- Different adjustment factors are applied to different types of stress and in different combinations
- Adjustment factors > 1.0 may be neglected, those < 1.0 must be used

Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber

		ASD only	ASD and LRFD											LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor	Resistance Factor	Time Effect Factor
													K_F	ϕ	
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	2.54	0.85	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	2.70	0.80	λ
$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-	2.88	0.75	λ
$F_c' = F_c$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-	2.40	0.90	λ
$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b	1.67	0.90	-
$E' = E$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-	1.76	0.85	-

Wood Design: Adjustment Factors

Size Factors, C_F

Grades	Width (depth)	F_b Thickness (breadth) 2" & 3"	F_b Thickness (breadth) 4"	F_t	F_c
Select Structural, No.1 & Btr, No.1, No.2, No.3	2", 3", & 4"	1.5	1.5	1.5	1.15
	5"	1.4	1.4	1.4	1.1
	6"	1.3	1.3	1.3	1.1
	8"	1.2	1.3	1.2	1.05
	10"	1.1	1.2	1.1	1.0
	12"	1.0	1.1	1.0	1.0
	14" & wider	0.9	1.0	0.9	0.9
Stud	2", 3", & 4"	1.1	1.1	1.1	1.05
	5" & 6"	1.0	1.0	1.0	1.0
	8" & wider	Use No.3 Grade tabulated design values and size factors			
Construction, Standard	2", 3", & 4"	1.0	1.0	1.0	1.0
Utility	4"	1.0	1.0	1.0	1.0
	2" & 3"	0.4	—	0.4	0.6

Table N3 Time Effect Factor, λ (LRFD Only)

Load Combination[2]	λ
1.4D	0.6
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	0.7 when L is from storage 0.8 when L is from occupancy 1.25 when L is from impact[1]
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$	0.8
$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$	1.0
$1.2D + 1.0E + L + 0.2S$	1.0
$0.9D + 1.0W$	1.0
$0.9D + 1.0E$	1.0

1. Time effect factors, λ, greater than 1.0 shall not apply to connections or to structural members pressure-treated with water-borne preservatives (see Reference 30) or fire retardant chemicals.
2. Load combinations and load factors consistent with ASCE 7-10 are listed for ease of reference. Nominal loads shall be in accordance with N.1.2. D = dead load; L = live load; L_r = roof live load; S = snow load; R = rain load; W = wind load; and E = earthquake load.

Wood Design: Adjustment Factors

Table 2.3.2 Frequently Used Load Duration Factors, C_D[1]

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load
Impact[2]	2.0	Impact Load

1. Load duration factors shall not apply to reference modulus of elasticity, E, reference modulus of elasticity for beam and column stability, E_{min}, nor to reference compression perpendicular to grain design values, $F_{c\perp}$, based on a deformation limit.
2. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives (see Reference 30), or fire retardant chemicals. The impact load duration factor shall not apply to connections.

Table 2.3.5 Format Conversion Factor, K_F (LRFD Only)

Application	Property	K_F
Member	F_b	2.54
	F_t	2.70
	F_v, F_{rt}, F_s	2.88
	F_c	2.40
	$F_{c\perp}$	1.67
	E_{min}	1.76
All Connections	(all design values)	3.32

Table 2.3.3 Temperature Factor, C_t

Reference Design Values	In-Service Moisture Conditions[1]	C_t		
		$T \le 100°F$	$100°F < T \le 125°F$	$125°F < T \le 150°F$
F_t, E, E_{min}	Wet or Dry	1.0	0.9	0.9
F_b, F_v, F_c, and $F_{c\perp}$	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

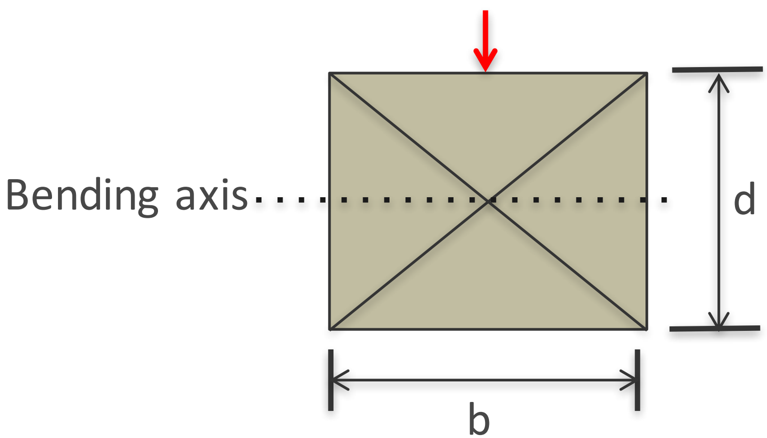
1. Wet and dry service conditions for sawn lumber, structural glued laminated timber, prefabricated wood I-joists, structural composite lumber, and wood structural panels are specified in 4.1.4, 5.1.4, 7.1.4, 8.1.4, and 9.3.3, respectively.

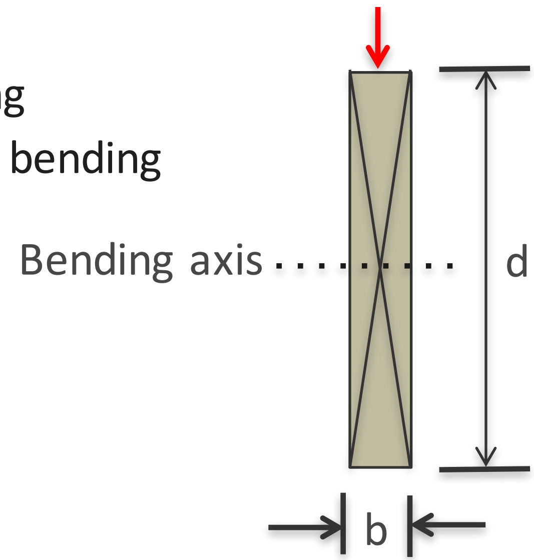
Wood Design: Section Properties

Section properties: geometric properties for a given cross-section which are used in determining the resulting stresses of applied forces

B = width of member, parallel to axis of bending

D = depth of member, perpendicular to axis of bending





Area
Shear, Compression, Tension

$$A = bd$$

Elastic Section Modulus
Bending

$$S = \frac{bd^2}{6}$$

Moment of Inertia
Bending, Deflection, Vibration

$$I = \frac{bd^3}{12}$$

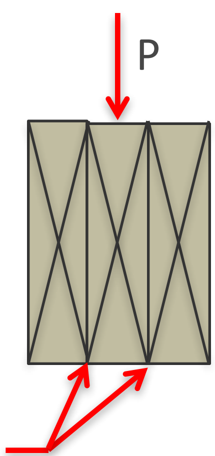
Wood Design: Multi-Ply Members

Multi-ply member design:

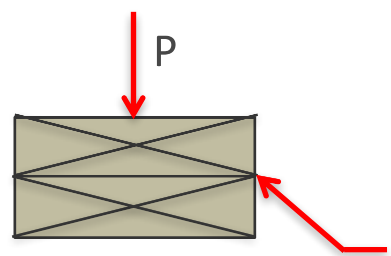
- When interface planes are parallel to direction of load, can assume all plies to act as one member if adequately connected together to distribute loads among all plies.
- When interface planes are perpendicular to direction of load, also need to consider “shear flow”

$$q = \frac{VQ}{I}$$

Shear Flow Equation



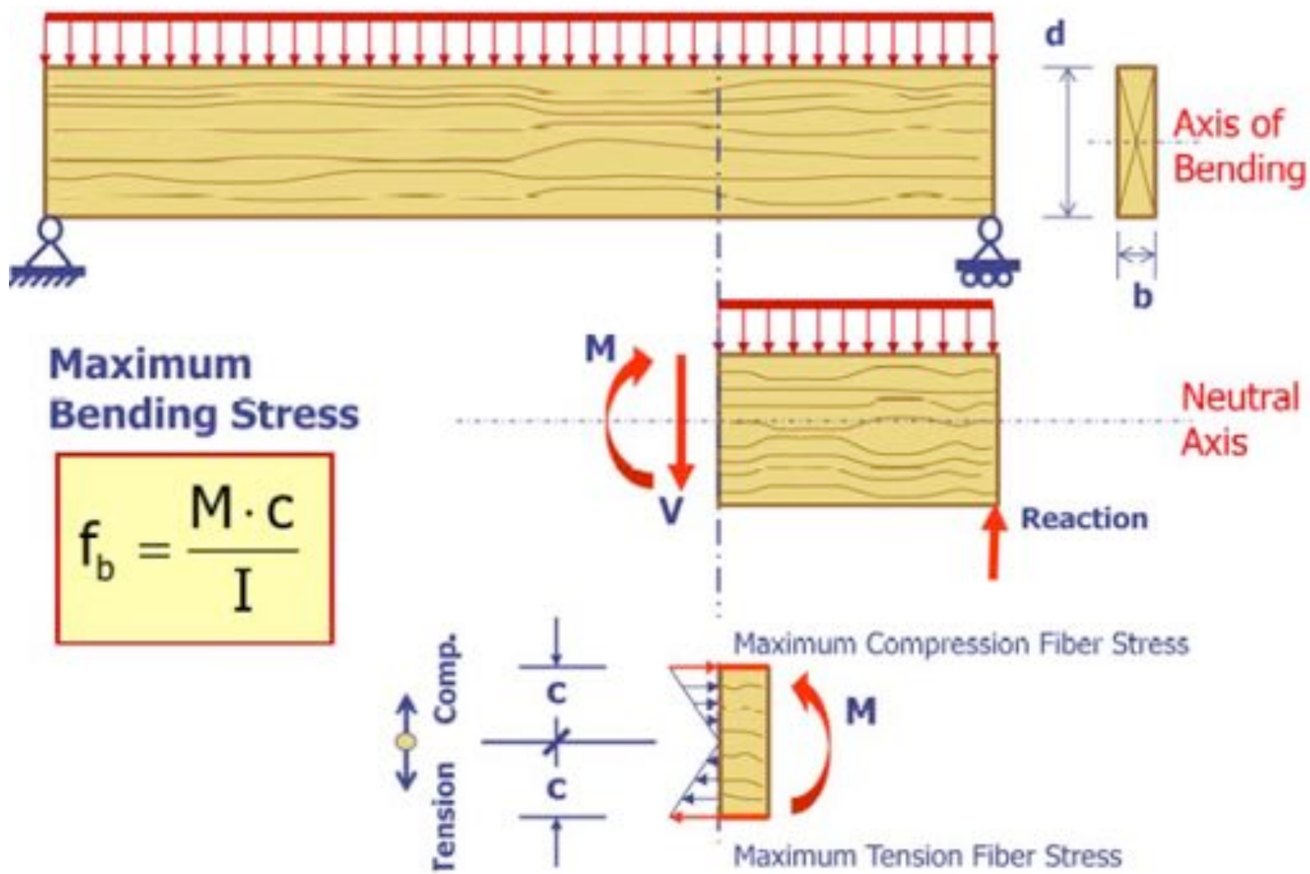
Fasteners distribute loads uniformly to all plies



Fasteners designed for shear flow to avoid slip at interface

Wood Design: Bending

Bending (or moment) design analyzes a member's ability to resist forces which cause it to bend. These forces typically are vertical loads applied to the strong axis.



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Wood Design: Bending

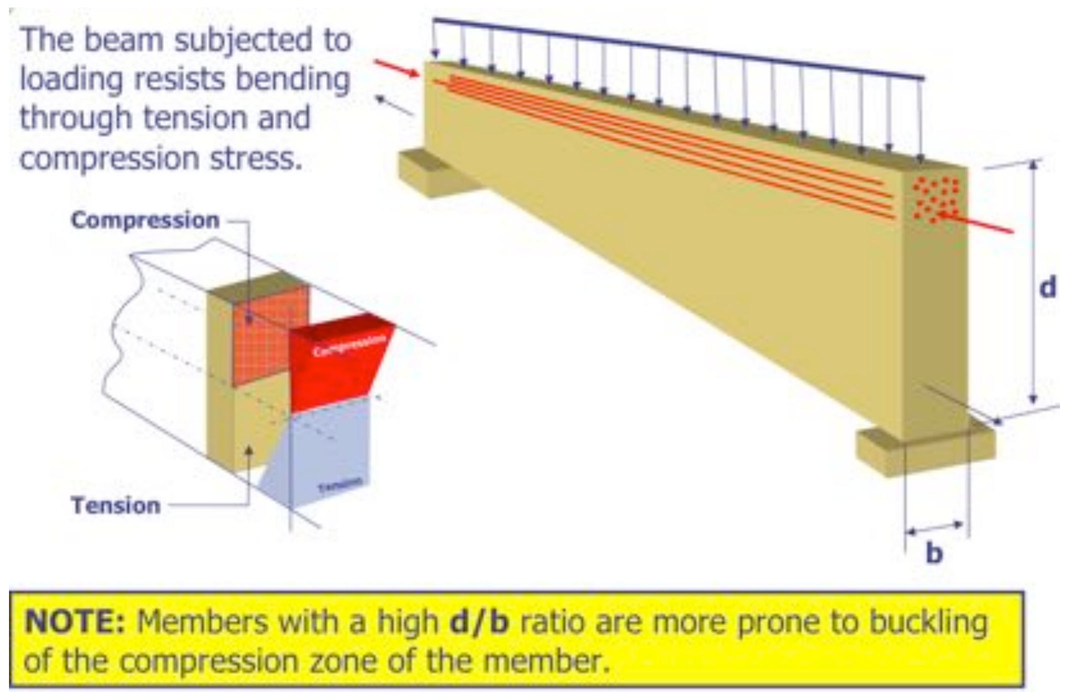
- For simply supported members, use equations of statics

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

Simply supported, uniformly loaded

$$M = \frac{PL}{4}$$

Simply supported, concentrated load at mid-span



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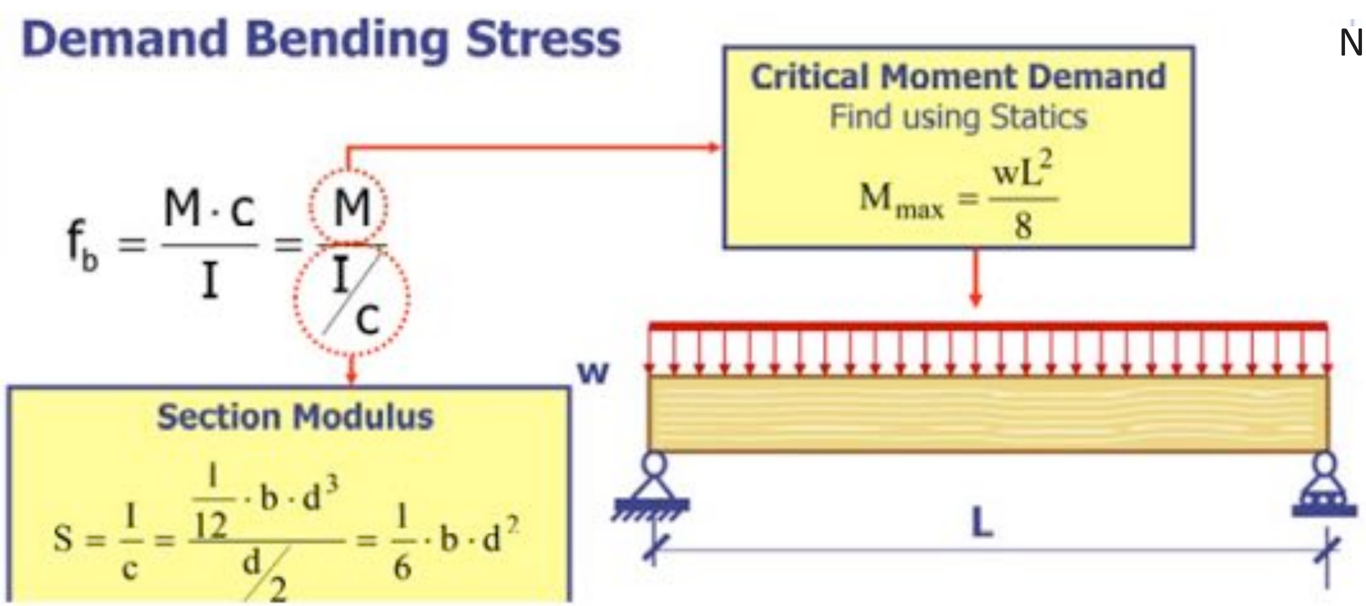
Wood Design: Bending

- Calculate actual design bending stress (ASD)

$$f_b = \frac{M}{S}$$

NDS 3.3.2



Note: If the beam is notched or tapered, the section modules will be different at different beam locations and the stresses at the notch/taper may need to be considered.

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Wood Design: Bending

- Compare actual design stress to allowable design stress

$$f_b \leq F'_b$$

- Bending capacity is dependent on bracing of compression edge of member. Common overlooked applications of this include wall headers (especially garage door headers) and members supporting concentrated loads only. The Beam Stability Factor C_L accounts for this (NDS 3.3.3.8):

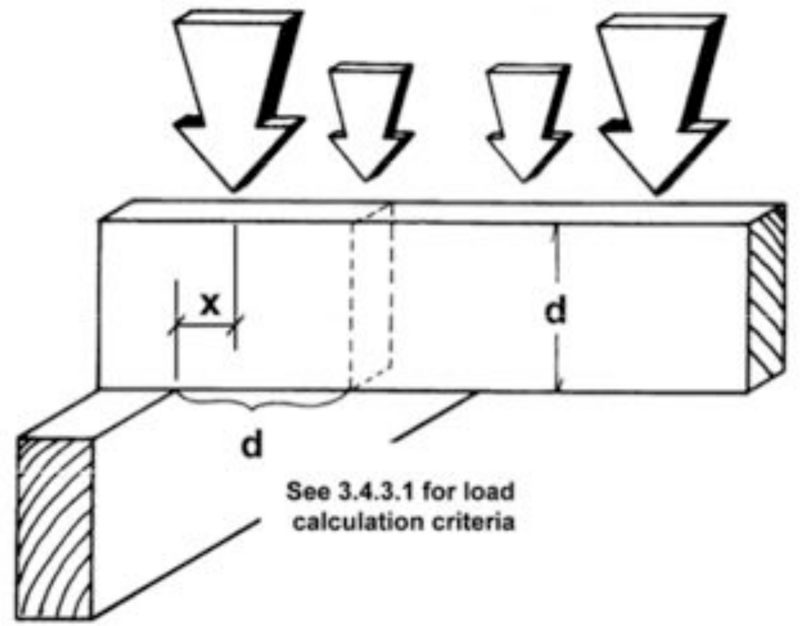
$$C_L = \frac{1 + \left(F_{bE}/F_b^*\right)}{1.9} - \sqrt{\left[\frac{1 + \left(F_{bE}/F_b^*\right)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \quad (3.3\text{-}6)$$

Wood Design: Shear

Shear design analyzes a member's ability to resist forces which cause it to shear, or break off, perpendicular to grain in the direction of the applied load. These forces typically are vertical loads applied to the member's strong axis.

- Uniform loads within d of support may be neglected; concentrated loads within d of supported may be reduced for shear check in certain conditions (NDS 3.4.3.1)

Figure 3C Shear at Supports



Wood Design: Shear

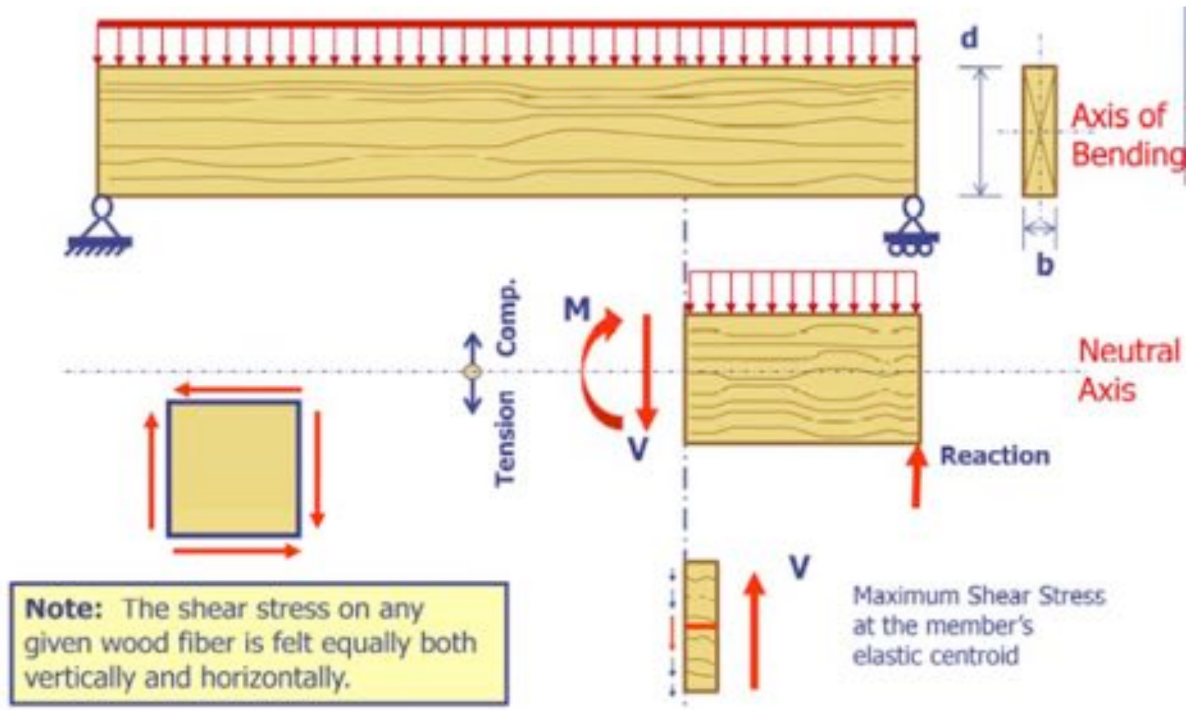
- For simply supported members, use equations of statics

$$V = \frac{P}{2}$$

Simply supported, concentrated load at mid-span

$$V = \frac{wL}{2}$$

Simply supported, uniformly loaded

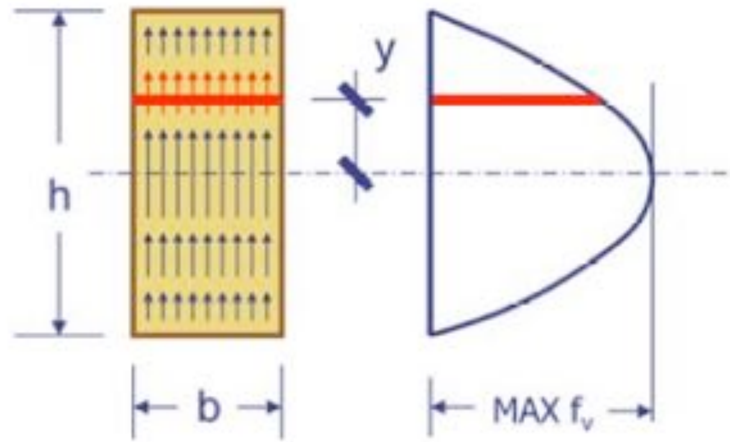


Wood Design: Shear

- Calculate actual design shear stress, f_v

$$f_v = \frac{3V}{2A}$$

NDS 3.4.2



Maximum Shear Stress occurs at the axis of bending (y = 0) $f_v = \frac{3}{2} \cdot \frac{V}{A}$

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- Compare actual stress, f_v, to allowable F'_v

$$f_v \leq F'_v$$

Wood Design: Shear & Notching

Notching affects a member's shear capacity

- For rectangular members notched on their tension face, adjusted shear capacity is (NDS 3.4.3.2 (a)):

$$V_r' = \left[\frac{2}{3} F_v' b d_n\right] \left[\frac{d_n}{d}\right]^2$$

Where:

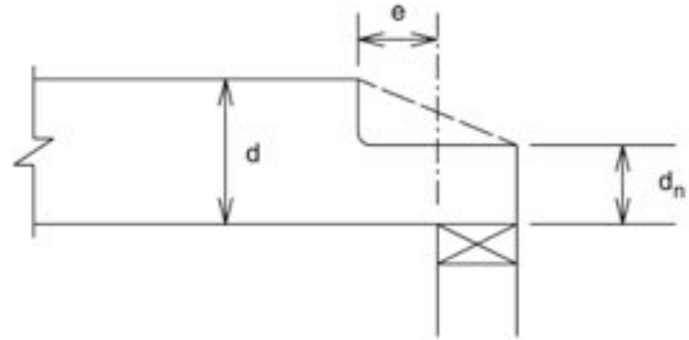
- V_r' = Adjusted design shear
- F_v' = Adjusted shear design value
- d = depth of unnotched member
- d_n = depth of member remaining at notch

Wood Design: Shear & Notching

- For ends of members notched on their compression face (NDS 3.4.3.2 (e)):

$$V_r' = \frac{2}{3} F_v' b \left[d - \left(\frac{d-d_n}{d_n} \right) e \right]$$

Figure 3D Bending Member End-Notched on Compression Face

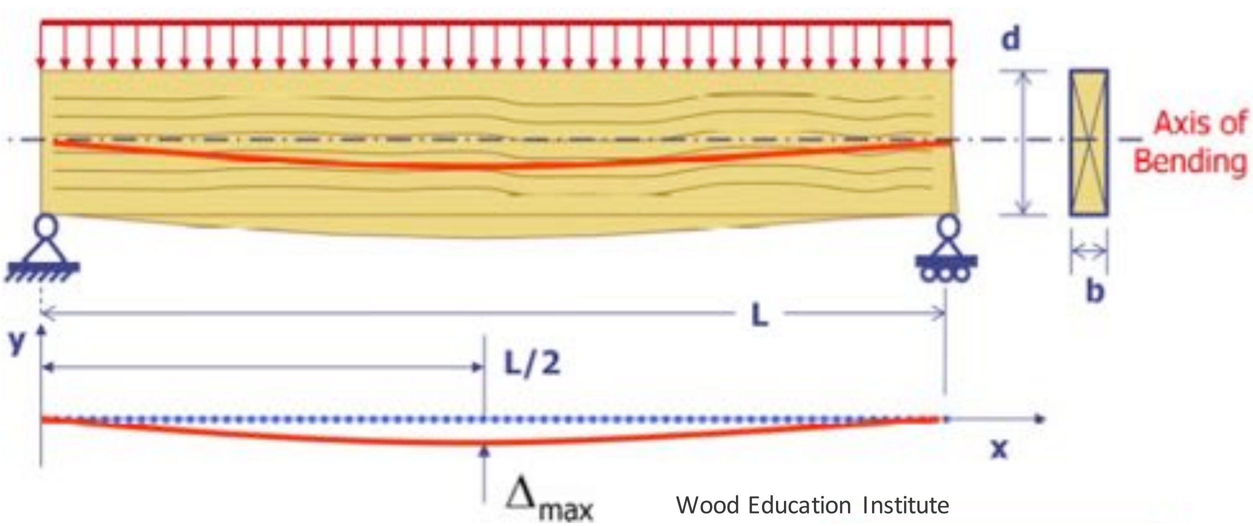


Where:

- V_r' = Adjusted design shear
- F_v' = Adjusted shear design value
- d = depth of unnotched member
- d_n = depth of member remaining at notch
- e = notch extension beyond inner face of support

Wood Design: Deflection

Deflection is a measurement of a wood member's displacement when subjected to design loads (compared to an unloaded condition). Displacement is typically checked for members supporting vertical loads applied to the strong axis.



$$\Delta = \frac{5wL^4}{384EI}$$

$$\Delta = \frac{PL^3}{48EI}$$

- For simply supported members, use equations of statics

Wood Design: Deflection

- Calculate actual deflection & compare to criteria (IBC Table 1604.3) for general, some can exceed minimum limits based on finishes, surrounding features

$$\Delta \leq \frac{L}{360}$$

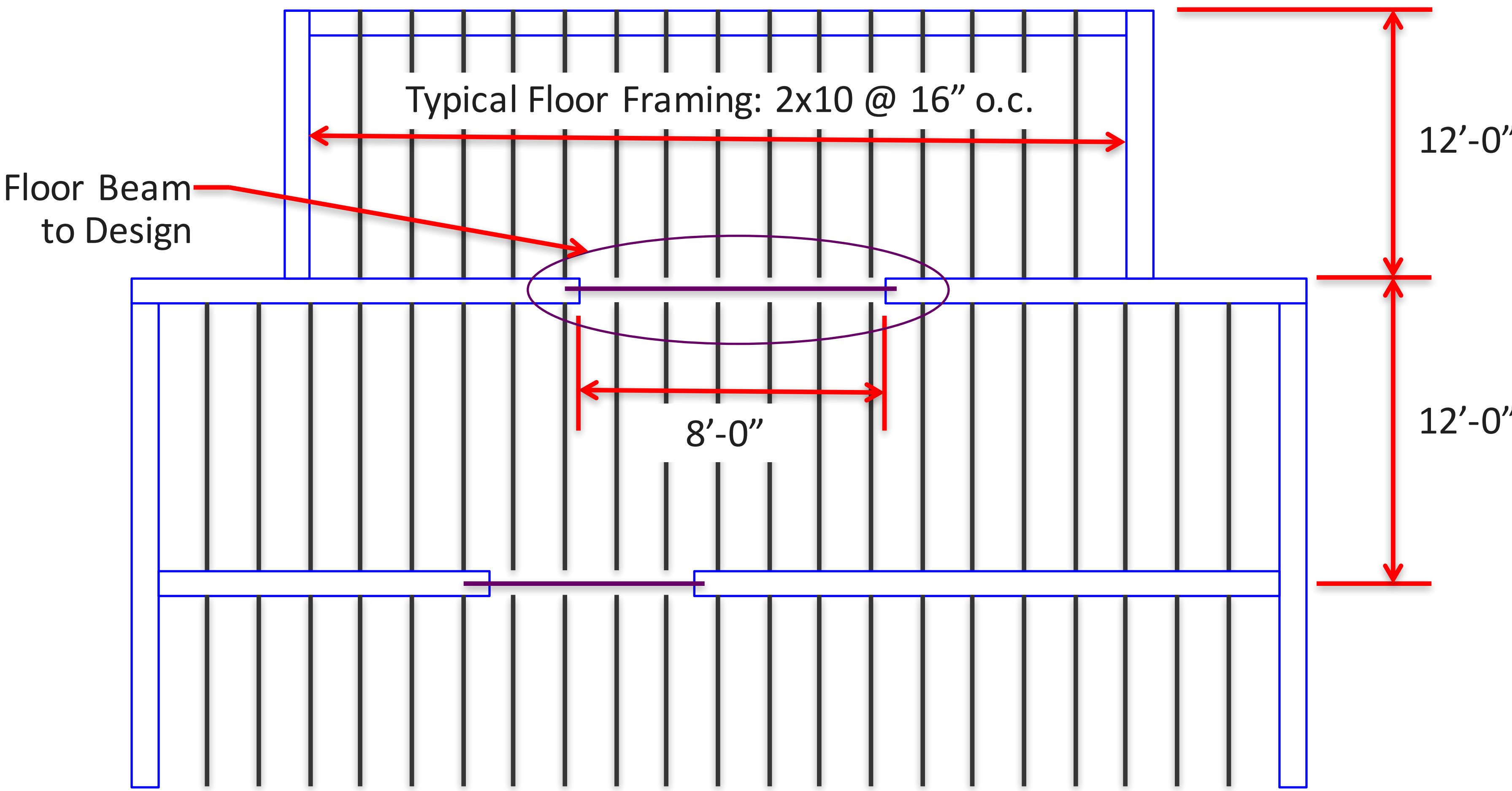
CONSTRUCTION	L	S or W [f]	$D + L$ [d, g]
Roof members:[e]			
Supporting plaster or stucco ceiling	$l/360$	$l/360$	$l/240$
Supporting nonplaster ceiling	$l/240$	$l/240$	$l/180$
Not supporting ceiling	$l/180$	$l/180$	$l/120$
Floor members	$l/360$	—	$l/240$
Exterior walls and interior partitions:			
With plaster or stucco finishes	—	$l/360$	—
With other brittle finishes	—	$l/240$	—
With flexible finishes	—	$l/120$	—
Farm buildings	—	—	$l/180$
Greenhouses	—	—	$l/120$

Beam Design Example

Office Building Floor Framing Plan

Assume Live Load = 50 psf, Dead Load = 30 psf

All framing is Douglas-Fir Larch #2



Beam Design Example

Loading Conditions:

- Tributary Width = 12/2 + 12/2 = 12 ft
- Uniform Live Load = 12 ft x 50 psf = 600 lb/ft
- Uniform Dead Load = 12 ft x 30 psf = 360 lb/ft
- Uniform Total Load = 600 + 360 = 960 lb/ft
- Span = 8 ft

- Douglas-Fir Larch #2: F_b = 900 psi, F_v = 180 psi, E = 1,600,000 psi per NDS 2012 Supplement

$$M = \frac{wL^2}{8} = \frac{(960\frac{lb}{ft})(8\,ft)^2}{8} = 7{,}680\,lb \cdot ft$$

Beam Design Example

Try 3-2x12 Beam; Bending Check:

$$S = \frac{bd^2}{6} = \frac{(4.5\ in)(11.25\ in)^2}{6} = 94.9\ in^3$$

$$f_b = \frac{M}{S} = \frac{(7{,}680\ lb \cdot ft)(\frac{12\ in}{1\ ft})}{94.9\ in^3} = 971\ psi$$

$$F_b' = F_b \cdot C_R = (900\ psi)(1.15) = 1{,}035\ psi$$

$$f_b < F_b' \therefore OK\ for\ bending$$

Beam Design Example

Check 3-2x12 Beam for Shear:

$$V = \frac{w(L-2d)}{2} = \frac{(960\,\frac{lb}{ft})(8\,ft - (2)\left(\frac{11.25\,in}{12\,\frac{in}{ft}}\right))}{2} = 2{,}940\,lb$$

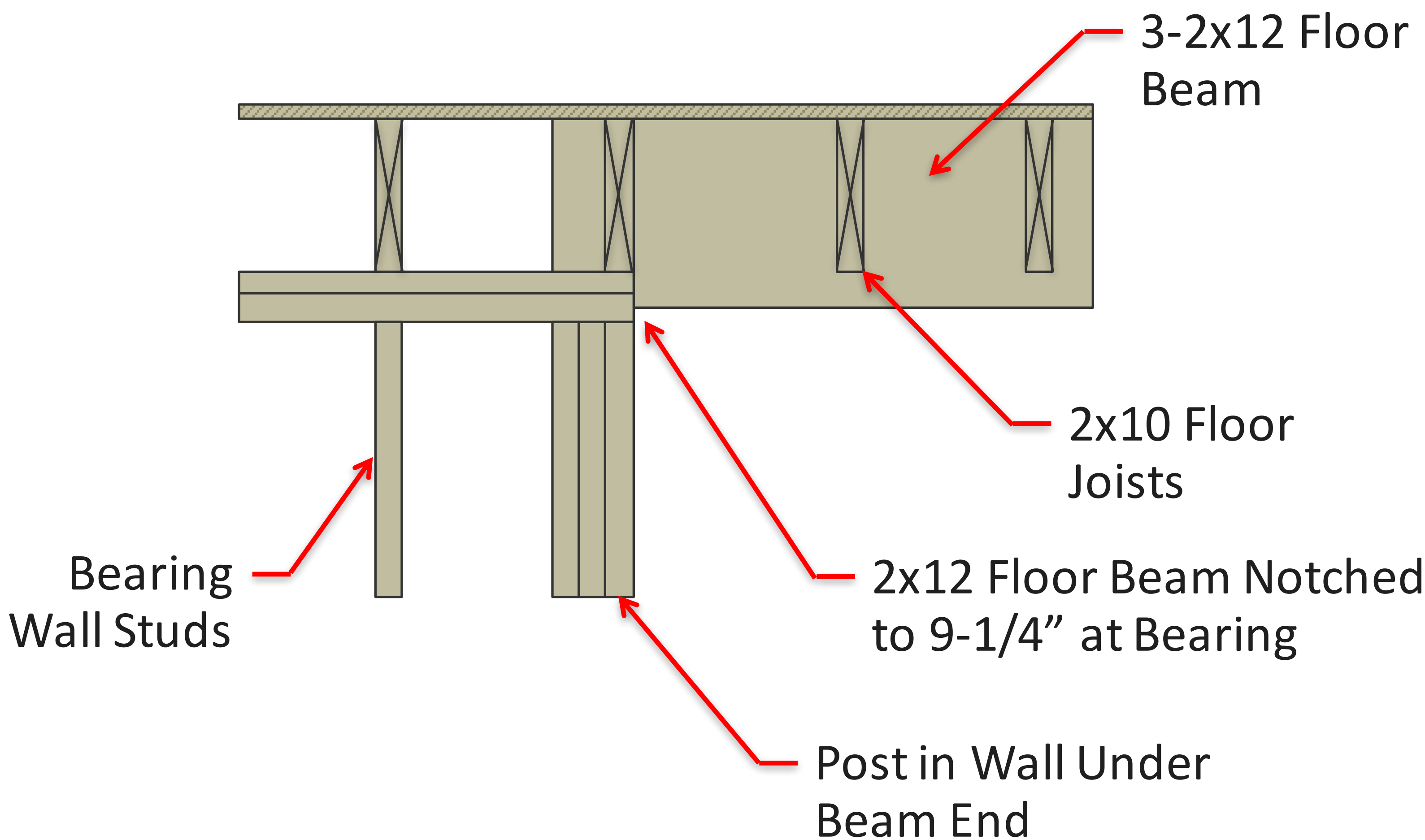
$$f_v = \frac{3V}{2A} = \frac{(3)(2{,}940\,lb)}{(2)(4.5\,in)(11.25\,in)} = 87\,psi$$

$$F_v' = 180\,psi$$

$$f_v < F_v' \therefore OK\ for\ shear$$

Beam Design Example

What if we notch the 3-2x12 beam down to 9-1/4" depth at bearing locations to maintain uniform wall plate elevations?



Beam Design Example

Check Notched 3-2x12 beam for Shear:

$$V_r' = \left[\frac{2}{3} F_v' b d_n\right] \left[\frac{d_n}{d}\right]^2 =$$

$$\left[\frac{2}{3} (180\,psi)(4.5\,in)(9.25\,in)\right] \left[\frac{9.25\,in}{11.25\,in}\right]^2 = 3{,}377\,lb$$

$$V = 2{,}940\,lb$$

$$V < V_r' \therefore OK\ for\ notched\ shear$$

Beam Design Example

Check 3-2x12 beam for Deflection:

$$I = \frac{bd^3}{12} = \frac{(4.5\ in)(11.25\ in)^3}{12} = 534\ in^4$$

$$\Delta_{TL} = \frac{5wL^4}{384EI} = \frac{5\left(960\,\frac{lb}{ft}\right)\left(\frac{1\,ft}{12\,in}\right)[(8\,ft)(12\,\frac{in}{ft})]^4}{384(1{,}600{,}000\ psi)(534\ in^4)} = 0.10\ in$$

$$\Delta_{All\ TL} \leq \frac{L}{240} = \frac{(8\,ft)\left(12\,\frac{in}{ft}\right)}{240} = 0.40\ in$$

Beam Design Example

Check 3-2x12 beam for Deflection:

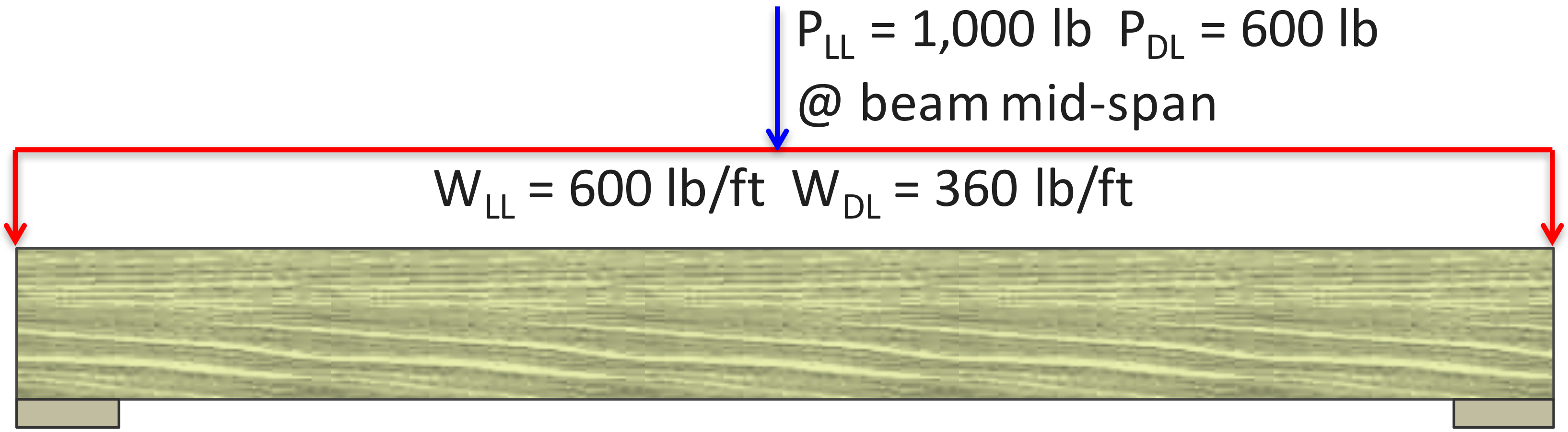
$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5\left(600\ \frac{lb}{ft}\right)\left(\frac{1\ ft}{12\ in}\right)[(8\ ft)(12\ \frac{in}{ft})]^4}{384(1{,}600{,}000\ psi)(534\ in^4)} = 0.06\ in$$

$$\Delta_{All\ LL} \leq \frac{L}{360} = \frac{(8\ ft)\left(12\ \frac{in}{ft}\right)}{360} = 0.27\ in$$

$$\Delta_{LL} < \Delta_{All\ LL} \quad \Delta_{TL} < \Delta_{All\ TL} \therefore OK\ for\ deflection$$

Beam Design Example

What happens if we introduce a concentrated load to the floor beam:



$$M_{total} = M_{uniform} + M_{concentrated} \qquad M_{uniform} = 7{,}680\ lb \cdot ft$$

$$M_{concentrated} = \frac{PL}{4} = \frac{(1{,}600\ lb)(8\ ft)}{4} = 3{,}200\ lb \cdot ft$$

Beam Design Example

Check 3-2x12 beam for concentrated and uniform loads:

$$M_{total} = M_{uniform} + M_{concentrated} = 7{,}680 + 3{,}200 = 10{,}880\ lb \cdot ft$$

$$f_b = \frac{M}{S} = \frac{(10{,}880\ lb \cdot ft)(\frac{12\ in}{1\ ft})}{94.9\ in^3} = 1{,}376\ psi$$

$$F_b' = F_b \cdot C_R = (900\ psi)(1.15) = 1{,}035\ psi$$

$$f_b > F_b' \therefore Inadequate\ for\ bending$$

Beam Design Example

Try 4-2x12 beam for concentrated and uniform loads:

$$S = \frac{bd^2}{6} = \frac{(6\ in)(11.25\ in)^2}{6} = 126.6\ in^3$$

$$f_b = \frac{M}{S} = \frac{(10{,}880\ lb \cdot ft)(\frac{12\ in}{1\ ft})}{126.6\ in^3} = 1{,}031\ psi$$

$$F_b' = F_b \cdot C_R = (900\ psi)(1.15) = 1{,}035\ psi$$

$$f_b < F_b' \therefore OK\ for\ bending$$

Beam Design Example

Try 4-2x12 beam for concentrated and uniform loads; notched shear:

$$V_{concentrated} = \frac{P}{2} = \frac{1{,}600\ lb}{2} = 800\ lb$$

$$V_{total} = V_{uniform} + V_{concentrated} = 2{,}940 + 800 = 3{,}740\ lb$$

$$V_r' = \left[\frac{2}{3}F_v{}'bd_n\right]\left[\frac{d_n}{d}\right]^2 =$$

$$\left[\frac{2}{3}(180\ psi)(6\ in)(9.25\ in)\right]\left[\frac{9.25\ in}{11.25\ in}\right]^2 = 4{,}503\ lb$$

$$V < V_r' \therefore OK\ for\ notched\ shear$$

Beam Design Example

Try 4-2x12 beam for concentrated and uniform loads; deflection:

$$I = \frac{bd^3}{12} = \frac{(6\ in)(11.25\ in)^3}{12} = 712\ in^4$$

$$\Delta_{conc.TL} = \frac{PL^3}{48EI} = \frac{(1{,}600\ lb)[(8\ ft)\left(12\frac{in}{ft}\right)]^3}{48(1{,}600{,}000\ psi)(712\ in^4)} = 0.03\ in$$

$$\Delta_{TL\ total} = \Delta_{TL\ uniform} + \Delta_{TL\ concentrated} = 0.10 + 0.03 = 0.13\ in$$

$$\Delta_{All\ TL} \le \frac{L}{240} = \frac{(8\ ft)\left(12\frac{in}{ft}\right)}{240} = 0.40\ in$$

Beam Design Example

Try 4-2x12 beam for concentrated and uniform loads; deflection:

$$\Delta_{conc.\ LL} = \frac{PL^3}{48EI} = \frac{(1{,}600\ lb)[(8\ ft)\left(12\frac{in}{ft}\right)]^3}{48(1{,}600{,}000\ psi)(712\ in^4)} = 0.02\ in$$

$$\Delta_{LL\ total} = \Delta_{LL\ uniform} + \Delta_{lL\ concentrated} = 0.06 + 0.02 = 0.08\ in$$

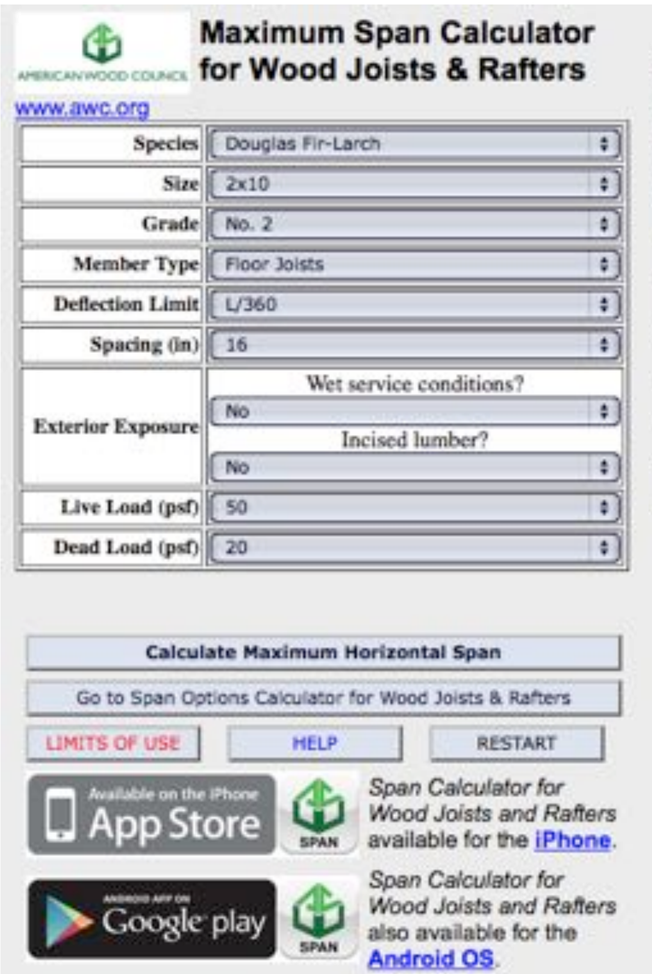
$$\Delta_{All\ LL} \leq \frac{L}{360} = \frac{(8\ ft)\left(12\frac{in}{ft}\right)}{360} = 0.27\ in$$

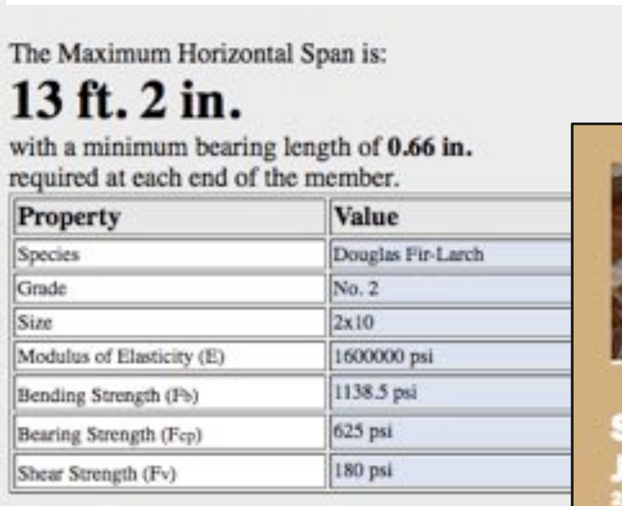
$$\Delta_{LL} < \Delta_{All\ LL} \quad \Delta_{TL} < \Delta_{All\ TL} \therefore OK\ for\ deflection$$

Beam Design Aids

American Wood Council Span Calculator:
http://www.awc.org/calculators/index.php

American Wood Council Span Tables:
http://www.awc.org/technical/spantables/index.php



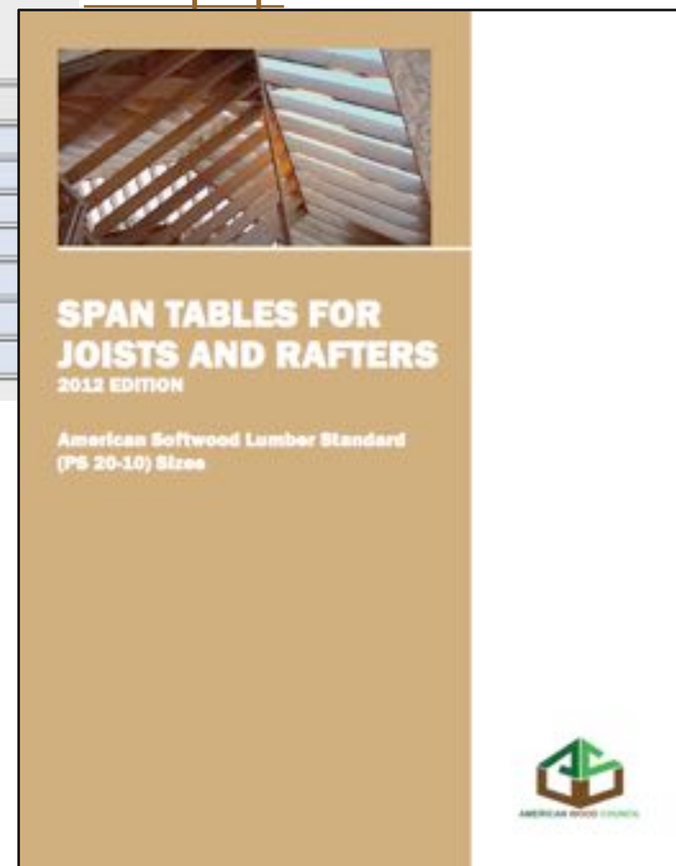


The Maximum Horizontal Span is:

13 ft. 2 in.

with a minimum bearing length of **0.66 in.** required at each end of the member.

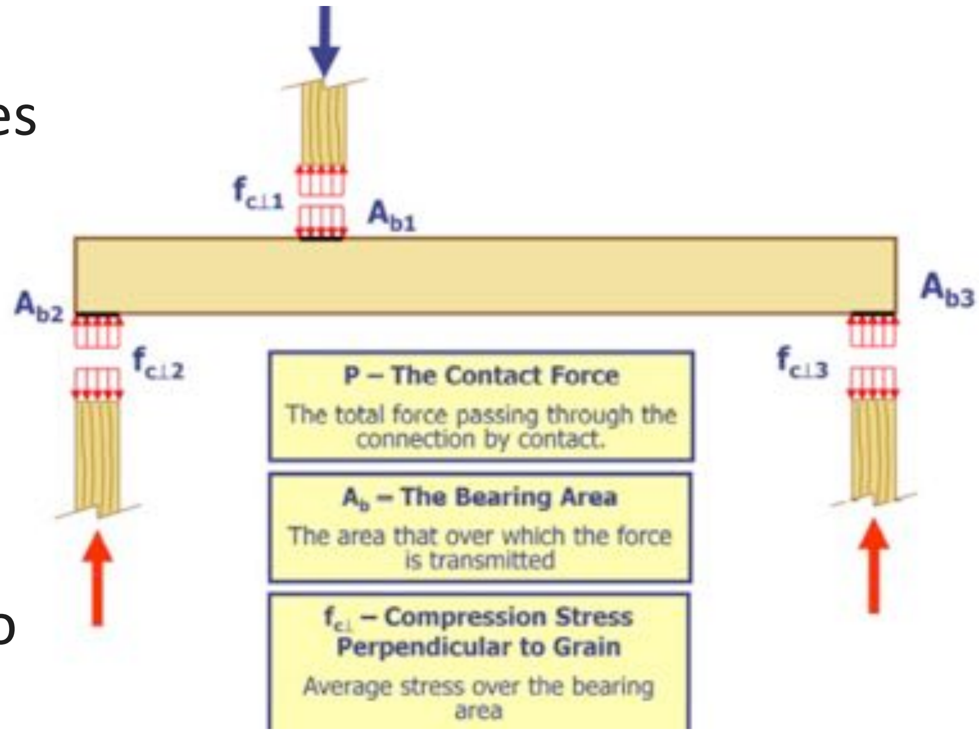
Property	Value
Species	Douglas Fir-Larch
Grade	No. 2
Size	2x10
Modulus of Elasticity (E)	1600000 psi
Bending Strength (Fb)	1138.5 psi
Bearing Strength (Fcp)	625 psi
Shear Strength (Fv)	180 psi



Wood Design: Compression (Bearing)

Compression perpendicular to grain, or bearing design analyzes a member's ability to resist forces applied to one of it's faces, or surfaces, without crushing.

- Consider points of support and concentrated loads
- Consider loading direction to grain



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Wood Design: Compression (Bearing)

- Calculate actual bearing stress

$$f_{c\perp} = \frac{P}{A_{brg}}$$

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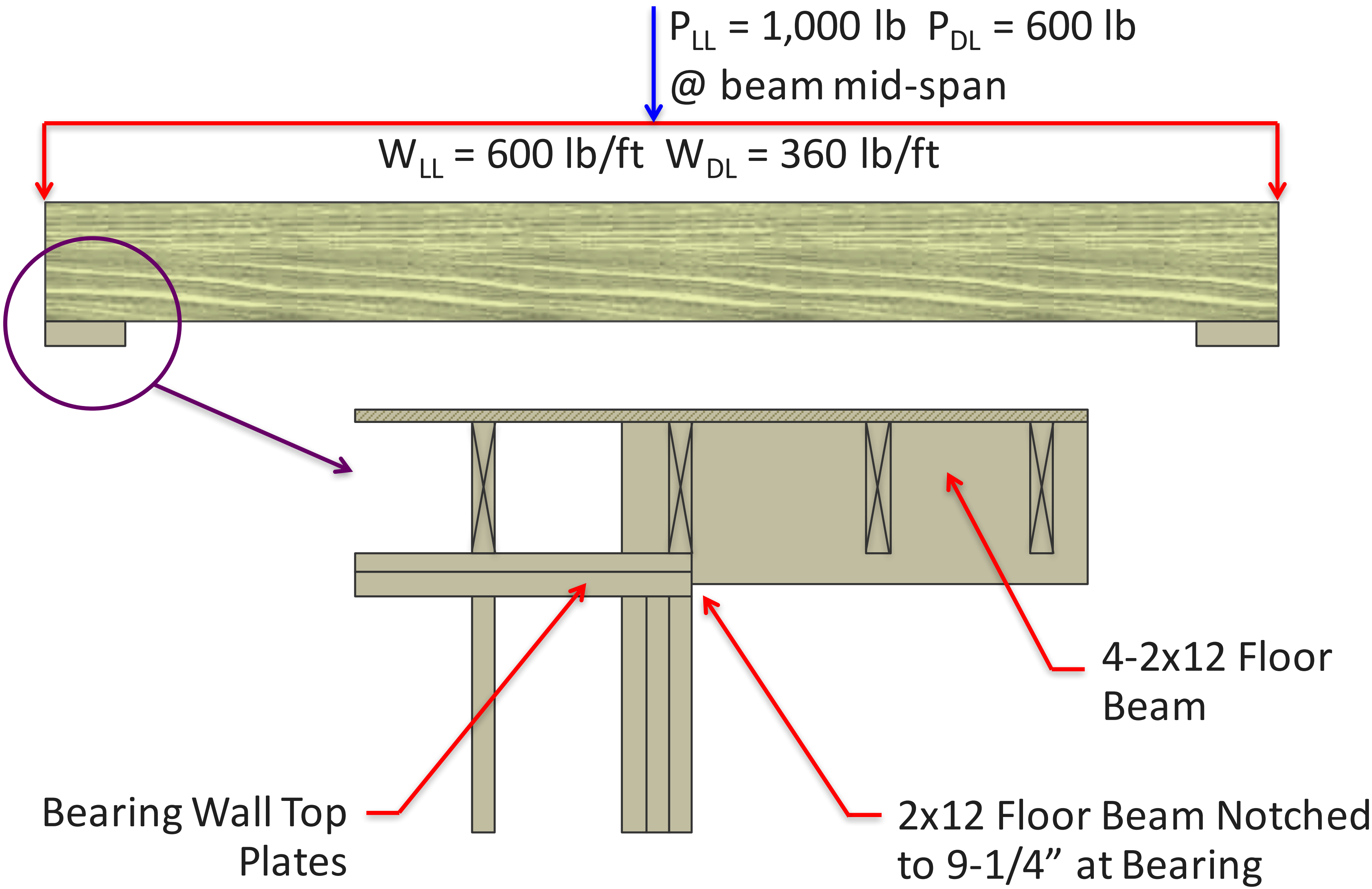
Where:

- P = Support reaction or applied load
- A_{brg} = Bearing Area

- Compare actual bearing stress to allowable

$$f_{c\perp} \leq F'_{c\perp}$$

Bearing Design Example



Bearing Design Example

Reaction at Beam End:

$$V = \frac{wL}{2} + \frac{P}{2} = \frac{(960\,\frac{lb}{ft})(8\,ft)}{2} + \frac{1{,}600\ lb}{2} = 4{,}640\ lb$$

Need to Look at Bearing Area

- Of Bottom of Beam
- Of Wall Top Plates

4-2x12 Floor Beam: Width = 6”, Species = DFL #2, F_{cperp} = 625 psi

2x6 Bearing Wall: Width = 5-1/2”, Species = SPF #2, F_{cperp} = 425 psi

Bearing Design Example

Beam width exceeds wall top plate width; can only use wall top plate width. Determine required bearing length:

$$L_{brng} = \frac{V}{b * F_{cperp}}$$

Bearing width will be same for both beam and wall top plates (5-1/2"). Wall plates will control as allowable bearing stress is lower.

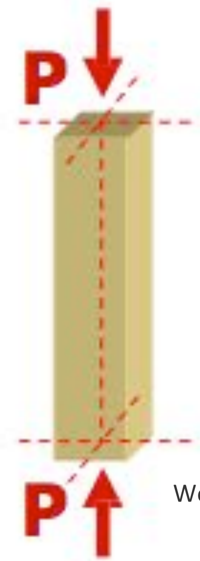
$$L_{brng} = \frac{V}{b * F_{cperp}} = \frac{4{,}640\ lb}{(5.5\ in)(425\ psi)} = 2.0\ in$$

Beam needs to extend 2" minimum onto wall top plate

Wood Design: Compression

Compression parallel to grain design analyzes a member's ability to resist compressive forces applied to its longitudinal axis without buckling or failing. Common applications are columns and truss members.

$$f_c = \frac{P}{A}$$



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- Calculate actual compression stress & compare to allowable

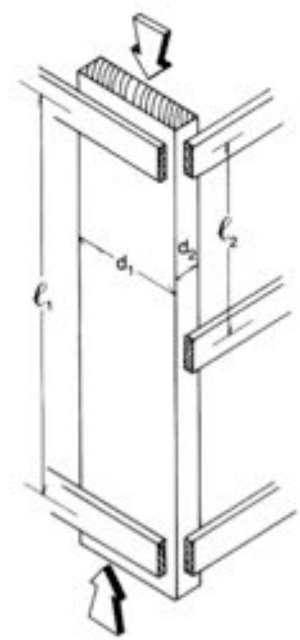
$$f_c \leq F'_c$$

Wood Design: Compression

- Unbraced length in both axes need to be considered
- NDS Column Stability Factor C_p accounts for unbraced lengths (NDS 3.7.1)

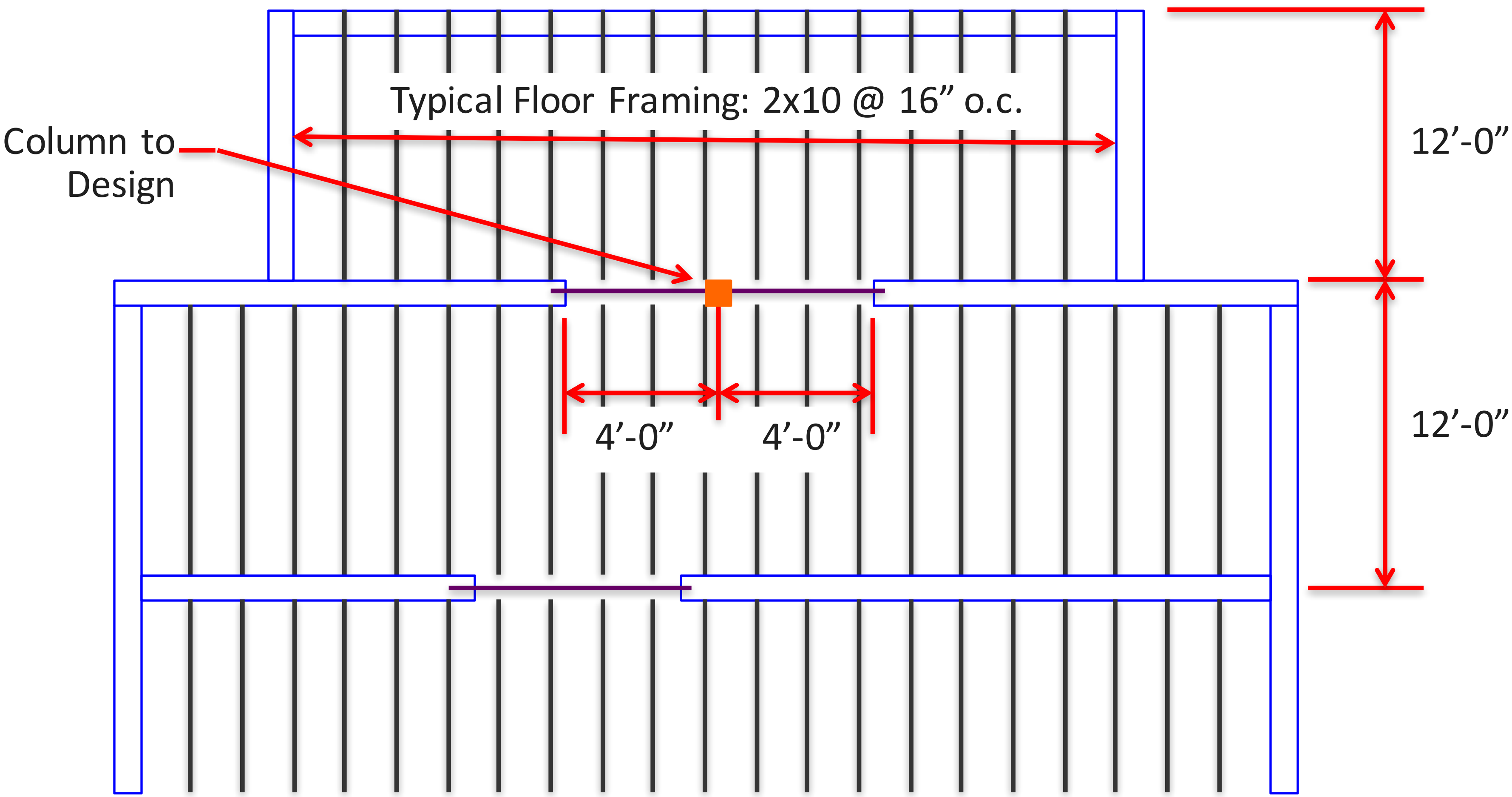
$$C_P = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

Figure 3F Simple Solid Column



Column Design Example

Using same office building floor framing plan and loading conditions from previous example, add column at beam mid-span



Column Design Example

Loading Conditions:

- Tributary Area = (12/2 + 12/2) + (4/2 + 4/2) = 48 ft^2
- Uniform Live Load = 48 ft^2 x 50 psf = 2,400 lb
- Uniform Dead Load = 48 ft^2 x 30 psf = 1,440 lb
- Concentrated Live Load = 1,000 lb
- Concentrated Dead Load = 600 lb
- Total Load = 2,400 + 1,440 + 1,000 + 600 = 5,440 lb
- Height = 10 ft

- Try 6x6 column (use Timber 5x5 or larger reference design values):

Douglas-Fir Larch #2: F_b = 750 psi, F_c = 700 psi, E = 1,300,000 psi, E_{min} = 470,000 psi per NDS 2012 Supplement

Column Design Example

- Assume end connections are modeled as pins, column buckling coefficient K = 1.0
- L_e/d = [(10 ft)(12 in/ft)]/5.5 in = 21.8 < 50, OK

$$F_{cE} = \frac{0.822E_{min}}{(\frac{l_e}{d})^2} = \frac{(0.822)(470{,}000\ psi)}{(21.8)^2} = 813\ psi$$

$$C_p = \frac{1+(\frac{F_{cE}}{F_{c^*}})}{2c} - \sqrt{\left[\frac{1+(\frac{F_{cE}}{F_{c^*}})}{2c}\right]^2 - \frac{\frac{F_{cE}}{F_{c^*}}}{c}} = \frac{1+\left(\frac{813\ psi}{700\ psi}\right)}{(2)(0.80)} - \sqrt{\left[\frac{1+\frac{813\ psi}{700\ psi}}{(2)(0.80)}\right]^2 - \frac{\frac{813\ psi}{700\ psi}}{0.80}} = 0.74$$

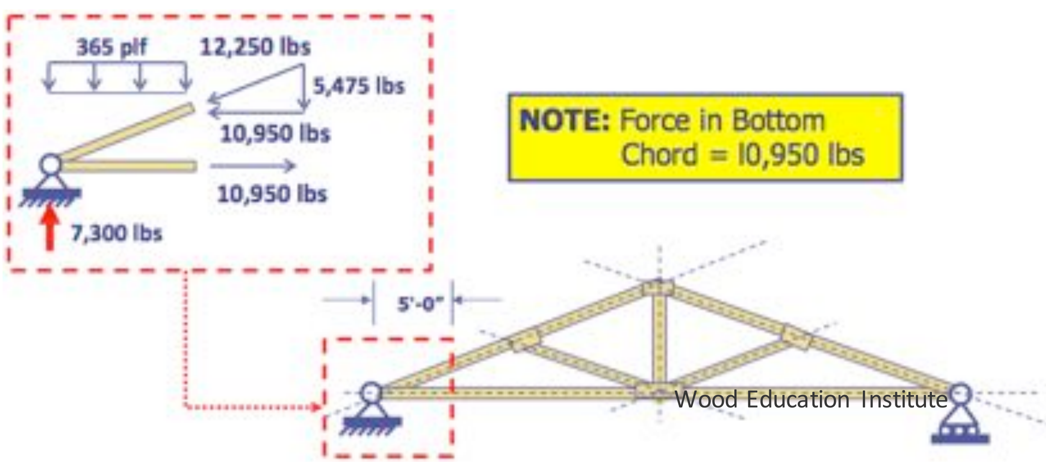
$$F_c' = F_c \cdot C_p = (700\ psi)(0.74) = 518\ psi$$

$$f_c = \frac{P}{A} = \frac{5{,}440\ lb}{(5.5\ in)(5.5\ in)} = 180\ psi \qquad f_c < F_c' \therefore OK\ for\ compression$$

Wood Design: Tension

Tension parallel to grain design analyzes a member's ability to resist tension (pulling) forces applied to its longitudinal axis without failing.

- Unbraced length for tension design does not need to be considered
- Avoid tension perpendicular to grain



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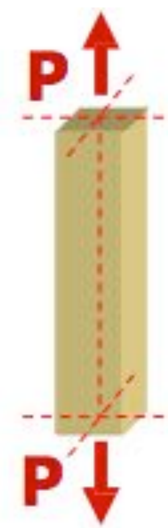
$$f_t = \frac{P}{A}$$



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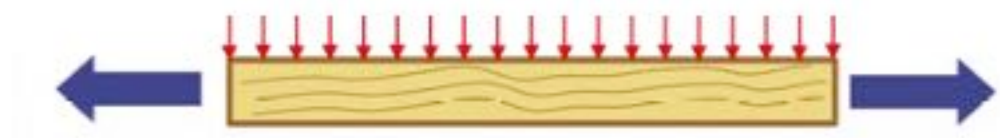
- Calculate actual tension stress & compare to allowable

$$f_t \leq F_t'$$

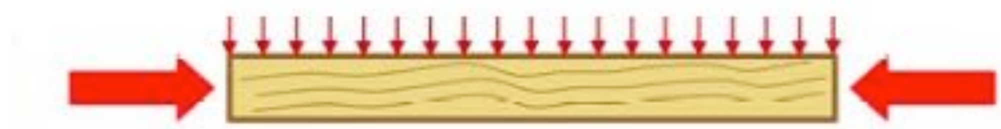
Wood Design: Combined Loads

Members subject to both bending and axial loads (tension or compression) shall be adequately designed to meet a series of interaction equations per NDS 3.9.1 (bending & tension) & NDS 3.9.2 (bending & compression):

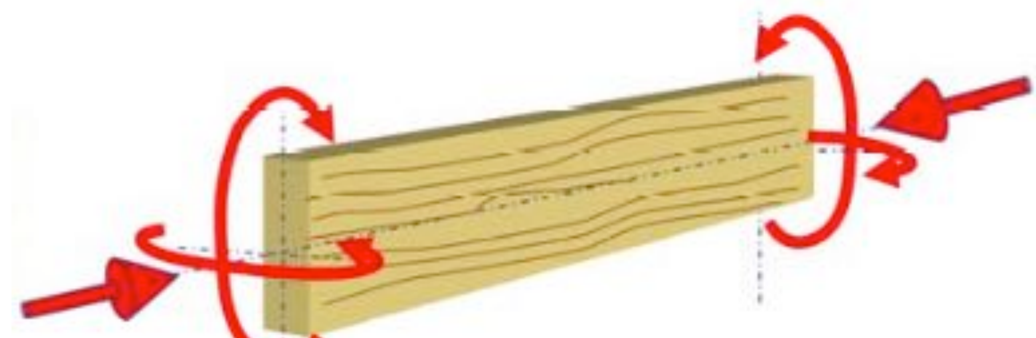
Tension + Bending



Compression + Bending



Axial + Biaxial Bending



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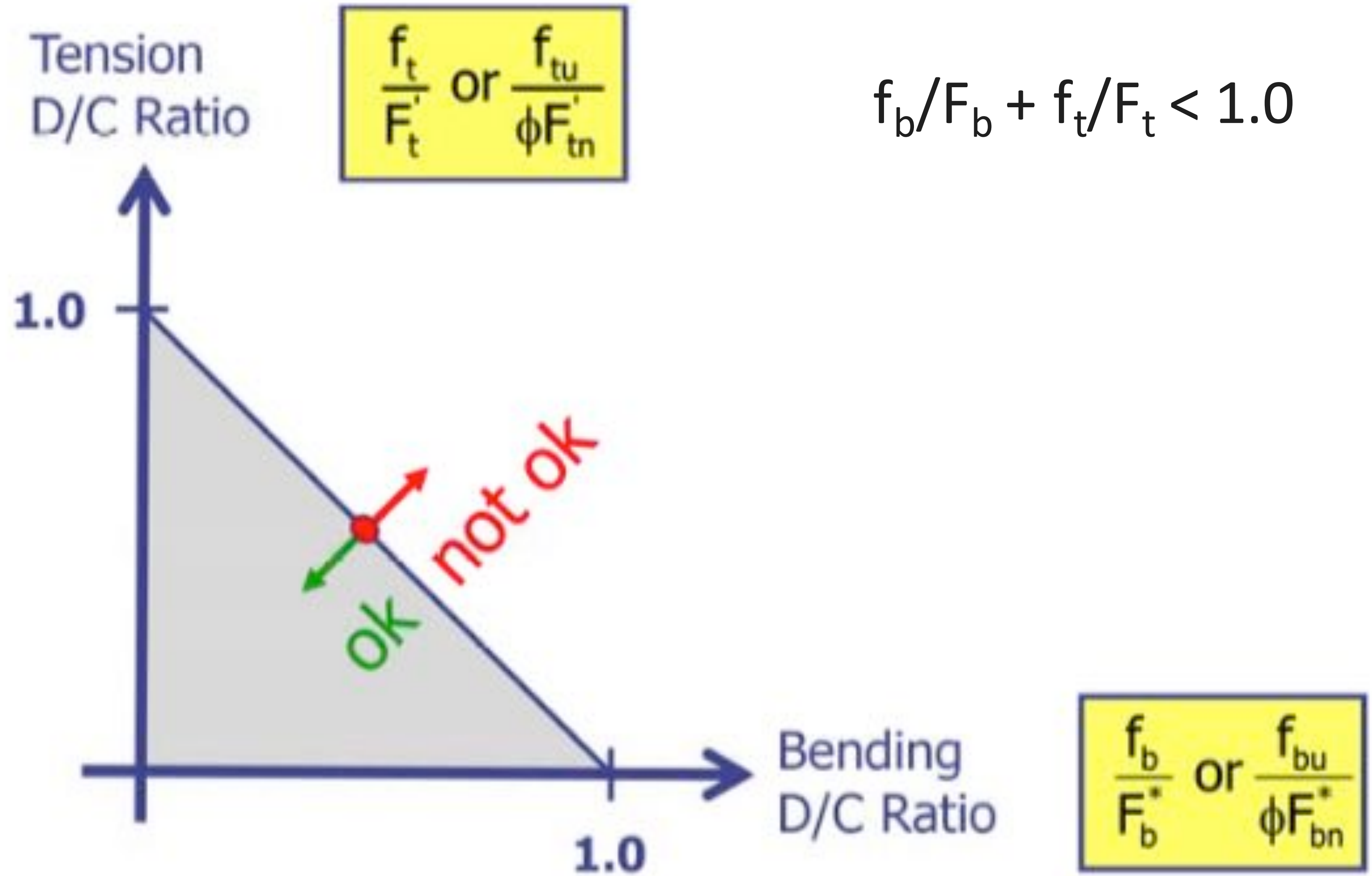
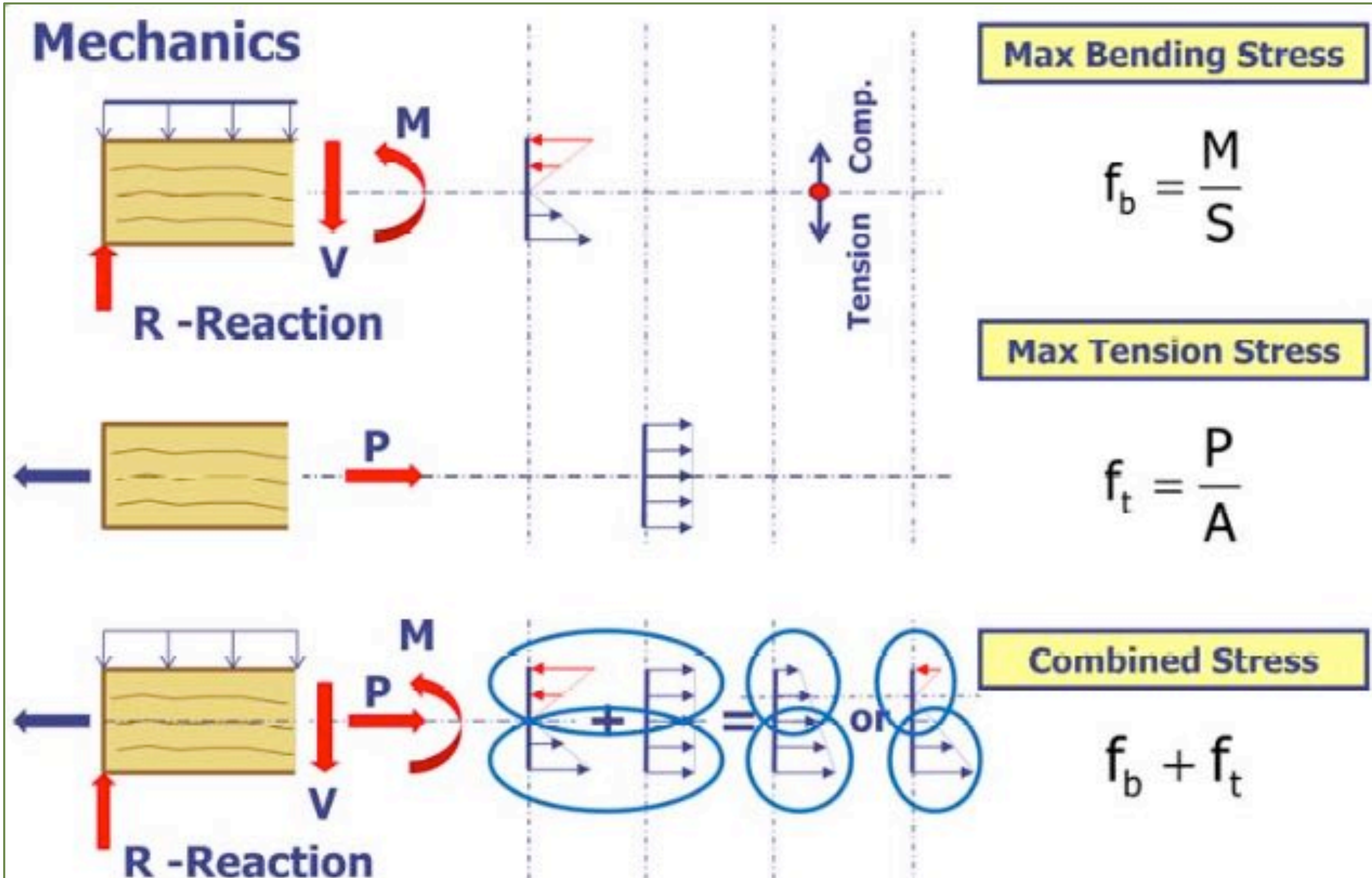
Wood Design: Combined Loads

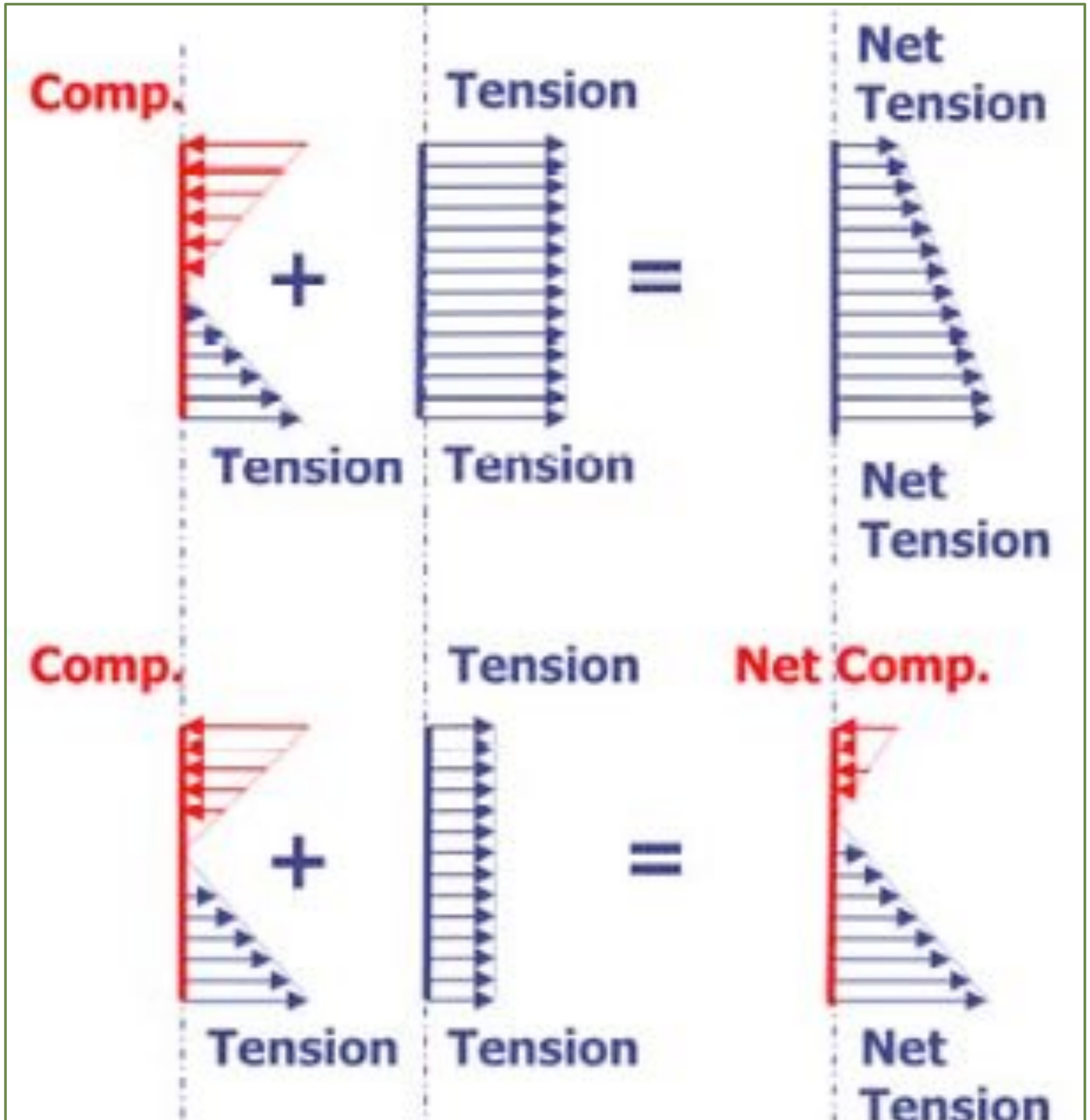


$f_b/F_b + f_t/F_t < 1.0$

Wood Design: Combined Loads



Wood Design: Combined Loads



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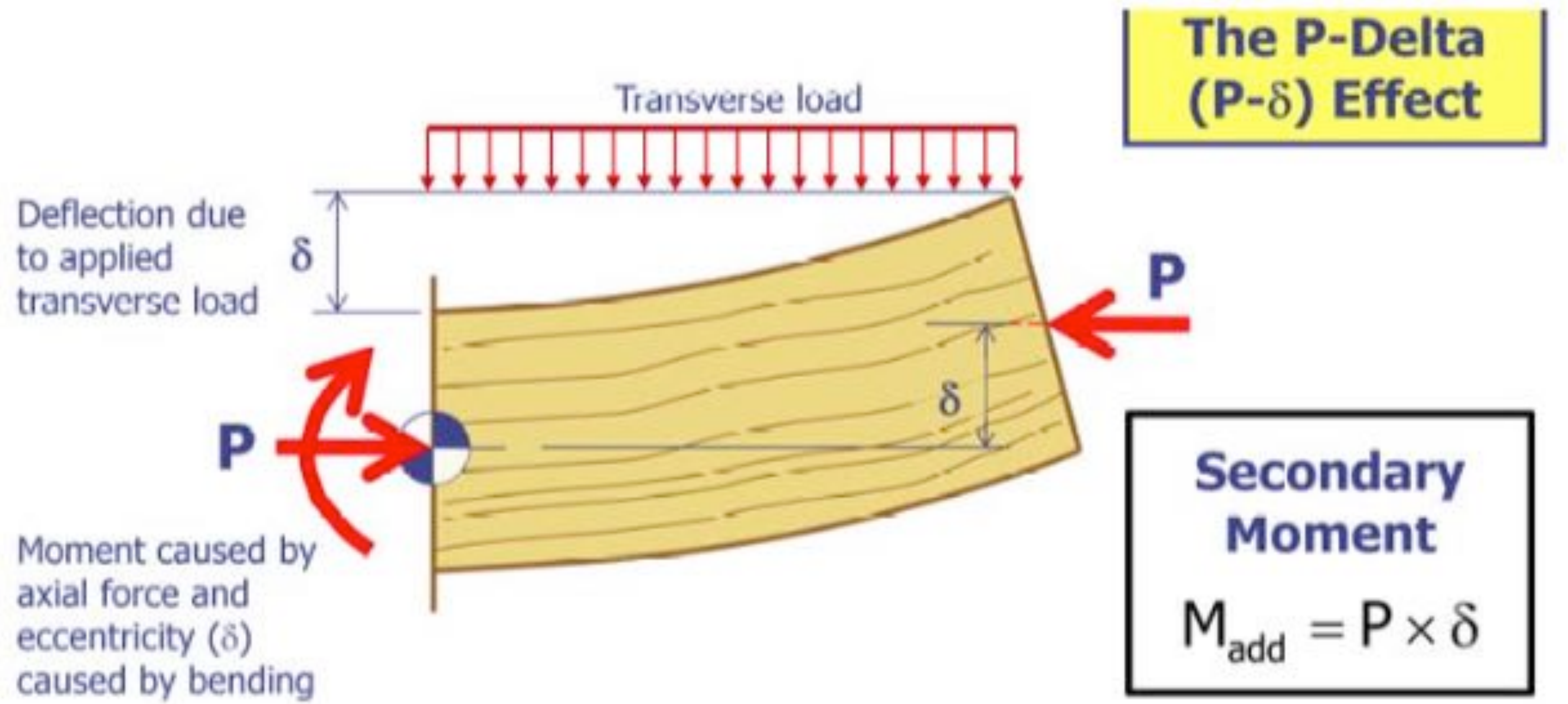
Wood Design: Combined Loads



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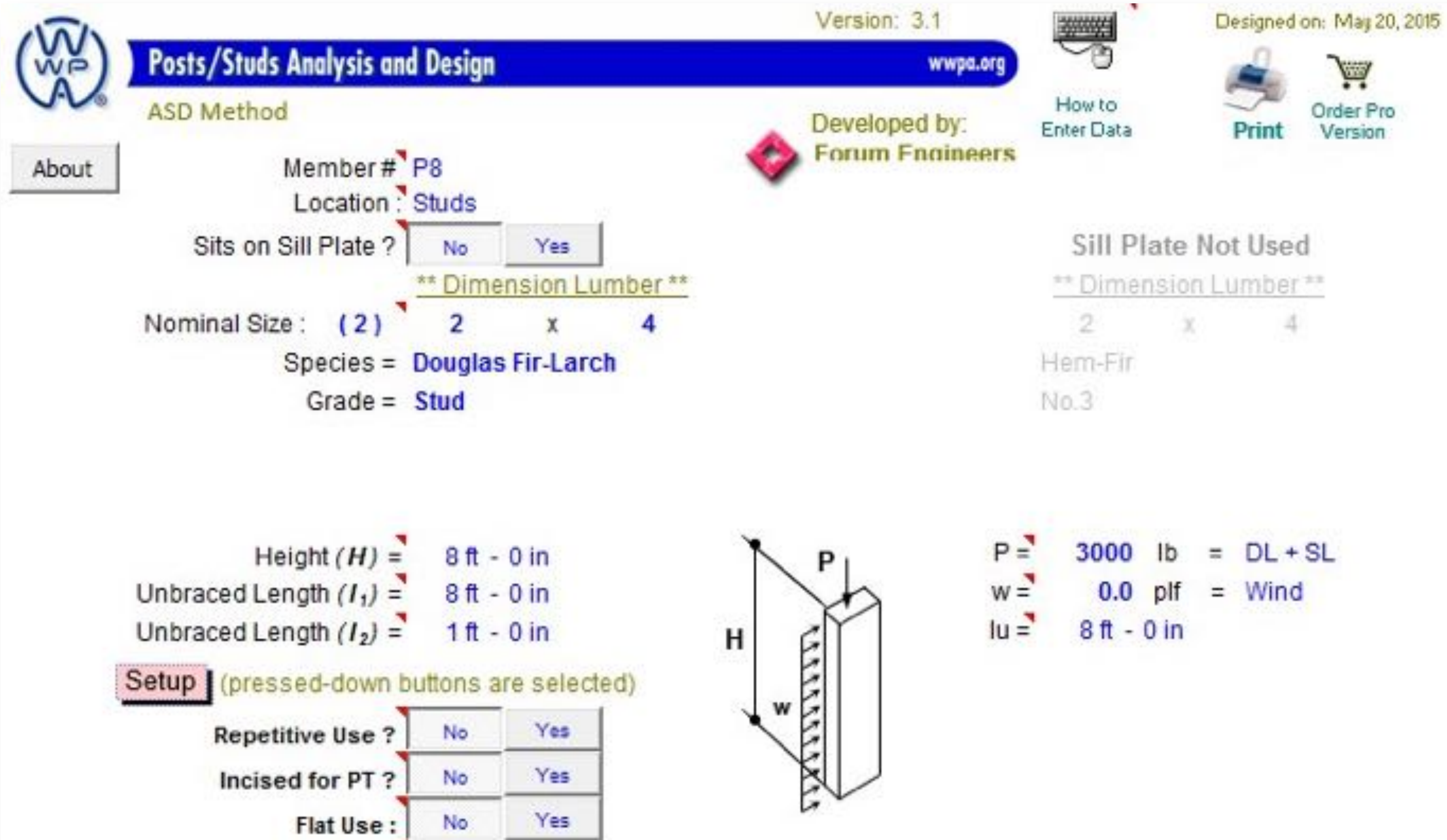
Combined Loads Design Aids:

Western Wood Products Association (WWPA) Design Suite:

http://www.wwpa.org/TECHGUIDE/DesignSoftware/tabid/859/Default.aspx



Wood Connections: Design

Wood connections generally resist 2 types of forces: shear (lateral) and withdrawal. Connections typically consist of one or more of the following:

- Fastener (nail, screw, bolt, etc.)
- Connector (steel side plate, light gauge steel angle, wood splice plate, etc.)

Michael Stenstrom, Strenstrom-Schneider, Inc.

Wood Connections: Design

Connection design is a function of a number of factors including:

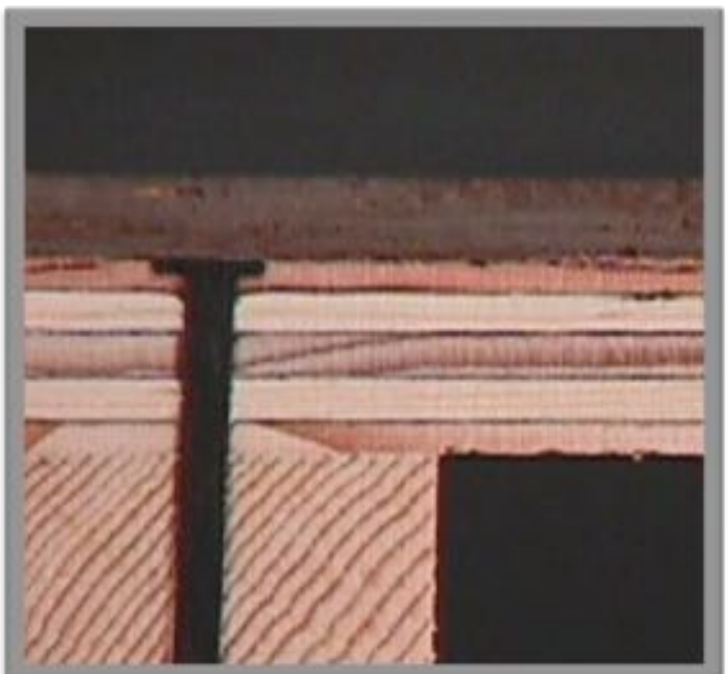
- Wood dowel bearing strength
- Wood member(s) thickness
- Angle to grain
- Fastener (diameter, penetration, bending capacity)
- Edge/End Distance
- Spacing

NDS provides references design values and adjustment factors

Wood Connections: Options

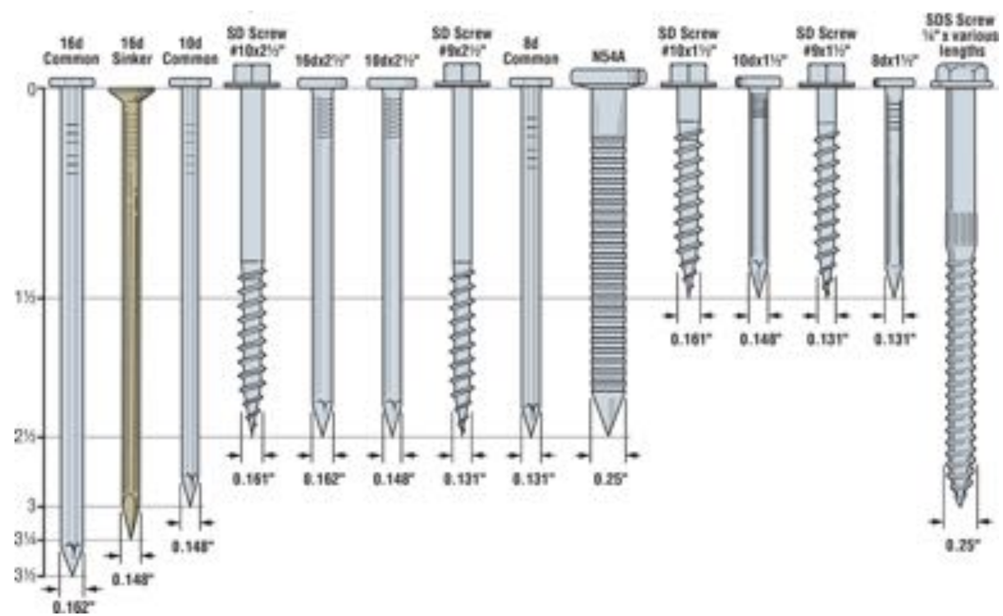
Wood connection categories:

- Dowel type fasteners (bolts, lag screws, wood screws, nails, spikes, drift bolts, drift pins)
- Split rings & shear plates
- Timber rivets
- Spike grids



Wood Connections: Dowel Type Fasteners

- Dowel type fasteners, particularly nails, are most common fastener in wood construction
- Other dowel type fasteners include bolts, lag screws, wood screws, spikes, drift bolts, and drift pins
- Used as sole connectors or in conjunction with connecting member (steel side plate, prefabricated hardware, etc.)
- IBC Section 2304.9 provides fastener schedules for standard connections



Dowel Type Fasteners: Withdrawal Design

Withdrawal connection design:

- Connections in withdrawal resist forces which would attempt to pull a fastener out of the wood
- Withdrawal not permitted for nails & spikes in end grain of wood
- NDS Chapter 11 provides reference withdrawal design values for nails, wood screws, lag screws, and ring shank nails

Dowel Type Fasteners: Withdrawal Design

Withdrawal connection design:

- Capacity is a function of:
    - Fastener diameter
    - Fastener penetration
    - Wood properties (SG)

Table 11.2A Lag Screw Reference Withdrawal Design Values, W[1]

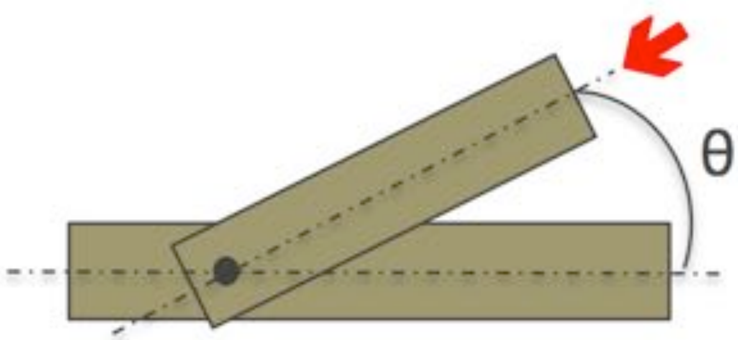
Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 11.2.1.1).

Specific Gravity, G[2]	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752

Dowel Type Fasteners: Shear Design

Shear (Lateral) connection design:

- Shear connections resist forces which would attempt to move (or displace) one member relative to its connected member
- Used to resist gravity (vertical) loads and lateral (horizontal) loads
- NDS Chapter 11 provides reference lateral design values for bolts, nails, wood screws, rink shank nails, and lag screws
- Edge distance, end distance, spacing requirements



$$F_{e\theta} = \frac{F_{e\parallel} F_{e\perp}}{F_{e\parallel} sin^2\theta + F_{e\perp} cos^2\theta}$$

Dowel Type Fasteners: Shear Design

Shear (Lateral) connection design:

- Capacity is a function of:
  - Fastener diameter, penetration, mechanical strength
  - Wood properties (SG)
  - Wood member(s) angle to grain

Table 11.3.3 Dowel Bearing Strengths, F_e, for Dowel-Type Fasteners in Wood Members

Specific[1] Gravity, G	Dowel bearing strength in pounds per square inch (psi)[2]										
	F_e	$F_{e\parallel}$	$F_{e\perp}$								
	D<1/4"	1/4" ≤ D ≤ 1"	D=1/4"	D=5/16"	D=3/8"	D=7/16"	D=1/2"	D=5/8"	D=3/4"	D=7/8"	D=1"
0.58	6100	6500	5550	4950	4500	4200	3900	3500	3200	2950	2750
0.57	5900	6400	5400	4850	4400	4100	3800	3400	3100	2900	2700
0.56	5700	6250	5250	4700	4300	4000	3700	3350	3050	2800	2650
0.55	5550	6150	5150	4600	4200	3900	3650	3250	2950	2750	2550
0.54	5350	6050	5000	4450	4100	3750	3550	3150	2900	2650	2500
0.53	5150	5950	4850	4350	3950	3650	3450	3050	2800	2600	2450
0.52	5000	5800	4750	4250	3850	3550	3350	3000	2750	2550	2350
0.51	4800	5700	4600	4100	3750	3450	3250	2900	2650	2450	2300
0.50	4650	5600	4450	4000	3650	3400	3150	2800	2600	2400	2250

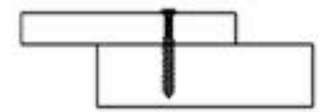
Dowel Type Fasteners: Shear Design

Shear (Lateral) capacity:

- NDS Chapter 11 Tables Provide Shear Capacity Tables

Table 11L WOOD SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]

for sawn lumber or SCL with both members of identical specific gravity (tabulated lateral design values are calculated based on an assumed length of wood screw penetration, p, into the main member equal to 10D)



Side Member Thickness	Wood Screw Diameter	Wood Screw Number	G=0.67 Red Oak	G=0.55 Mixed Maple Southern Pine	G=0.5 Douglas Fir-Larch	G=0.49 Douglas Fir-Larch(N)	G=0.46 Douglas Fir(S) Hem-Fir(N)	G=0.43 Hem-Fir	G=0.42 Spruce-Pine-Fir	G=0.37 Redwood (open grain)	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods	G=0.35 Northern Species
t_s in.	D in.		lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1/2	0.138	6	88	67	59	57	53	49	47	41	40	38
	0.151	7	96	74	65	63	59	54	52	45	44	42
	0.164	8	107	82	73	71	66	61	59	51	50	48
	0.177	9	121	94	83	81	76	70	68	59	58	56
	0.190	10	130	101	90	87	82	75	73	64	63	60
	0.216	12	156	123	110	107	100	93	91	79	78	75
	0.242	14	168	133	120	117	110	102	99	87	86	83
5/8	0.138	6	94	76	66	64	59	53	52	44	43	41
	0.151	7	104	83	72	70	64	58	56	48	47	45
	0.164	8	120	92	80	77	72	65	63	54	53	51
	0.177	9	136	103	91	88	81	74	72	62	61	58
	0.190	10	146	111	97	94	88	80	78	67	65	63
	0.216	12	173	133	117	114	106	97	95	82	80	77
	0.242	14	184	142	126	123	115	106	103	89	87	84

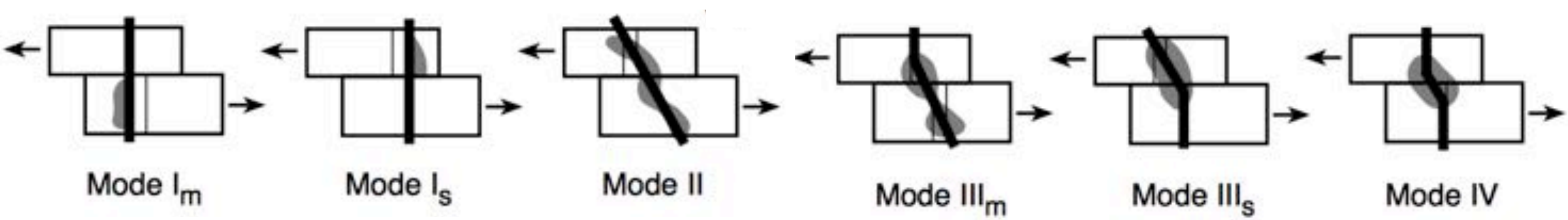
Dowel Type Fasteners: Shear Design

Shear (Lateral) capacity:

- Connections consist of one side member and one main member (single shear) or two side members and one main member (double shear)
- Shear capacity is calculated based on least of 6 failure modes (NDS 11.3.1)
- Side member(s), main member, or fastener can fail

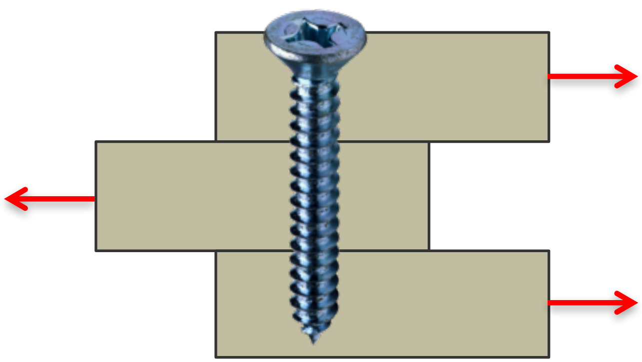
Single Shear Connection

Source: Wood Handbook, USDA Forest Service

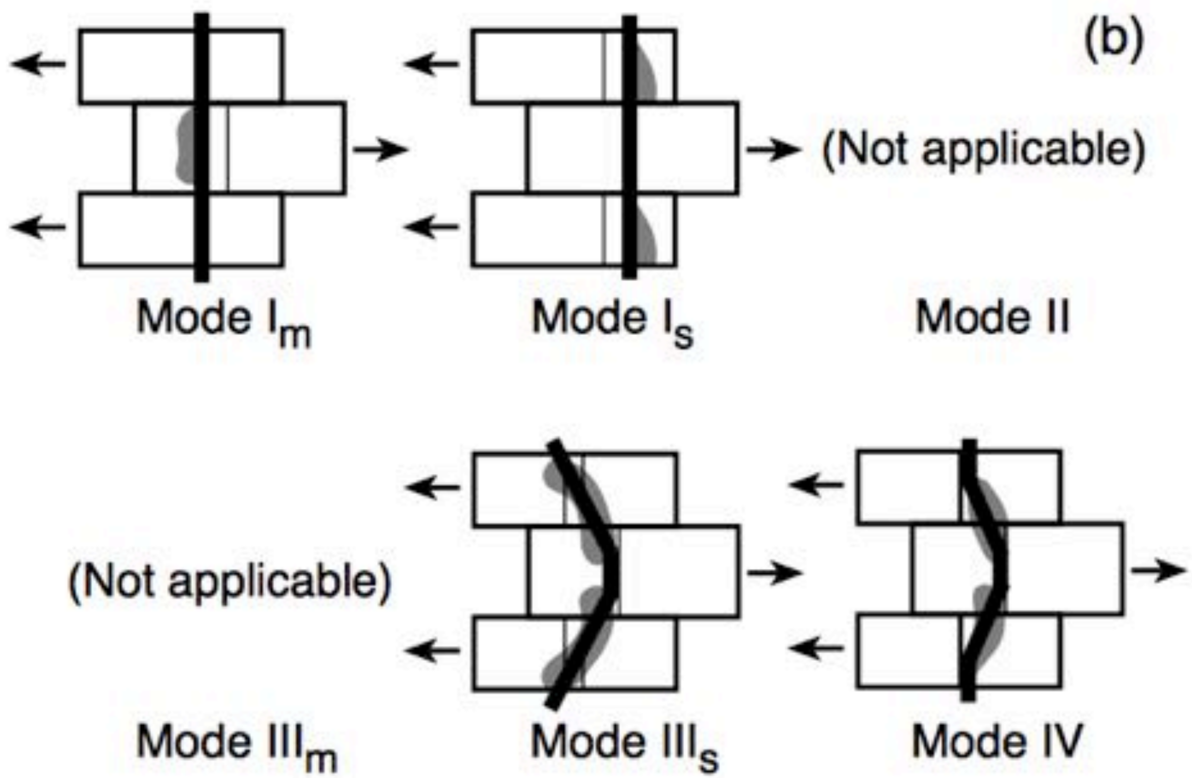


Dowel Type Fasteners: Shear Design

Double shear connection design:



Double Shear Connection

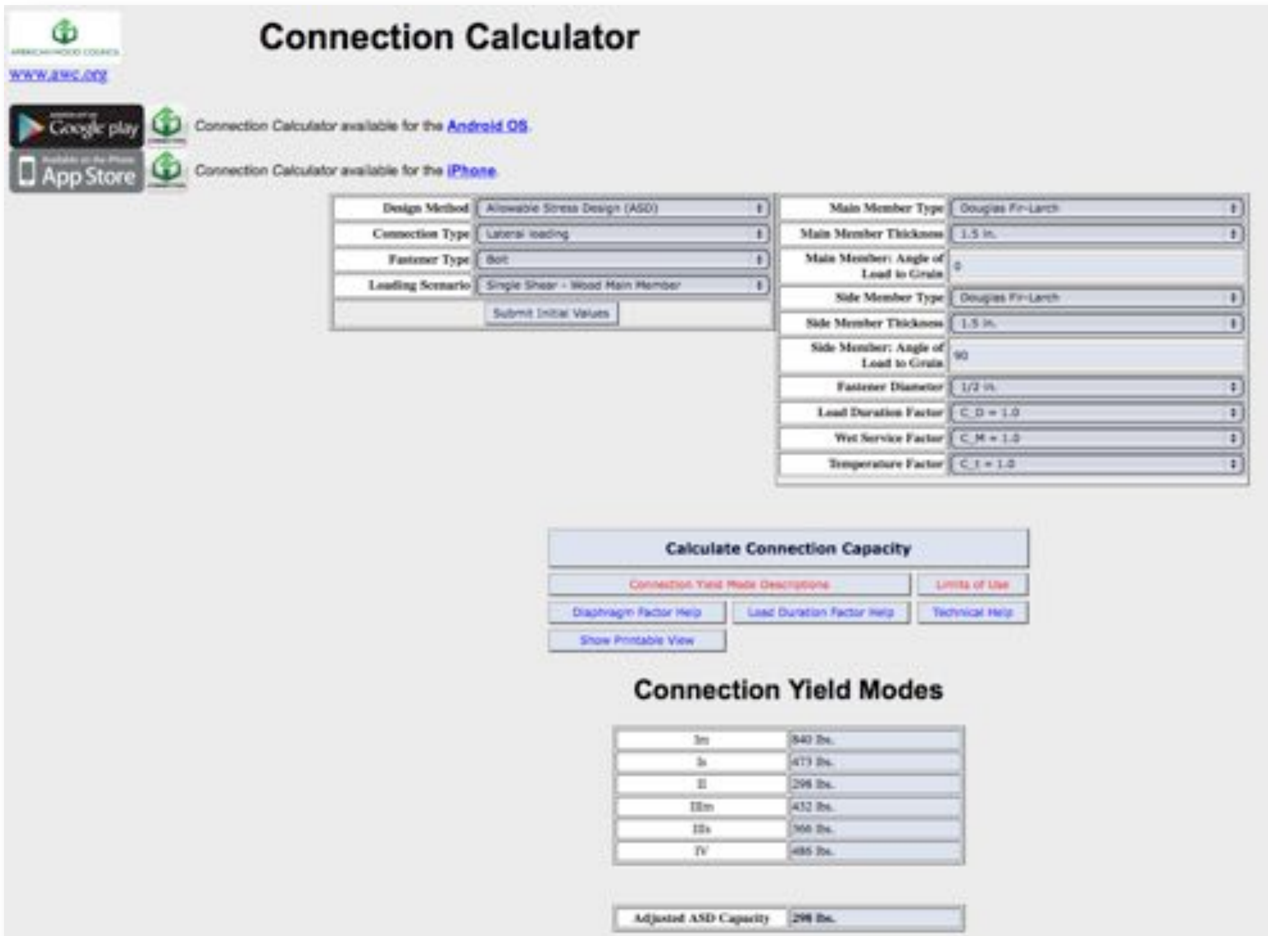


Source: Wood Handbook, USDA Forest Service

Connection Design Aids

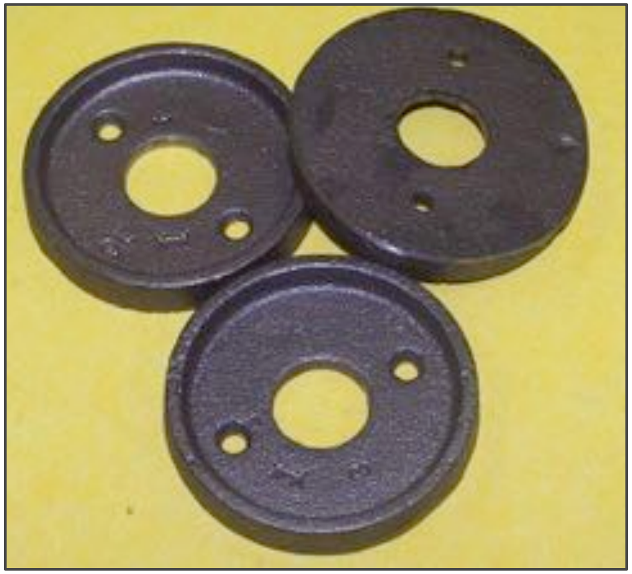
American Wood Council Connection Calculator for shear design of bolts, lag screws, wood screws, and nails:

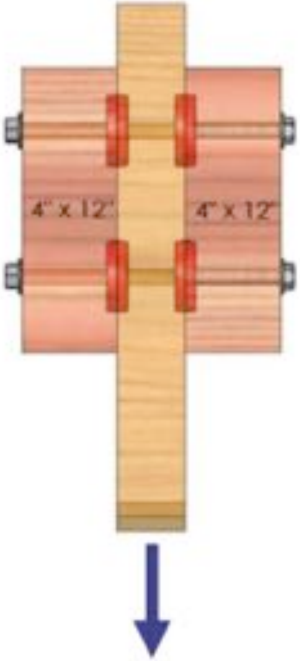
http://www.awc.org/calculators/connections/ccstyle.asp



Wood Connections: Split Rings & Shear Plates

- Act as large diameter bolts (bearing area)
- Split in ring allows for shrinkage
- Note-malleable iron washer for bolt to wood connection
- Split Ring: wood to wood; Shear Plate: wood to steel
- Commonly used in steel plated glulam trusses
- NDS Chapter 12 provides design equations & reference design values

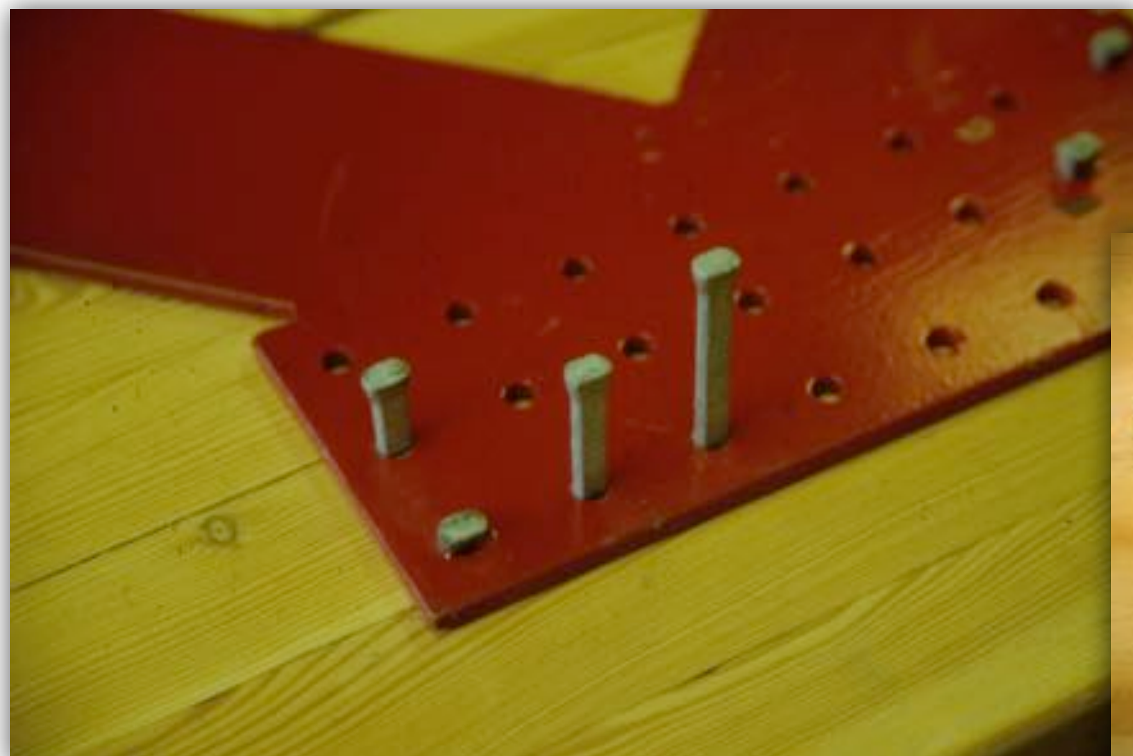

Shear plates

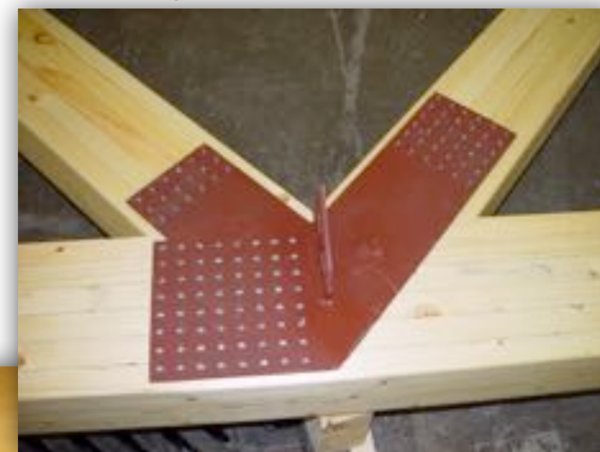



Split Ring

Wood Connections: Timber Rivets

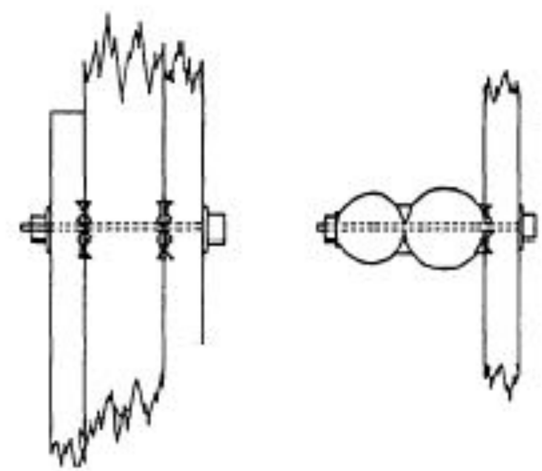
- Oval shaped nails with narrow side parallel to grain
- Allows closer spacing of rivets - reduces splitting
- NDS Chapter 13 limits use: only for steel side plates connecting to glulam members





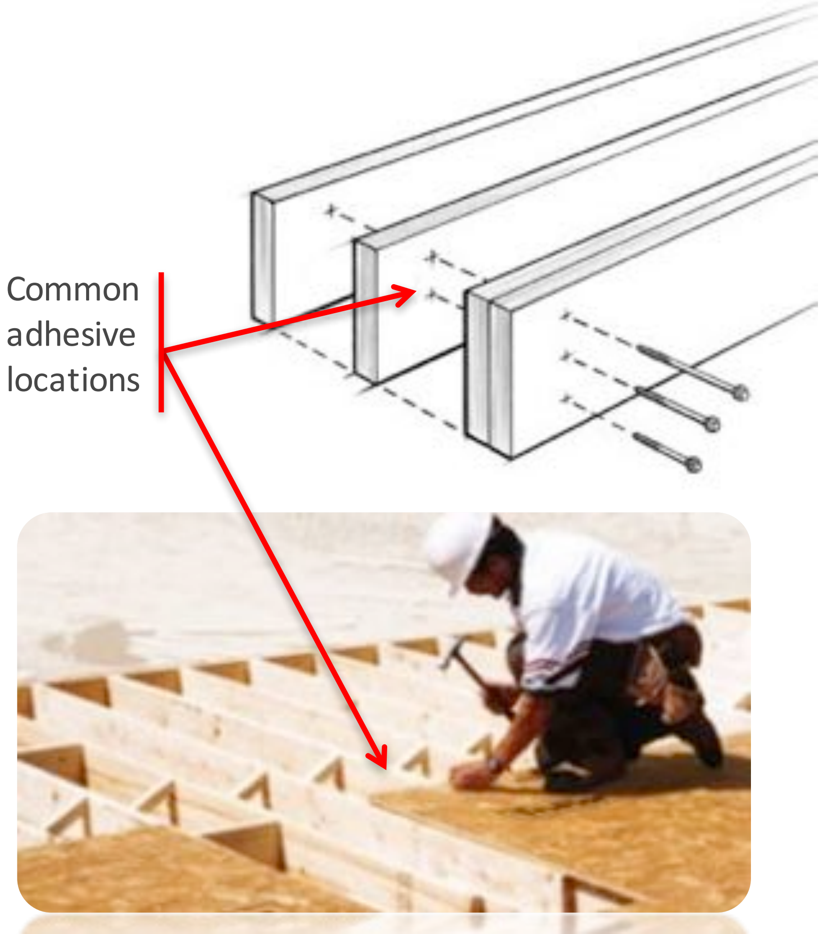
Wood Connections: Spike Grids

- Malleable iron grids with blunt teeth or spikes protruding outward from both sides
- Square or circular, flat or curved (one one side or both)
- Teeth ~1-1/4” long, in ~ 4” square grid pattern



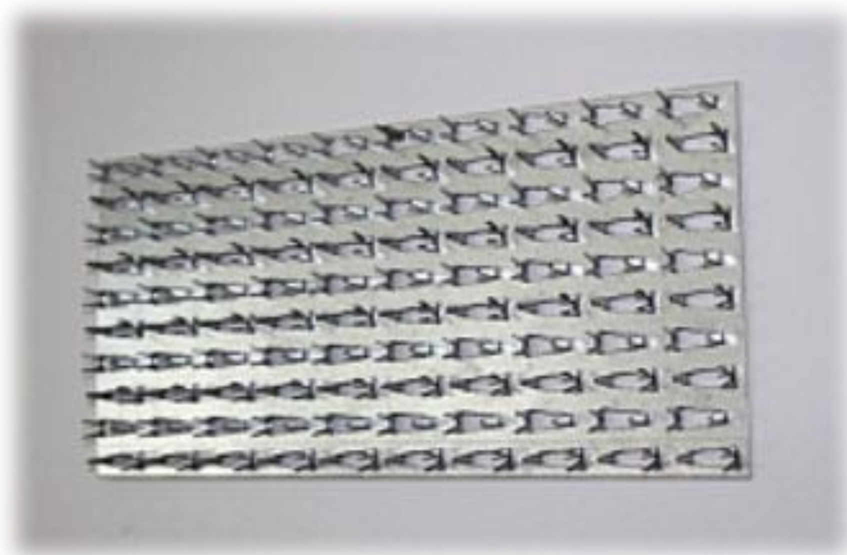
Wood Connections: Other

- Adhesives: typically not included in structural capacity
- Staples: no capacities in NDS, IBC provides shear wall & diaphragm capacities for staples



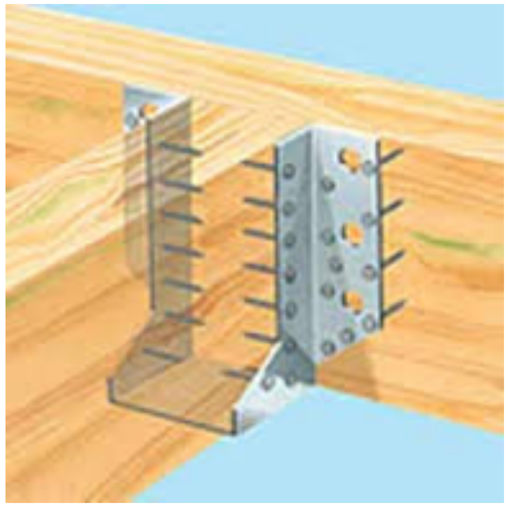
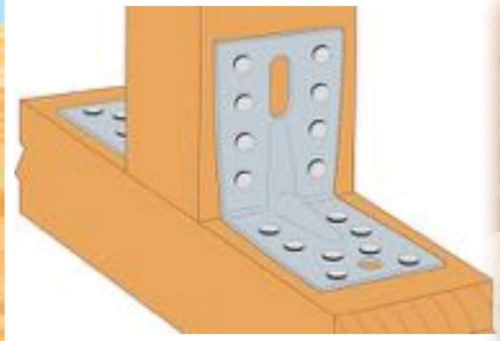
Wood Connections: Other

- Metal Plate Connectors: used in trusses, joining members in same plane
- Traditional timber pegs

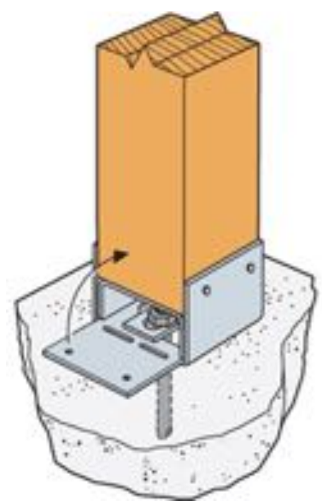
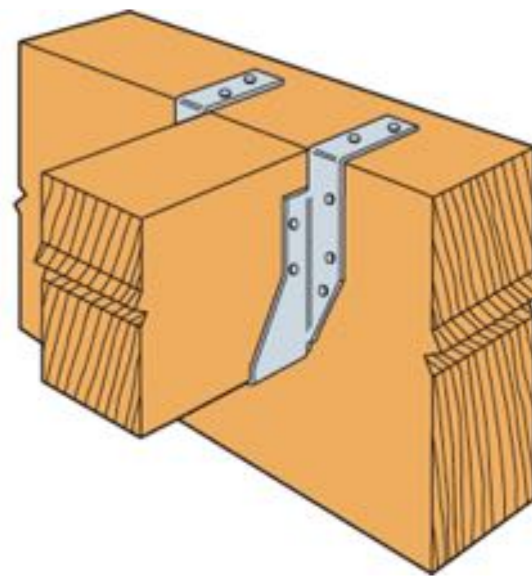


Wood Connections: Prefabricated Hardware

Manufacturers provide capacities based on testing & fastener limits

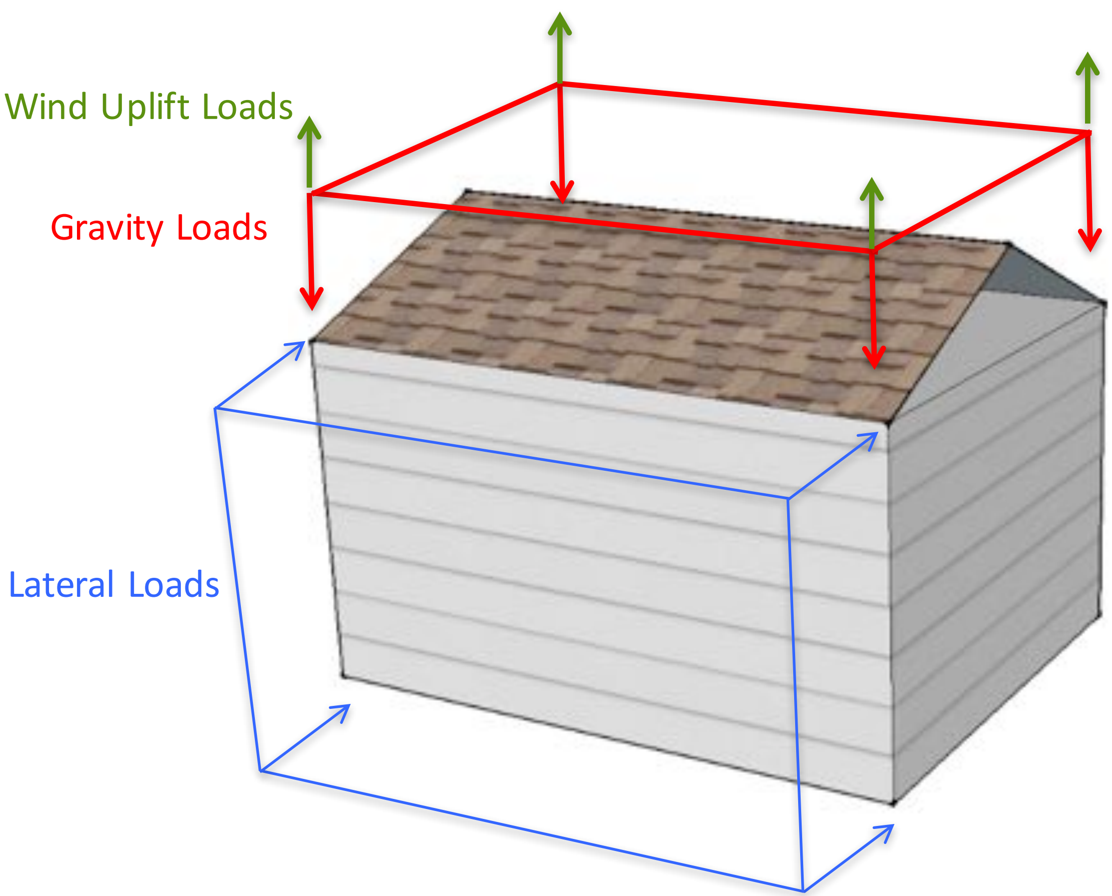


Structural Wood Design: Lateral Loads

For structural building design, two main loading directions exist: gravity (vertical) and lateral (horizontal)

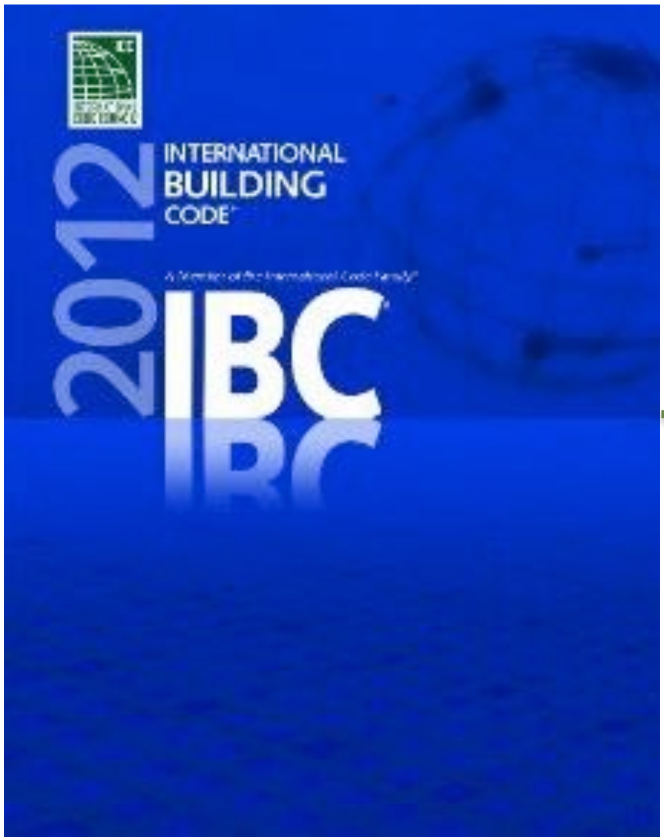
- This presentation will focus on structural wood design for lateral loads
- Lateral loads include wind and seismic



Outline

- Lateral Loads: Demand & Capacity
- WSP Systems: Diaphragms & Shear Walls
- WSP Shear Wall Components
- Other Wood Framed LFRS

Lateral Load Demand



IBC: Base Code – References ASCE 7 for determination of wind and seismic Forces

ASCE 7: Referenced Standard. Provides information required to determine wind and seismic forces on a structure

Lateral Load Capacity

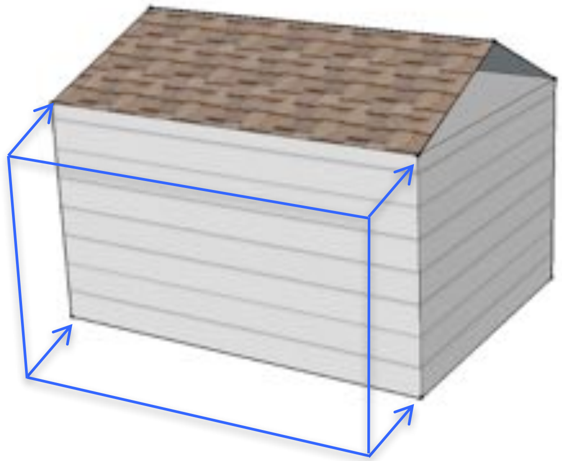
IBC: References Special Design Provisions for Wind & Seismic (SDPWS) for capacities of <u>most</u> wood framed lateral systems. IBC provides capacity of stapled WSP and gypsum shear walls

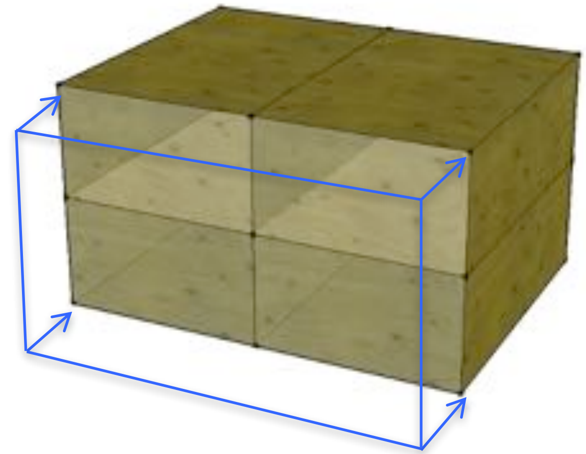
SDPWS: Provides capacities of most wood-framed vertical and horizontal lateral force resisting systems

Proprietary Products which have 3rd party testing reports may also be submitted

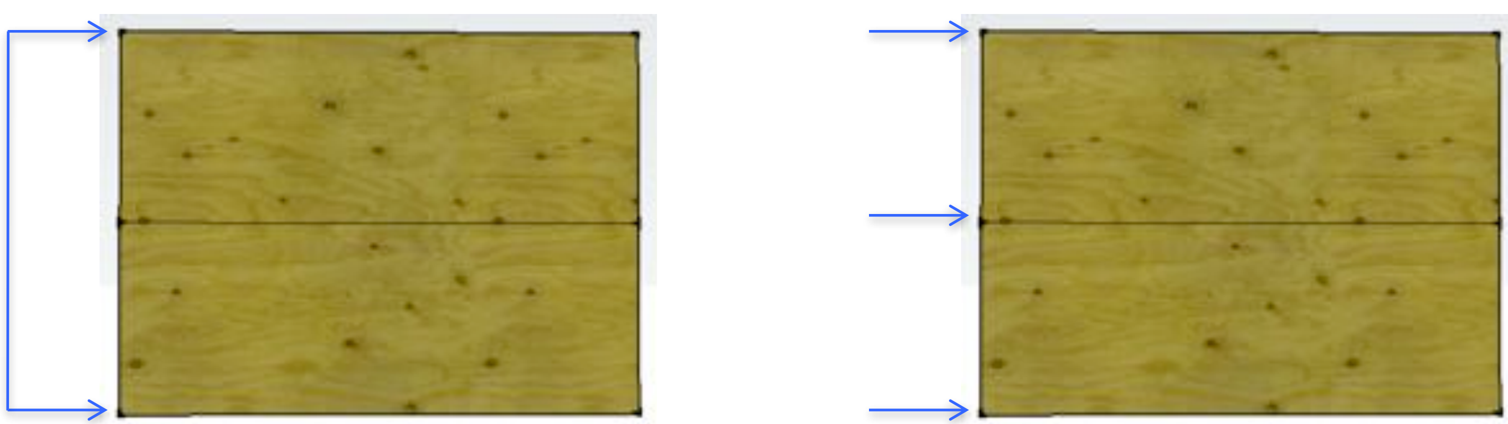
WSP Shear Wall & Diaphragm Design

- Lateral loads on a building are modeled as uniform surface loads





- Wall framing distributes surface loads to diaphragms (floors & roof)

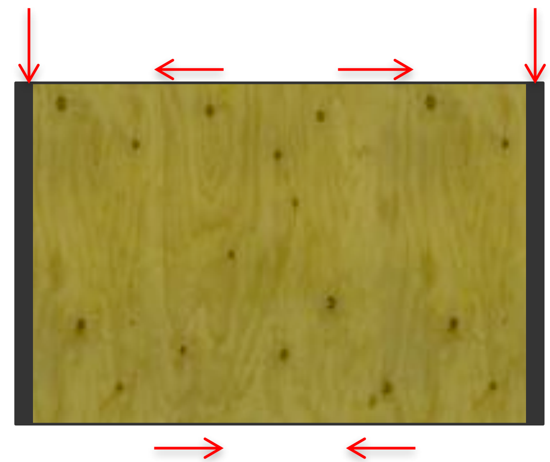


Elevation View

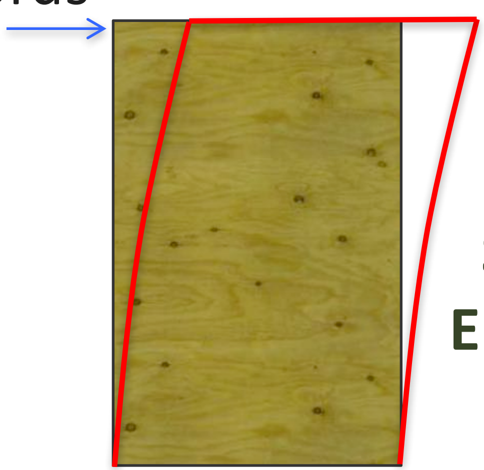
WSP Shear Wall & Diaphragm Design

- Uniform diaphragm loads are distributed to shear walls

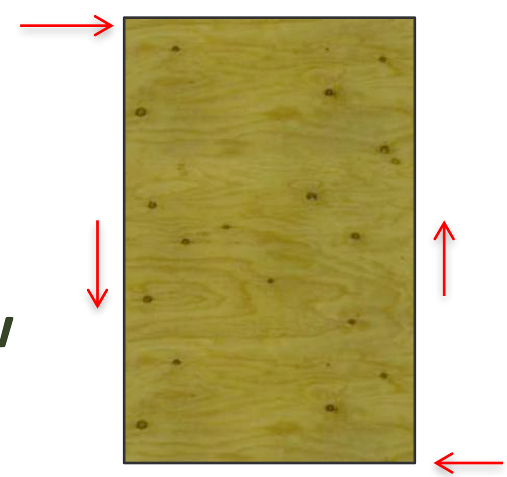
Diaphragm - Plan View



- Shear wall resists applied load through shear panel & boundary chords

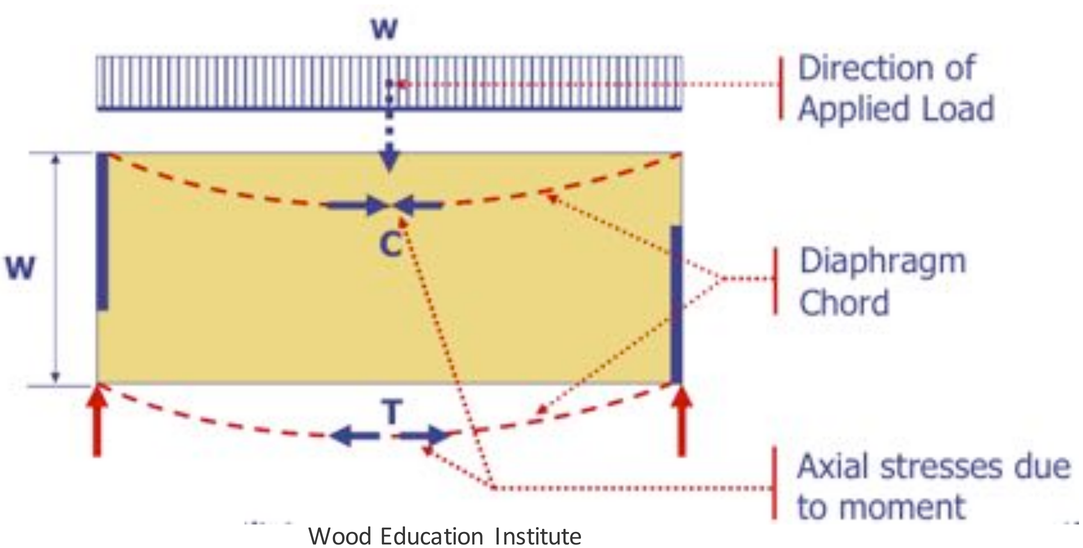


Shear Wall – Elevation View



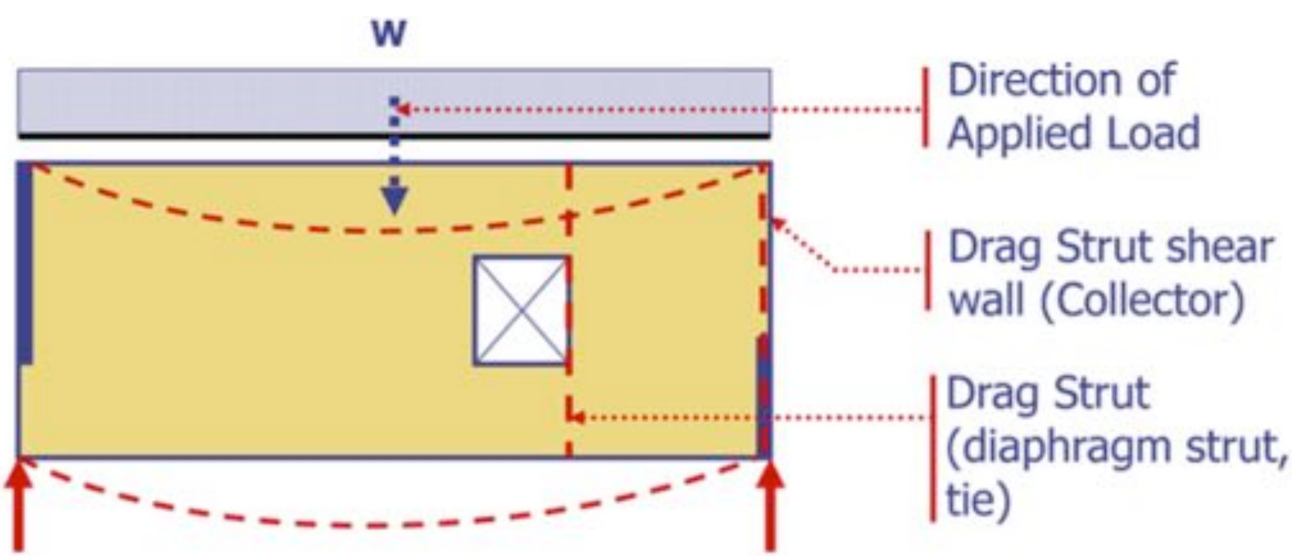
Diaphragm Design

- Diaphragm: Roof, floor, or other membrane or bracing system acting to transfer lateral forces to the vertical resisting elements
- Diaphragm loads are generally uniform loads, resisted by the diaphragm in bending, similar to a horizontal deep beam
- Diaphragm bending results in tension/compression in chords perpendicular to load



Diaphragm Design

- Reactions at diaphragm ends transfer load to shear wall through shear in panels
- Principles of shear resistance, panel attachment, boundary elements similar to shear wall design
- Drag struts carry or “drag” diaphragm loads into shear walls



Diaphragm Design: Capacity

- Capacities listed in AWC's Special Design Provisions for Wind and Seismic
- Sheathed diaphragm most common. Can also use horizontal and diagonal decking
- Unblocked diaphragms most common. Adding blocking at panel edges increases diaphragm capacity

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

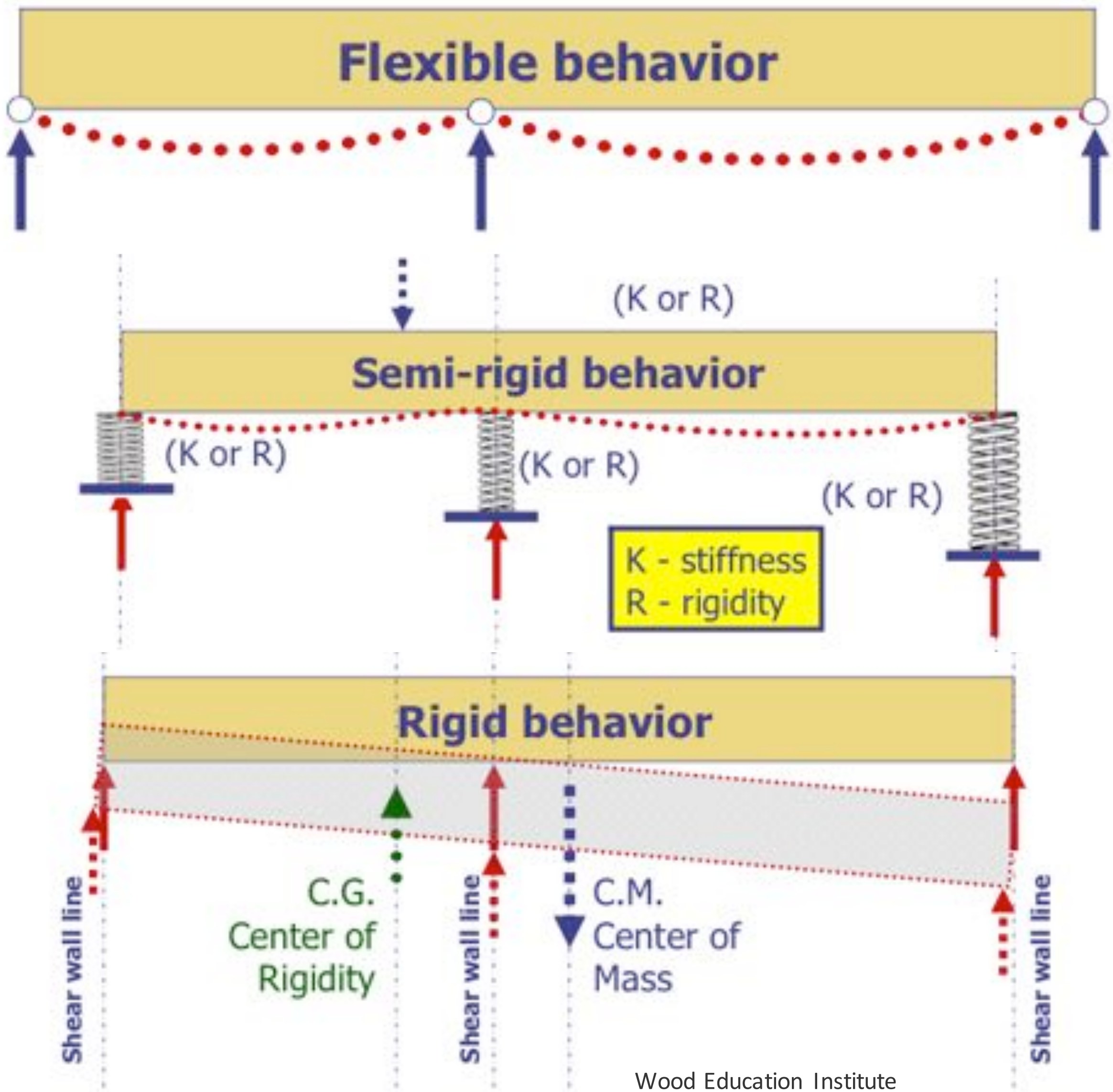
Blocked Wood Structural Panel Diaphragms[1,2,3,4]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	A SEISMIC												B WIND			
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)												Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
					6			4			2-1/2			2			6	4	2-1/2	2
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)												Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
					6			6			4			3			6	6	4	3
					v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
						OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15	520	700	1050	1175
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	590	785	1175	1330
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	755	1010	1485	1680
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295

Diaphragm Design: Flexibility

- Diaphragms can be idealized as flexible, semi-rigid, or rigid
- Light wood frame diaphragms traditionally idealized as flexible, a function of diaphragm construction and deflection
- Trends in mid-rise, multi-family buildings toward fewer exterior shear walls move into semi-rigid & rigid modeling
- ASCE 7 Section 12.3.1 Provides Diaphragm Flexibility definitions

Diaphragm Design: Flexibility



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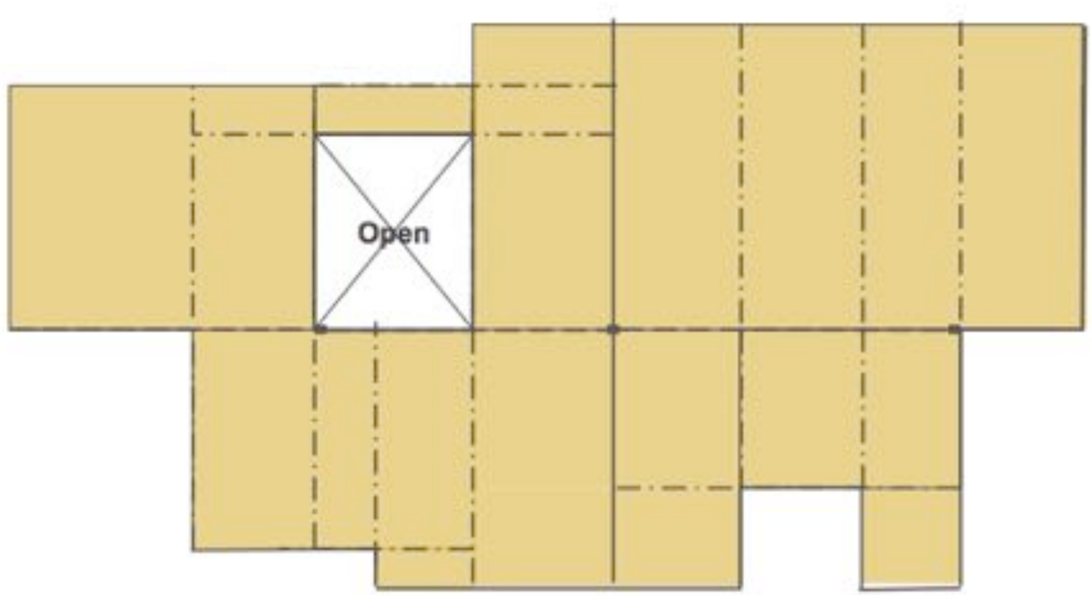
Additional Diaphragm Considerations

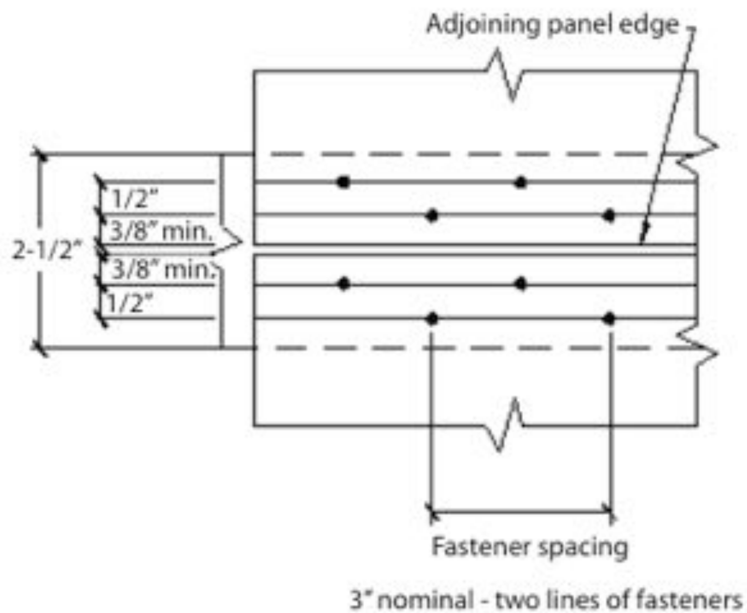
- Typical floor plan results in diaphragm offsets, re-entrant corners, discontinuities, openings
- Diaphragm openings, discontinuities = higher concentrated, localized loads
- Code requirements for diaphragm length to width ratios must be met



Additional Diaphragm Considerations

- Higher concentrated loads = closer panel edge fastener spacing, larger chord & strut loads, may require blocked diaphragm
- Diaphragm deflections may need to be calculated

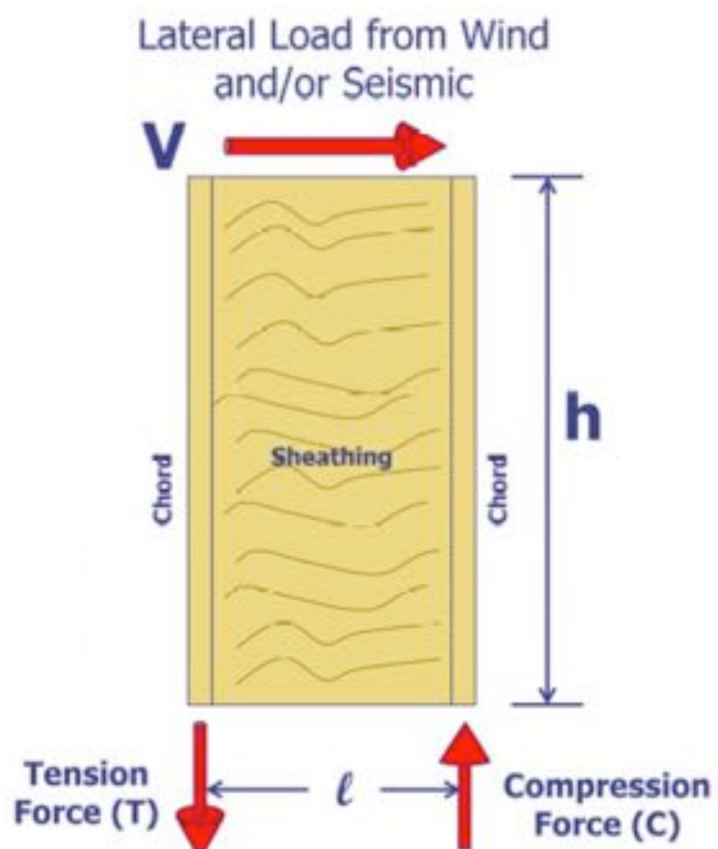
Figure 4C High Load Diaphragm



$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W}$$

WSP Shear Wall Design

- Shear wall: vertical components of a building's lateral force resisting system
- Shear wall transfers lateral loads from diaphragm above to wall/foundation below



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WSP Shear Wall Design

- Diaphragm transfers lateral concentrated load to top of wall
- Wall resists load through unit shear (panel) & resisting moment (end posts/hold downs)
- 3 shear wall design components:
  - Sheathing panel edge fasteners (unit shear)
  - Base of wall anchorage (unit shear)
  - End post & hold down sizes (resisting moment)

$$v = \frac{V}{l}$$

Unit shear

$$T = C = \frac{V * h}{l}$$

Resisting moment

WSP Shear Wall Design: Capacity

- Capacities listed in AWC's Special Design Provisions for Wind and Seismic (SDPWS)
- Blocked shear walls most common. SDPWS has reduction factors for unblocked shear walls
- Note that capacities are given as nominal: must be adjusted by a reduction or resistance factor to determine allowable unit shear capacity (ASD) or factored unit shear resistance (LRFD)

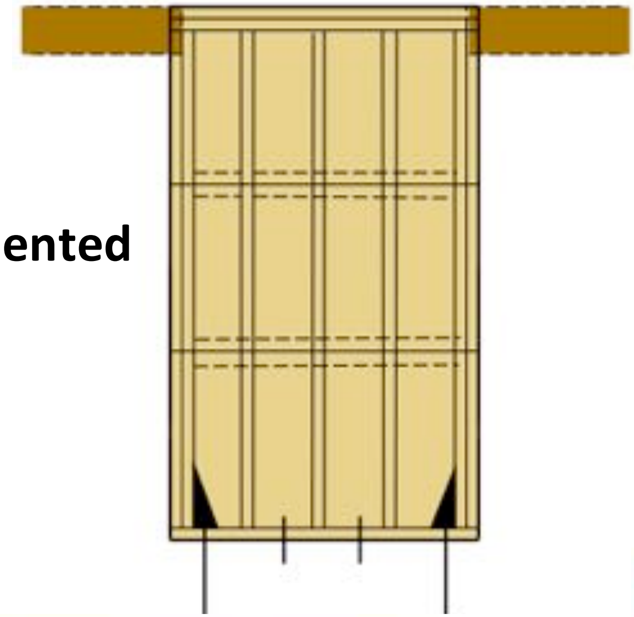
Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

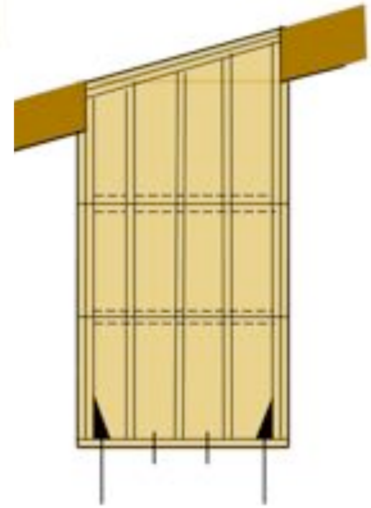
Wood-based Panels[4]

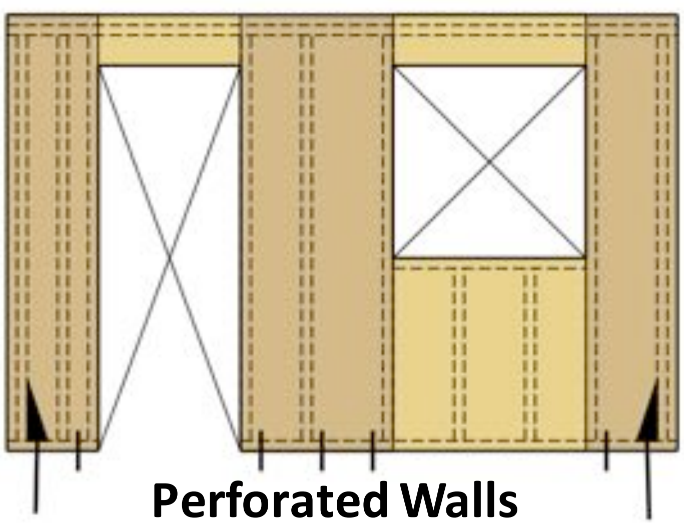
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC												B WIND			
				Panel Edge Fastener Spacing (in.)												Panel Edge Fastener Spacing (in.)			
				6			4			3			2			6	4	3	2
				v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
			Nail (common or galvanized box)		OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Wood Structural Panels - Structural I[4,5]	5/16	1-1/4	6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260

Engineered Shear Wall Systems w/ WSP

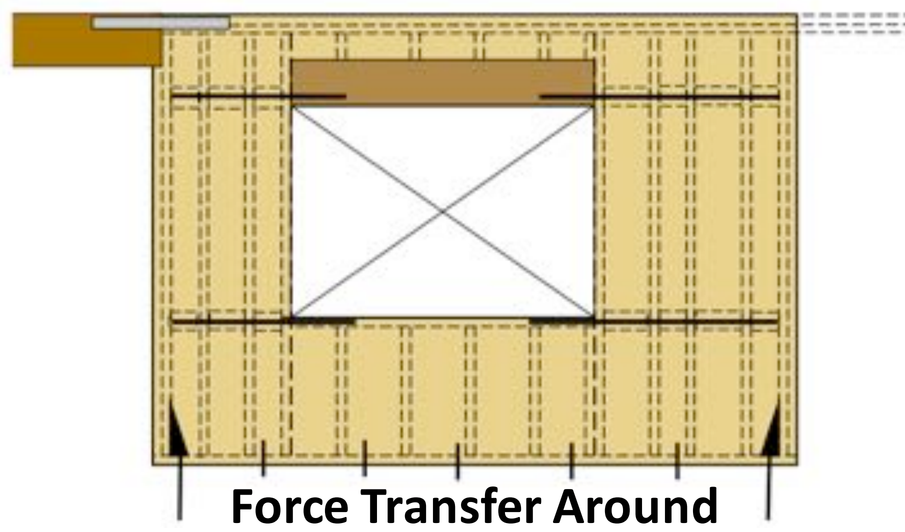
Solid or Segmented Walls







Perforated Walls



Force Transfer Around Openings Walls

Shear Walls: Additional Considerations

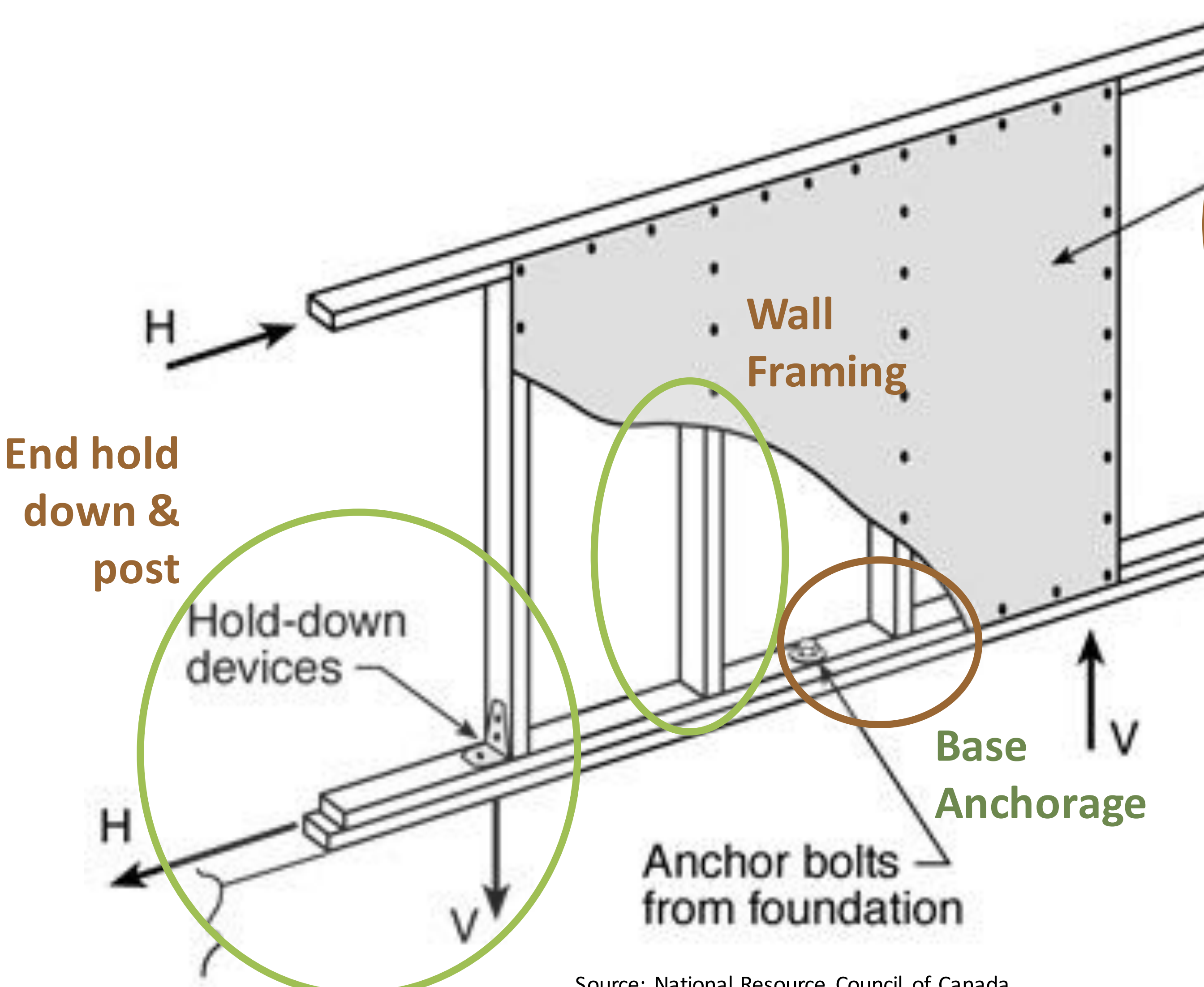
- Aspect Ratio: Height to Width Limitations (SDPWS)
- Some systems may only be used in lower Seismic Design Categories (SDPWS)
- Strength limitations with some systems (horizontal board sheathing, gypsum)
- Shear Wall deflection should be considered

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

Table 4.3.4 Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type	Maximum h/b_s Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1[1]
Particleboard, blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum wallboard	2:1[2]
Portland cement plaster	2:1[2]
Structural Fiberboard	3.5:1[3]

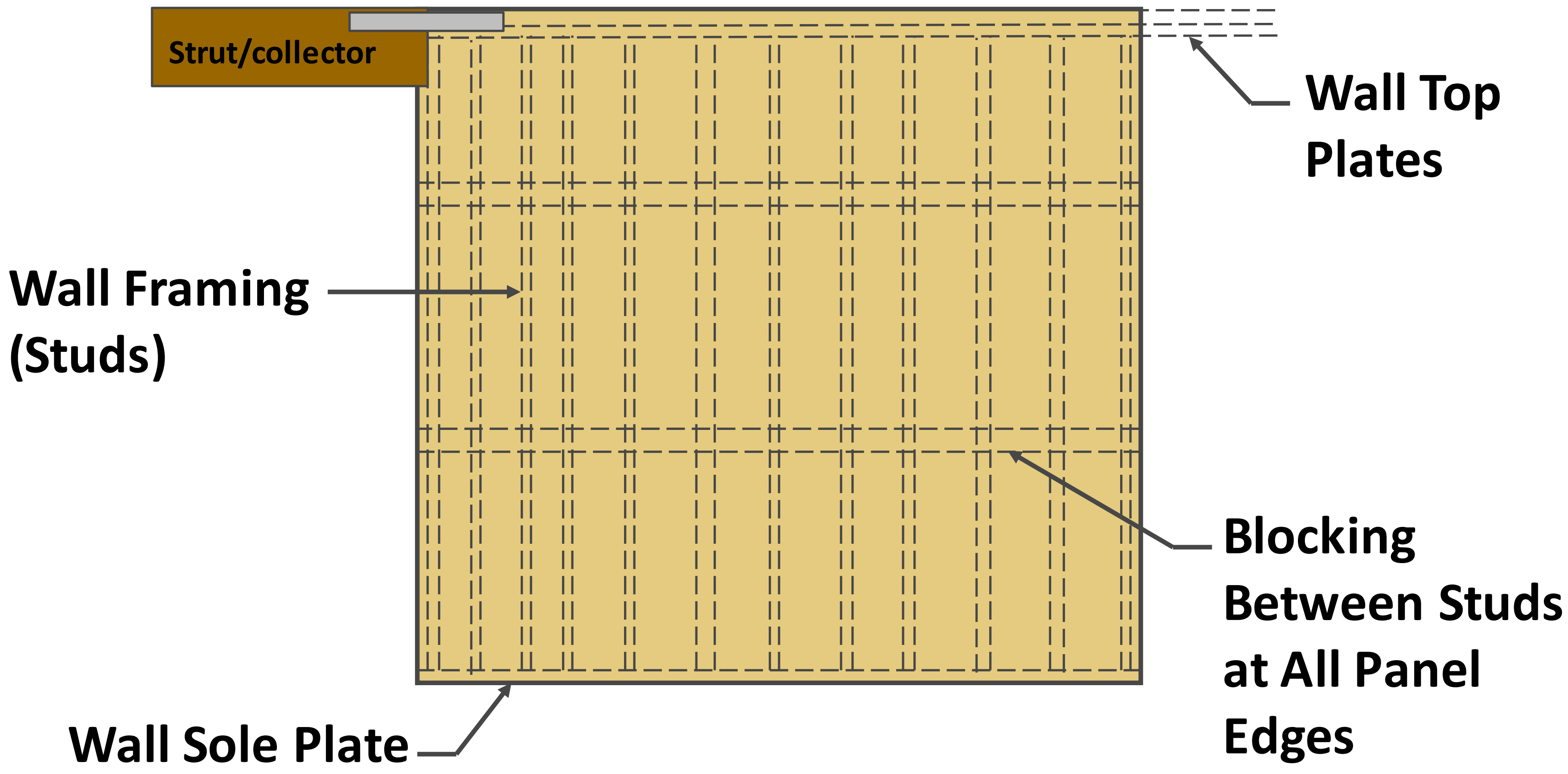
WSP Shear Wall Components



Source: National Resource Council of Canada

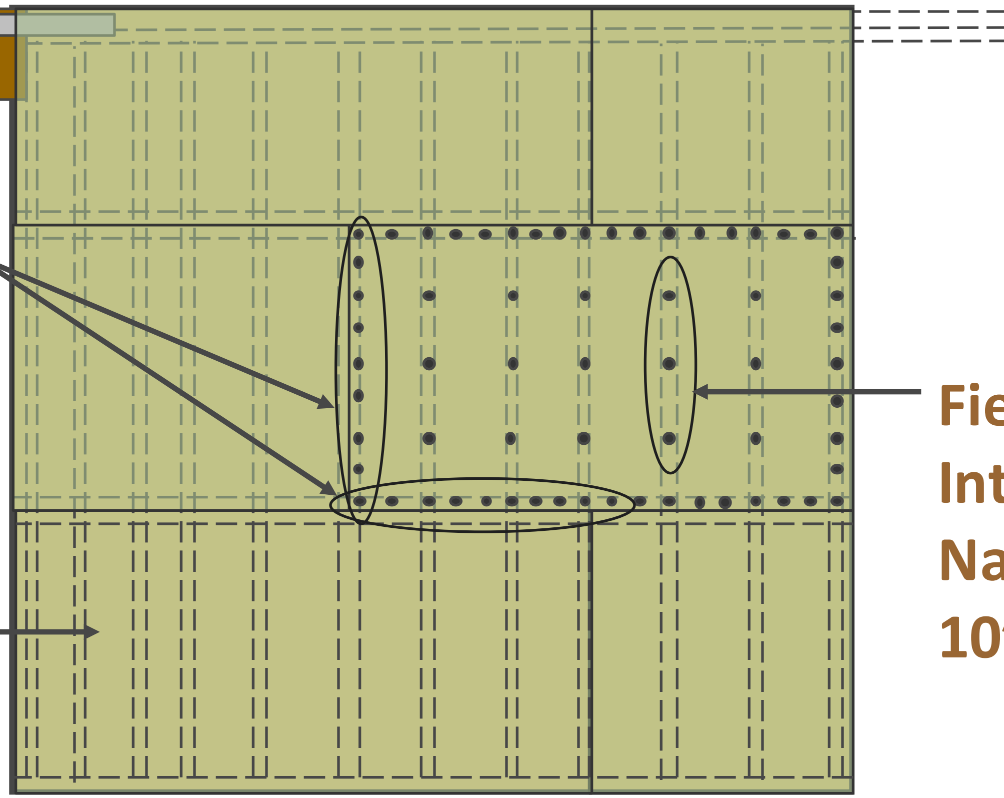
Source: Norbord

Shear Wall Components: Wall Framing



Note: Can use "un-blocked" wall but capacities can be significantly lower: SDPWS 4.3.3

Shear Wall Components: WSP & Fasteners



Panel Edge Nailing – Typ. 2" – 6" o.c.

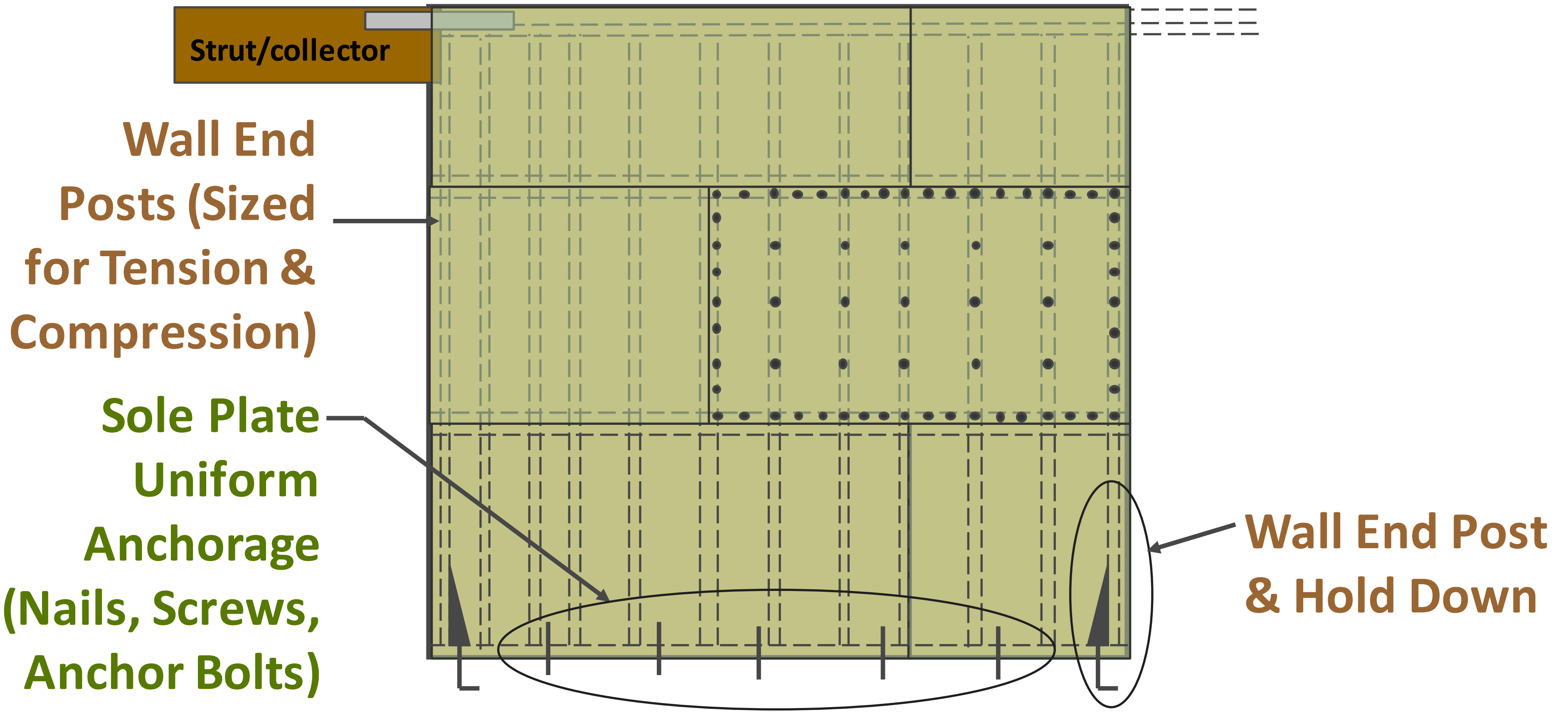
Field or Intermediate Nailing – Typ. 10" – 12" o.c.

Sheathing Panels OSB or Plywood

Field or Intermediate Nailing: Attaches panel to intermediate wall framing (studs) not along panel edges

Boundary Nailing: Attaches all 4 edges of every panel to wall framing (studs, blocking, top & sole plates)

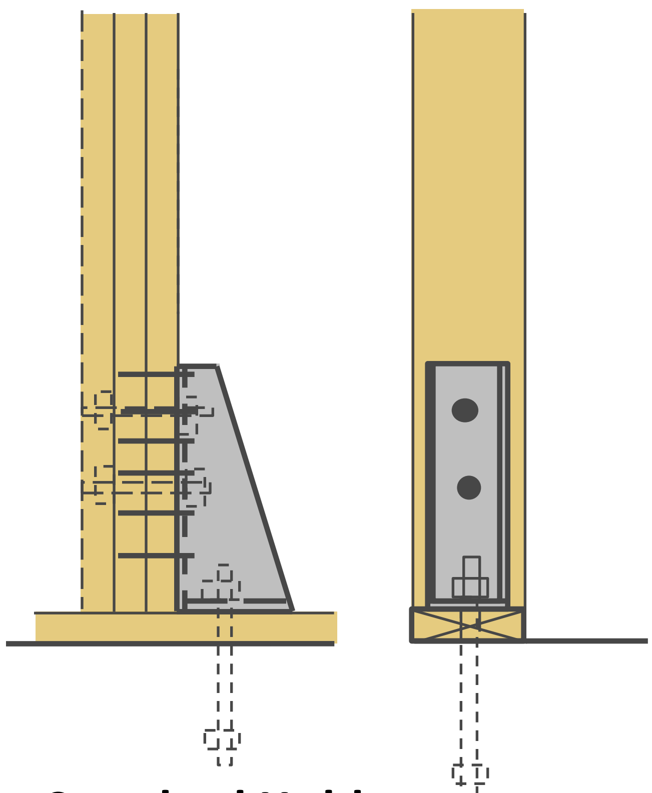
Shear Wall Components: Base Anchorage, End Posts & Hold Downs



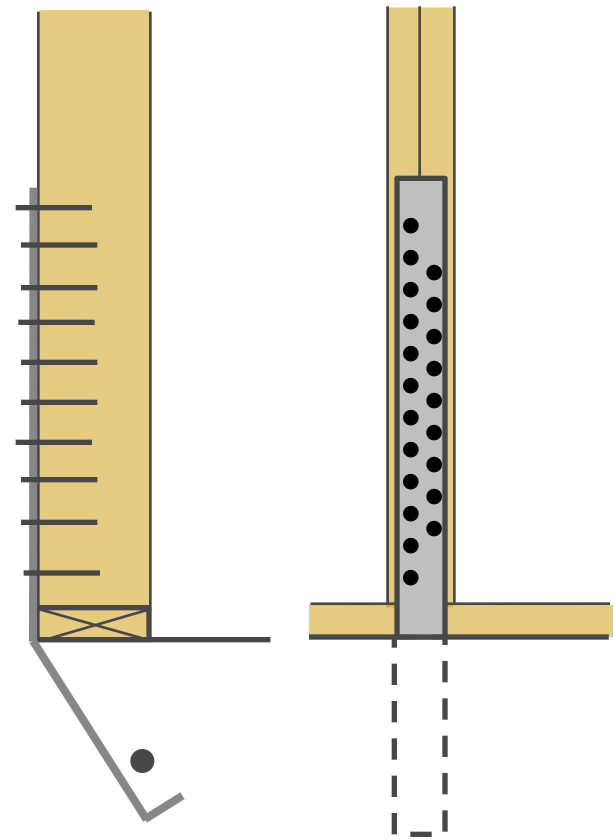
Sole Plate Uniform Anchorage: Transfers shear from wall sole plate to floor/wall or foundation below.

Wall End Post & Hold Down: Transfers vertical tension & compression forces to floor/wa foundation below.

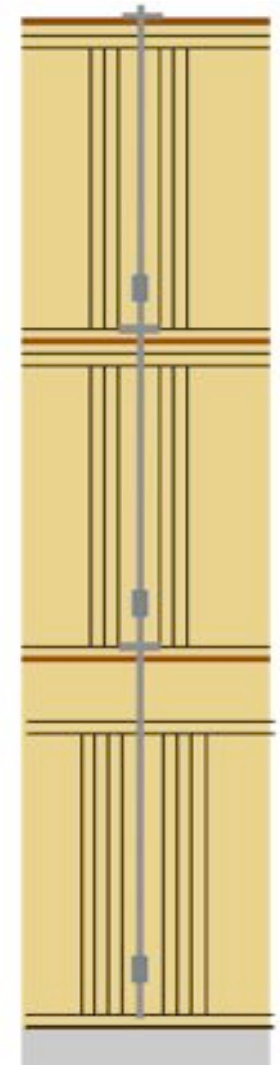
Shear Wall Components: Hold Downs



Standard Hold Down Installation (Bucket Style)

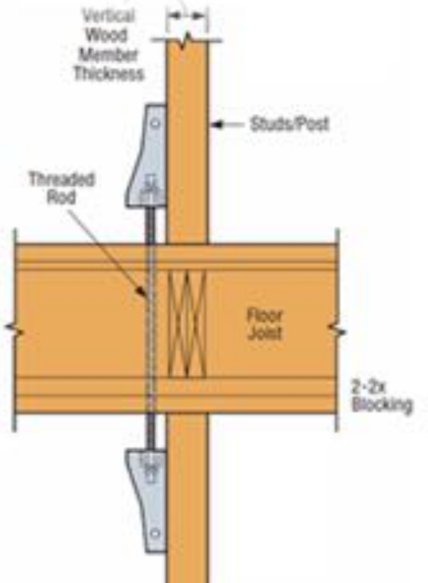


Strap Hold Down Installation



Continuous Rod - Automatic Tensioning Systems

Shear Wall Components: Hold Downs

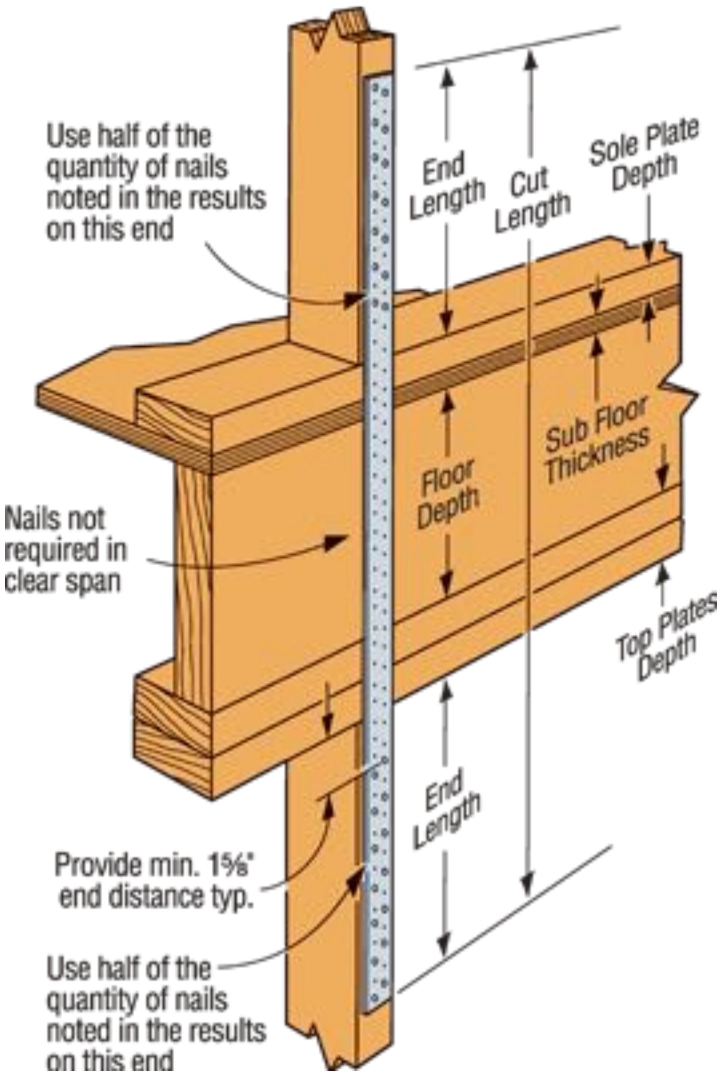


Source: strongtie.com

Bucket Style

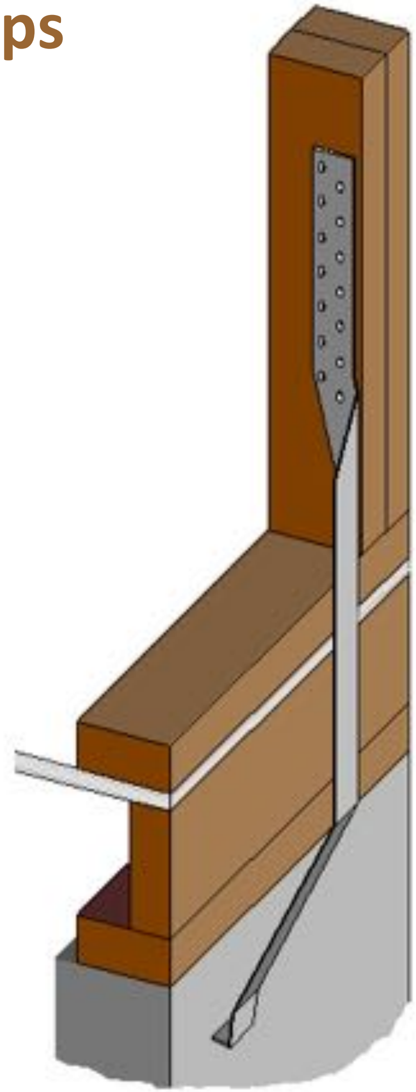
Source: DartDesignInc.com

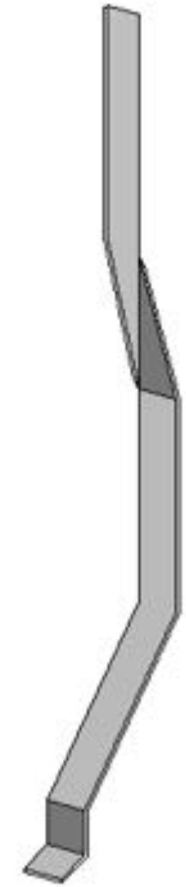
Shear Wall Components: Hold Downs



Source: strongtie.com

Straps





Shear Wall Components: Hold Downs

Continuous Rod Tie Downs with Shrinkage Compensation Devices

Prescriptive Shear Wall Systems – IBC 2308

IBC 2308.3,9,11,12: Prescriptive Braced Wall Lines

- Provides Braced Wall Spacings, Components, Fasteners
- Limitations:
  - Building Height:
    - 3 Stories max for SDC A and B
    - 2 Stories max for SDC C
    - 1 Story max for SDC D and E
    - Floor to Floor max = 11’-7”
    - Bearing Wall Stud Length max = 10’-0”
  - Loads
    - Max DL = 15 psf, Max LL = 40 psf
    - Max Ground Snow = 50 psf, Max Wind V_{asd} = 100 mph
  - Others: see IBC 2308.2

Prescriptive Shear Wall Systems – IBC 2308

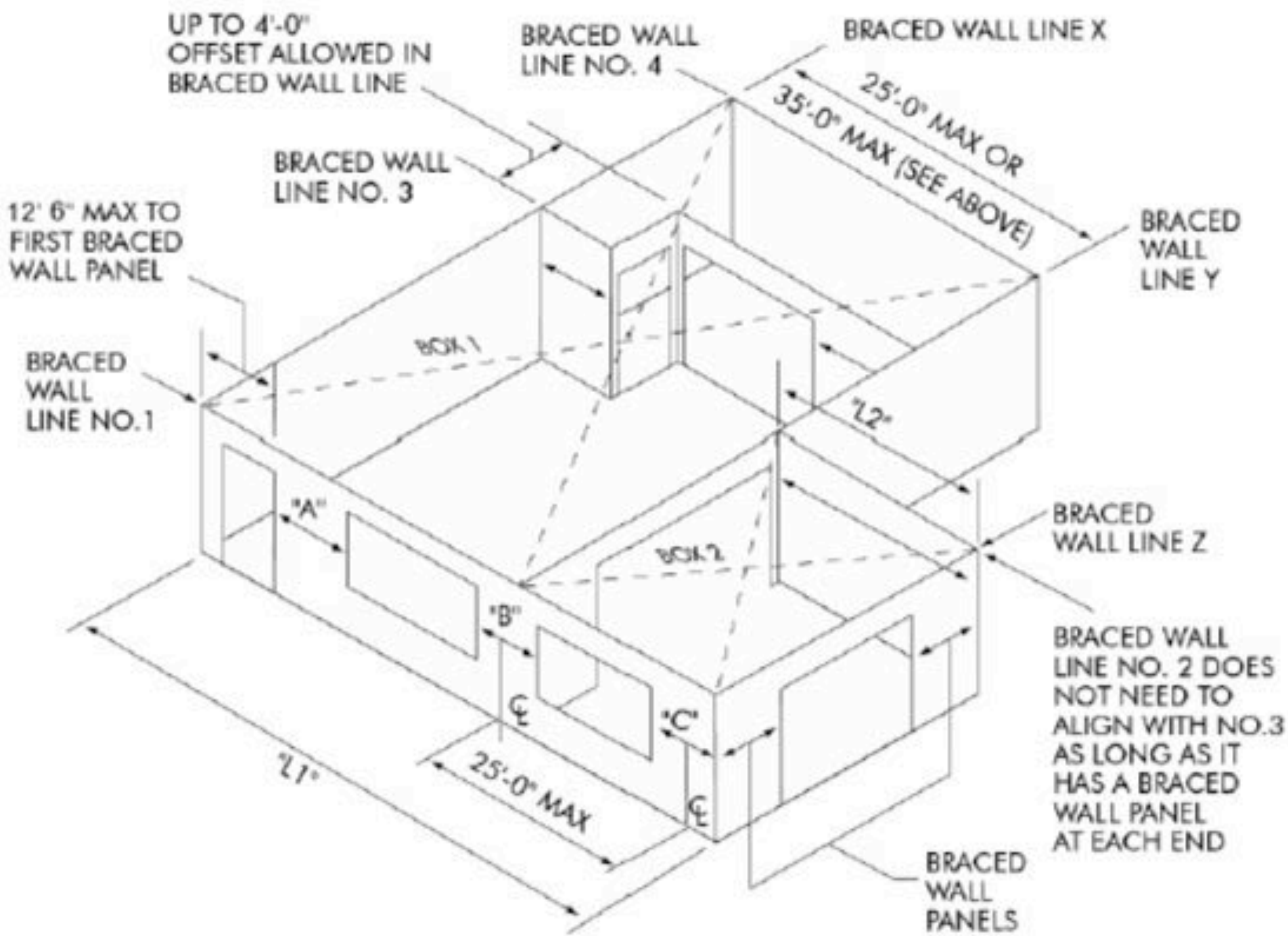
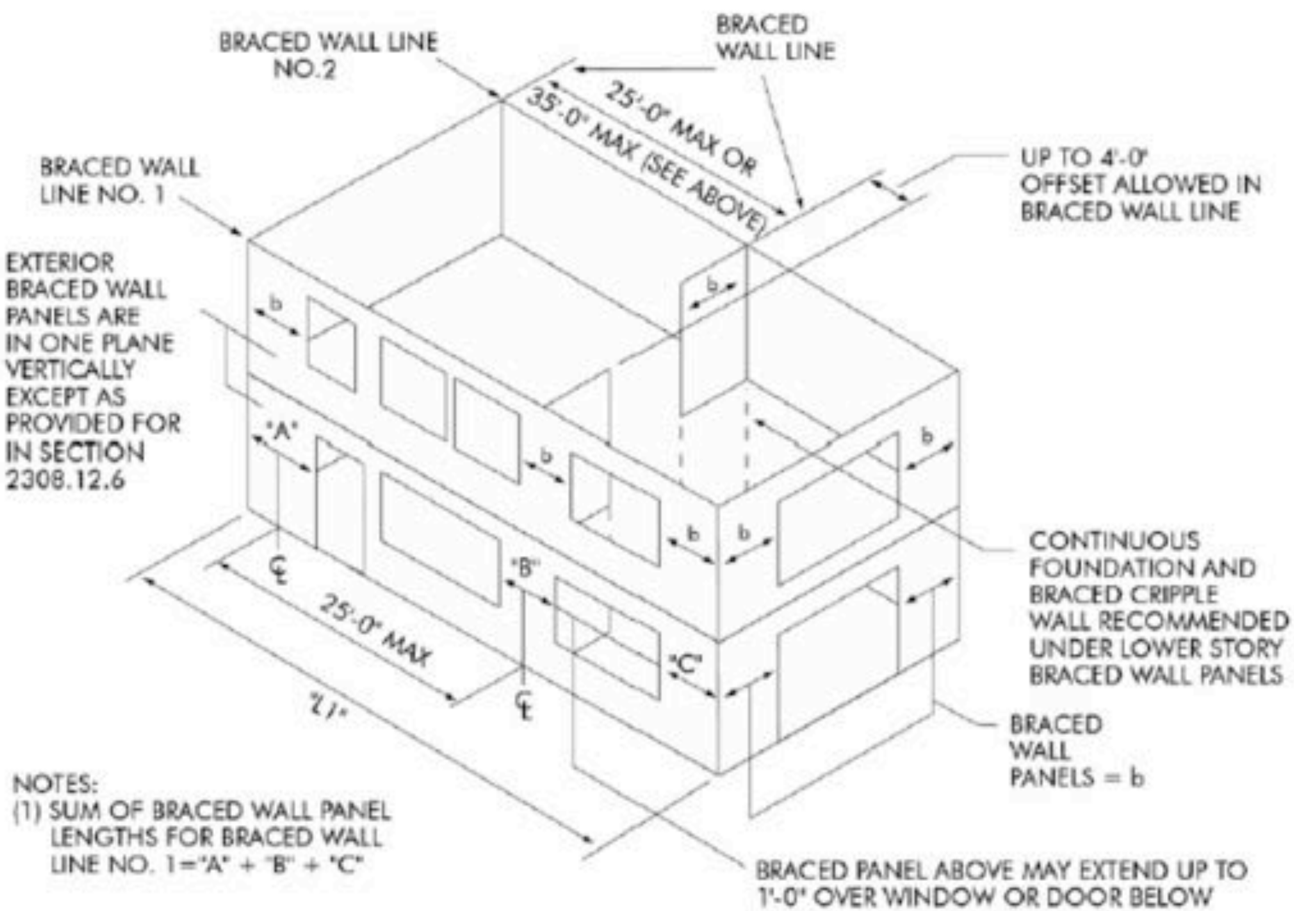


FIGURE 2308.9.3 BASIC COMPONENTS OF THE LATERAL BRACING SYSTEM

Prescriptive Shear Wall Systems – IBC 2308



For SI: 1 foot = 304.8 mm.

FIGURE 2308.9.3 BASIC COMPONENTS OF THE LATERAL BRACING SYSTEM

Prescriptive Shear Wall Systems – IBC 2308

SEISMIC DESIGN CATEGORY	MAXIMUM WALL SPACING (feet)	REQUIRED BRACING LENGTH, b
A, B and C	35′ -0″	Table 2308.9.3(1) and Section 2308.9.3
D and E	25′ -0″	Table 2308.12.4

TABLE 2308.9.3(1) BRACED WALL PANELS[a]

SEISMIC DESIGN CATEGORY	CONDITION	CONSTRUCTION METHODS[b, c]								BRACED PANEL LOCATION AND LENGTH[d]
		1	2	3	4	5	6	7	8	
A and B	One story, top of two or three story	X	X	X	X	X	X	X	X	Located in accordance with Section 2308.9.3 and not more than 25 feet on center.
	First story of two story or second story of three story	X	X	X	X	X	X	X	X	
	First story of three story	—	X	X	X	X[e]	X	X	X	
C	One story or top of two story	—	X	X	X	X	X	X	X	Located in accordance with Section 2308.9.3 and not more than 25 feet on center.
	First story of two story	—	X	X	X	X[e]	X	X	X	Located in accordance with Section 2308.9.3 and not more than 25 feet on center, but total length shall not be less than 25% of building length[f].

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

a. This table specifies minimum requirements for braced panels that form interior or exterior braced wall lines.

b. See Section 2308.9.3 for full description.

c. See Sections 2308.9.3.1 and 2308.9.3.2 for alternative braced panel requirements.

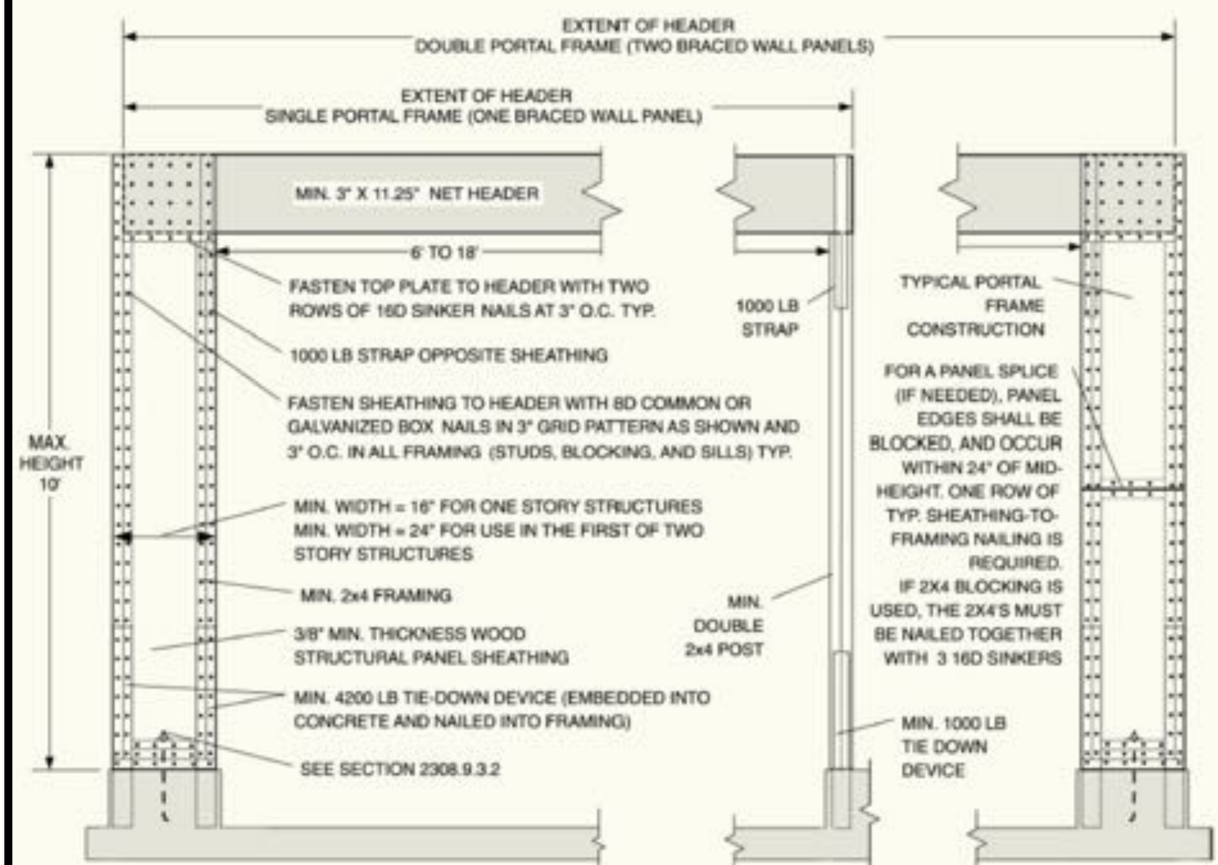
d. Building length is the dimension parallel to the braced wall length.

e. Gypsum wallboard applied to framing supports that are spaced at 16 inches on center.

f. The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.

Prescriptive Portal Frame Systems

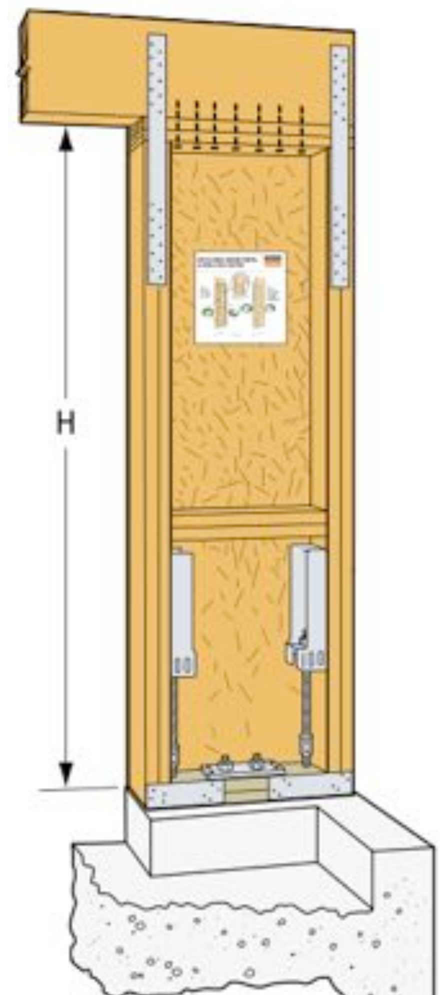
Prescriptive Code Portal Frames IBC 2308.9.3.2



For SI: 1 foot = 304.8 mm; 1 inch = 25.4 mm; 1 pound = 4.448 N.

FIGURE 2308.9.3.2 ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

Proprietary Portal Frames



Source: strongtie.com

Engineered Shear Wall Systems w/ WSP

Stapled Shear Walls

- Capacities in IBC 2306

Mid-Ply Shear Walls

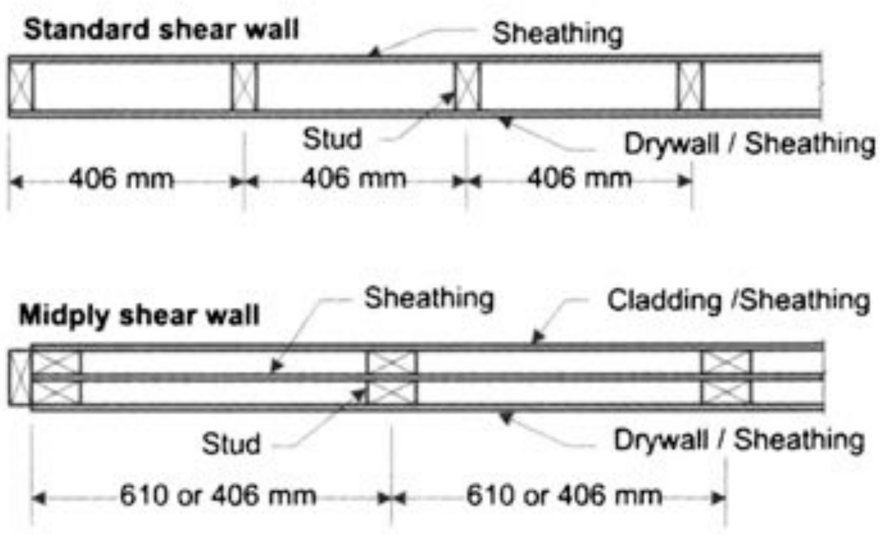


Fig. 1. Cross section of typical standard shear wall and midply wall

Source: Journal of Structural Engineering, 2007

Source: nees.org

Non-WSP Engineered Shear Wall Systems

Gypsum Shear Walls

Proprietary Trussed Shear

Non-WSP Engineered Shear Wall Systems

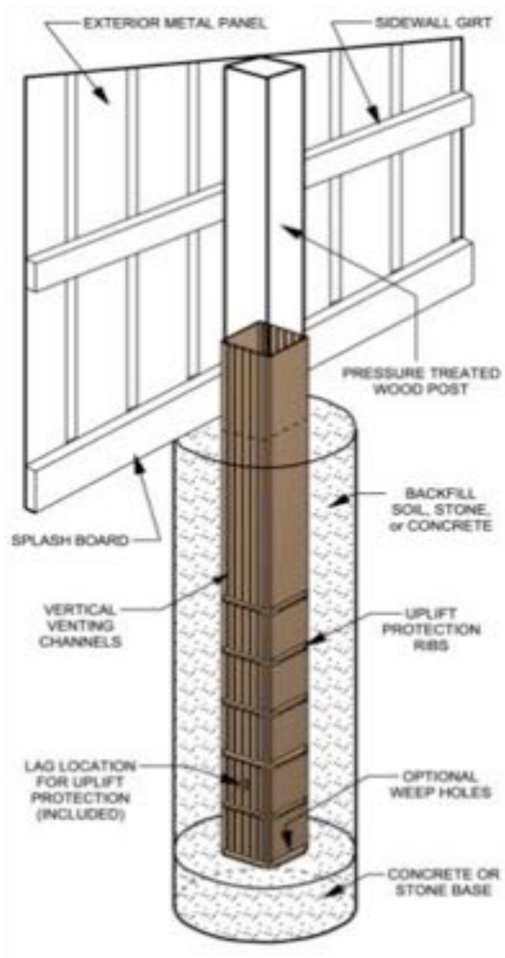
Horizontal & Diagonal Board Sheathing


Source: firstdayonpei

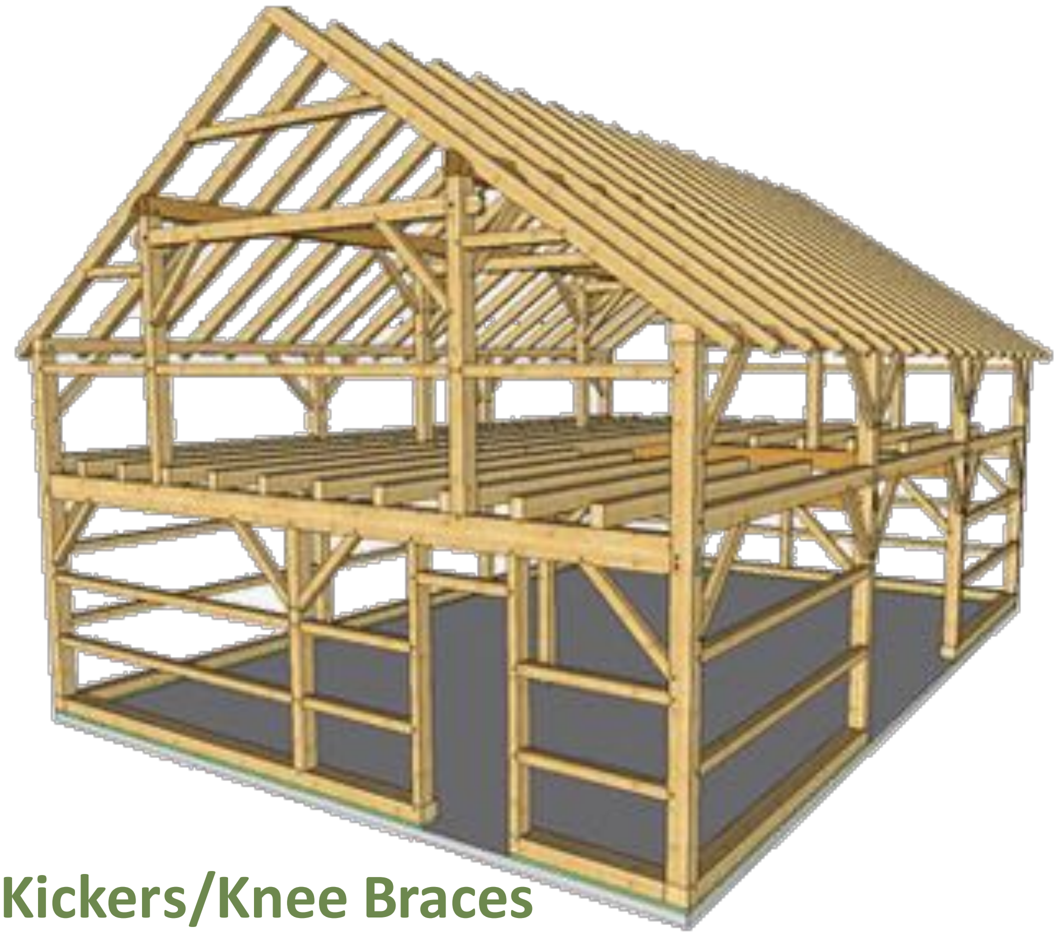

Source: johnotvos

Capacities in AWC's SDPWS Table 4.3D

Post Frame Buildings – Lateral Options



Embedded/Cantilever Columns



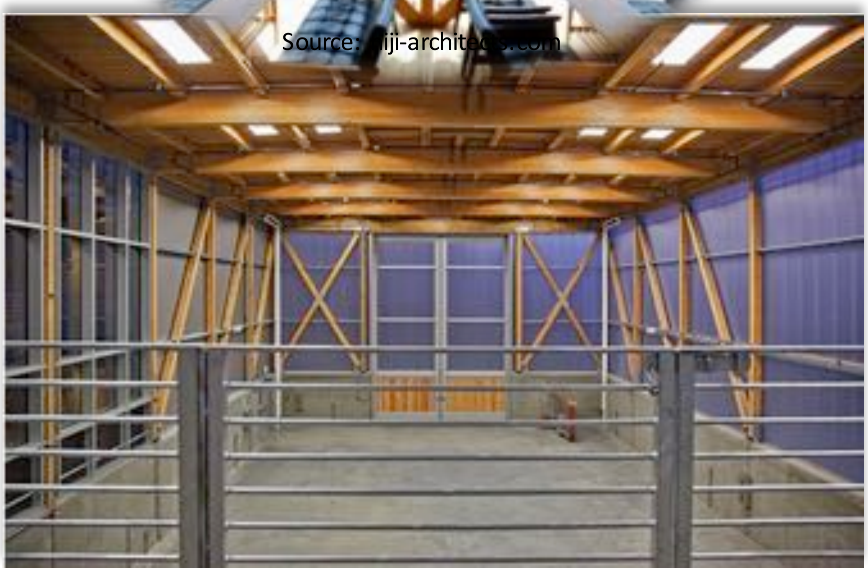
Source: newenglandbarn.com

- Kickers/Knee Braces
- Sheathed Walls/Roof
- Steel Rod X-Bracing
- Others

Heavy Timber Braced Frames (HTBF)

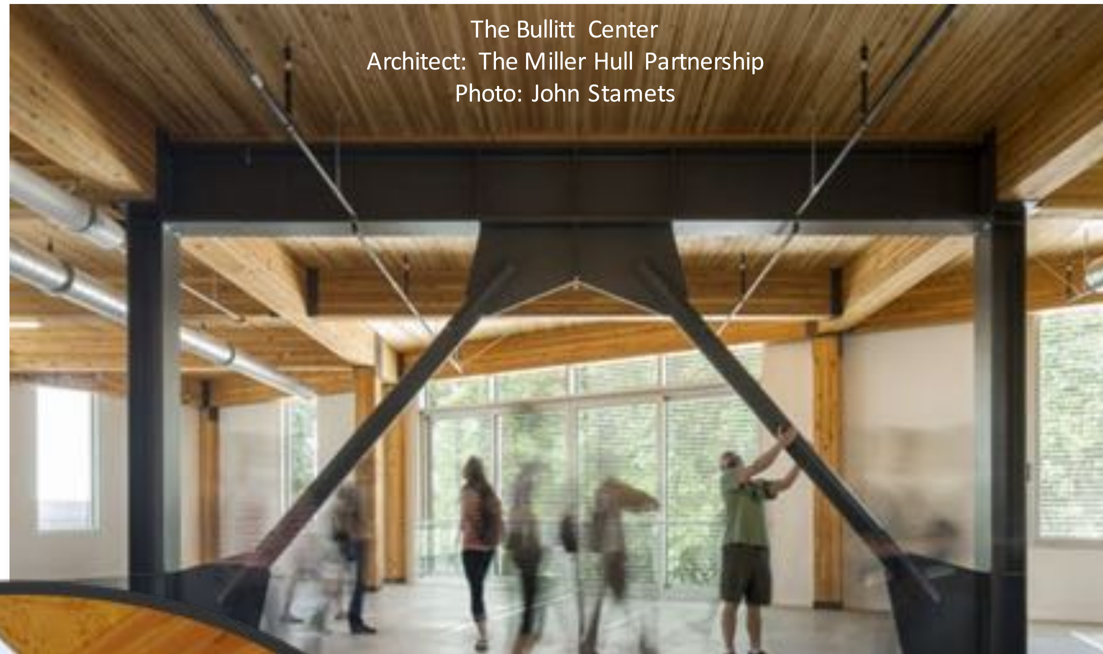
Heavy timber braced frames are becoming a preferred alternative vertical/lateral resisting system due to cost, performance and aesthetics.

Source: [illegible]iji-archite[illegible].com



Source: naturallywood.com

Hybrid Wood/Steel Braced Frames



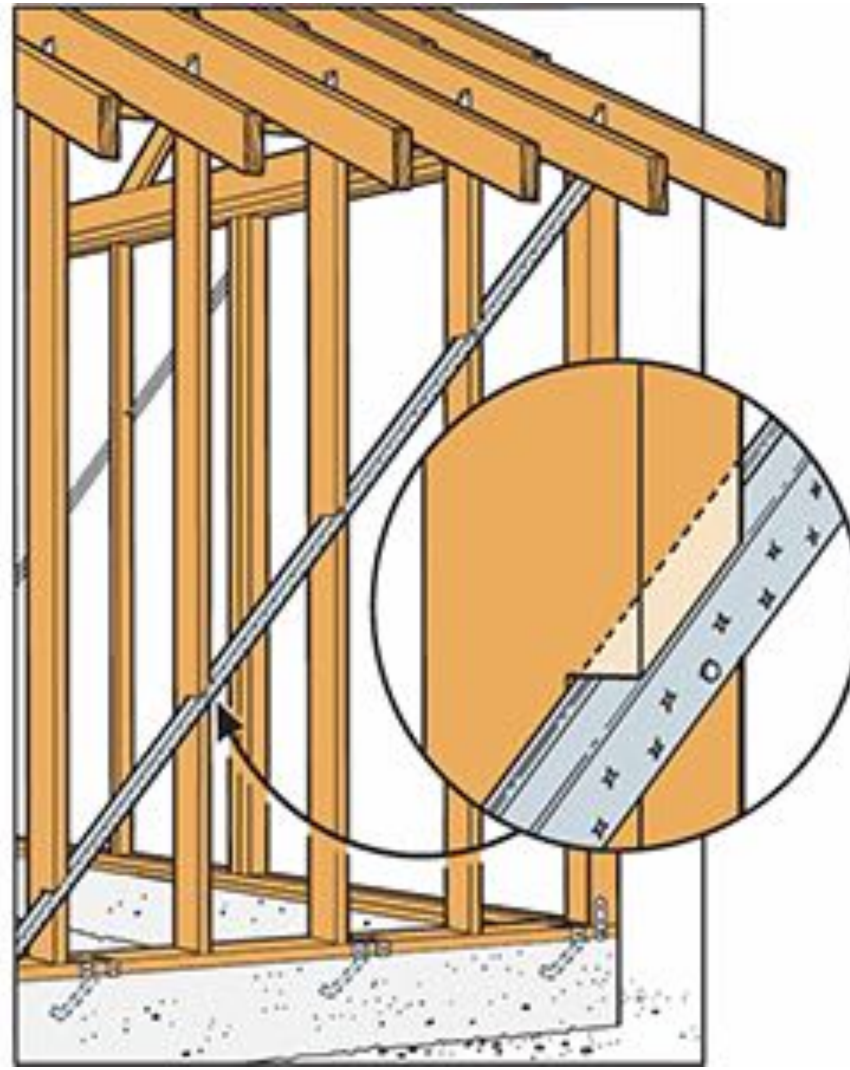
Hybrid Wood/Steel Proprietary Systems

Source: hardyframe.com

Hybrid Wood/Steel Proprietary Systems

Bracing installed in a saw kerf

Source: hardyframe.com



Source: strongtie.com

Questions?

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