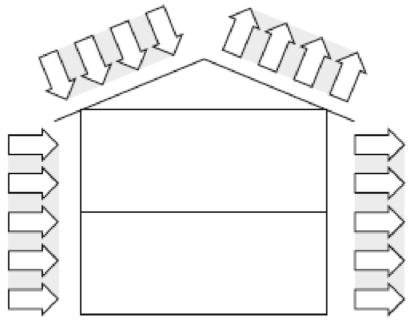
Lecture #5 – Shear wall and Diaphragm Design

Y. H. Chui

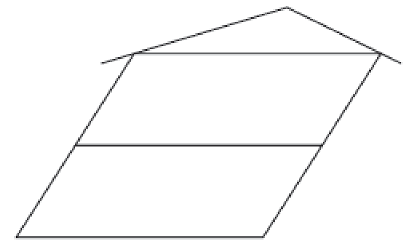
University of Alberta

Lateral Load Resisting Structure – Vertical bracing system

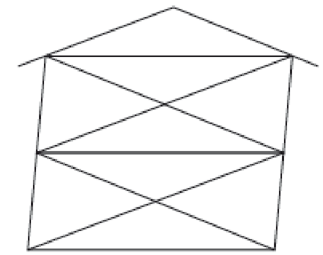
Function of bracing system



Wind forces on a building.

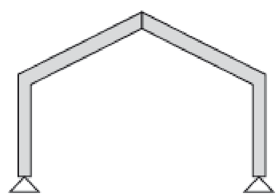


Without vertical bracing, the building will sway excessively and become unstable.

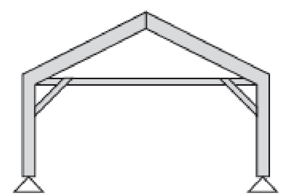


With vertical bracing, the building sway will be minimized and wind forces are transferred to the foundation.

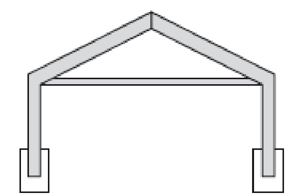
Type of wood bracing system



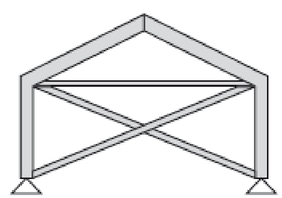
Rigid frame (Arch)



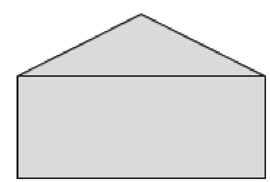
Knee brace



Pole frame

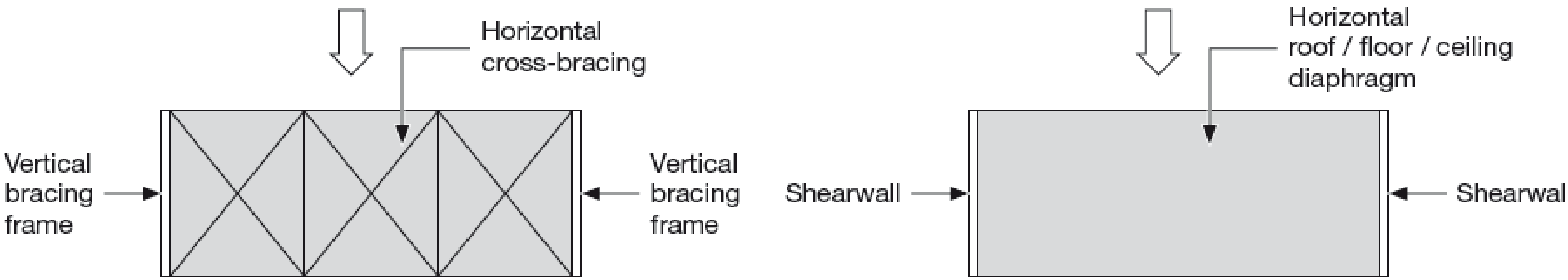


Cross-brace



Shearwall

Lateral Load Resisting Structure – Horizontal bracing system



Plan view of building

Contents

- Performance of wood frame construction
  - Experience from earthquakes and hurricanes
- Understanding load path
- Design of light wood-frame lateral load resisting assemblies
  - Failure modes
  - Design principles for diaphragms
  - Design principles for shear walls

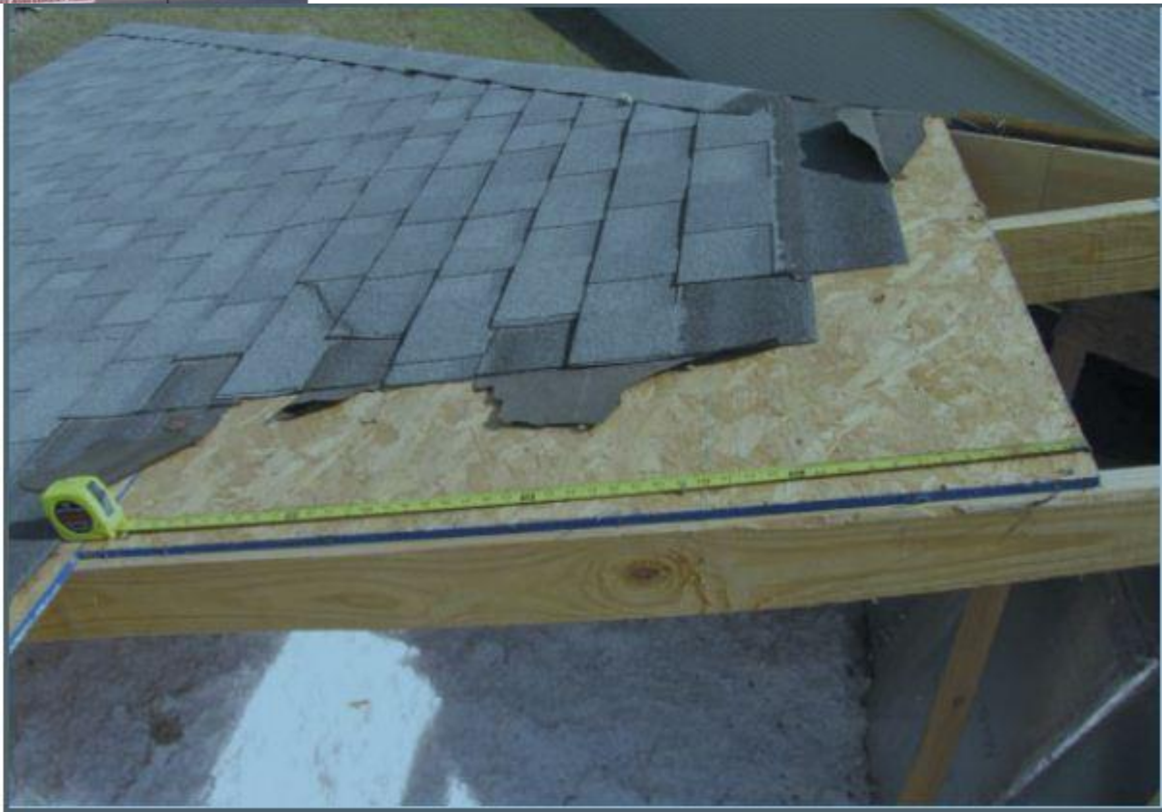
Performance of wood frame houses

- Hurricanes
- Earthquakes

Damage to Housing in Hurricanes

Critical Details: Roof Sheathing

Gable end bracing details



The APA Disaster Assessment Team evaluates a case of roof sheathing loss in Slidell, Louisiana after Hurricane Katrina.

Source: APA

Built after Hurricane Andrew (1998)

Source: APA

Seismic Performance of Wood-Frame Buildings

- Strength and stiffness
  - High strength-to-weight ratio
  - Plywood or OSB Shearwalls are effective in resisting the racking forces
  - Non-structural elements contribute to the lateral resistance
- Ductility
  - Allows building to dissipate energy
- Redundancy
  - Numerous load paths provide an extra level of safety

Performance of Wood-Frame Buildings in Past Earthquakes

- Recent California earthquakes
  - San Fernando Earthquake 1971 (Richter scale 6.6)
  - Loma Prieta Earthquake 1989 (7.1)
  - Northridge Earthquake 1994 (6.7)
- The majority of the wood-frame houses performed well from life safety point of view
- Lessons learned from past earthquakes

Connections to Foundations

This wooden residence sustained major damage when it moved off its foundation in the 1989 Loma Prieta Earthquake.

Weak and brittle sheathing material

Stucco failure

Success of full plywood coverage after Northridge earthquake

Undamaged modern Canadian-style wood-frame buildings after Japan’s 1995 Kobe earthquake

Weak First Storey

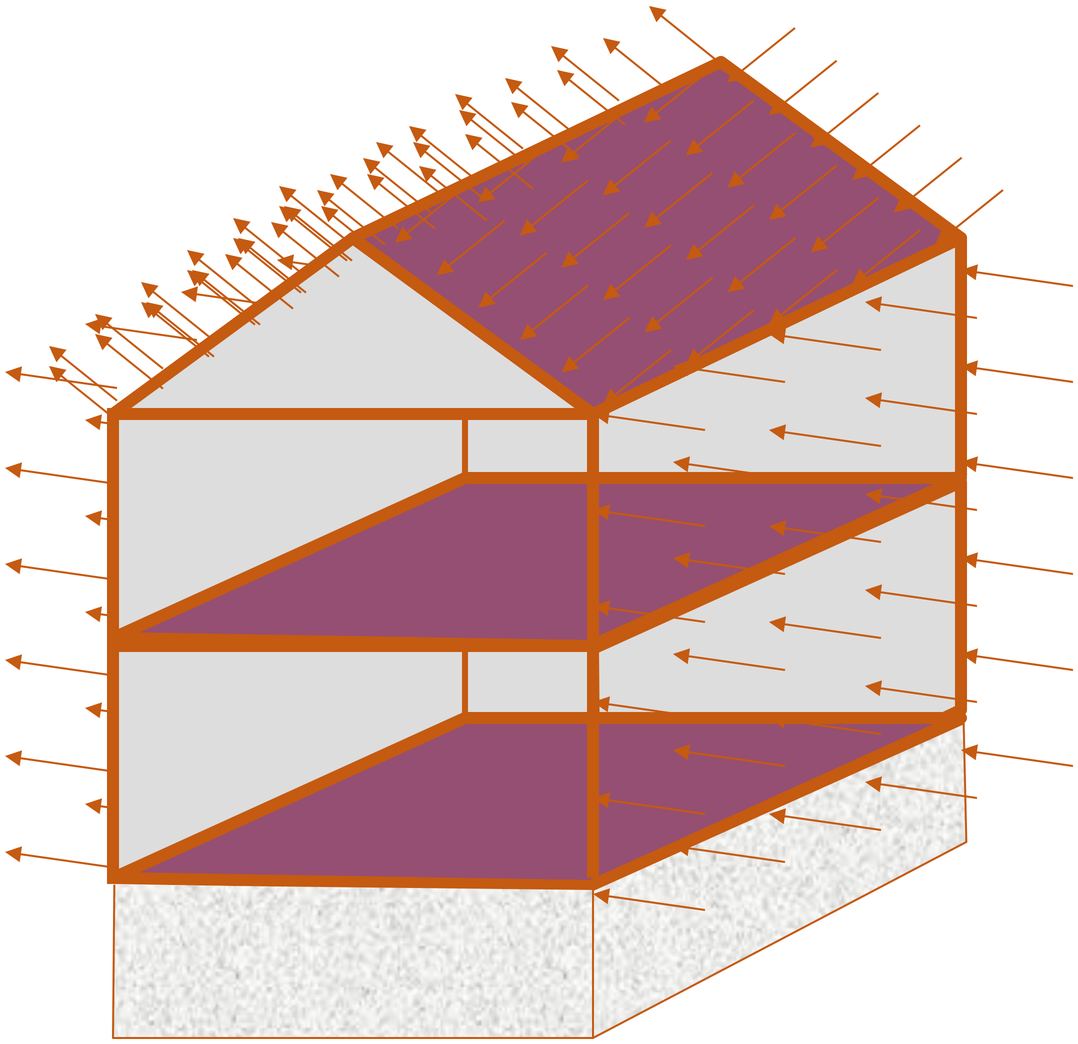
Openings in first storeys created soft storeys that led to failures

Research – Shake table test on 6-storey building (Neeswood Project)

https://www.youtube.com/watch?v=hSwjkG3nv1c

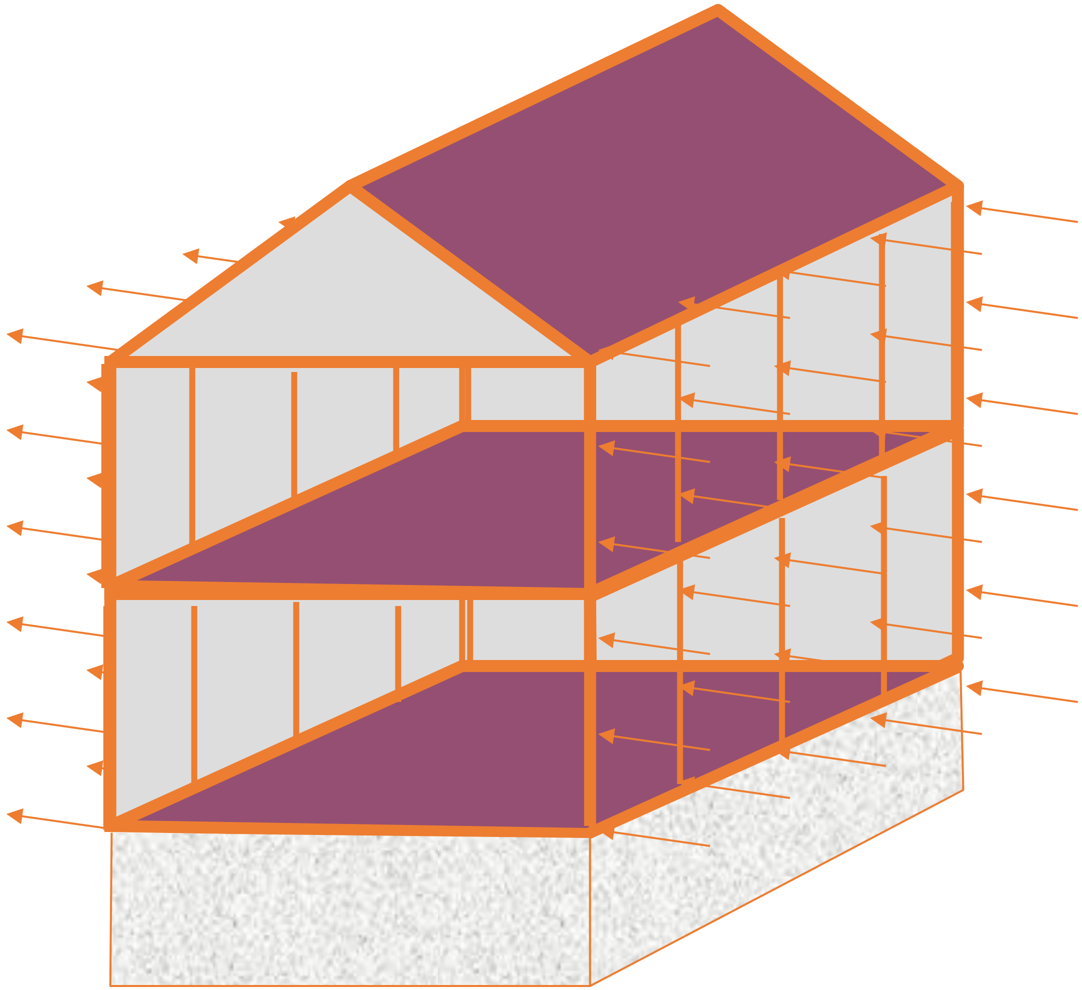
Understanding the Load Paths

Lateral Load Path - Wind Loads

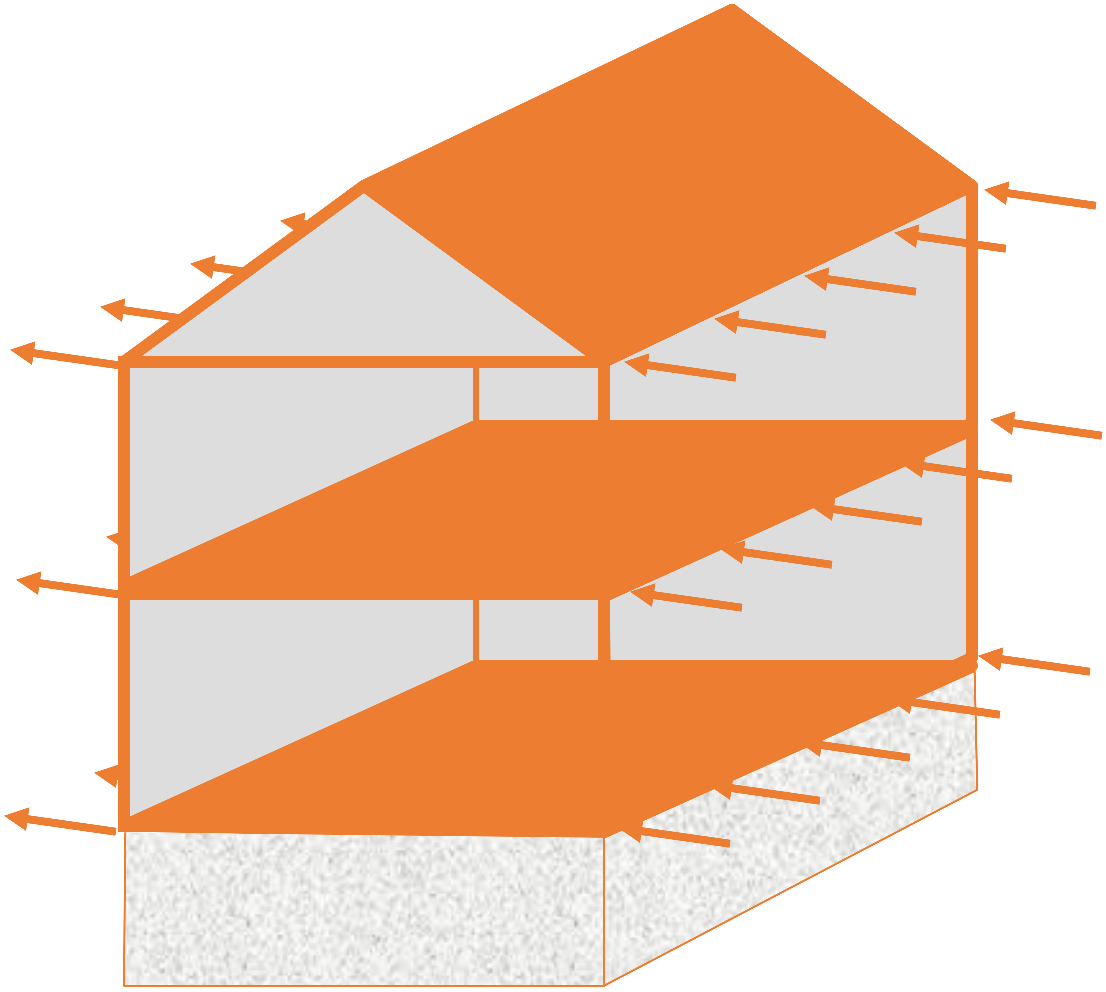


Pressure
or
Suction

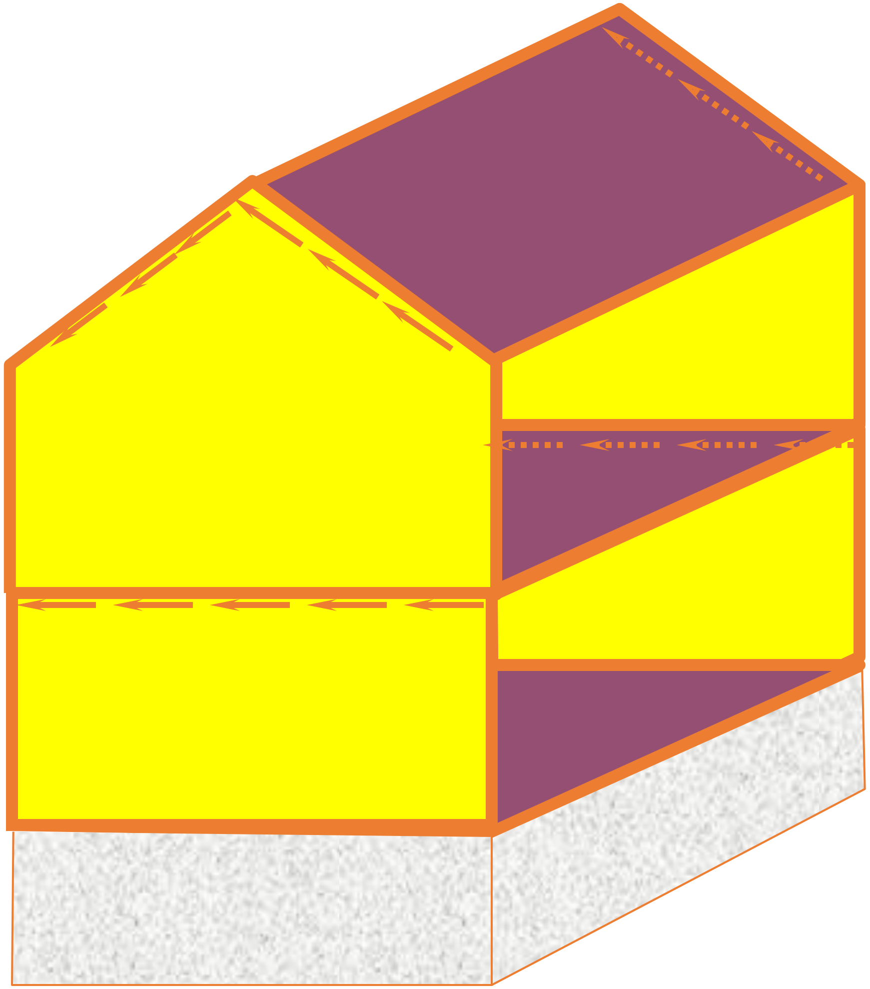
Wall Sheathing/Cladding to Studs



Studs to Diaphragms

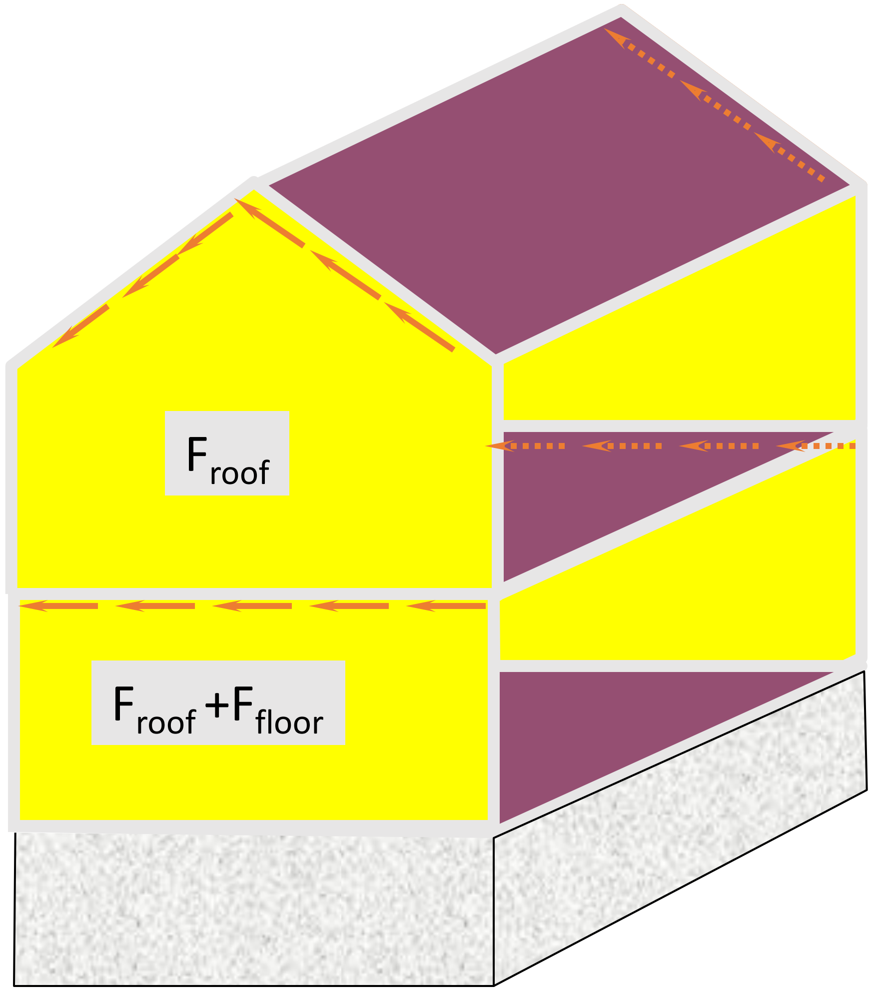


Diaphragm to shear walls



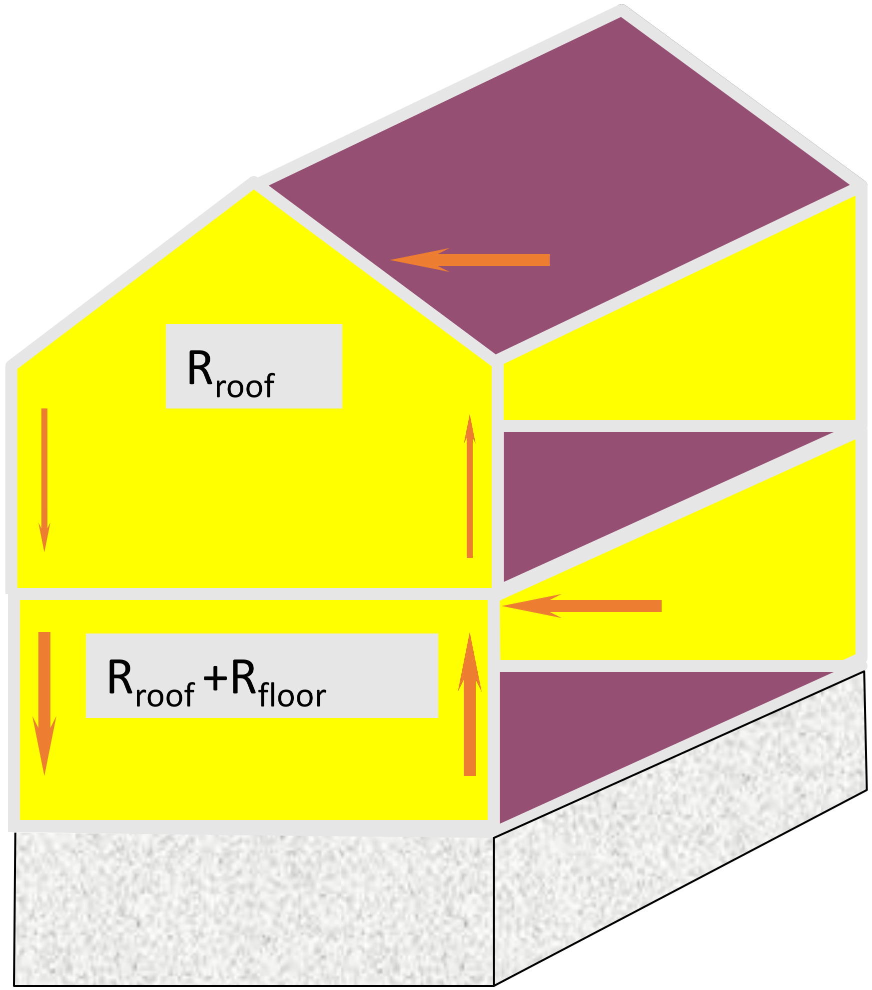
Shear Forces

Upper Storey Shear walls to Lower Storey Shear walls



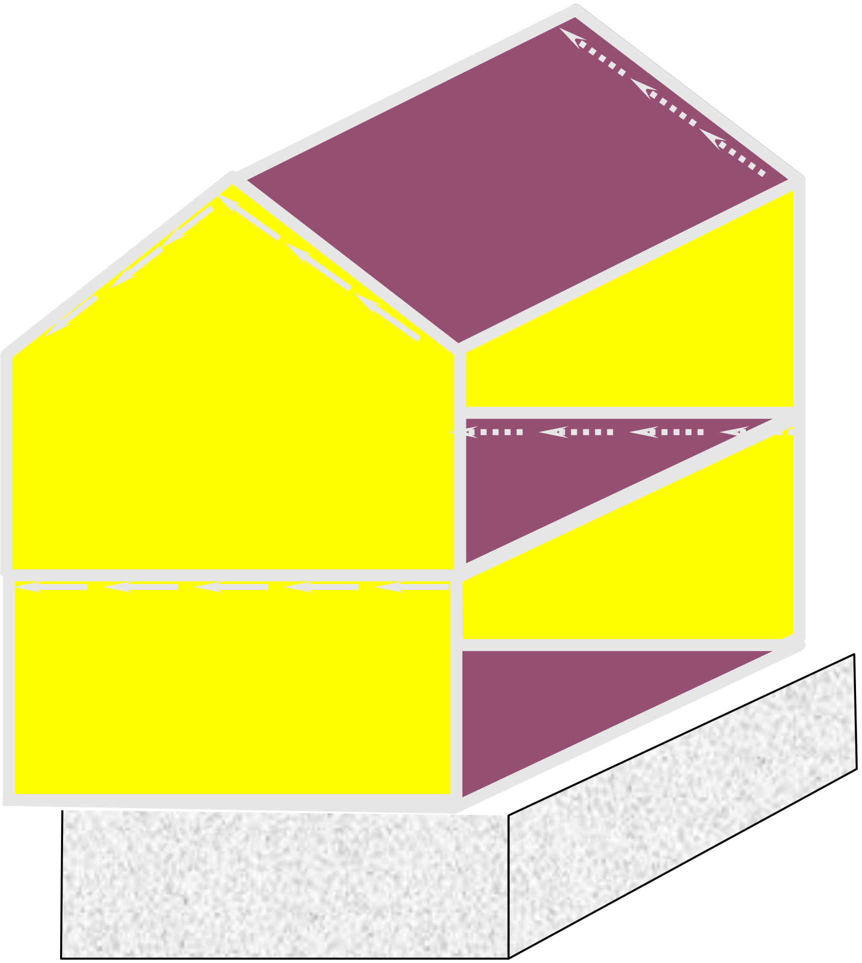
Shear Forces
"F"

Upper Storey Shear walls to Lower Storey Shear walls

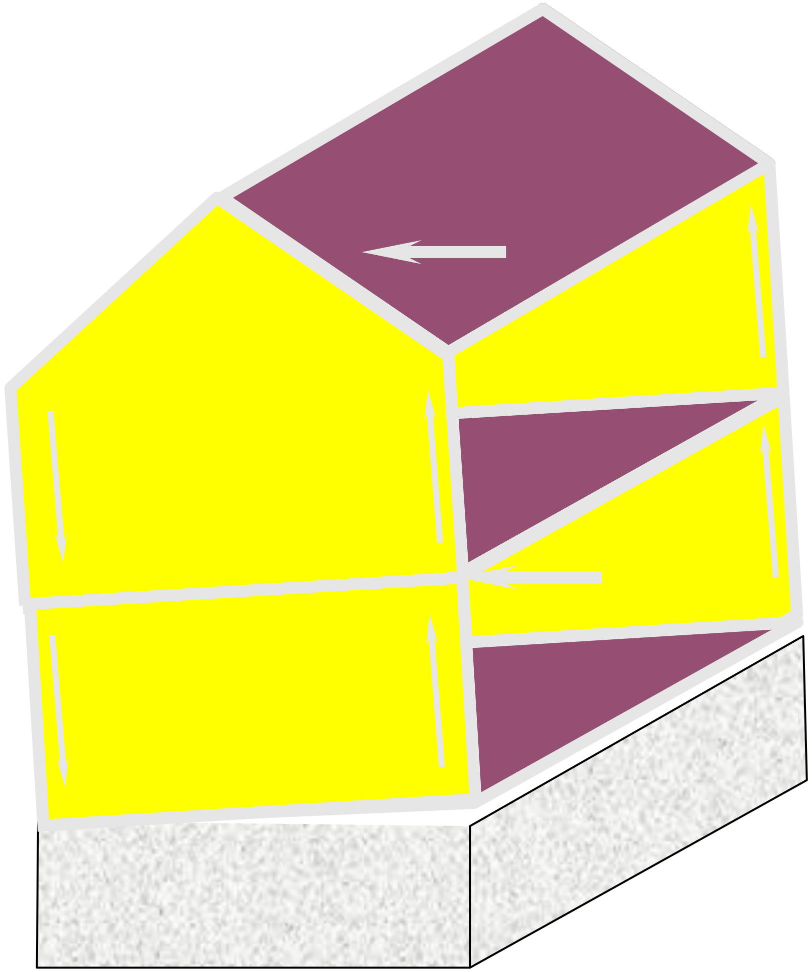


Overturning Forces, “R”

Connection to foundation

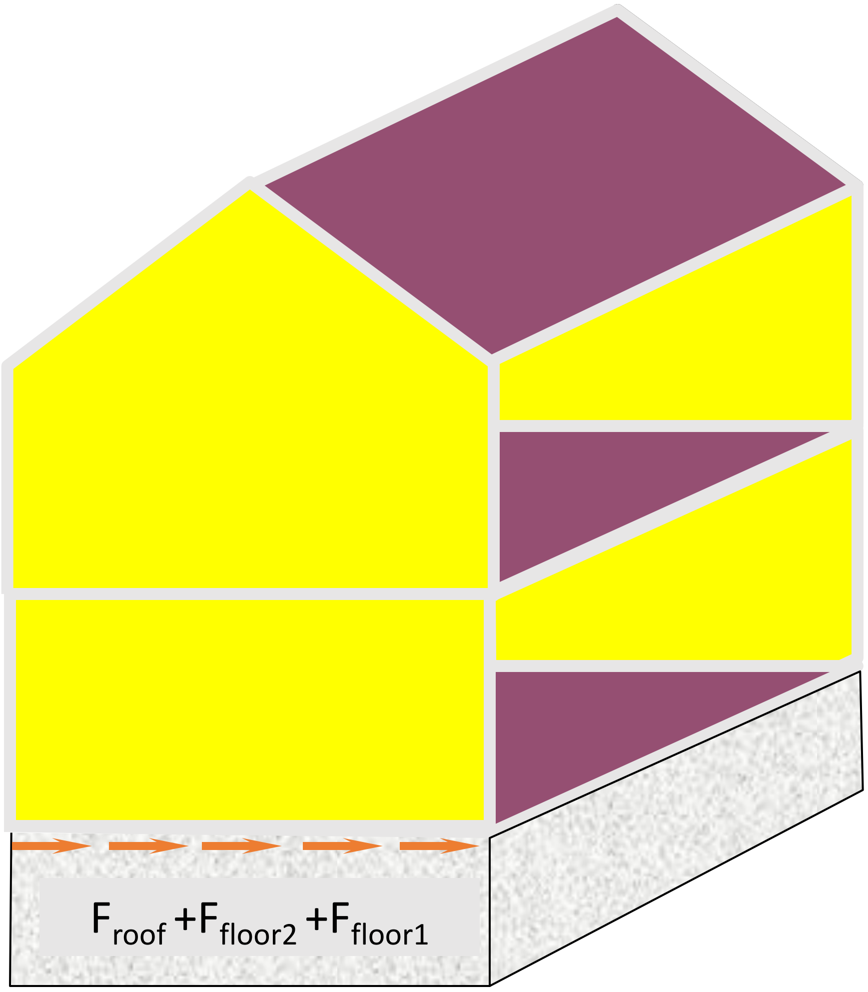


Shear Forces



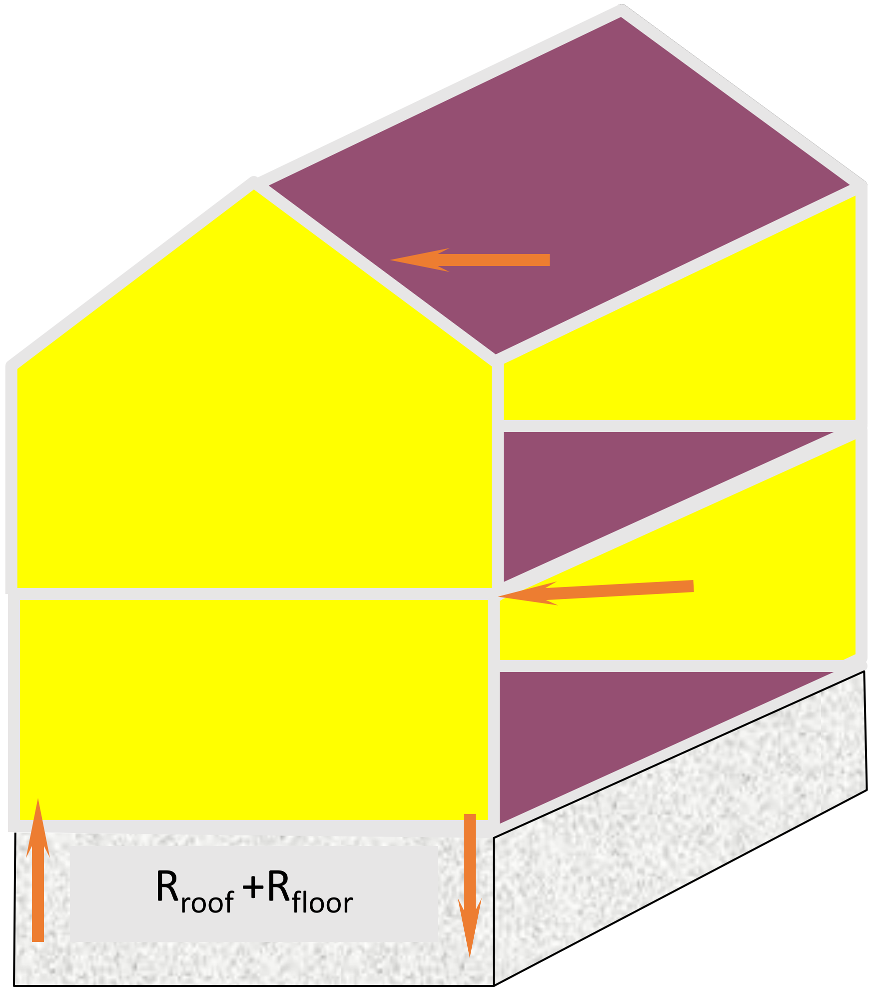
Overturning Forces

Shear walls to Foundation



Shear Forces

Shear walls to Foundation



Overturning Forces

Members and connections in the load paths

- Assemblies:
  - Roof diaphragm
  - Floor diaphragm
  - Shear wall

Chord members
Sheathing-framing connections

- Connections between assemblies:
  - Shear wall-diaphragm
  - Shear wall-foundation
  - Diaphragm-foundation

Shear connections
Hold-down connections

Lateral Load - Wind and Earthquake

- Loads are dynamic in nature
- In National Building Code of Canada Part 4, If certain conditions are met, simplified Equivalent Static Procedure can be used for design
- For wood construction, these conditions are generally met, hence Equivalent Static Procedure is used

All connections and assemblies must be designed!

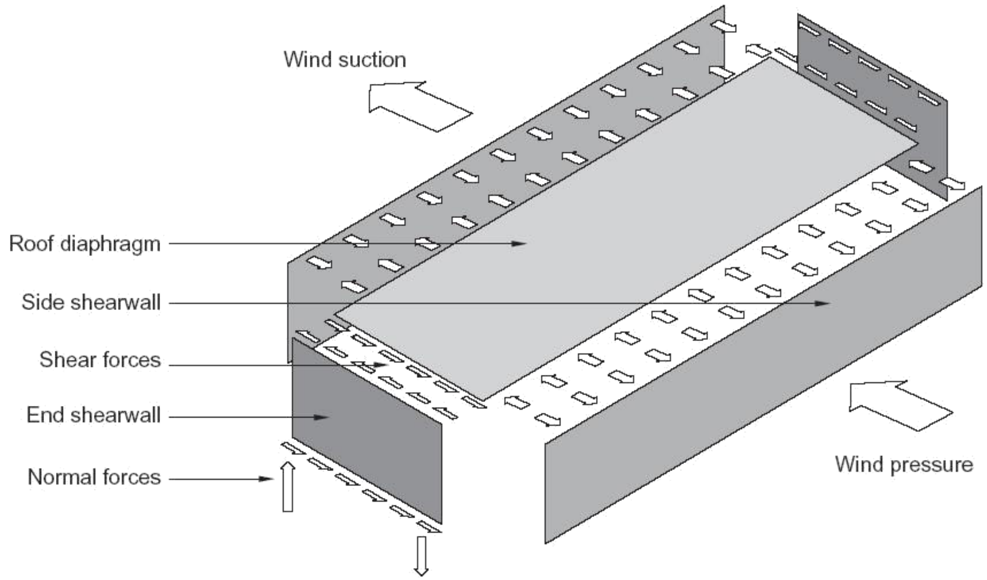
- In light wood frame buildings, lateral loads are resisted by shearwalls and diaphragms (as lateral load resisting assemblies).
- Each load path must be considered in design to ensure that:
  - Connections are adequate to transfer loads between assemblies
  - Assemblies are adequate to resist applied lateral load without failure

- This lecture focuses on design of assemblies

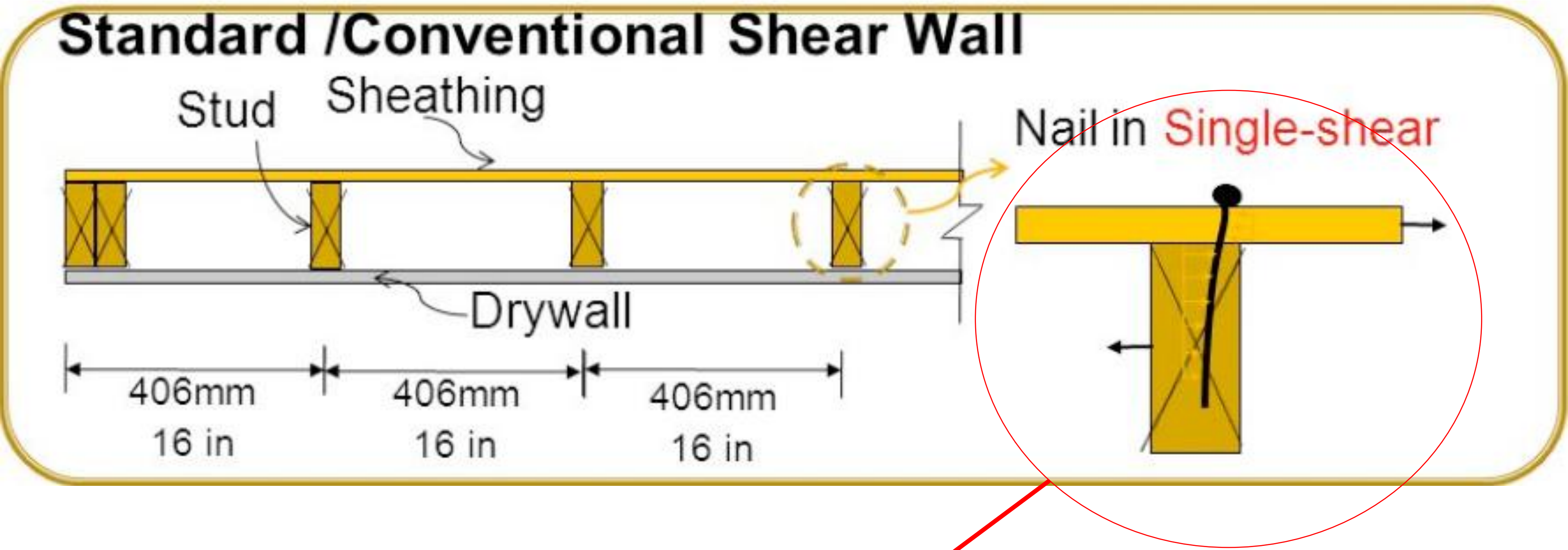
Shear wall and diaphragm action

Plate type structural elements designed to transmit force in their own plane:

- Diaphragm:
  - Horizontal element
    - Floor
    - Roof
- Shearwall:
  - Vertical element
    - Walls

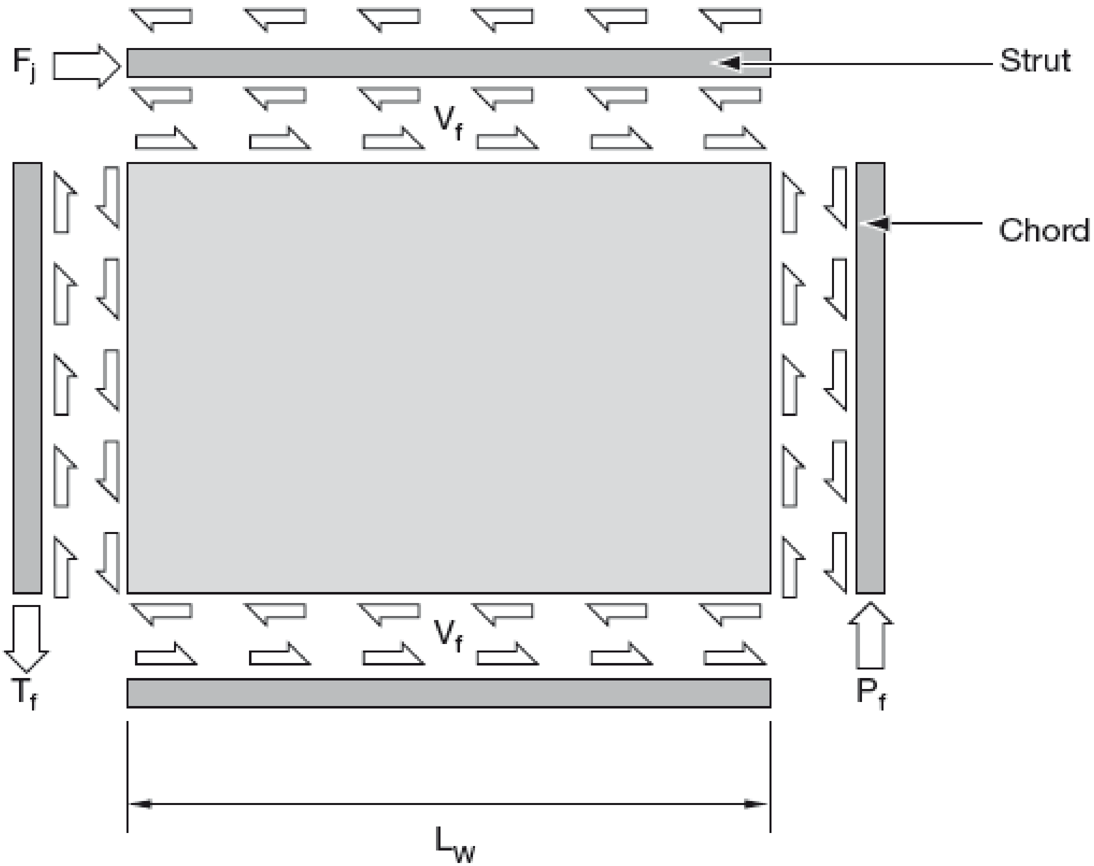


Each is assumed to act as a system, and design is to determine sheathing panels, framing members and sheathing-to-framing connection (nailed)



This combination determines the shear strength of shear walls and diaphragms

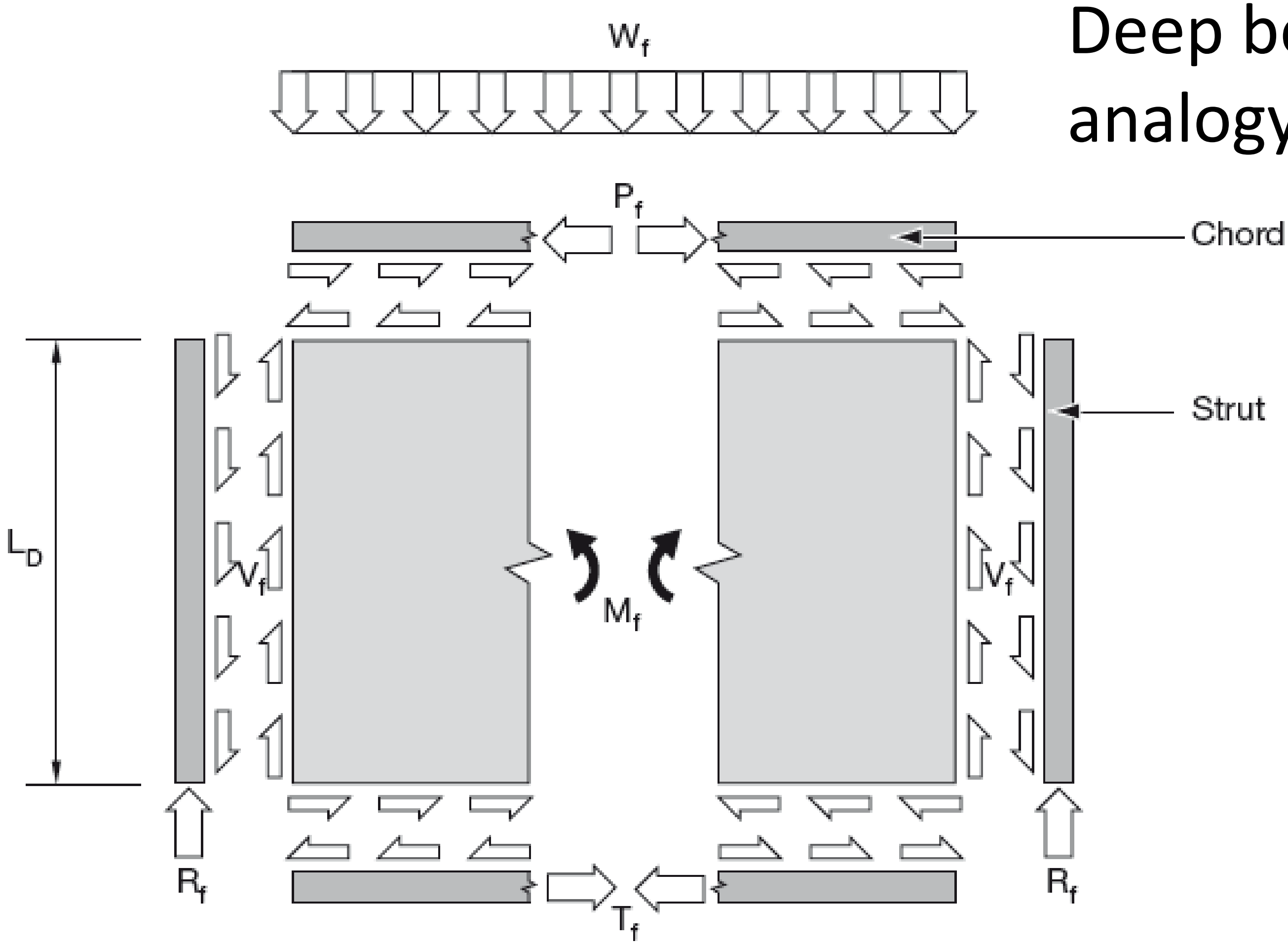
Free-body diagram for shear wall



Elevation of shearwall segment

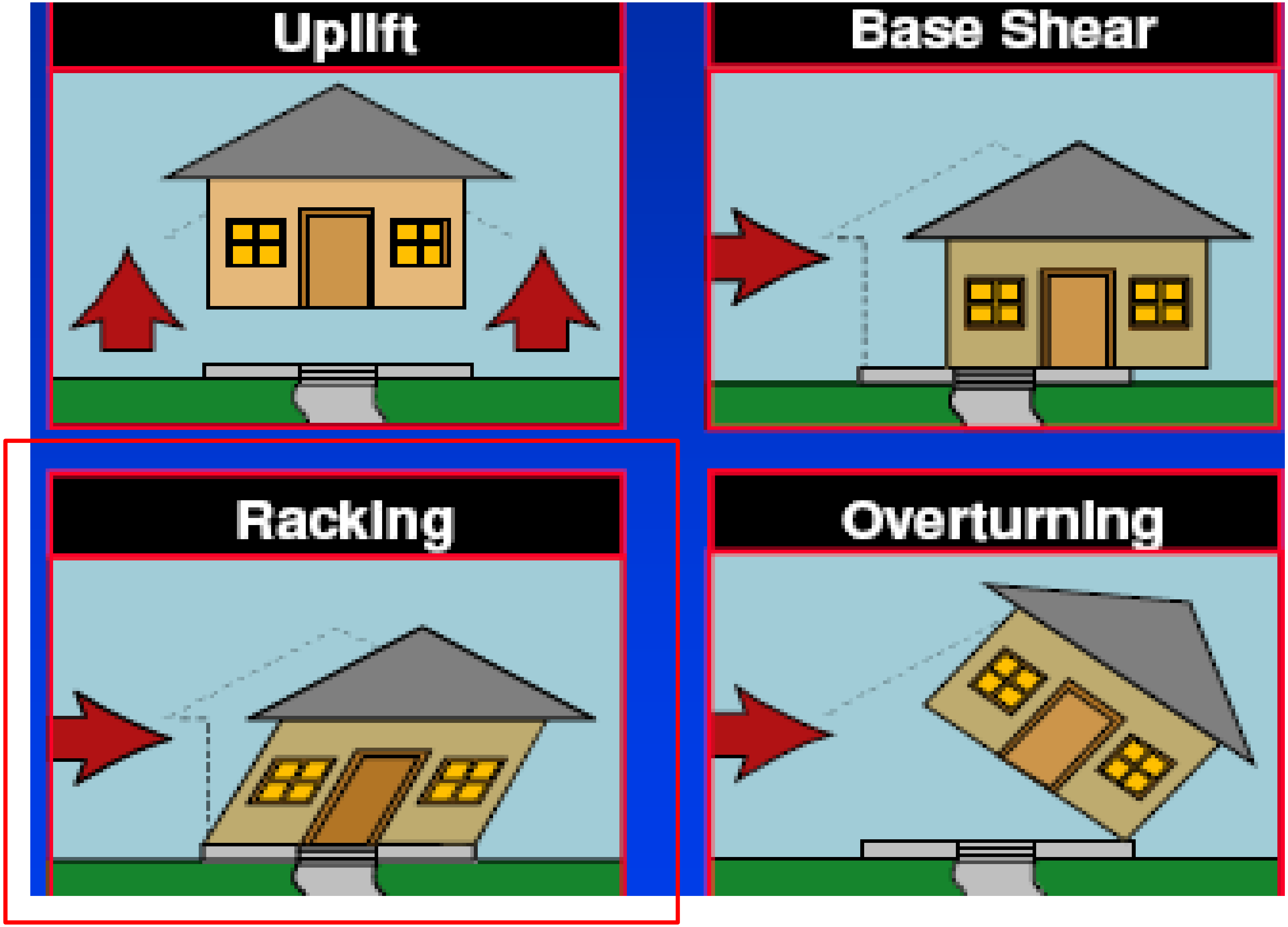
Free-body diagram for diaphragm

Deep beam analogy



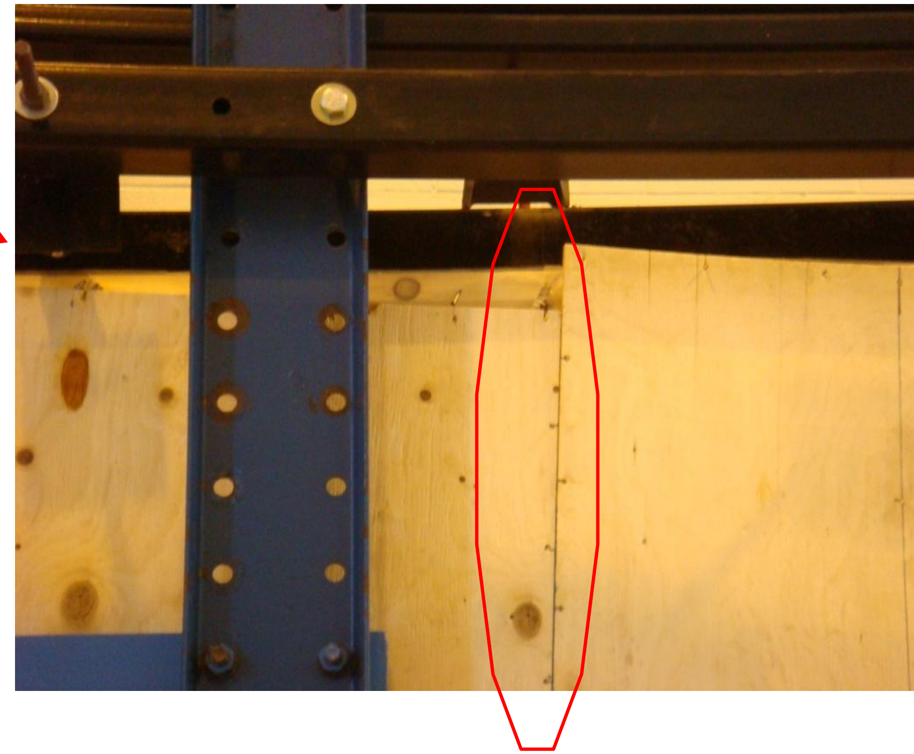
Plan of diaphragm

Failure modes in buildings under lateral load



Courtesy of APA

Failure modes in shear walls



Yielding of nails around the perimeter

Failure modes in shear walls

Nail Yielding

Nail bending/ripping through sheathing

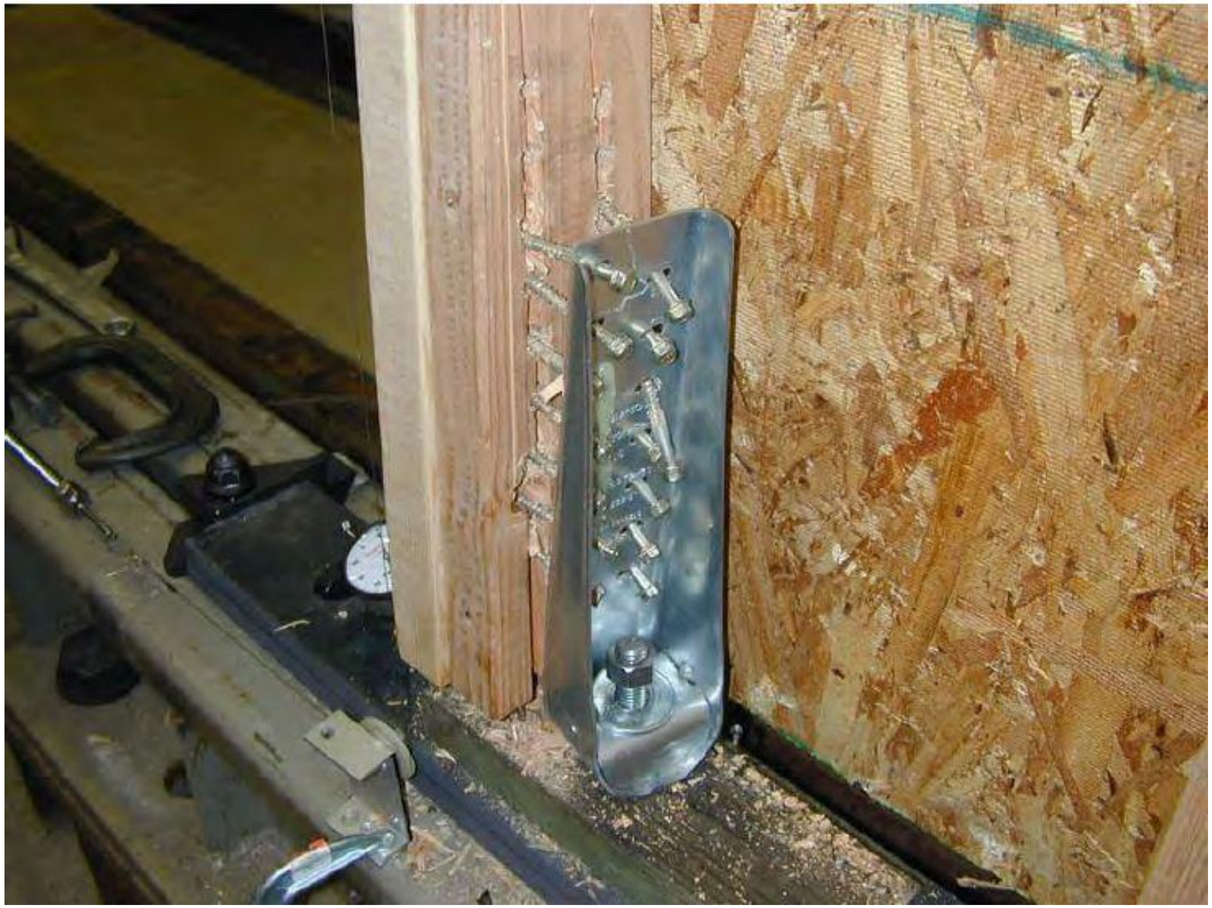
Source: APA & AWC

Failure modes in shear walls

Failure of boundary members

Source: APA & AWC

Failure modes in shear walls



End stud failure at hold-down

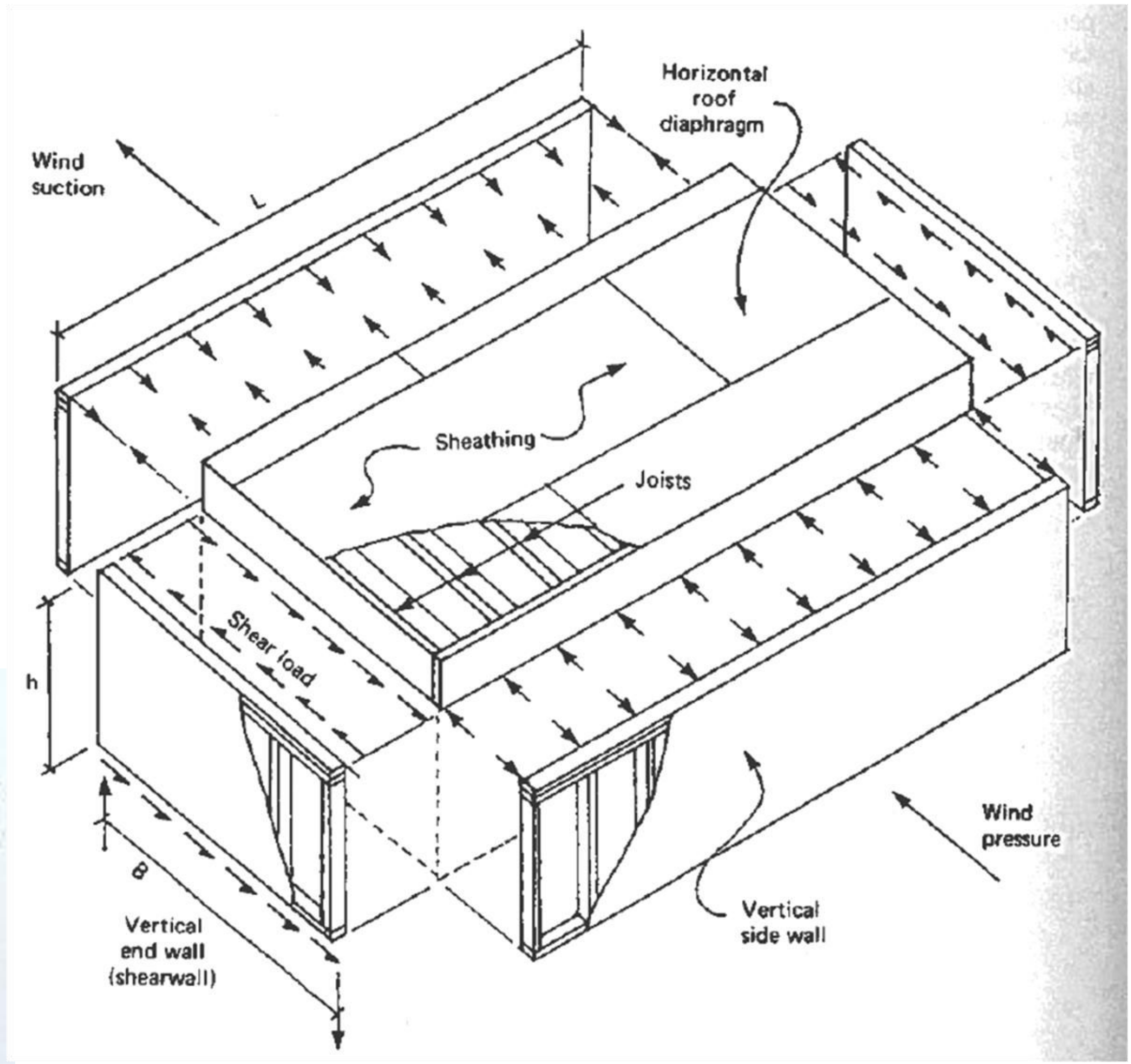
Source: AWC

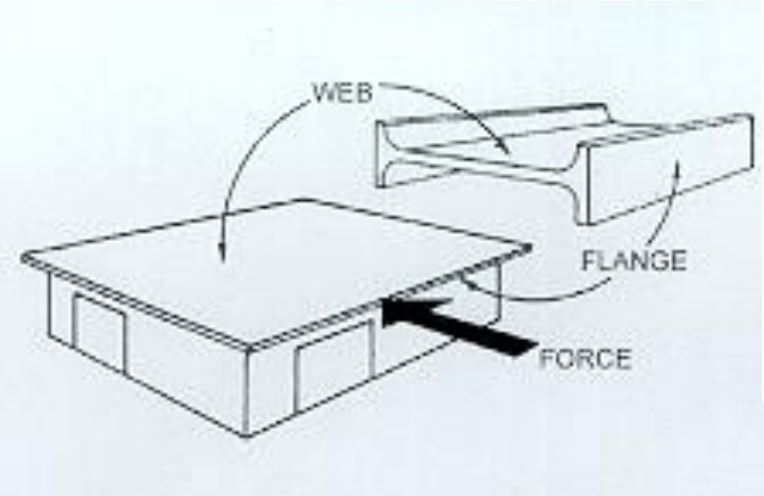
Design principles of CSA O86-14 for light wood frame shear walls and diaphragms

Diaphragm Design

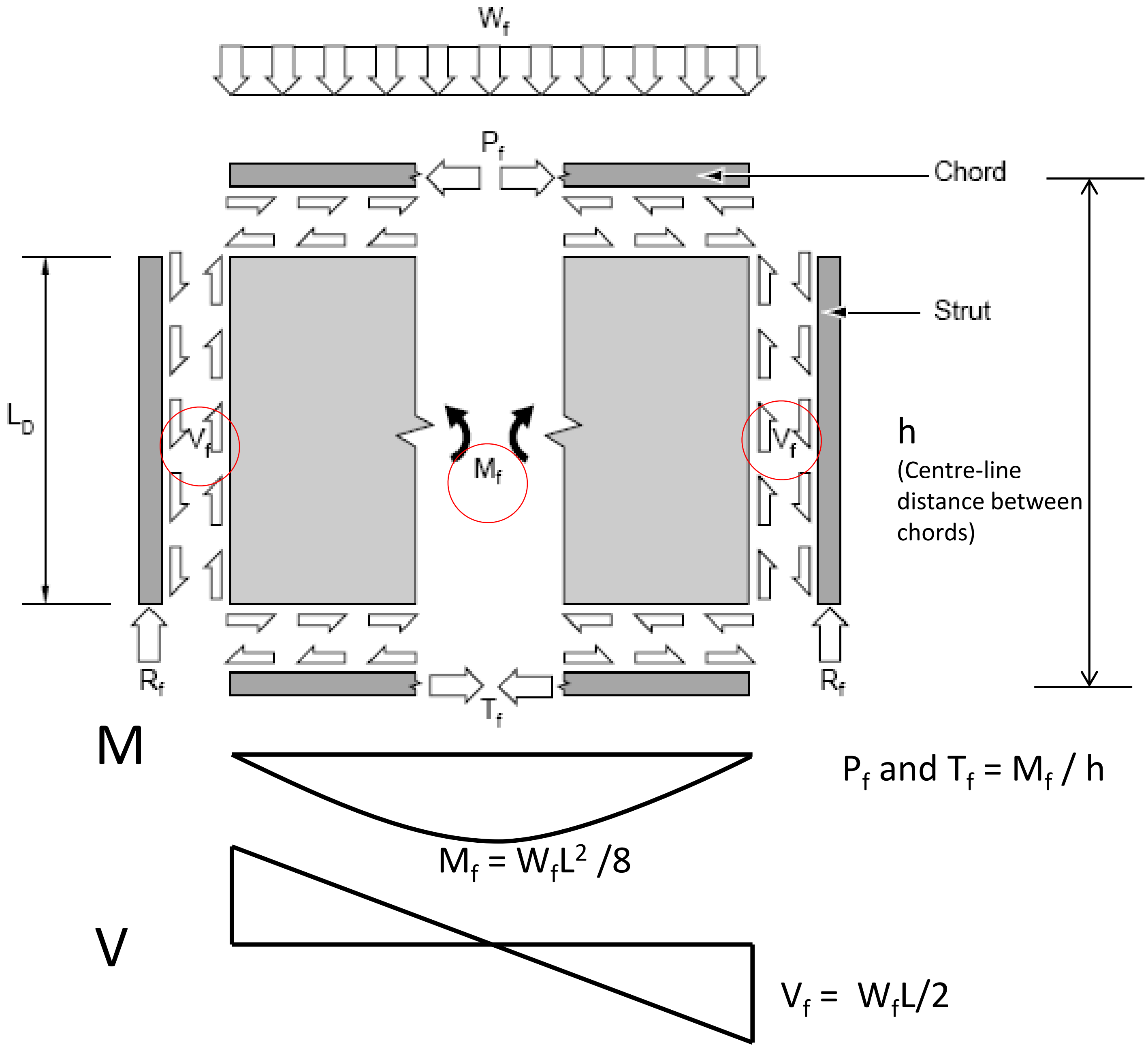
Deep beam analogy

- Shear resistance : sheathing (Web)
- Flexural resistance: chords (Flanges)





Calculation of applied force



Shear resistance of diaphragm

Shear resistance provided by sheathing-to-framing joint leading to slippage of sheathing panels relative to floor joists

11.5.2 Shear resistance of diaphragms

2 possible failure modes

(a) $V_{rs} = \phi v_d J_D J_s J_f J_{ud} L_D$ Sheathing-to-framing joint failure

where

ϕ = 0.8

v_d = N_u/s, for diaphragms sheathed with plywood or OSB, kN/m

= specified shear strength for diaphragms sheathed with diagonal lumber sheathing (Clause 11.5.5), kN/m

N_u = lateral strength resistance of sheathing-to-framing connection along panel edges, per fastener (see Clause 12.9.4 for nails), N

J_D = factor for diaphragm and shearwall construction (Clause 12.9.4.1)

J_s = fastener spacing factor (Clause 11.4.1)

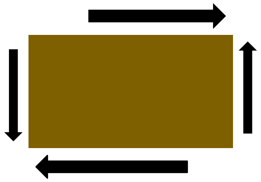
s = fastener spacing along panel edges, mm

J_f = fastener row factor for blocked diaphragm (Clause 11.4.2)

J_{ud} = strength adjustment factor for unblocked diaphragms (Clause 11.4.3)

L_D = dimension of diaphragm parallel to direction of factored load, m

(b) $V_{rs} = \phi v_{pb} K_D K_s K_T L_D$ Sheathing panel buckling – only occurs in thin panels



where

ϕ = 0.8

K_S = service condition factor (Table 9.4.2)

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb} \frac{\pi^2 t^2}{3000b} \left(B_{a,0} B_{a,90}{}^3\right)^{\frac{1}{4}}$$

Lateral strength of nailed joint, N_u

12.9.4.1

The factored lateral strength resistance of the nail or spike connection, N_r, shall be taken as follows:

$$N_r = \phi N_u n_F n_S J_F$$

where

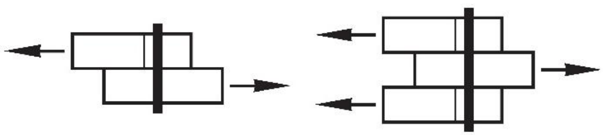
ϕ = 0.8

$N_u = n_u (K_D K_{SF} K_T)$

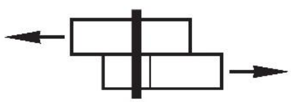
where

n_u = unit lateral strength resistance, N (Clause 12.9.4.2)

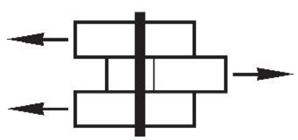
(a) $f_1 d_F t_1$



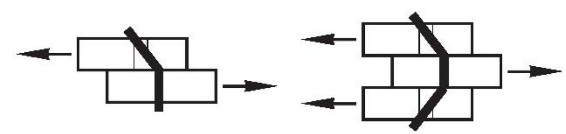
(b) $f_2 d_F t_2$



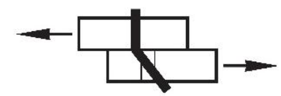
(c) $\frac{1}{2} f_2 d_F t_2$



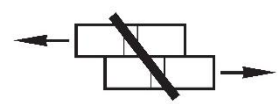
(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$



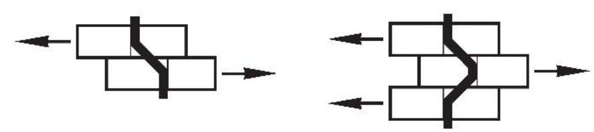
(e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$



(f) $f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$



(g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$



Modification factors for diaphragm shear

J_D = Diaphragm and shear wall construction factor (Cl. 12.9.4.1) = 1.3

Increased by 30% to account for the fact that more than one fastener resists applied load.

J_S = Fastener spacing factor (Cl. 11.4.2)

11.4.1 Fastener spacing factor, J_s

The shear strength of shearwalls and diaphragms built with structural wood-based panels and lumber framing shall be multiplied by the fastener spacing factor, J_s, given as follows:

$J_s = 1.0$ for $s \geq 150$ mm

$$= 1 - \left(\frac{150 - s}{150}\right)^{4.2} \text{ for } 50 \text{ mm} \leq s < 150 \text{ mm}$$

To account for the possibility that the framing members are cracked due to close fastener spacing (50mm-150mm).

Modification factors for diaphragm shear

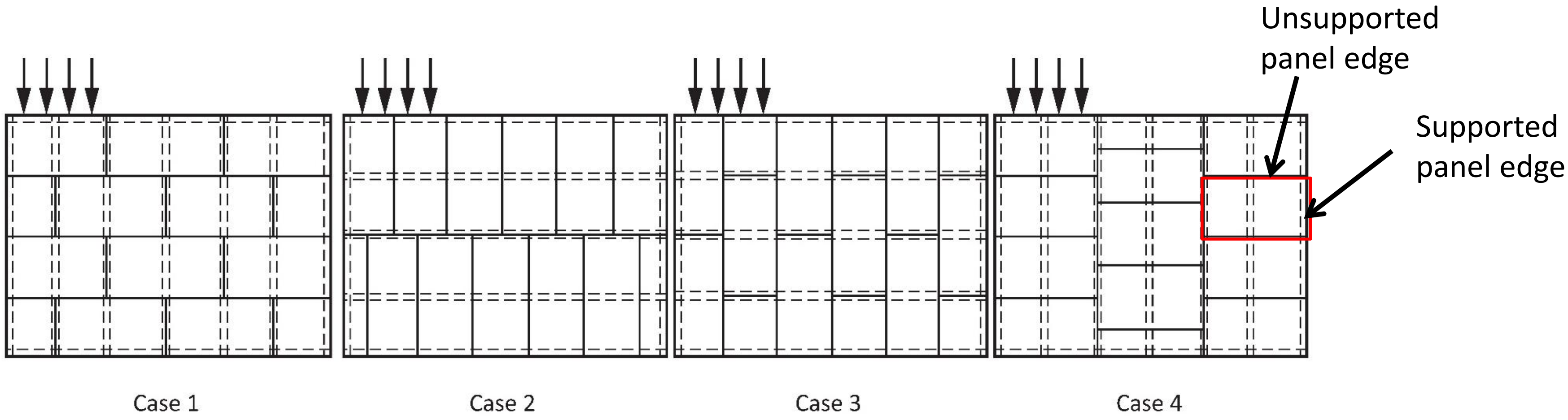
J_{ud} = Strength adjustment factor for unblocked diaphragms (Cl. 11.4.3)

Table 11.4.3
Strength adjustment factor, J_{ud}, for unblocked diaphragms*

Configuration case†	J_{ud}
1	0.89
2, 3 and 4	0.67

**The shear strength of an unblocked diaphragm shall be calculated by multiplying the strength adjustment factor by the specified shear strength of a blocked diaphragm with fasteners spaced at 150 mm on centre along panel edges and 300 mm on centre along intermediate framing members.*

Unblocked diaphragm does not have all sheathing panel edges supported



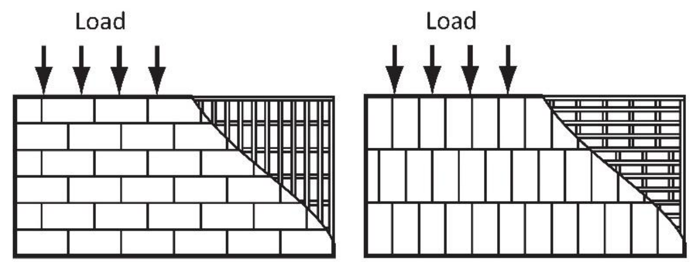
Modification factors for diaphragm shear

J_f = Fastener row factor for blocked diaphragm (Cl 11.4.2)

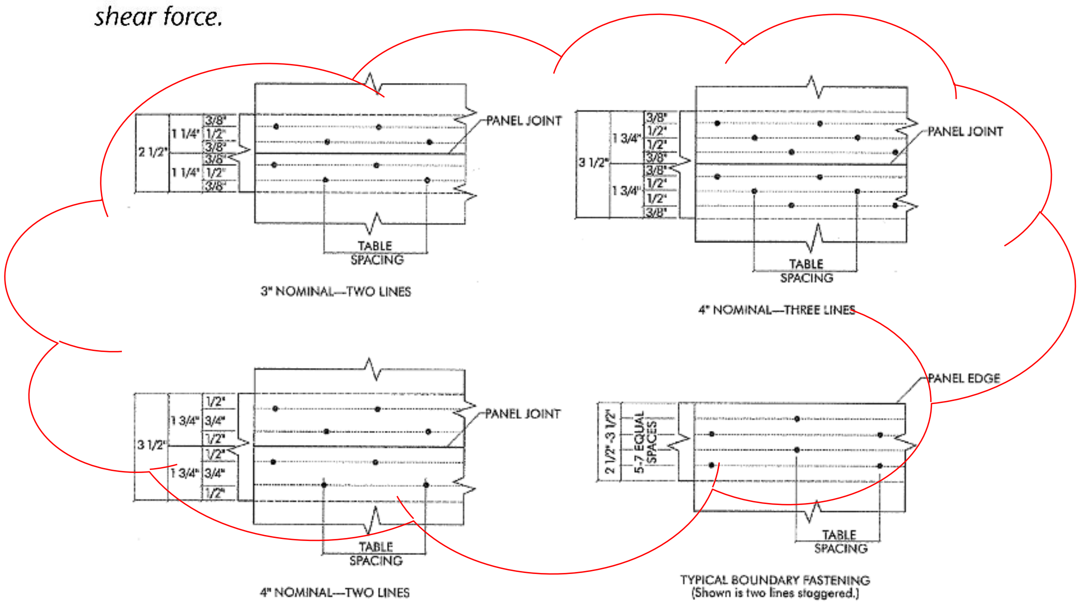
Table 11.4.2
Fastener row factor, J_f, for blocked diaphragms

Number of rows	Minimum thickness of framing member, mm	J_f
1	38	0.89
	64*	1.00
2	64*	1.78
	89†	2.00
3	89†	2.67

**Or two* 38 mm *thick members connected to transfer the factored shear force.*
†*Or three* 38 mm *thick members connected to transfer the factored shear force.*



Blocked diaphragm has all sheathing panel edges supported on joist or blocking



Panel buckling resistance, v_{pb}

(c) $V_{rs} = \phi v_{pb} K_D K_S K_T L_s$

where

ϕ = 0.8

K_S = service condition factor (Table 9.4.2)

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb} \frac{\pi^2 t^2}{3000b} \left(B_{a,0} B_{a,90}{}^3\right)^{\frac{1}{4}}$$

where

K_{pb} = panel buckling factor

$$= 1.7(\eta + 1)\exp\left(\frac{-\alpha}{0.05\eta + 0.75}\right) + (0.5\eta + 0.8)$$

where

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

$$\eta = \frac{2\ B_v}{\sqrt{B_{a,0} B_{a,90}}}$$

where

a = larger dimension of panel, mm

b = smaller dimension of panel, mm

$B_{a,0}$ = axial stiffness of panel 0° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

$B_{a,90}$ = axial stiffness of panel 90° orientation, see Tables 9.3A, 9.3B and 9.3C, N/mm

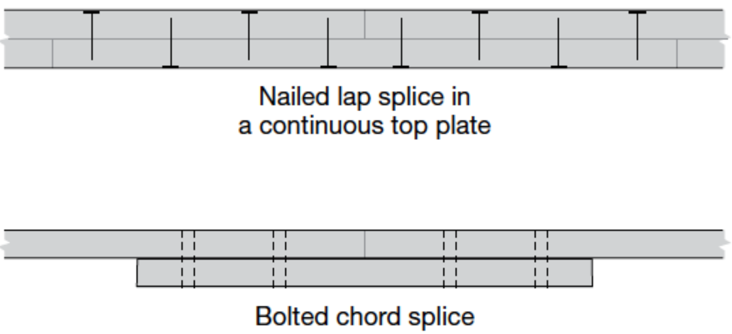
B_v = shear-through-thickness rigidity, see Tables 9.3A, 9.3B and 9.3C, N/mm

t = panel thickness, mm

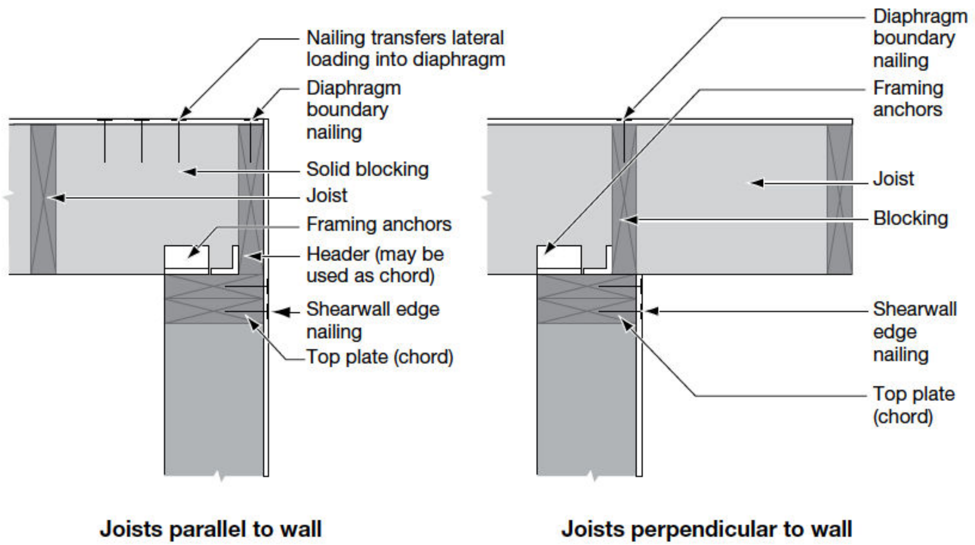
Buckling resistance of a sheathing panel (plywood or OSB) is a function of the axial moduli and shear modulus of the panel.

Other design checks for diaphragm

- Chord member forces caused by bending moment: T_f and P_f (according to provisions for lumber tension and compression members)
- Deflection of diaphragm (Cl. 11.7.2)
- Connection in chord members
- Connection to supporting shear wall

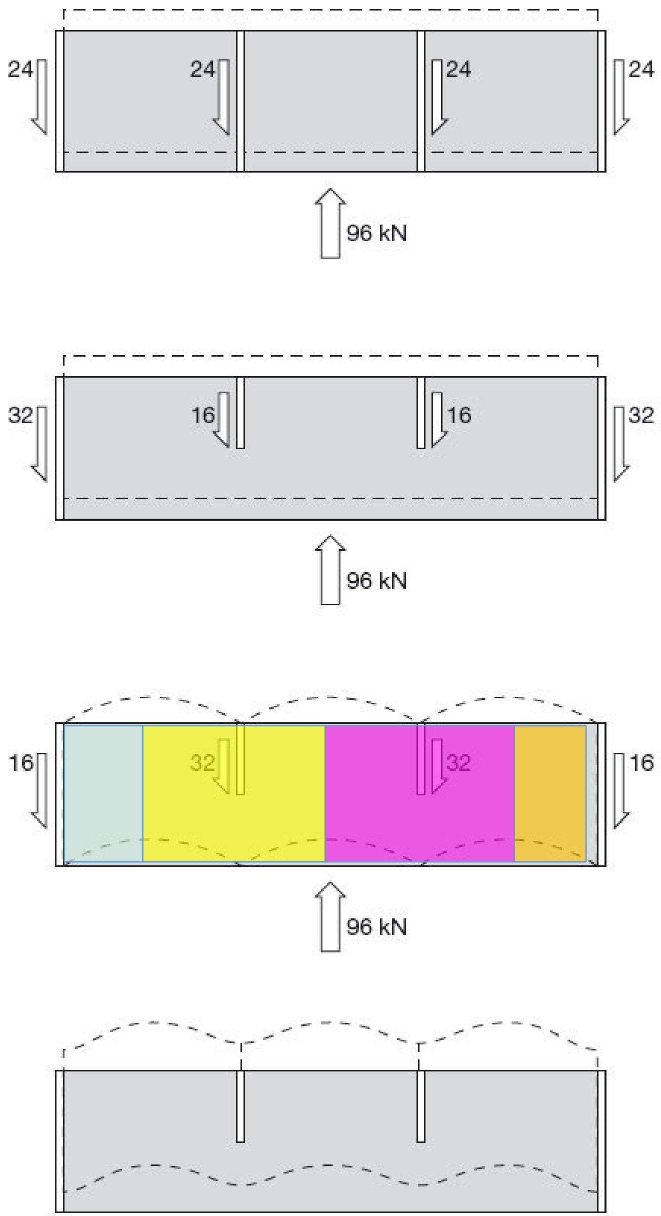


Connection in chord members



Connection to shear wall

Distribution of forces to shear wall



Case 1
Rigid diaphragm
(Shearwalls of equal stiffness)

Case 2
Rigid diaphragm
(Interior shearwalls half as stiff as exterior shearwalls)

Rigid diaphragm – depends on shear wall stiffness

Case 3
Flexible diaphragm
(Shearwalls are very rigid)

Flexible diaphragm – based on tributary area

More critical of the two approaches for each shear wall

Case 4
Semi-rigid diaphragm and shearwalls

Semi-rigid – real case but complex

More sophisticated method

2014 paper published in American Society of Civil Engineering

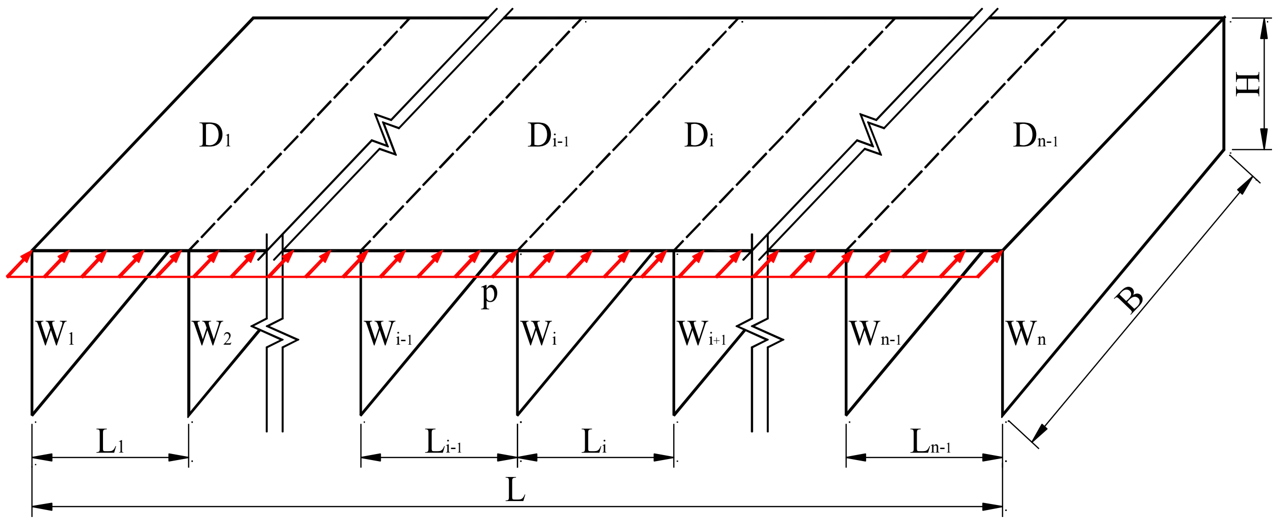
Load Distribution in Timber Structures Consisting of Multiple Lateral Load Resisting Elements with Different Stiffnesses

Zhiyong Chen, A.M.ASCE[1]; Ying H. Chui[2]; Chun Ni, M.ASCE[3]; Ghasan Doudak, M.ASCE[4]; and Mohammad Mohammad, M.ASCE[5]

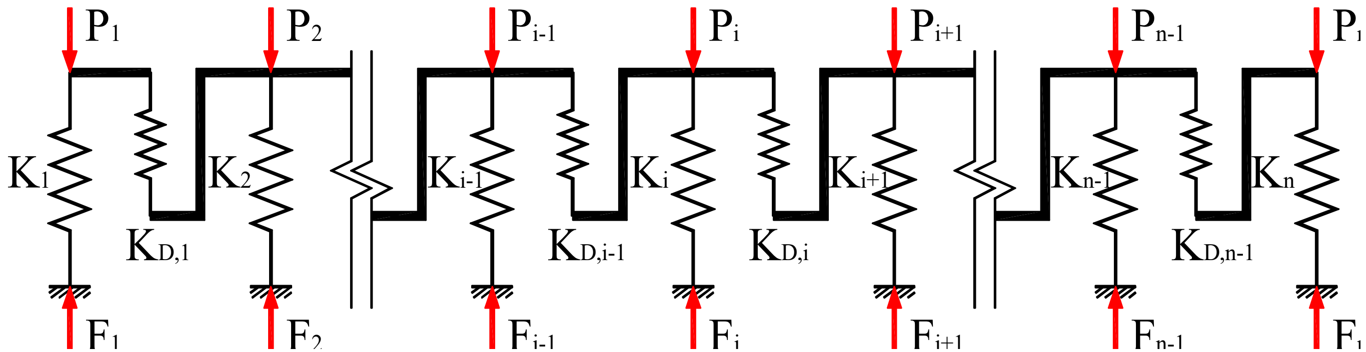
Abstract: It is well known that the stiffness properties of diaphragms and lateral load resisting elements (LLREs) influence the load distribution between LLREs under lateral load induced by earthquake or wind. Where a more sophisticated method of calculating the load distribution in a lateral load resisting system is used, often it is based on the concept of a beam on an elastic foundation. This approach could be tedious to apply in design when there are more than a few LLREs. A multiple-spring model (MSM), whereby the translational springs are used to model the diaphragm stiffnesses and the stiffnesses of the LLREs, is proposed. The model was validated with test and finite-element results of a particular benchmark building. The lateral load distribution between LLREs with various stiffness ratios of diaphragm to LLRE was also investigated. The results show that, contrary to common belief, the forces transferred by a semirigid diaphragm to supporting LLREs may be higher than those predicted by flexible and rigid-diaphragm assumptions. Therefore, using the envelope force approach may lead to underestimation of the design forces in the shear walls. **DOI:** 10.1061/(ASCE)CF.1943-5509.0000587.

Author keywords: Timber structures; Lateral load resisting elements; Diaphragm flexibility; Load distribution.

More sophisticated method



Single-storey building

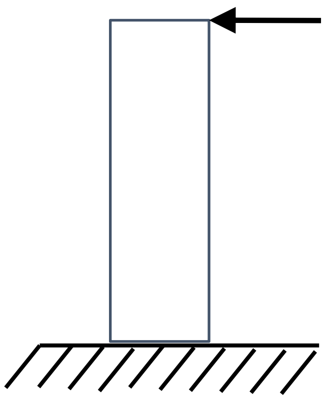


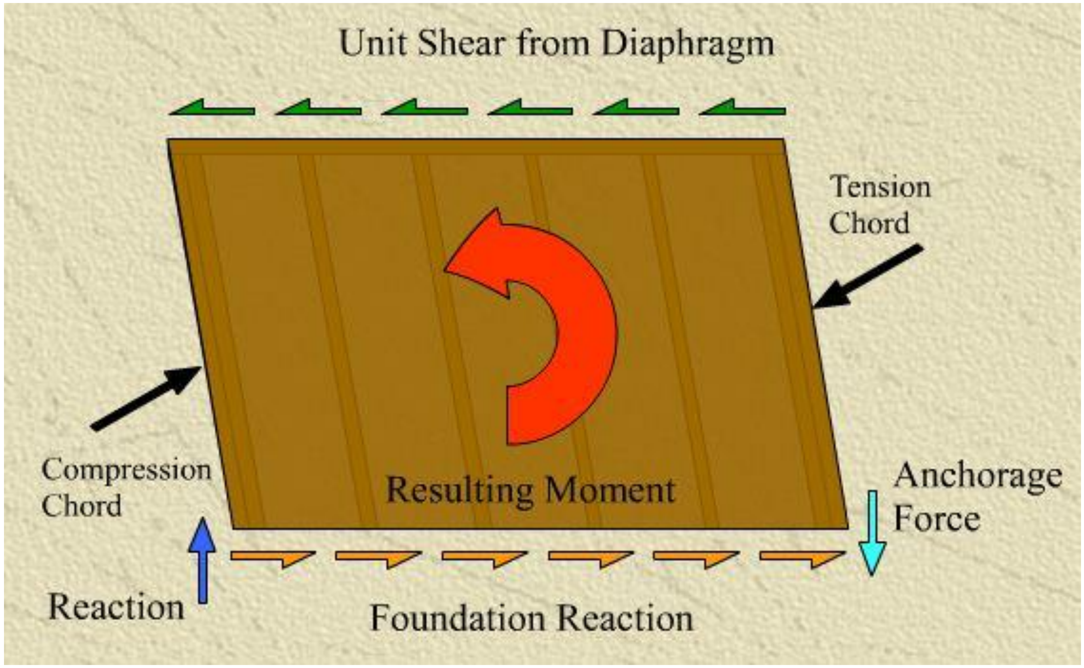
Multiple spring model for a single-storey building

Shear wall design

Vertical cantilevered member

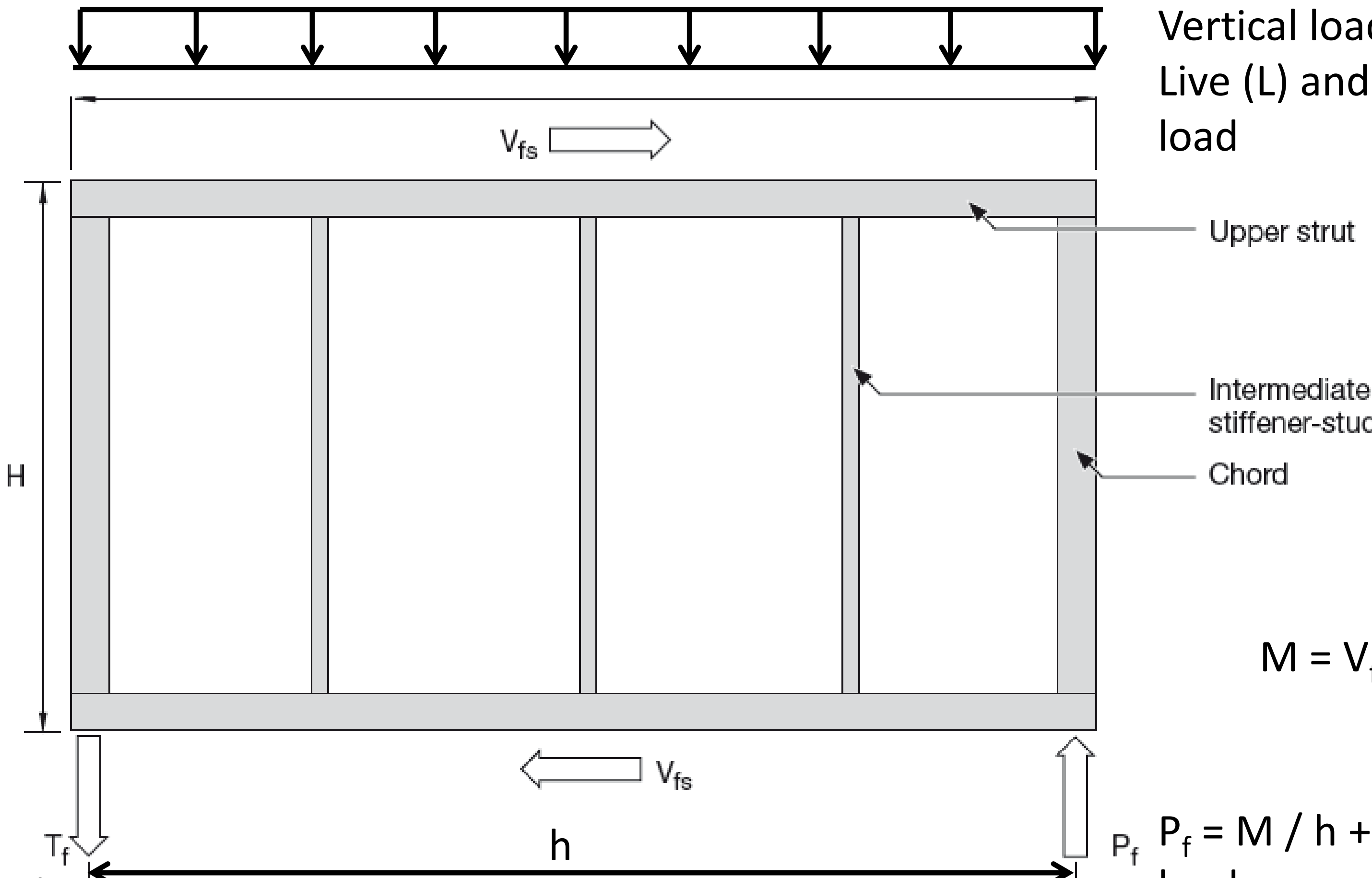
- Shear resistance:
  - Sheathing-to-framing connection
- Flexural resistance
  - Shear wall chords





Calculation of chord forces

Vertical load due to Live (L) and Snow (S) load

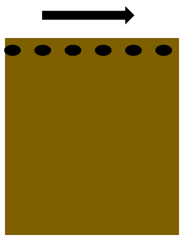


$M = V_{fs} \times H$

$P_f = M / h$ + Vertical load component

$T_f = M / h$
(allowed to subtract vertical load component)

Shear resistance of shear wall segment – sheathing-to-framing connection



11.5.1 Shear resistance of shearwalls

(b) $V_{rs} = \phi\, v_d\, J_D\, n_s\, J_{us}\, J_s\, J_{hd}\, L_s$

where

ϕ = 0.8

v_d = N_u/s, for shearwall segment sheathed with plywood or OSB, kN/m

= specified shear strength for shearwall segment sheathed with diagonal lumber sheathing (Clause 11.5.5), kN/m

n_S = number of shear planes in sheathing-to-framing connection for walls sheathed with wood panels (see Clause 11.5.3.4); n_S = 1.0 for lumber sheathing

N_u = lateral strength resistance of sheathing-to-framing connection along panel edges, per fastener (see Clause 12.9.4 for nails), N

J_D = factor for diaphragm and shearwall construction (Clause 12.9.4.1) (As for diaphragm)

J_s = fastener spacing adjustment factor (Clause 11.4.1) (As for diaphragm)

s = fastener spacing along panel edges, mm

J_{us} = strength adjustment factor for unblocked shearwalls (Clause 11.4.4)

J_{hd} = hold-down effect factor for shearwall segment (Clause 11.4.5)

L_s = length of shearwall segment parallel to direction of factored load, m

Lateral strength of nailed joint, N_u

12.9.4.1

The factored lateral strength resistance of the nail or spike connection, N_r, shall be taken as follows:

$$N_r = \phi N_u n_F n_S J_F$$

where

$\phi = 0.8$

$N_u = n_u (K_D K_{SF} K_T)$

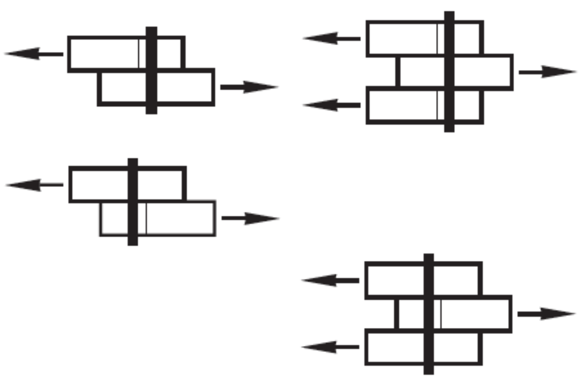
where

n_u = unit lateral strength resistance, N (Clause 12.9.4.2)

(a) $f_1 d_F t_1$

(b) $f_2 d_F t_2$

(c) $\frac{1}{2} f_2 d_F t$

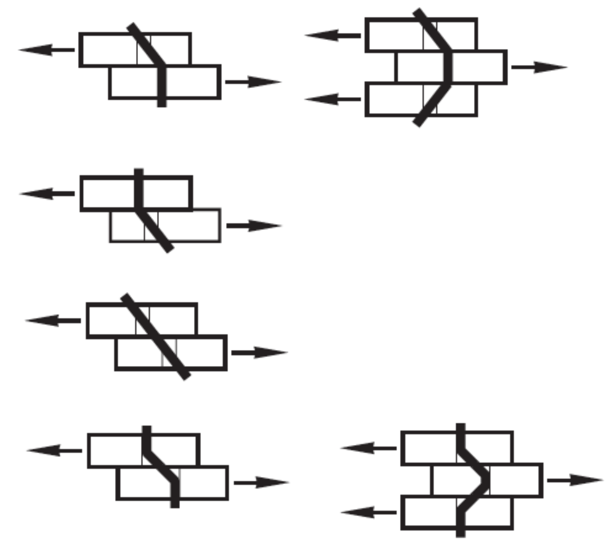


(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$

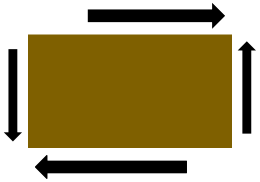
(e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$

(f) $f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$

(g) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_3}{(f_1 + f_3)} \frac{f_y}{f_1}}$



Shear resistance of shear wall segment – buckling of sheathing panel



(c) $V_{rs} = \phi\, v_{pb} K_D K_S K_T\, L_s$

where

ϕ = 0.8

K_S = service condition factor (Table 9.4.2)

v_{pb} = panel buckling strength of the most critical structural panel within the segment, kN/m

$$= K_{pb} \frac{\pi^2 t^2}{3000b} \left(B_{a,0} B_{a,90}{}^3\right)^{\frac{1}{4}}$$

where

K_{pb} = panel buckling factor

$$= 1.7(\eta + 1)\exp\left(\frac{-\alpha}{0.05\eta + 0.75}\right) + (0.5\eta + 0.8)$$

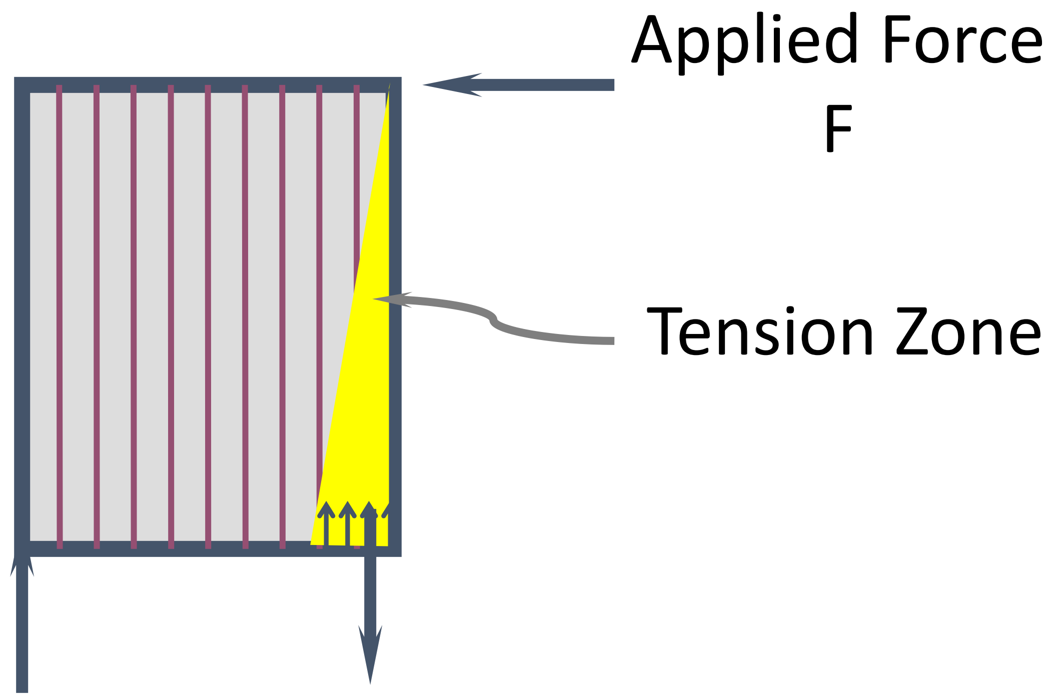
where

$$\alpha = \frac{a}{b}\left(\frac{B_{a,90}}{B_{a,0}}\right)^{\frac{1}{4}}$$

$$\eta = \frac{2\, B_v}{\sqrt{B_{a,0} B_{a,90}}}$$

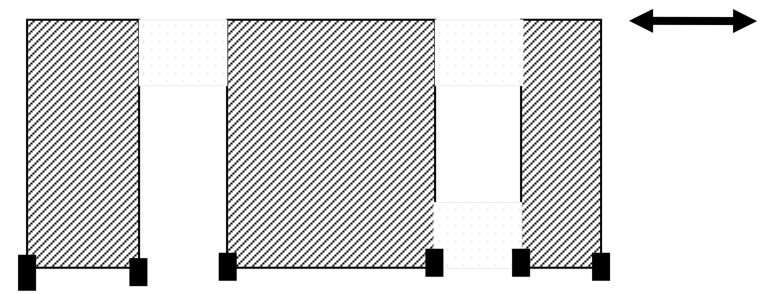
Shear wall segments without hold-downs

- Overturning tension force is resisted by sheathing panels and nails in sheathing-to-framing joints, thereby reducing shear strength of shear wall



Hold-down Effect Factor, J_{hd}

CSA O86 - 11.4.5



- **Case 1-** J_{hd} = 1 if hold-downs are designed to resist all of the tension forces due to overturning
- **Case 2-** if there are no hold downs at either end

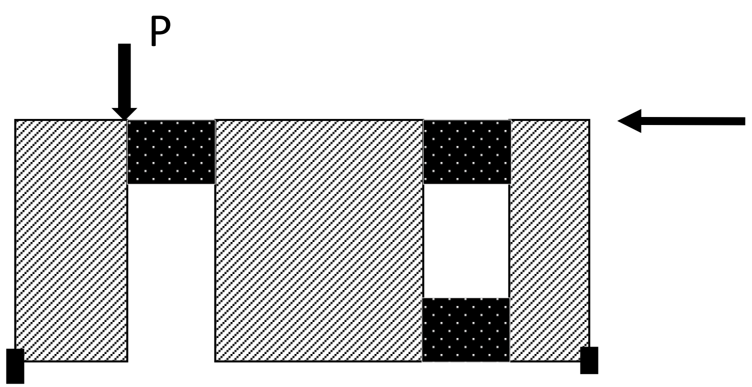
$$J_{hd} = \sqrt{1 + 2\frac{P_{ij}}{V_{hd}} + \left(\frac{H_S}{L_S}\right)^2} - \frac{H_S}{L_S} \leq 1.0$$

P_{ij} = factored uplift restraint force for storey i at bottom of end stud of shear wall j
V_{hd} = factored shear resistance of the shear wall assuming J_{hd} = 1
H_s and L_s are height and length of shear wall

- **Case 3-** If a lower storey segment is held at the bottom but not the top, and there is uplift restraint force at the top of end stud

$$J_{hd} = \frac{V_{hd} + P}{V_{hd}} \leq 1.0$$

P= uplift restraint force at top of end stud of shear wall (-ve)



Modification factors for shear wall

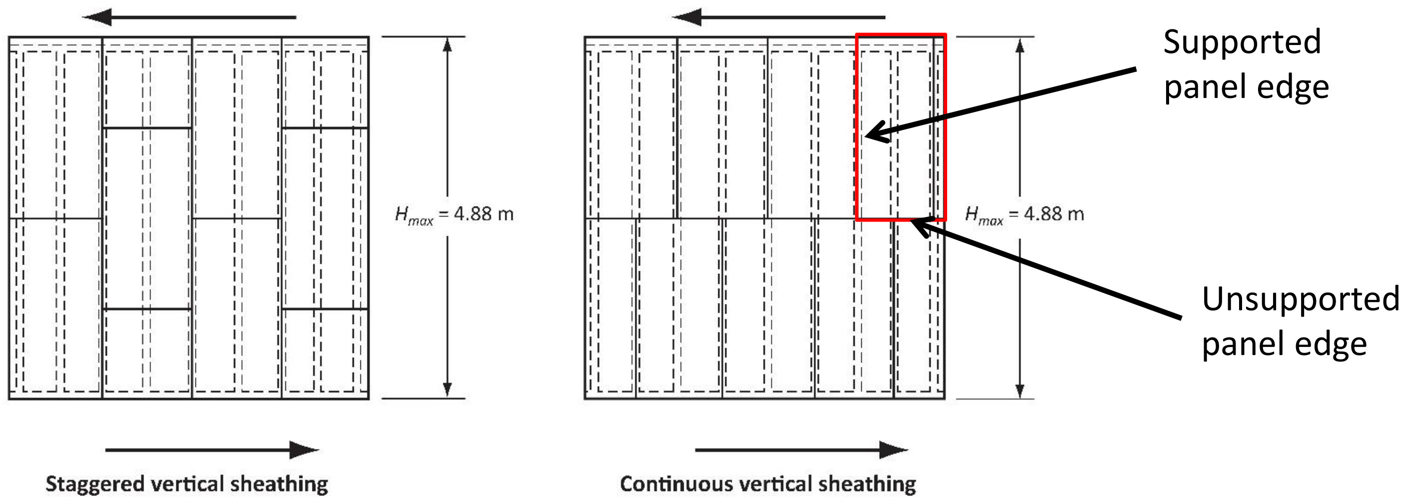
J_{us} = Strength adjustment factor for unblocked shear wall (Cl. 11.4.4)

Unblocked shearwall does not have all sheathing panel edges supported

Table 11.4.4
Strength adjustment factor, J_{us}, for unblocked shearwalls*

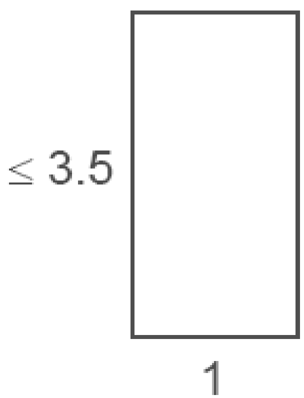
Fastener spacing at supported edges, mm	Fastener spacing at intermediate studs, mm	Stud spacing, mm			
		300	400	500	600
150	150	1.0	0.8	0.6	0.5
150	300	0.8	0.6	0.5	0.4

**The shear strength of an unblocked shearwall shall be calculated by multiplying the strength adjustment factor by the specified shear strength of a blocked shearwall with fasteners spaced at* 150 mm *on centre along panel edges and* 300 mm *on centre along intermediate framing members.*

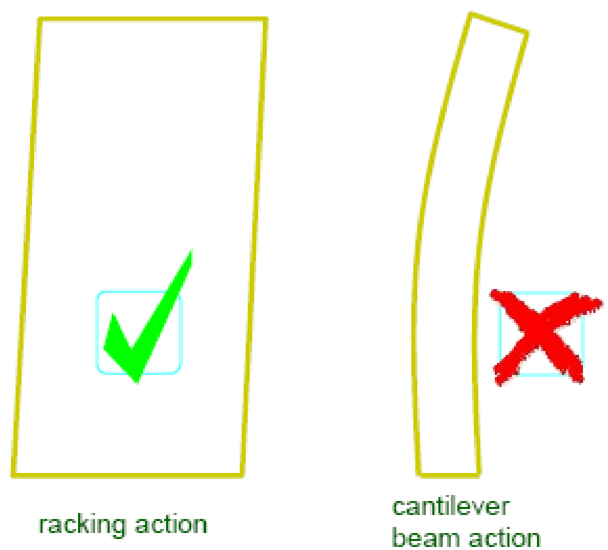


Shear wall aspect ratio

- Blocked shear wall
  - Maximum aspect ratio (height-to-length ratio): 3.5:1
- Unblocked shear wall
  - Maximum aspect ratio: 2:1
  - H≤4.88 m



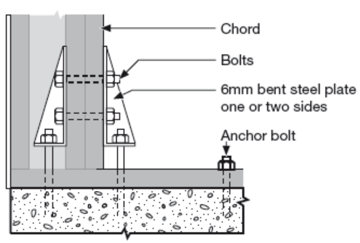
Blocked



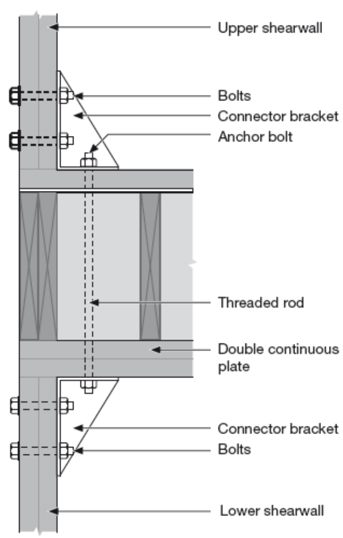
To prevent slender shear wall!!

Other design checks for shear wall

- End stud axial forces caused by applied bending moment: T_f and P_f (according to provisions for lumber tension and compression members)
- Deflection of shear wall (Cl. 11.7.1)
- Connection to supporting diaphragm or foundation
- Hold-down connection at end of shear wall segment (hold-down connectors are proprietary products)



Shearwall to foundation

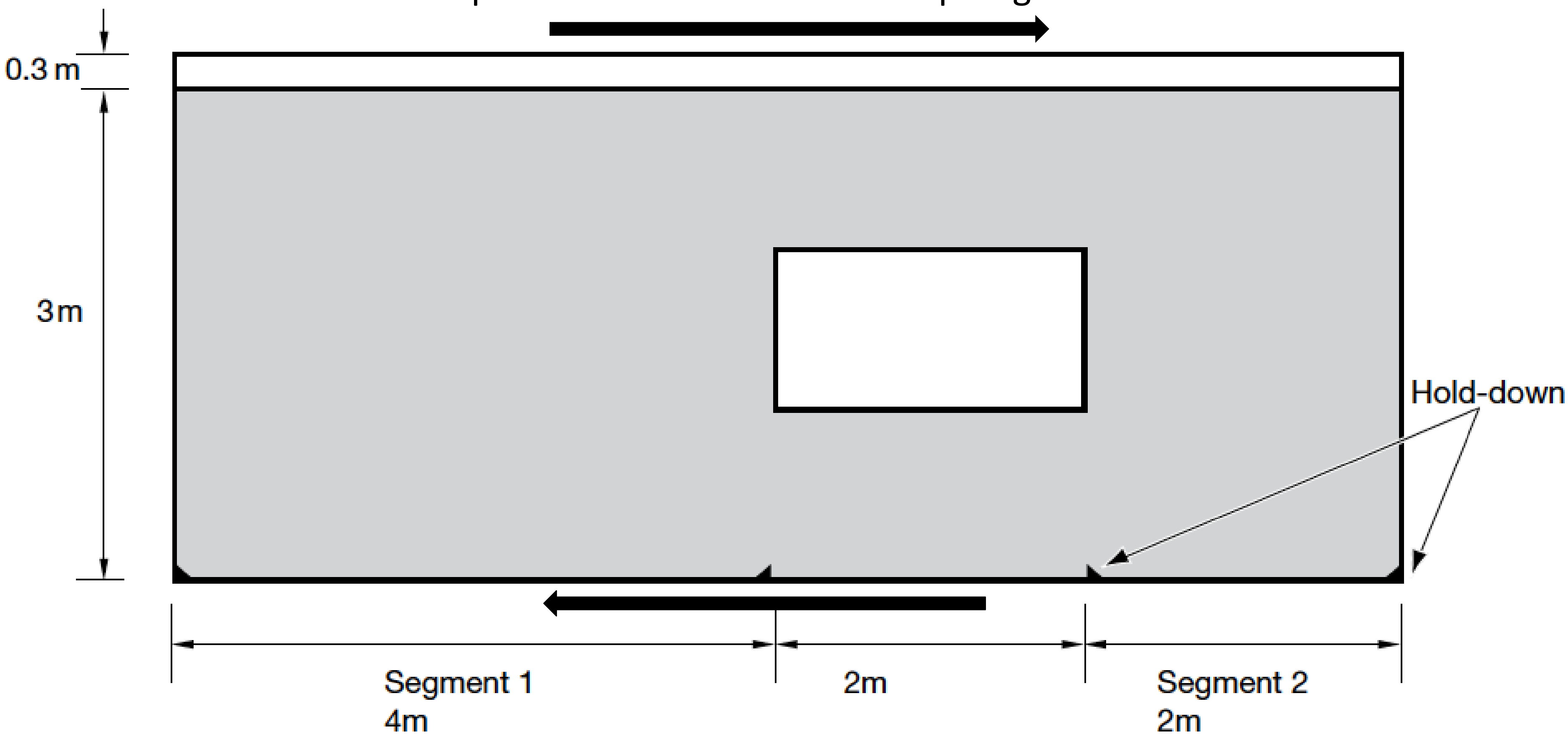


Shearwall to shearwall

Design Example – Shear wall

- Wind load only
- Dry service condition
- Hold-down at all corners

Specified shear force from diaphragm = 30 kN



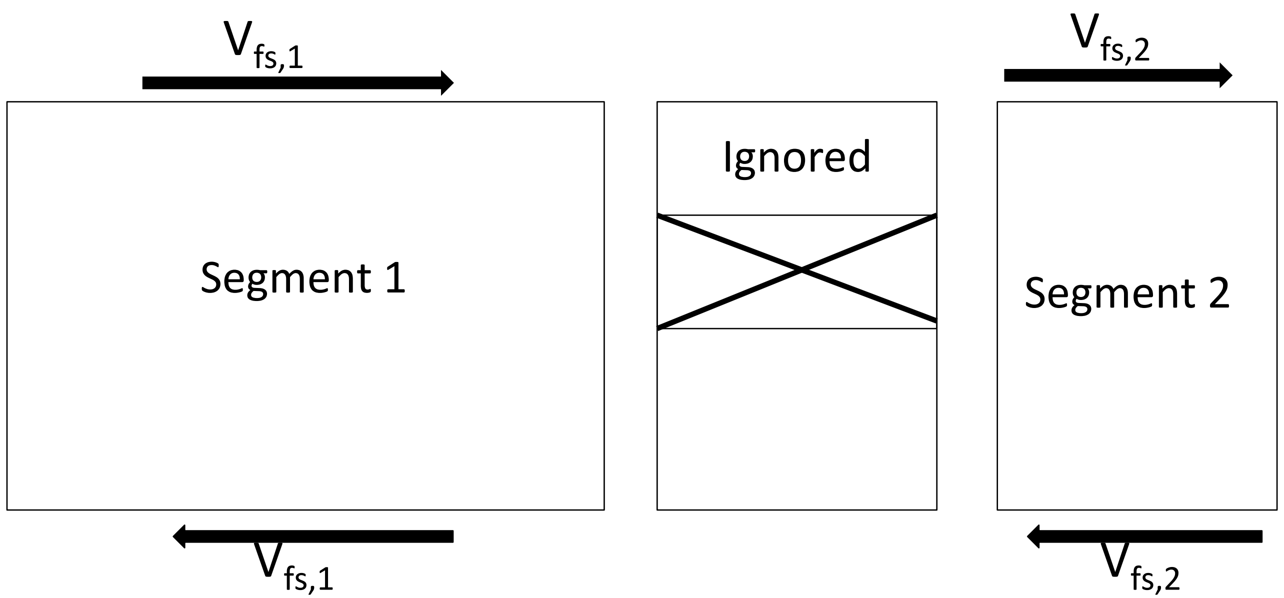
Factored shear wall force due to wind, V_{fs} = 1.4 x 30 kN = 42 kN

Distribution of shear force to shear wall segments

- Assumed to be proportional to strength of shear wall
- If segments 1 and 2 have the same construction $v_{rs,1} = v_{rs,2}$ (kN/m unit strength)
- Since strength proportional to length:
  - $V_{fs,1} = 28$ kN, $V_{fs,2} = 14$ kN

$$V_{fsj} = V_{fs} \times \frac{V_{rs,j}}{\sum V_{rs,j}}$$

$$V_{rs,j} = v_{rs,j} \times L_s$$



Trial shear wall details

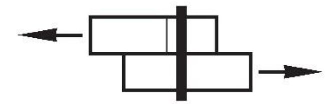
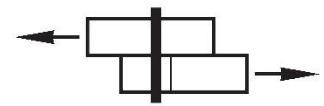
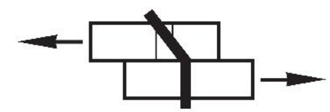
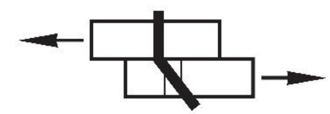
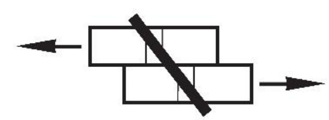
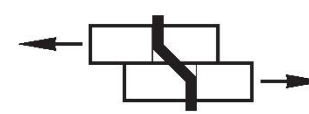
- Sheathing panel : 12.5mm Canadian Softwood Plywood (4-ply)
- Stud : spruce-pine-fir 38mm x 140mm at 600mm spacing
- Nail : 3mm diameter, 63mm length
- Nail spacing at panel edge : 75mm
- Blocked shear wall
- Hold-down at all corner locations

Calculation of nail joint strength, N_u – Clause 12.9.4

- Calculate n_u

d_F = 3mm
t_1 = 12.5mm
t_2 = 63 – 12.5 = 50.5mm
G = 0.42 for sheathing panel (CSP)
G = 0.42 for lumber stud (SPF)
J_X = 1.0 (not CLT)
f_1 = 30.6 MPa
f_2 = 20.4 MPa
f_3 = 22.4 MPa
f_y = 650 MPa

$$N_u = n_u\,(K_D\,K_{SF}\,K_T) = 566\text{ N}$$

Failure load			Failure mode
1146 N	(a)	$f_1 d_F t_1$	
3086 N	(b)	$f_2 d_F t_2$	
Govern $\boxed{566\text{ N}}$	(d)	$f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F} \right)$	
1263 N	(e)	$f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_2}{d_F} \right)$	
846 N	(f)	$f_1 d_F^2 \frac{1}{5}\left(\frac{t_1}{d_F} + \frac{f_2}{f_1}\frac{t_2}{d_F} \right)$	
673 N	(g)	$f_1 d_F^2 \sqrt{\frac{2}{3}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}}$	

Shear wall strength, V_{rs} based on joint failure – Clause 11.5.1 (b)

- For segment 1 (b) $V_{rs} = \phi v_d J_D n_s J_{us} J_s J_{hd} L_s$

  $\phi = 0.8$

  $s = 75$ mm

  $n_s = 1$

  $J_D = 1.3$

  $J_s = 1 - ((150 - s)/150)^{4.2} = 0.95$

  $J_{hd} = 1$

  $v_d = N_u / s = 7.46$ kN/m

  $L_s = 4$ m

$V_{rs,1} = 0.8 \times 1.3 \times 0.95 \times 7.46 \times 4\text{m} = 29.5\text{kN} > V_{fs,1} = 28\text{kN}$ Ok

Similarly for segment 2, $V_{rs,2} = 2\text{m} \times 7.37 \text{ kN/m} = 14.8 \text{ kN} > V_{fs,2} = 14 \text{ kN}$ Ok

Shear wall strength, V_{rs} based on panel buckling – Clause 11.5.1 (c)

- For segment 1

$$\text{(c)} \quad V_{rs} = \phi\, v_{pb} K_D K_S K_T\, L_s$$

$\phi = 0.8$

$t = 12.5$ mm

$K_D = K_S = K_T = 1$

$L_s = 4$ m

$a = 2440$mm

$b = 1220$ mm

See next slide

$B_{a,0} = 55{,}000$ N/mm

$B_{a,90} = 57{,}000$ N/mm

$B_v = 5{,}700$ N/mm

$\alpha = 2.02$

$\eta = 0.20$

$V_{pb} = 24.87$ kN/m

$V_{rs,1} = 0.8 \times 24.87 \times 4\text{m} = 79.6\text{kN}$

$> \; V_{fs,1} = 28$ kN

Similarly for segment 2,

$V_{rs,2} = 2\text{m} \times 19.9$ kN/m = 39.8 kN $\quad > \; V_{fs,2} = 14$ kN

Buckling ok

Sheathing panel design properties

Table 9.3B
Specified strength, stiffness, and rigidity capacities for standard constructions of regular grades of unsanded Canadian softwood plywood (CSP)

Nominal thickness, mm	No. of plies	Bending stiffness, $B_b = EI$, N•mm²/mm		Axial stiffness (in tension or compression), $B_a = EA$, N/mm		Shear-through-thickness rigidity, B_v, N/mm
		Orientation of applied force relative to face grain				
		0°	90°	0°	90°	0° and 90°
7.5	3	340 000	17 000	55 000	24 000	3 400
9.5	3*	610 000	27 000	55 000	28 000	4 300
12.5	3	1 400 000	79 000	81 000	39 000	5 700
	4*	1 300 000	190 000	55 000	57 000	5 700
	5	1 400 000	350 000	79 000	47 000	5 700

What else need to be checked?

CLT Lateral Load Resisting System Design (11.9)

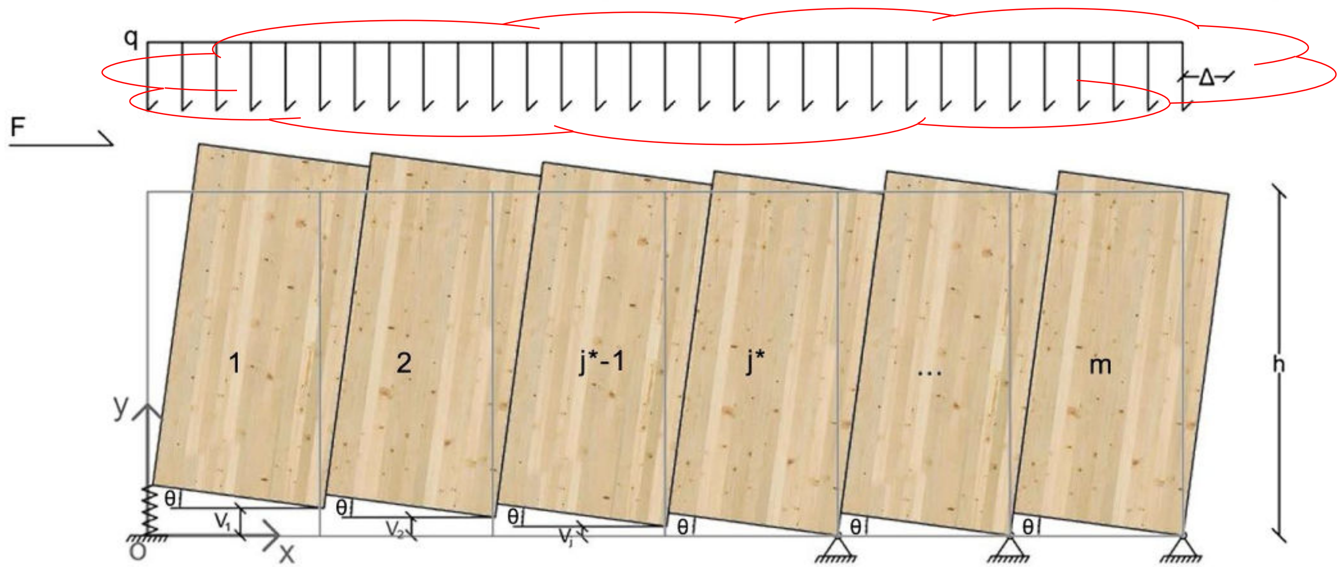
Shear wall and Diaphragm

11.9.1.1

Clause 11.9 shall apply to platform-type constructions not exceeding 30 m in height. For high seismic zones (i.e., $I_E \, F_a \, S_a(0.2) > 0.75$), the height shall be limited to 20 m. Alternative systems shall be designed in accordance with Clause 4.3.2 of this Standard and *NBC* subsection 4.1.8.

11.9.1.2

The factored shear resistance of CLT shearwalls shall be governed by the resistance of connections between the shearwalls and the foundations or floors, and connections between the individual panels, calculated using methods of mechanics, assuming each individual panel acts as a rigid body.



Seismic Resistance

11.9.2.1 General

Factors $R_d \leq$ 2.0 and R_o = 1.5 shall apply to platform-type CLT structures where the energy is dissipated through connections as specified in Clause 11.9.2.2 following the capacity design principles given in Clause 11.9.2.4, and wall panels act in rocking or in combination of rocking and sliding. Type 4 or 5 irregularities as defined in the *NBC* shall not be allowed. Other types of irregularities shall be dealt with in accordance with the *NBC*. CLT structures with wall panels with aspect ratios (height-to-length) less than 1:1 or acting in sliding only shall be designed with $R_d R_o$ = 1.3.

Note: *See the CWC* Commentary on CSA O86 *for further information.*

- R_d R_o= 3.0 if connection is able to dissipate energy (ductile) and capacity-based design is adopted
- By comparison, for wood shear wall R_d R_o=5.1

Seismic Resistance

11.9.2.2 Energy dissipative connections

Energy dissipative connections of CLT structures shall satisfy all of the following requirements:

(a) connections shall be designed so that a yielding mode governs the resistance;

(b) connections shall be at least moderately ductile in the directions of the assumed rigid body motions of CLT panels; and

(c) connections shall possess sufficient deformation capacity to allow for the CLT panels to develop their assumed deformation behaviour, such as rocking, sliding, or combination thereof.

Note: *For further information on moderately ductile connections, see the CWC* Commentary on CSA O86*.*

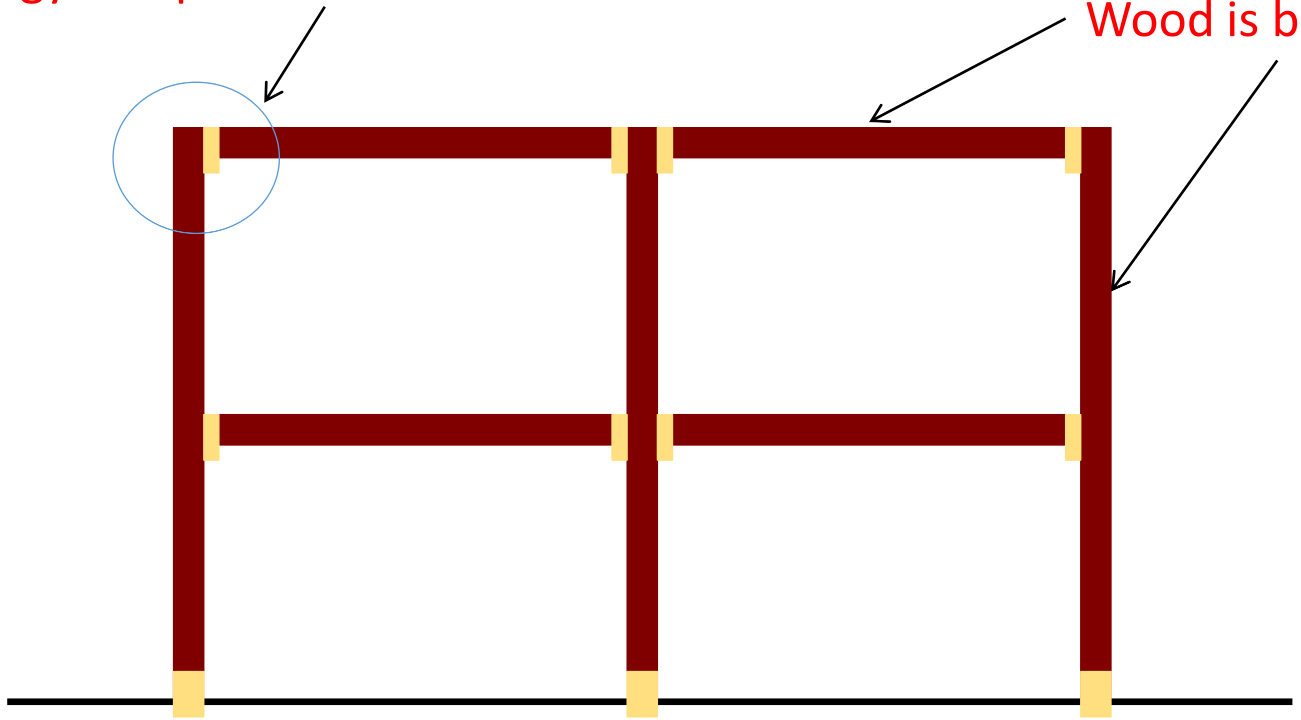
Information/data required to to allow designers to detail energy dissipative connections in CLT:

a) Yield vs brittle mode of failure is only defined for bolts and dowels in O86
b) How to define moderately ductile?
c) Deformation characteristics on connection are required

Capacity-based design

Mechanical connections are the only source of energy dissipation in timber structures

Wood is brittle



- Detail mechanical connections to fail by yielding
- Could also detail certain connection (e.g. beam-column) to yield before other connection (e.g. base)

Capacity-based design

11.9.2.4 Capacity design principles

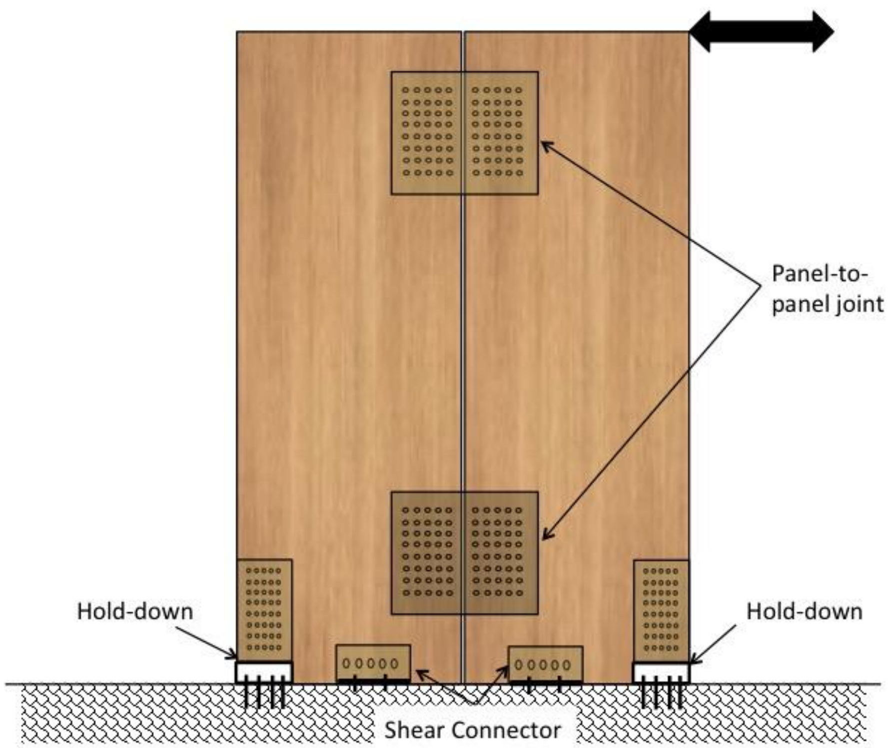
11.9.2.4.1 General

The following capacity design principles shall be used for seismic design of CLT structures. All inelastic deformations and energy dissipation shall occur in

(a) vertical joints between the panels in shearwalls;

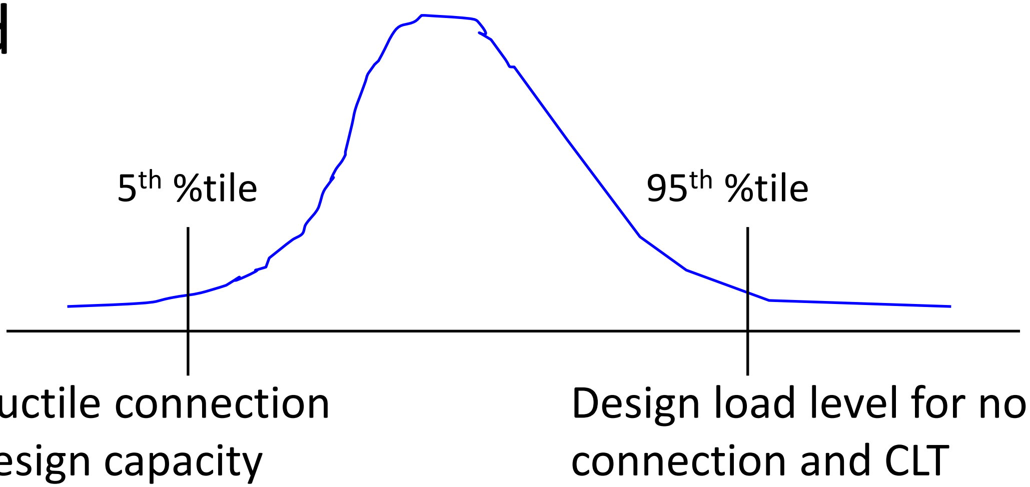
(b) shear connections between the shearwalls and the foundations or floors underneath; and

(c) hold-down connections, except for continuous steel rods (Clause 11.9.3).



Capacity-based design

Non-dissipative connections and CLT elements (i.e. brittle elements) shall be designed for a higher load to ensure **target** ductile connection failure in the event of overload



11.9.2.3 Non-dissipative connections

Non-dissipative connections shall be designed to remain elastic under the force and displacement demands that are induced in them when the energy-dissipative connections reach the 95th percentile of their ultimate resistance or target displacement, in accordance with engineering principles of equilibrium and displacement compatibility. The seismic design force need not exceed the force determined using $R_d R_o = 1.3$.

End
Lecture #5

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