Lecture #6 – Design of Connections

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Design of connections I

- Introduction of fasteners/connectors and fastening systems
- Design of connections with light fasteners

Connections in Timber Structures

- Connections are often the weakest components of a timber structure
- Timber connections capable of transferring large loads are a challenge to designers
- For heavy timber structures, member sizes can be governed by connection size

Factors for selecting connection design

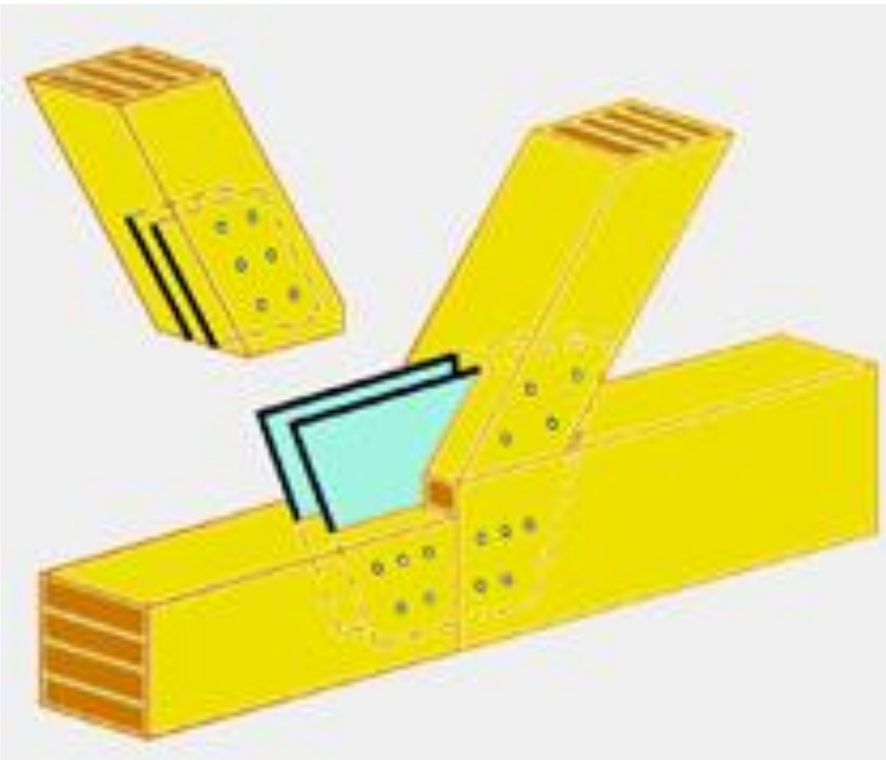
- **Cost**
- Structural – strength, stiffness and ductility
- Availability of fasteners
- Ease of fabrication
- Aesthetics
- Fire – exposed steel plates
- Durability
- Acoustics

Fabrication was a challenge for bolted connections!

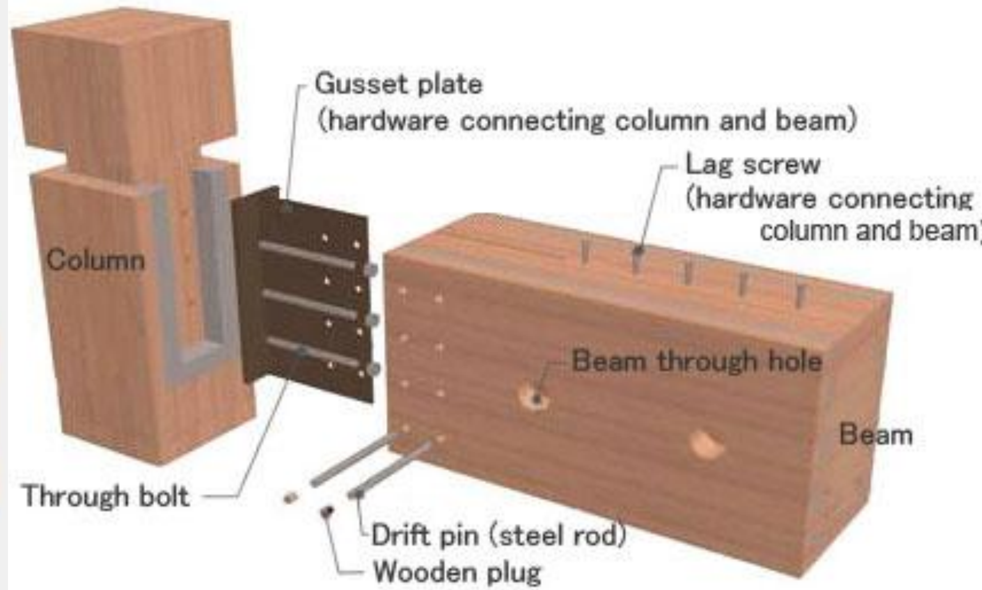
Fitting problems:

- Wood dimensions vary
- Precision drilling was difficult
- Large hole tolerance (2mm)

Precision machining is now possible



Precision - fraction of mm



Metal fitting at job site – sliding / slotting connection

Computer Numerical Control (CNC) Machines

- Machine can saw, drill, rout with mm or lower precision
- Revolutionized heavy timber construction

Mechanical Connections

- Nails & Spikes
- Bolts & Dowels
- Screws – lag and wood
- Timber connectors
- Timber rivets
- Truss plates
- Joist hangers
- Proprietary fasteners

Nails

Common fastener in residential construction

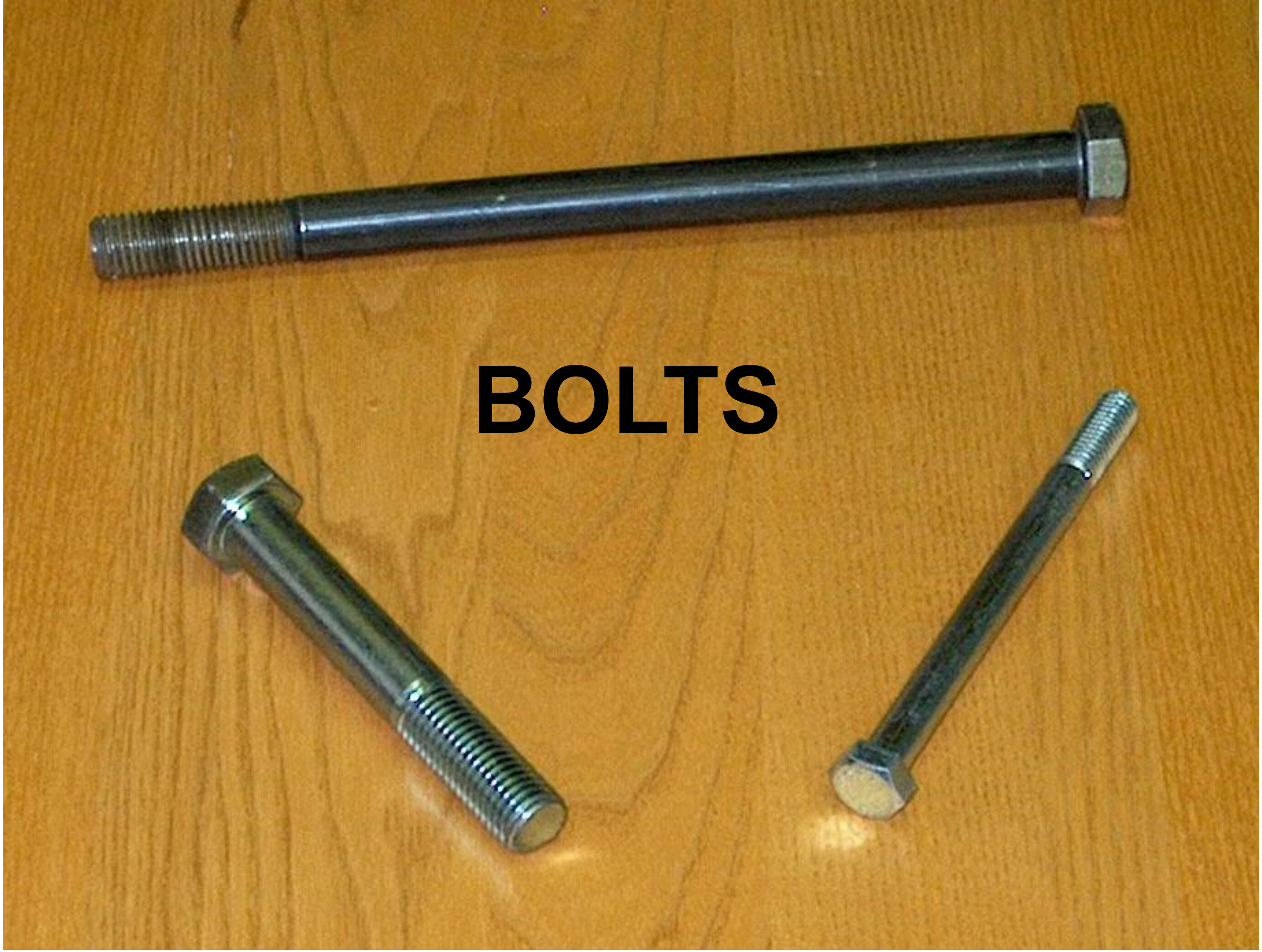
Nails (and spikes)

- Easy to apply on site
- Common diameters up to 4mm
- Transfer roughly 0.5 - 2 kN lateral design load per nail
- Simple tool required
- No need for pre-drilling
- Flexible, leading to ductile structures

Excellent seismic performance

Wood screws

- Similar to nails/spikes, and used where withdrawal strength is required
- Slower application than nails
- Common diameters up to 5mm
- Transfer roughly 0.5 - 2 kN lateral design load per screw
- No need for pre-drilling
- Flexible
- Could be brittle due to large tensile stress under cyclic load


BOLTS

Bolts

- Commonly used diameters are 6mm - 25mm, but can be up to 38mm
- Capable of transferring large load, roughly 2 kN – 30 kN per bolt
- Require precision pre-drilling
- Traditionally the only fastener used in heavy timber construction
- Small diameter bolts (6mm) behave in a ductile manner (yielding of fastener), but large bolts (>6mm) generally fail wood in a brittle manner (cracks in wood)

Wood-Wood-Wood connection

Wood-Steel-Wood & Steel-Wood-Steel Connections

Reasons for choice?

Steel dowels

- Smooth round steel pins with slightly reduced diameter near the ends
- Tight-fit without tolerance
- Designed as bolts

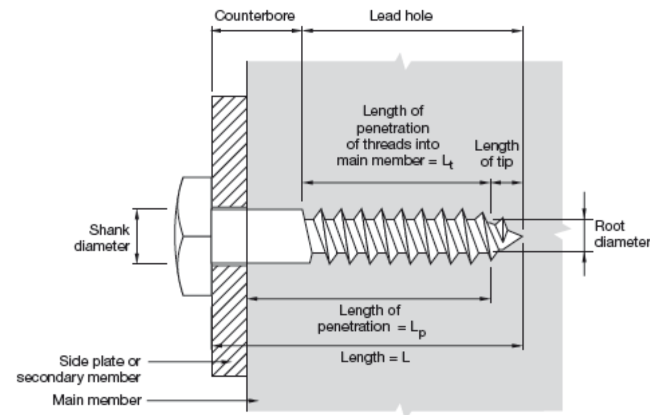
Dowel vs Bolt?

Wood-Steel-Wood Connection (Dowel)

Lag Screws

Lag Screws

- Transfer lateral load similar to bolt of similar size
- Installed through one face of a connection (preferred to bolt)
- Diameter ranged from 6mm – 25mm, length 3" – 12"
- Pre-drilling is not required, but lead hole is required especially for large screw and dense wood
- High withdrawal strength



Richmond Olympic Oval, Vancouver

TIMBER CONNECTORS

Split ring

Shear plate

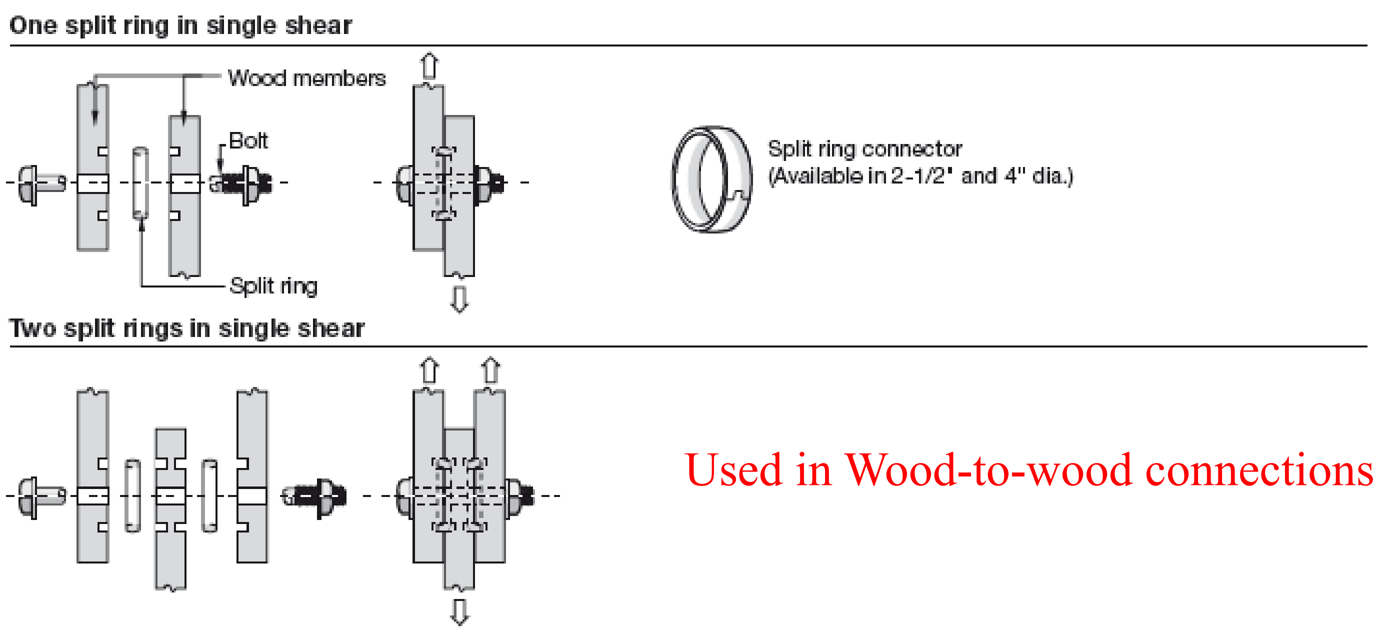

Split ring
A split ring transfers the load but needs a bolt or rod to keep the members together

Shear plate

Need bolt or rod to transfer load

Split ring

- Round connector (2.5” or 4”) to provide a large diameter connection on wood surface
- Could transfer 20 kN – 50 kN lateral design load per connector
- Special tool to cut grooves in two jointing members
- A bolt or lag screw is used to draw members together

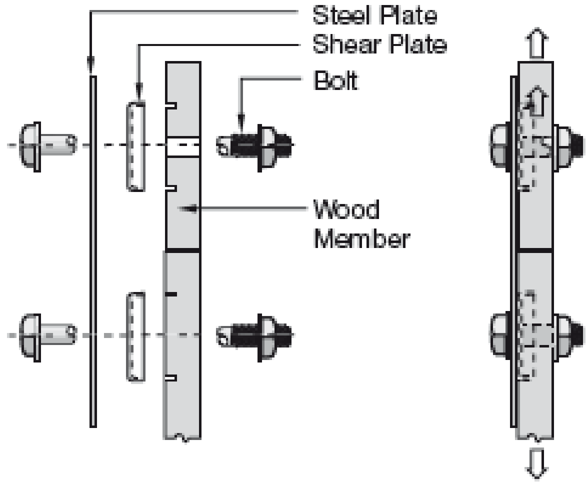


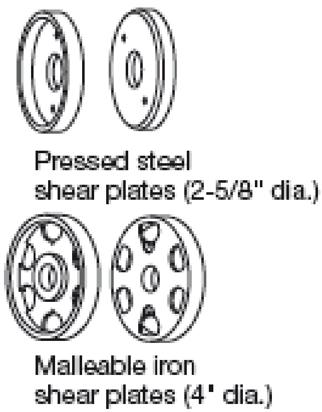
Used in Wood-to-wood connections

Shear plate

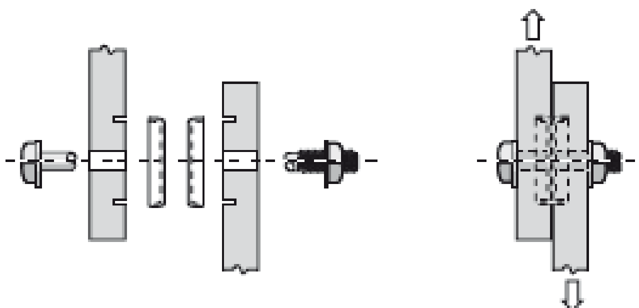
- Round connector (2-5/8” or 4”) to provide a large diameter connection on wood surface
- Could transfer 20 kN – 40 kN lateral design load per connector
- Special tool to cut grooves in wood jointing members
- A bolt or lag screw is used to draw members together

One shear plate in single shear



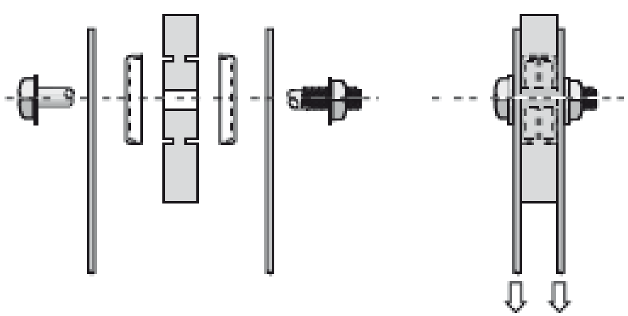


Two shear plates in single shear



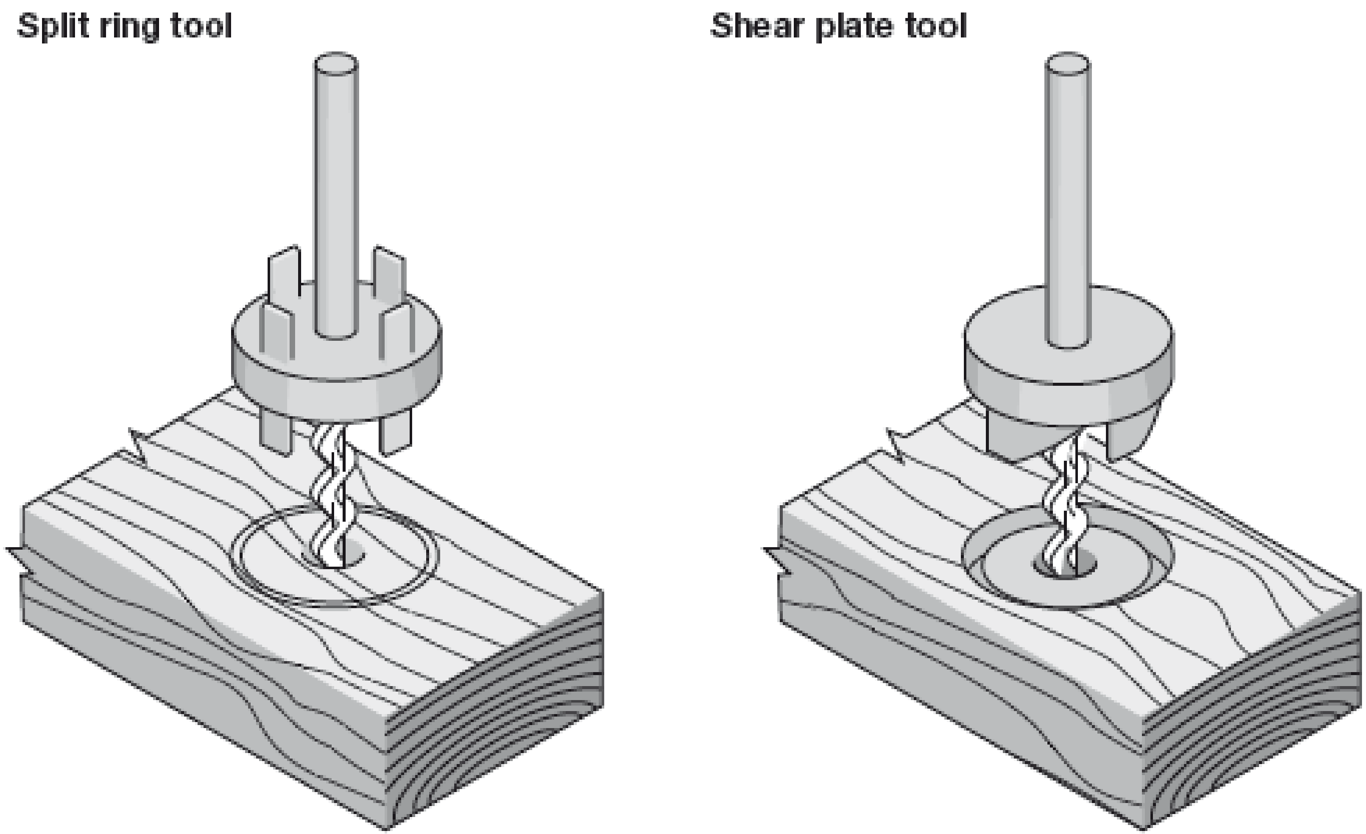
Used in Wood-to-wood or Wood-to-Steel connections

Two shear plates, each in single shear



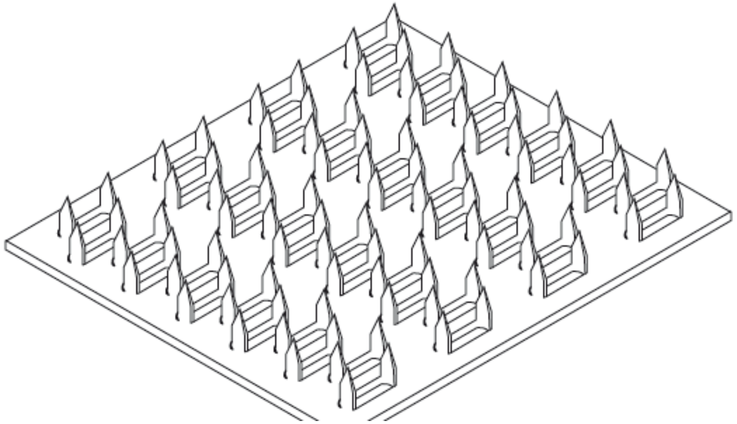
Shear plate is often attached to wood members in factory

Cutting of grooves for split ring and shear plate



Truss plates

- Light-gauge galvanized steel plate with press-out teeth
- Used in lumber truss fabrication primarily for floor and roof applications
- Strength is proportional to area of embedment
- Proprietary products - design properties are published by manufacturers



Timber Rivets

Tight-fitting through wedging action of head into steel

Simple tool, no predrilling

Strong and stiff connection – high load transfer per connection area

- Canadian invention in 1960's

Timber Rivets

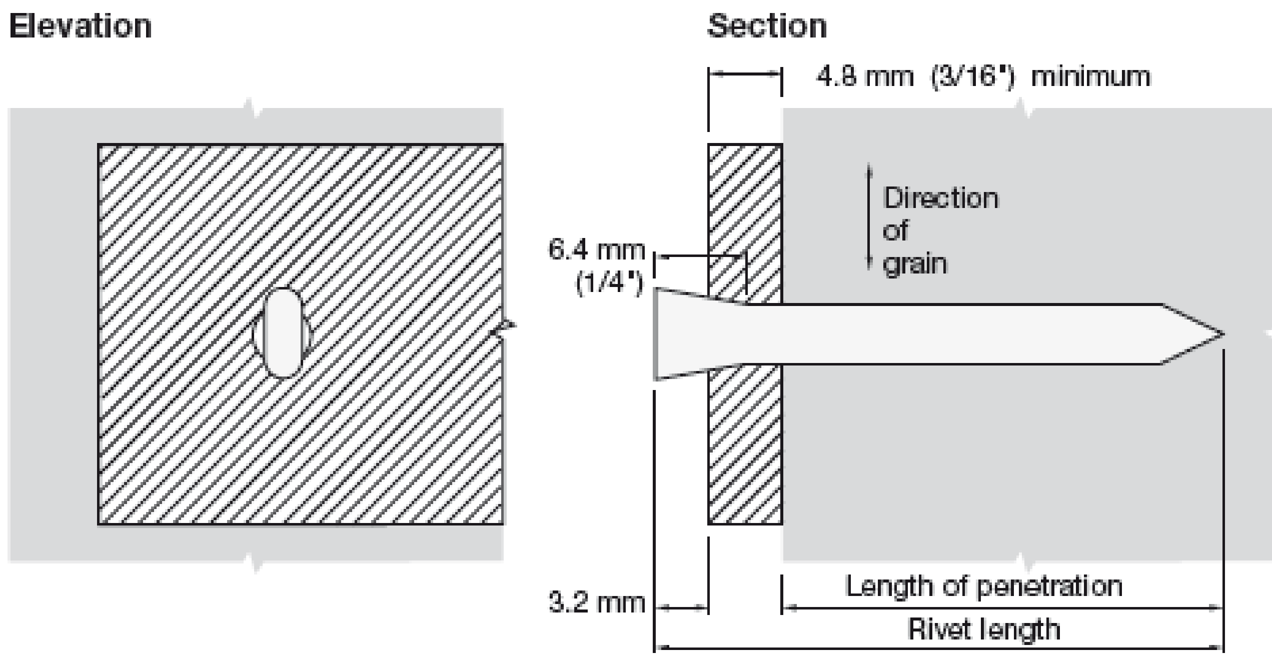
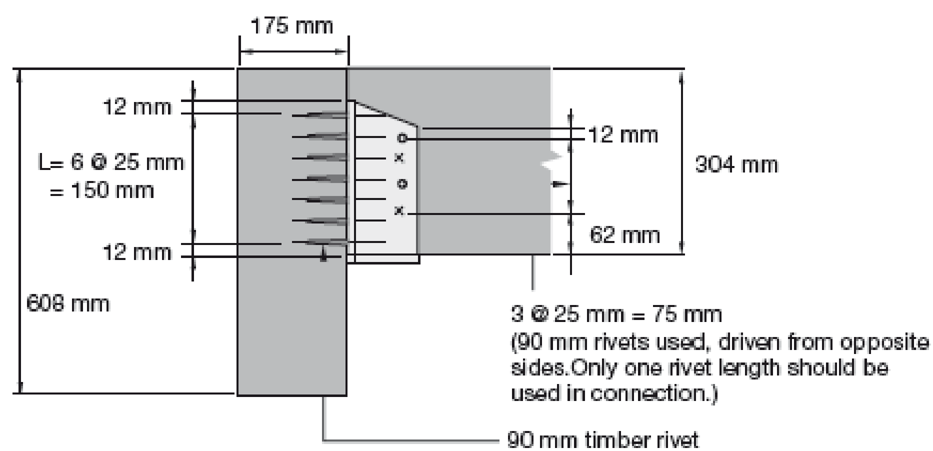
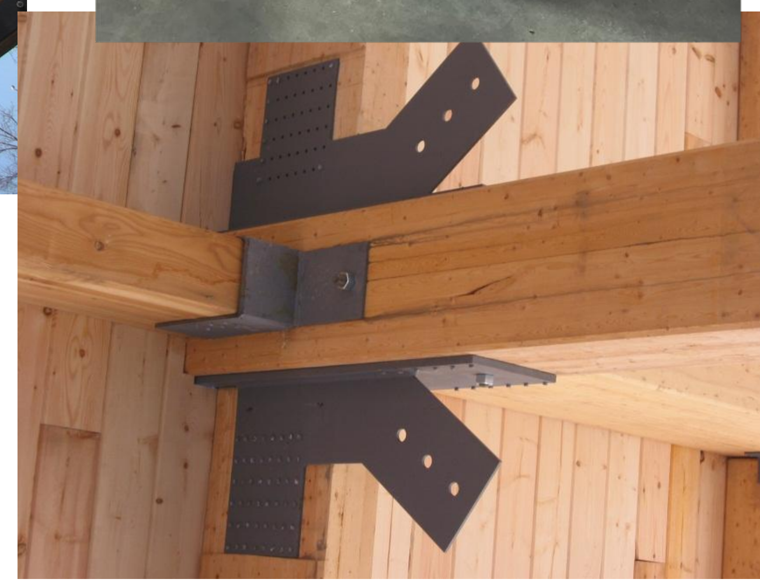


FIGURE 11.24

Beam hanger connection using timber rivets



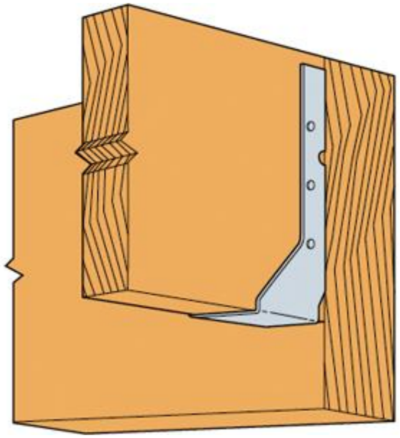


Timber rivets in a steel-wood connection

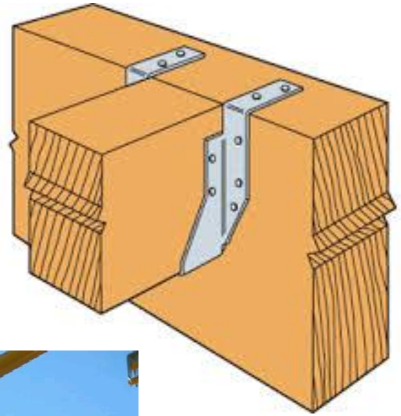
Joist hangers

Joist hangers

- Made from light-gauge galvanized steel
- Support joists in floor and roof
- Could be face-mount or top-mount
- Proprietary products with design properties specific to the manufacturer
- Heavy steel required for higher load transfer

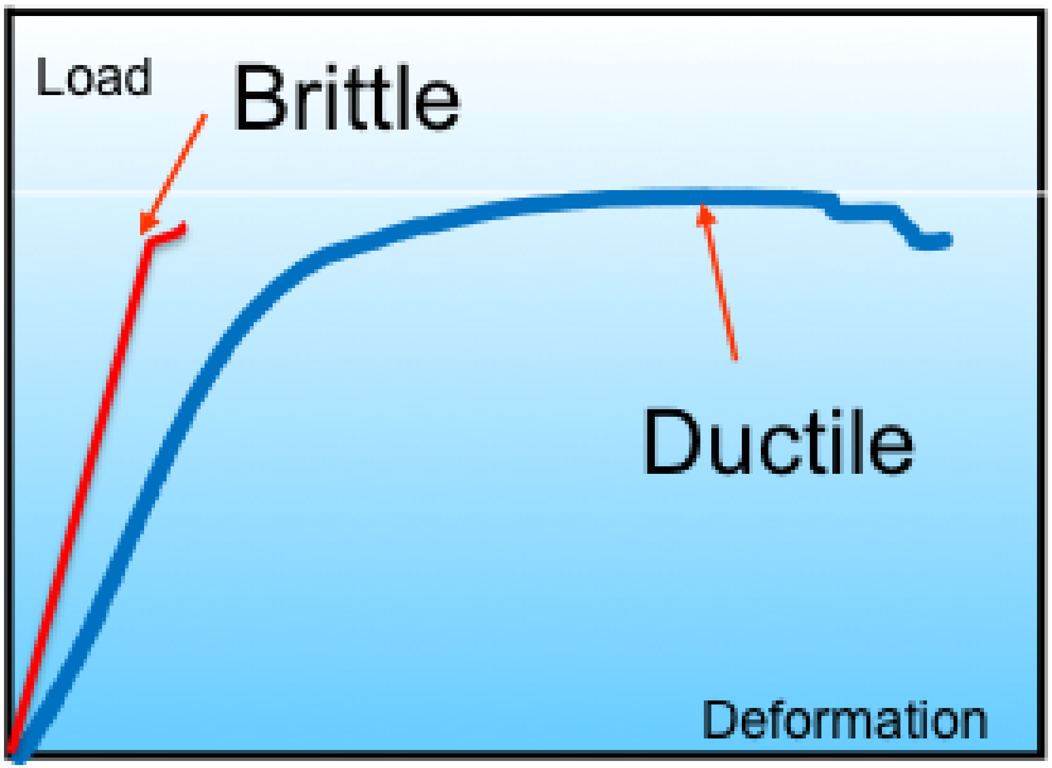


Face-mount



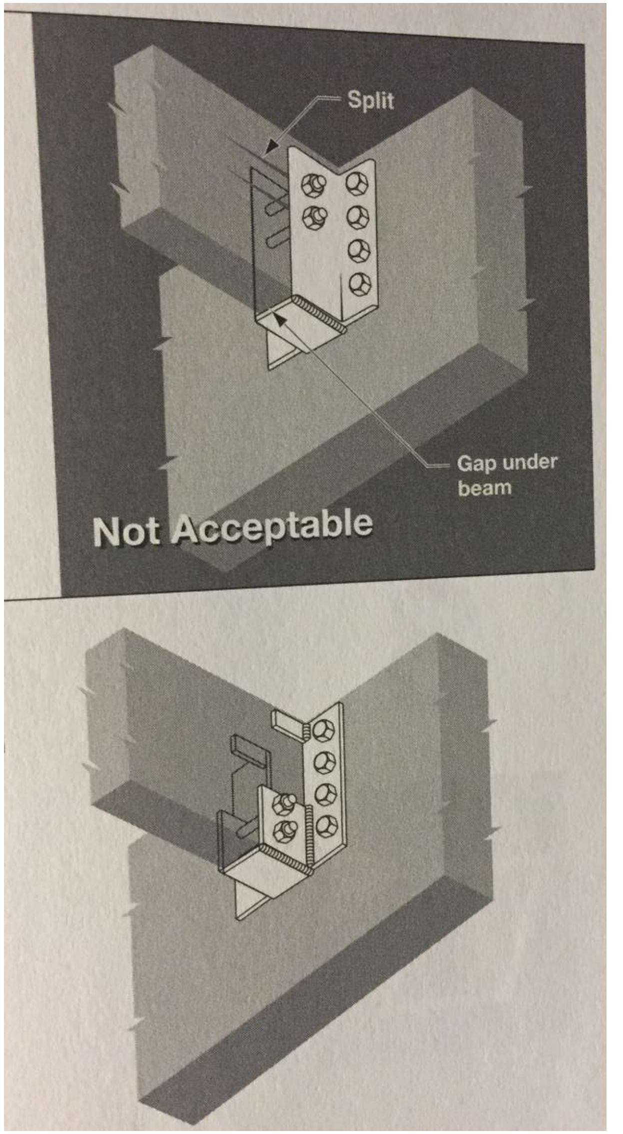
Top-mount

Slender fasteners, such as nails, wood screws and timber rivets, can lead to ductile connections - Preferred



The other types of connections <u>are brittle</u> and may cause sudden failures.

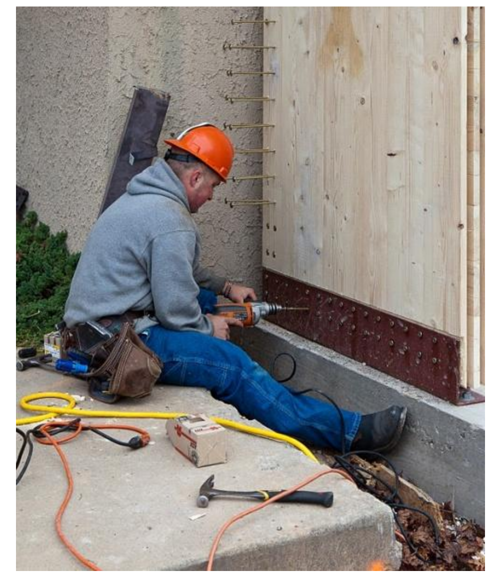
Avoid details which would result in tension perpendicular to grain under shrinkage



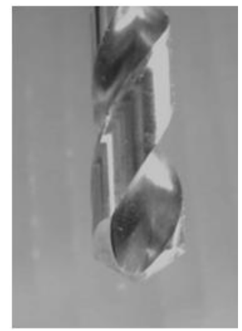
Innovative/Proprietary Connection Systems

Self-tapping Screws

- Proprietary fasteners with high yield strength > 800 MPa
- Self-tapping because the shaft is designed to act like a drill bit – no need for pre-drilling and fast installation



Various shank profiles by manufacturers



Drill bit

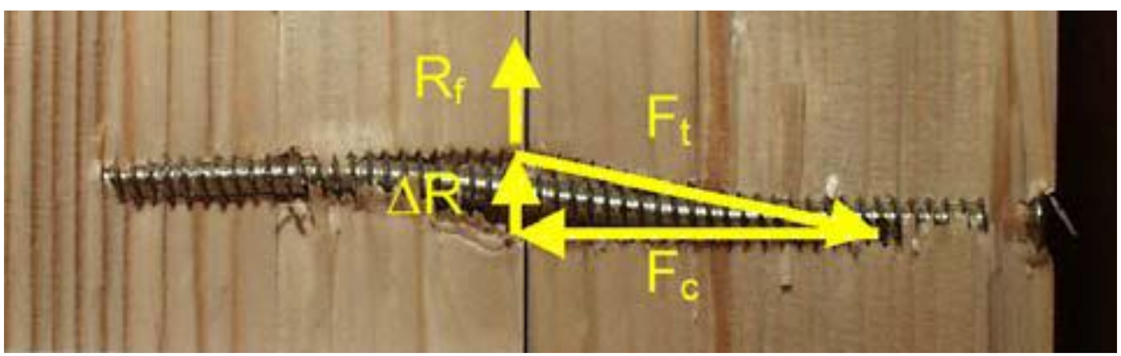
- Available in a variety of sizes and features
  - Diameter up to **<u>12mm</u>**
  - Length up to **<u>900mm</u>**
  - Full or partial thread

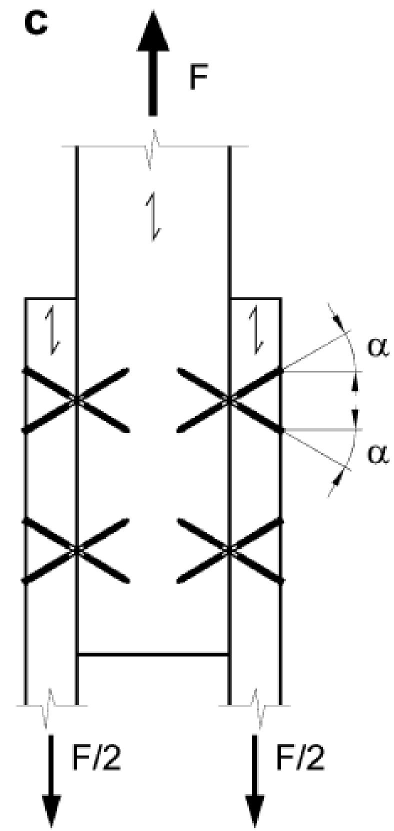
Fastener that has become the choice for fabricators and designers!!!

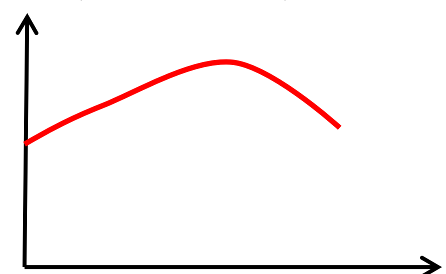
Source: Kevin Meechan
Courtesy: WoodWorks

Self-tapping Screws

- Lateral resistance can be improved by over 30% and stiffness increased by 3 to 5 times if the fasteners are inclined at 30° – 45°, due to the role played by withdrawal resistance of the thread







Glued connection systems

- Glued-in rod

Developed strength through withdrawal resistance

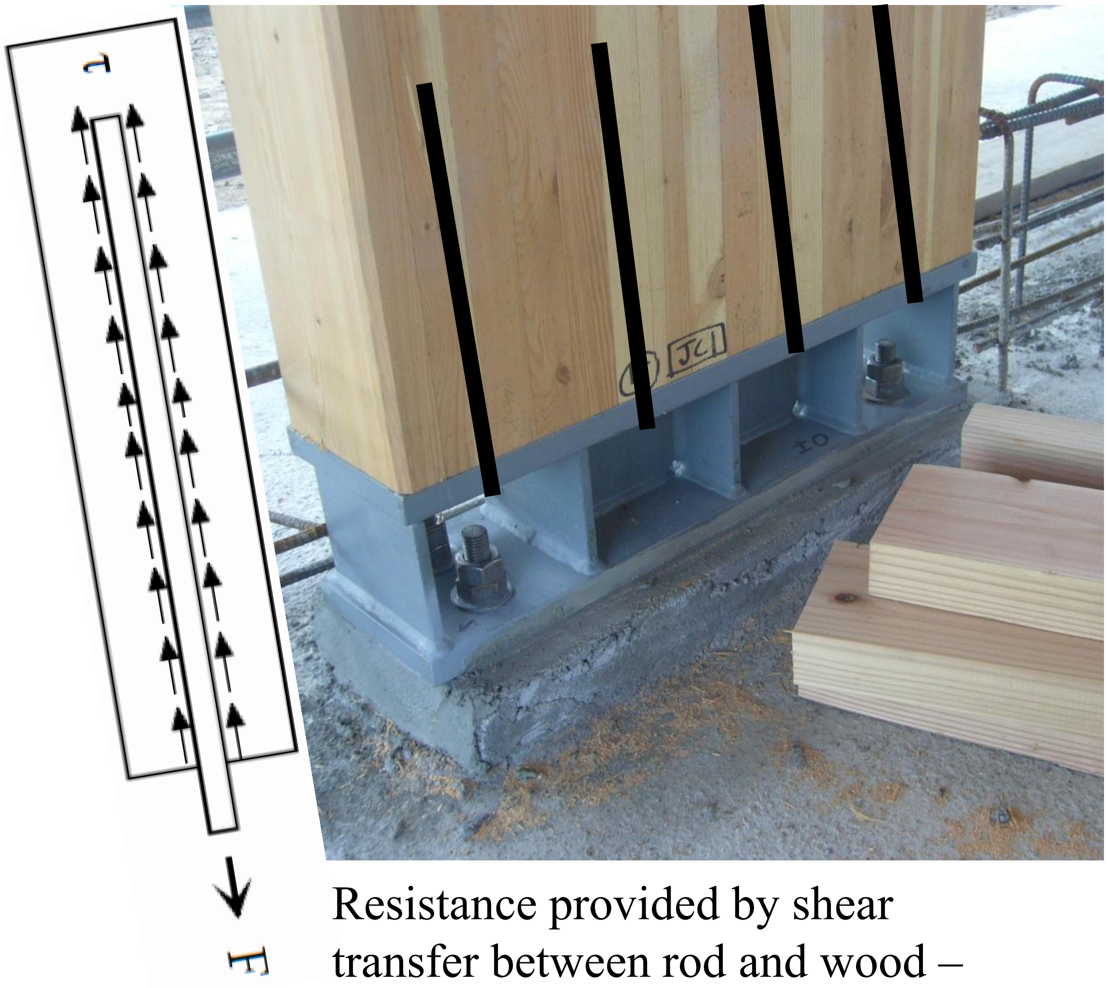
- Glued-in plate

HBV or HSK systems

Moment connections are traditionally avoided by designers

- Bolts are not desirable for moment connections as they have tendency to cause tension perpendicular to grain failure

Glued-in Rod Connection



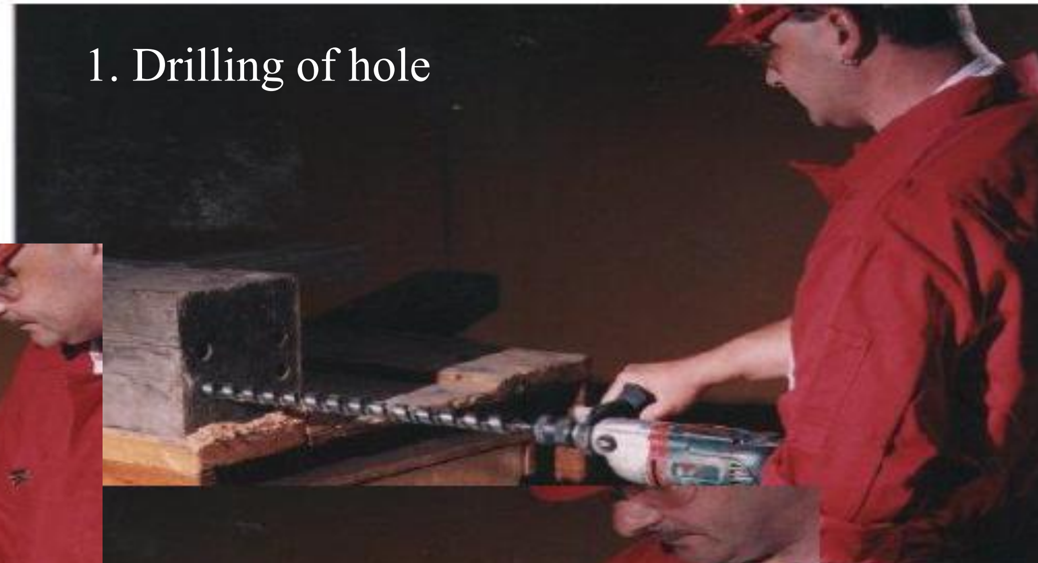
Resistance provided by shear transfer between rod and wood – long length leads to high capacity

Conventional bolted connection – limited by thickness of member & removal of wood material

Glued-in rod connection

- Glued-in rods installed in members in factory, which are then connected on site
- Can achieve strong moment connection and hold-down in heavy timber frames

Glued-in rod preparation



1. Drilling of hole

3. Injection of resin
(epoxy glue)

2. Threaded steel rod installation

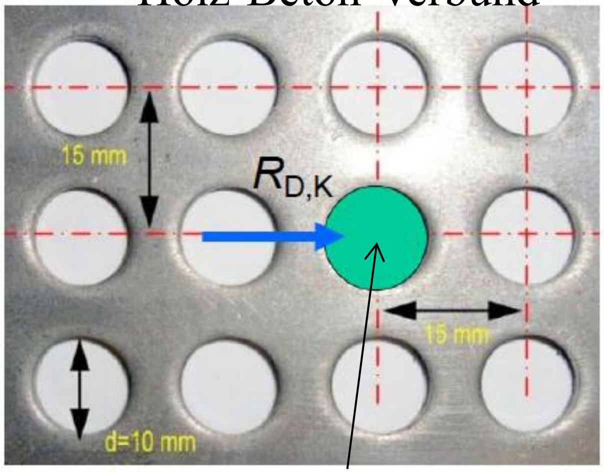
Application of glued-in rod connections

- One of the strongest connection methods around
- Aesthetically pleasing without exposed fasteners
- Not yet included in standardized design procedure
- Quality control procedure is under development
- Strong and stiff connection, but not ductile

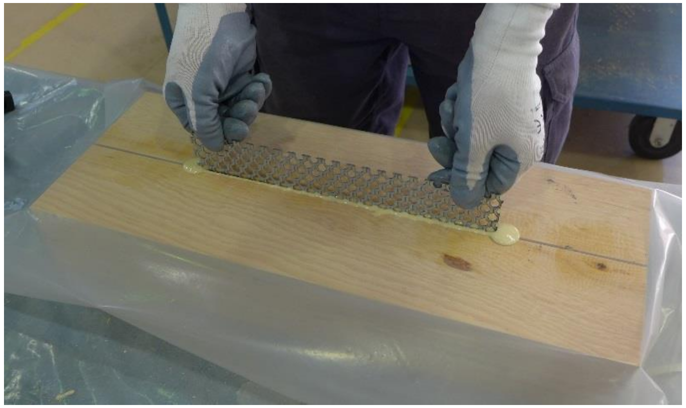
Glued-in plate connections (HBV System)

- Steel plate with round holes is inserted into slot in wood member and then filled with an adhesive
- Simple design as the capacity per adhesive dowel is 1.2 kN
- Strong connections can be achieved by specifying no. of plates and plate size

Holz-Beton-Verbund

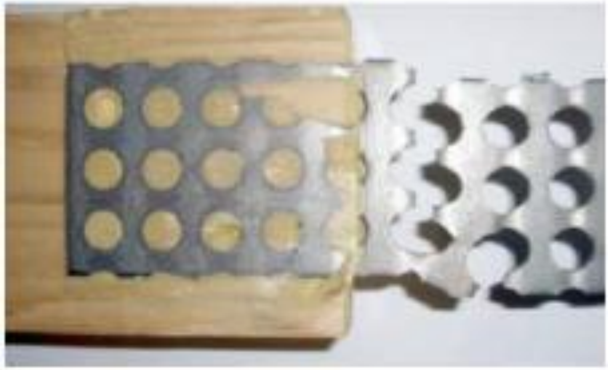


Adhesive dowel



Applications of glued plate connections

- Aesthetically pleasing without exposed fasteners
- Not yet included in standardized design procedure
- Can be applied to concrete-wood composite system
- Strict quality control is required and performed by skilled operator
- Strong, stiff and ductile connections

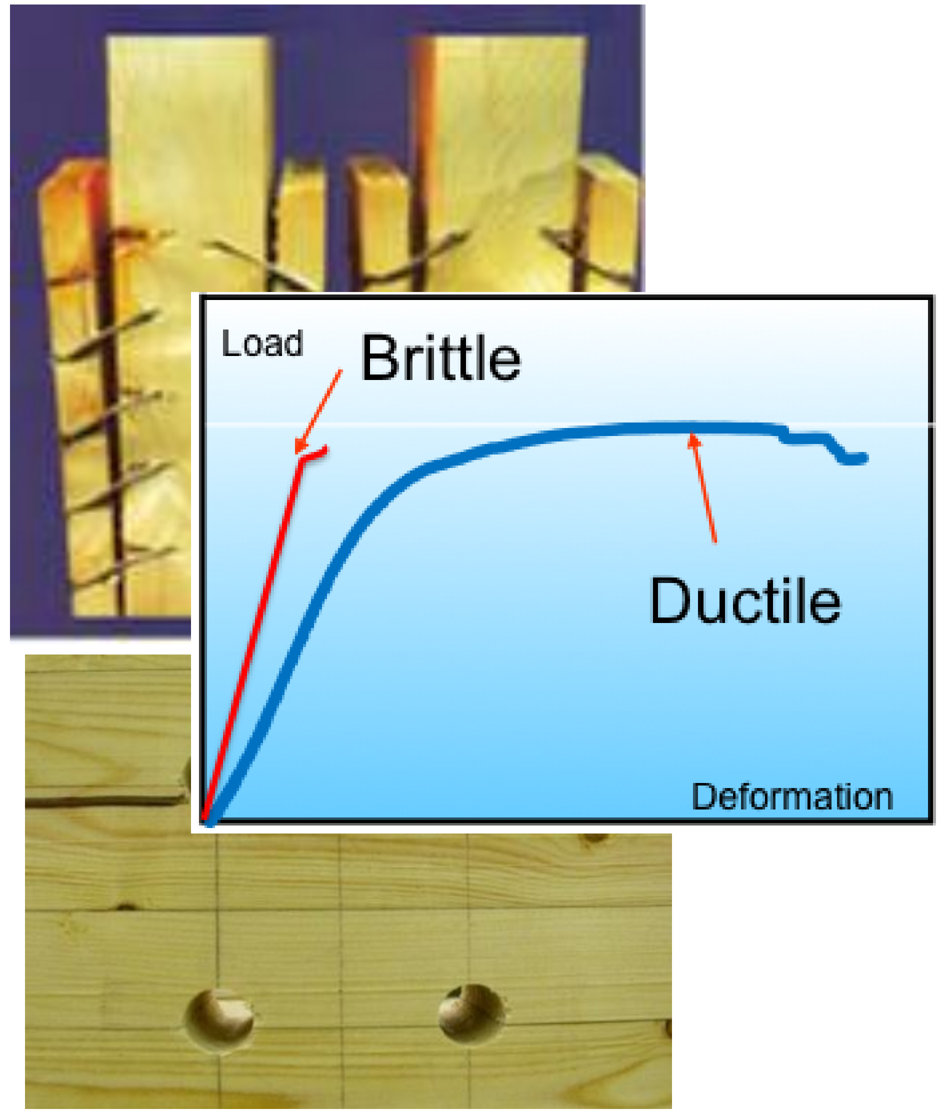


Connection with light fasteners

- Primary failure modes are ductile with yielding in either the fastener or wood
  - Nails and spikes
  - Lag screws (< ¼")
  - Wood screws
  - Small diameter bolts (< ¼")
  - Timber rivets

Response of timber connection to lateral load

- Ductile failure
  - Fastener bending
  - Wood crushing (embedment)
- Brittle failure
  - Wood splitting



Factors influencing failure modes

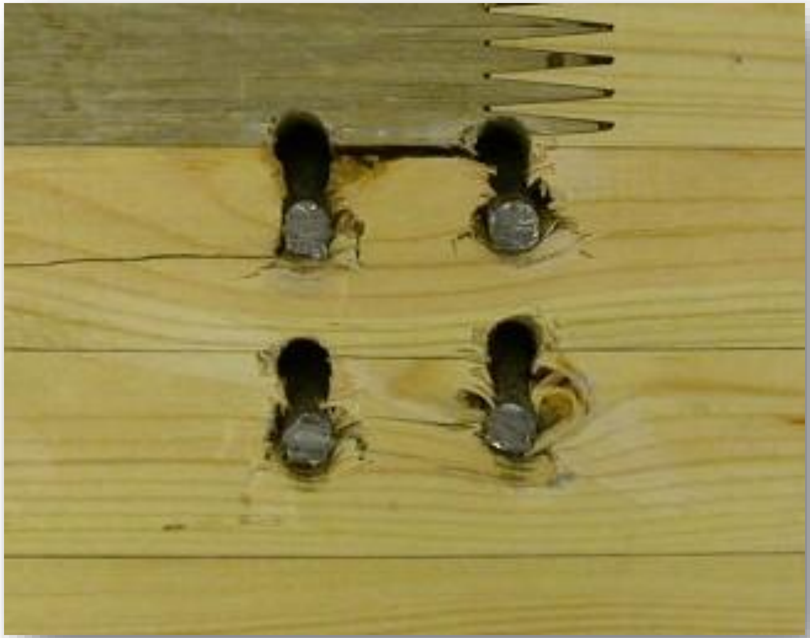
- Fastener
  - Slender ratio (bearing length / dia)
  - Stiffness and yield strength
- Wood
  - Embedment, tension perp, shear and splitting resistance
- Geometry / Orientation
  - Row spacing and end distance
  - Direction of loading

Ductile failure

Mechanics-based European Yield Model (EYM) to check (lower of 2 governs)

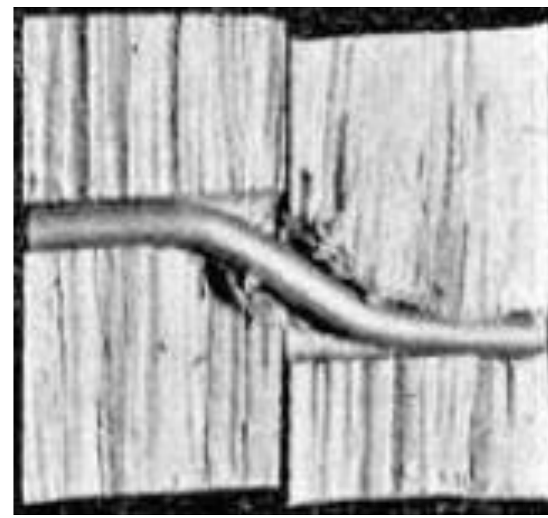
Yield of fastener

Crushing of wood fibre



European Yield Models (EYM)

- First proposed by K. W. Johansen (Theory of timber connections, 1949)
- Basis of all timber design standards in the World for predicting ductile modes of failure of connections containing dowel-type fasteners
- Theoretical basis:
  - Beam on elastic foundation
  - Wood and steel are elasto-plastic materials

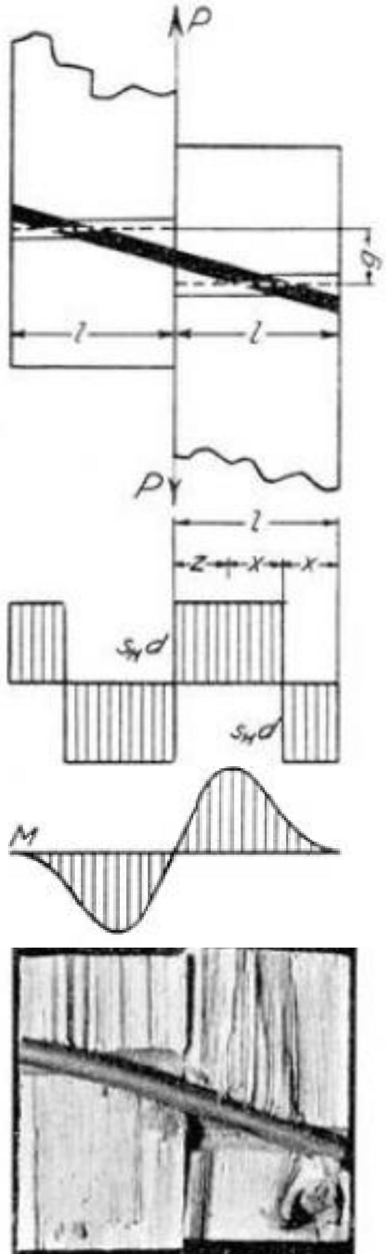


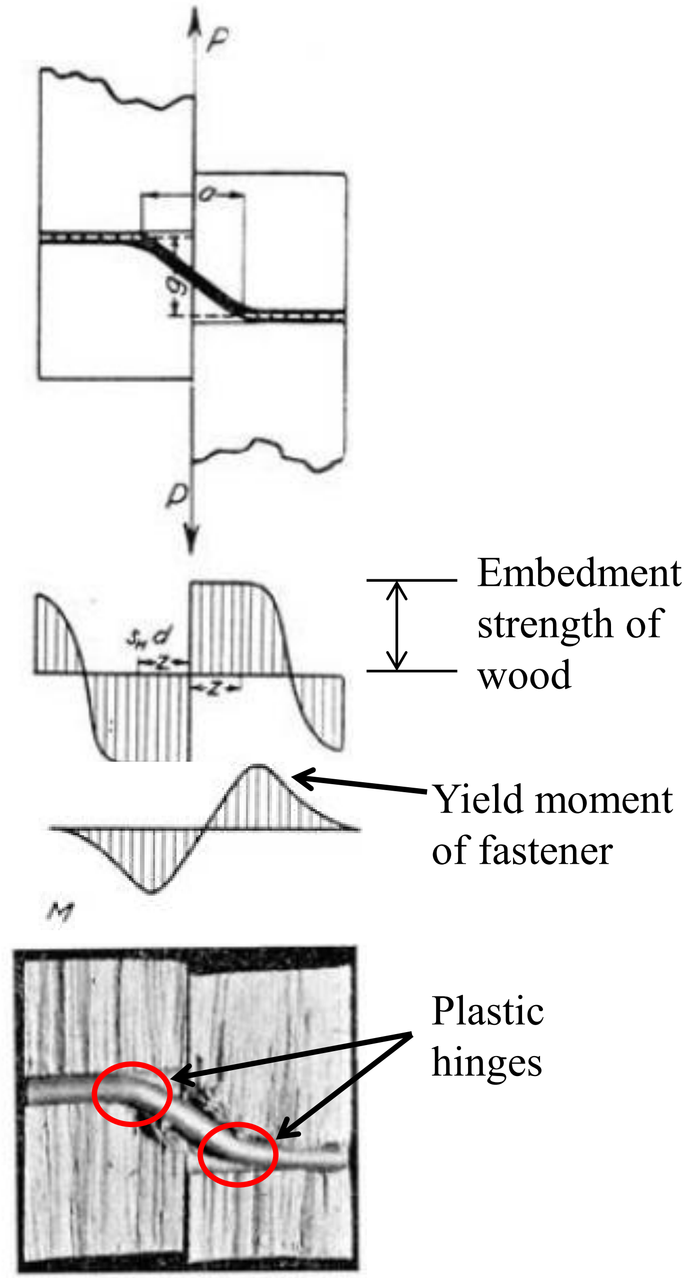
2-member joint

Stiff fastener | **Flexible fastener**

Bearing stress on wood

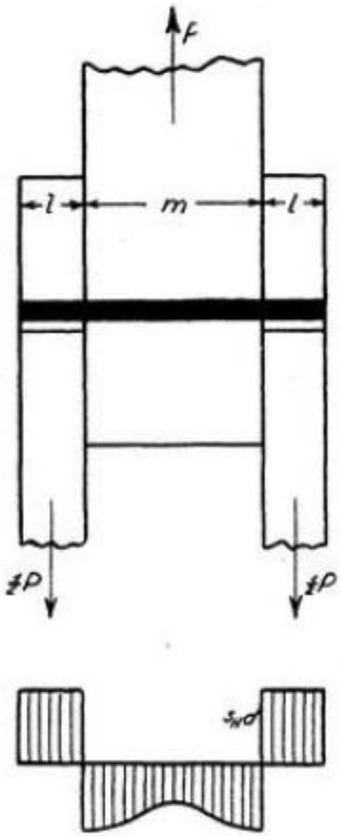
Bending moment in fastener

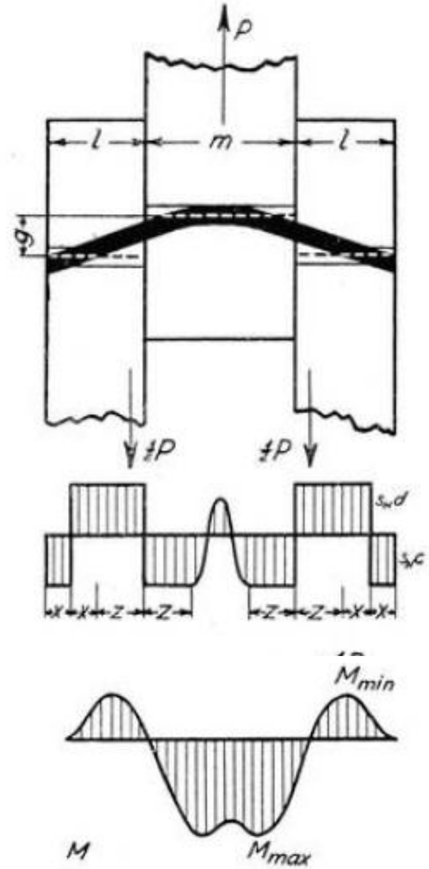


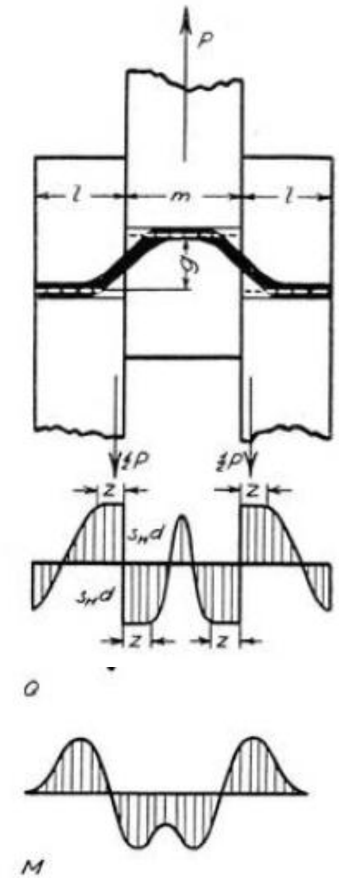


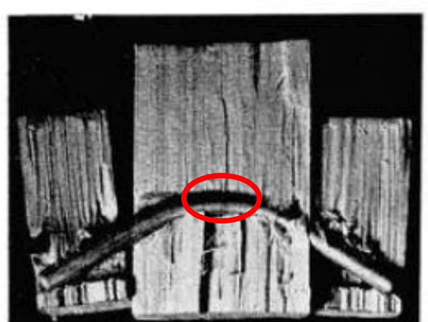
3-member joint

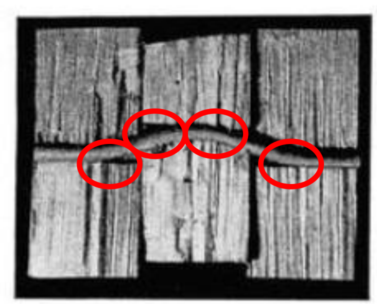
Thin member | **Stiff fastener** | **Flexible fastener**





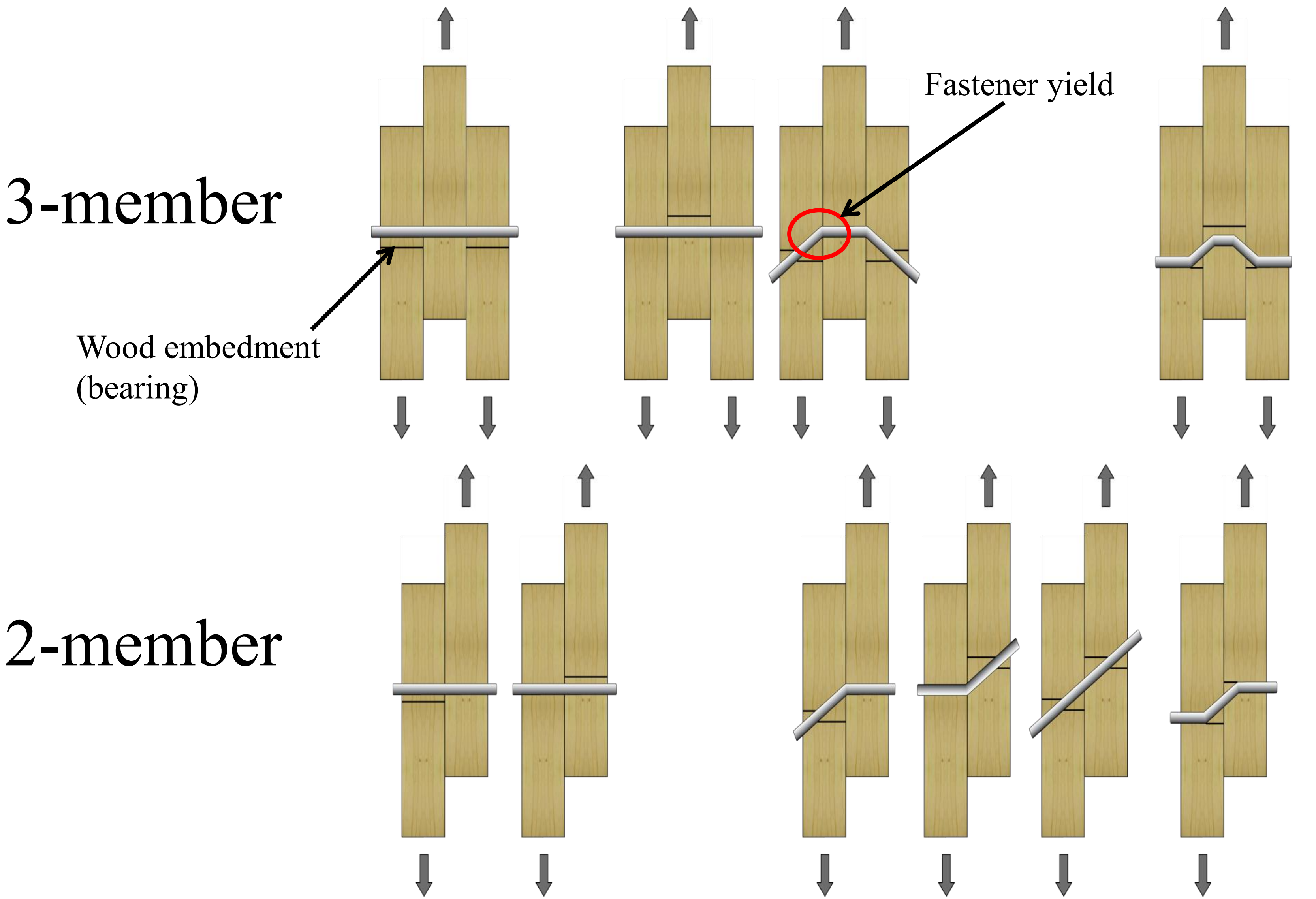






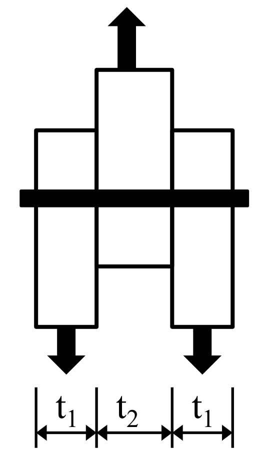
Plastic hinges

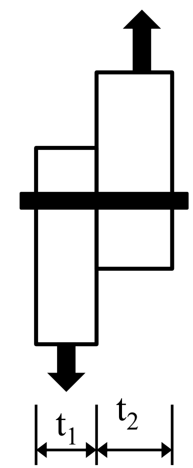
Possible yield failure modes



Mechanics-based equations for all possible modes

- Failure mode depends on:
  - Member thicknesses or fastener embedment lengths (t_1, t_2)
  - Embedment strengths of wood members (f_1, f_2)
  - Diameter (d_F) and Yield strength (f_y) of fastener





- European Yield Models (EYM) apply to connections containing round dowel fastener: nail, wood screw, lag screw, bolt, steel dowel

CSA O86 Equations

Mode

(a) $f_1 d_F t_1$

(b) $f_2 d_F t_2$

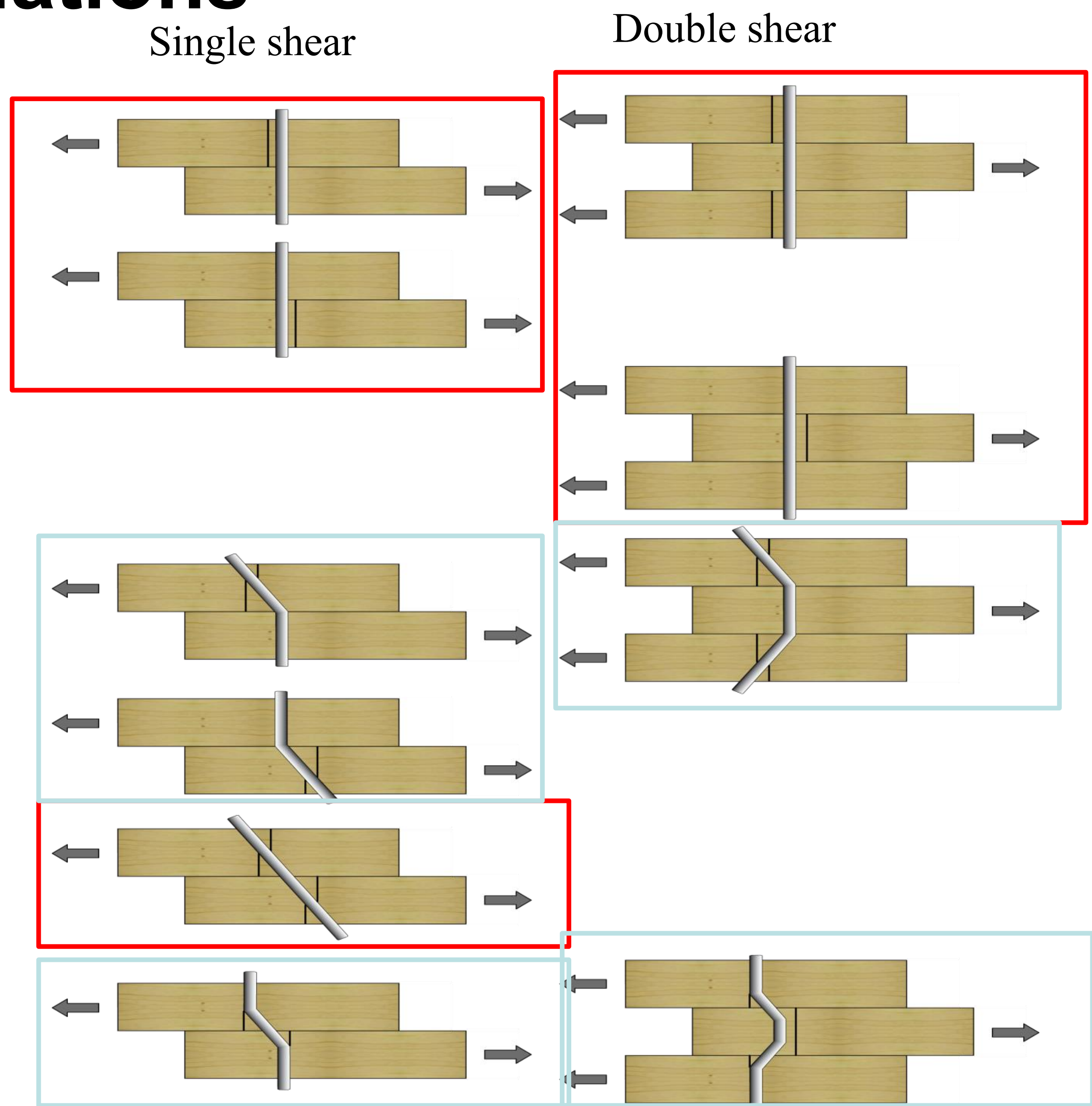
(c) $\frac{1}{2} f_2 d_F t_2$

(d) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F}\right)$

(e) $f_1 d_F^2 \left(\sqrt{\frac{1}{6}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_2}{d_F}\right)$

(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) $f_1 d_F^2 \sqrt{\frac{2}{3}\frac{f_2}{(f_1+f_2)}\frac{f_y}{f_1}}$



Bolted connections under lateral load

Check yield and brittle modes

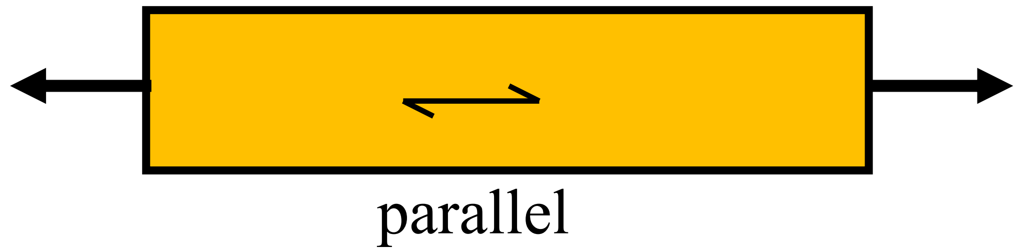
Factored resistance ≥ Factored load effect

Bolted connections under lateral load (12.4.4.2)

$$N_r \geq N_f \quad \textbf{(Yield failure)}$$

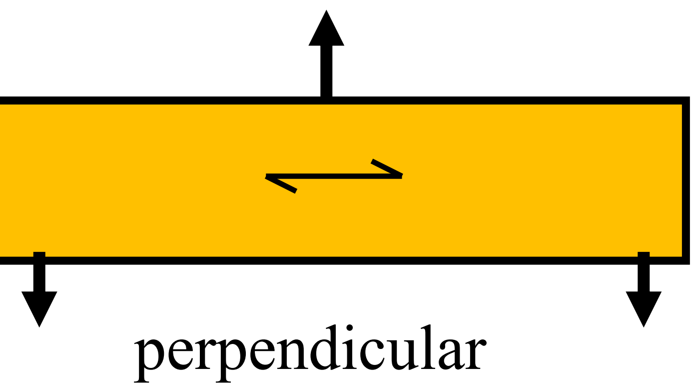
$$P_r \geq P_f$$

(Brittle)



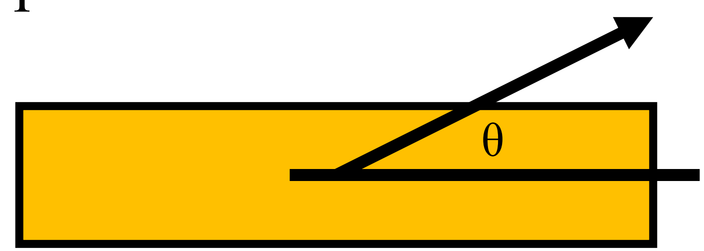
$$Q_r \geq Q_f$$

(Brittle)



$$N_f \leq \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

(Brittle)



Yielding/Bearing Resistance (12.4.4.3)

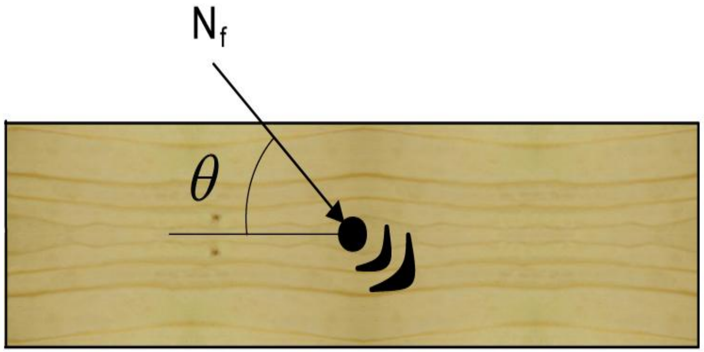
$$N_r = \phi_y\, n_u\, n_s\, n_f$$

Φ_y = 0.8

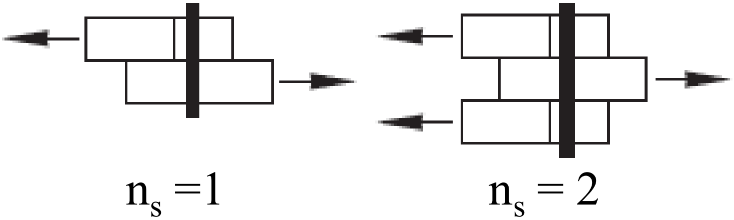
n_u = unit yielding resistance (EYM)

n_s = No. of shear planes

n_f = No. of fasteners



Bearing



Unit Lateral Load Resistance (n_u)

(a) $f_1\, d_F\, t_1$

(b) $f_2\, d_F\, t_2$

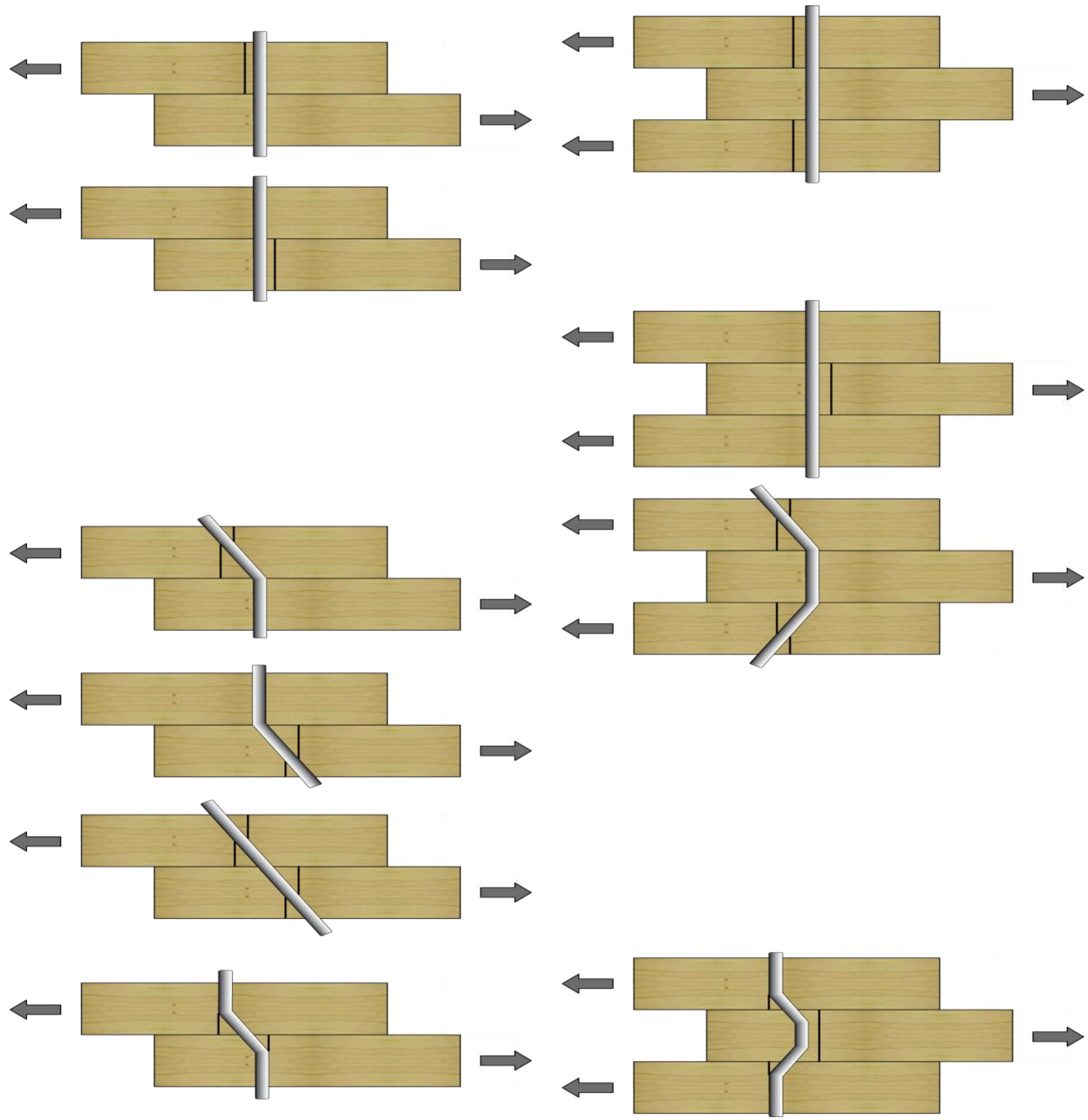
(c) $\frac{1}{2}\, f_2\, d_F\, t_2$

(d) $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6}\,\frac{f_2}{(f_1+f_2)}\,\frac{f_y}{f_1}} + \frac{1}{5}\,\frac{t_1}{d_F} \right)$

(e) $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6}\,\frac{f_2}{(f_1+f_2)}\,\frac{f_y}{f_1}} + \frac{1}{5}\,\frac{t_2}{d_F} \right)$

(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) $f_1 d_F{}^2 \sqrt{\frac{2}{3}\,\frac{f_2}{(f_1+f_2)}\,\frac{f_y}{f_1}}$



Embedment strength ($f_{i,P}$ & $f_{i,Q}$) in 12.4.4.3.3.

The embedment strength of member "i" is modified for the loading, service condition and treatment

$f_{i,P}$ and $f_{i,Q}$ are determined as follows:

- $f_{i,P} = 50\,G\,(1 - 0.01 d_F)\,J_x$ *// to grain*
- $f_{i,Q} = 22\,G\,(1 - 0.01 d_F)$ *per. to grain*

G: Mean specific gravity of wood member ([Table A.12.1])
d_F: Fastener diameter
J_x: 0.9 for CLT, 1.0 for all other cases

If loaded at an angle,

$$f_{i,\theta} = \frac{f_{i,P}\, f_{i,Q}}{f_{i,P} \sin^2\theta + f_{i,Q} \cos^2\theta} K_D K_{SF} K_T$$

Table A.12.1
Relative density values

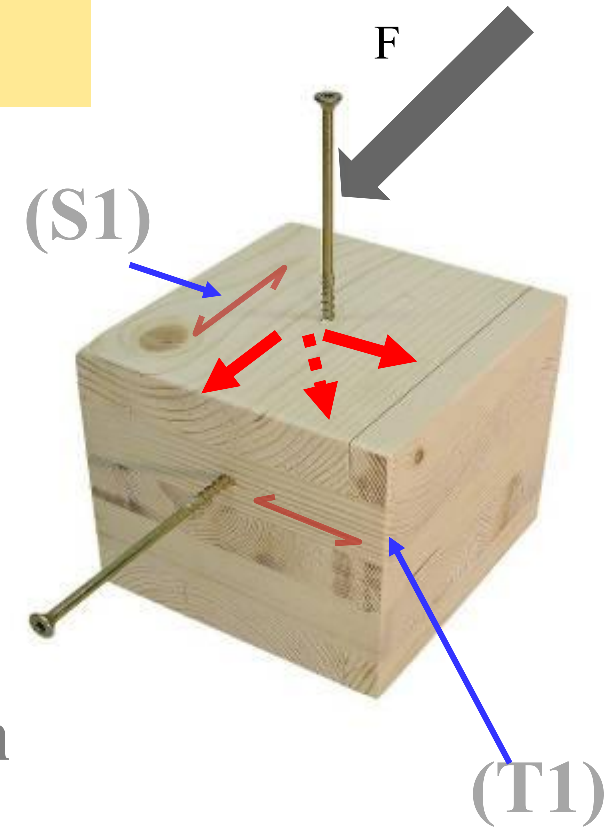
Visually graded lumber	Glued-laminated timber	MSR (or MEL) E Grades of S-P-F*	CLT	Mean oven-dry relative density
		13 800–16 500 MPa		0.50
D Fir-Larch	D Fir-Larch, Hem-Fir†		V1	0.49
		12 400–13 100 MPa		0.47
Hem-Fir	Hem-Fir†			0.46
	Spruce-Pine			0.44
Spruce-Pine-Fir		8 300–11 700 MPa	V2, E1	0.42
Northern Species			E3	0.35

Why connections in CLT are different to those in sawn timber and glulam?

Cross Lamination Effect

Different layers are loaded @ different angles due to X-lamination

- Critical for fasteners ≥ ¼” diameter



For Example

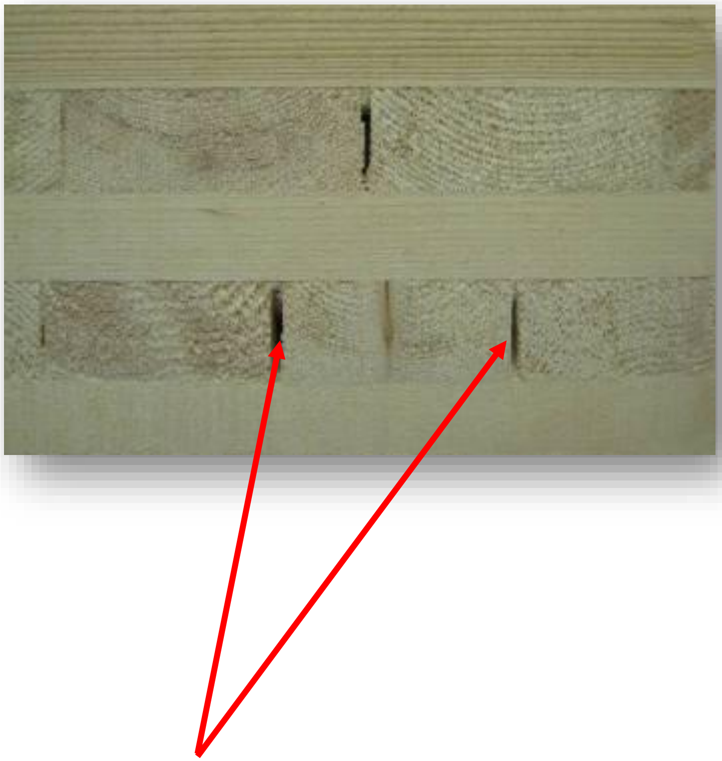
- Outer layer (S1) loaded // to Grain
- Transverse layer (T1) perp. to Grain

Why connections in CLT are different to those in sawn timber and glulam?

Also…

presence of specific CLT features such as:

- Unglued laminate edges (gaps)
- Sawn grooves to relieve drying stresses



Gaps and grooves

Not common to all CLT products

Embedment strength of steel plate

12.4.4.3.3.2 Non-wood-based materials

The embedment strength of non-wood-based materials, in MPa, shall be taken as follows:

(a) for steel:

$K_{sp} (\phi_{steel}/\phi_y) f_u$

where

K_{sp} = 3.0 for mild steel referenced in CSA S16

= 2.25 for cold-formed light gauge steel referenced in CSA S136

f_u = specified minimum tensile strength of steel, MPa

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g., for*

(a) ASTM A36/A36M steel, f_u = 400 MPa;

(b) CSA G40.21 steel, Grades 300W and 350W, f_u = 450 MPa; and

(c) cold-formed light gauge steel, Grade SS 230, f_u = 310 MPa.

ϕ_{steel} = resistance factor for steel plates in connections with bolts and dowels

= 0.8 for mild steel referenced in CSA S16

= 0.5 for cold-formed light gauge steel referenced in CSA S136

ϕ_y = resistance factor for yielding failures in wood members in connections with bolts and dowels

= 0.8

(b) for concrete or masonry: 125

Yield strength of fastener

12.4.4.3.3.3 Dowel or bolt yield strength

The yield strength of a dowel or bolt, f_y, in MPa, shall be taken as follows:

(a) ASTM A307, SAE J429 Grade 2 bolts and dowels: 310; or

(b) other CSA- or ASTM-compliant bolts and dowels:

$$\frac{f_{ym} + f_{um}}{2}$$

where

tensile f_{ym} (yield strength of bolts and dowels) and f_{um} (ultimate strength of bolts and dowels) are obtained from applicable material standards

Modification Factors for Connections

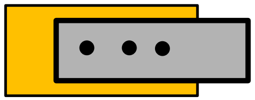
- Service condition factor, K_{SF} (Cl. 12.2.1.6)
- Load duration factor, K_D (as for member design)
- Treatment factor, K_T (No information provided)

12.2.1.8 Treatment factor, K_T

For connections containing wood-based members treated with fire-retardant or other strength-reducing chemicals, the strength capacities of connections shall be based on the documented results of tests that shall take into account the effects of time, temperature, and moisture content in accordance with Clause 4.3.2.

Note: *The effects of fire-retardant treatments can vary depending on manufacturing materials and processes. See the CWC* Commentary on CSA O86.

SWS



A

Table 12.2.1.6
Service condition factor, K_{SF}, for connections

Service conditions	Moisture content of wood when connection is fabricated: Dry (≤ 19%) Dry	Dry (≤ 19%) Wet	Green (> 19%) Dry	Green (> 19%) Wet	Connection detail	Angle of load to grain
Timber rivets						
Lateral loads	1.00	0.80	0.90	0.80	All	All
Withdrawal loads	1.00	*	0.60	*		
Split rings, shear plate connectors, and truss plates	1.00	0.67	0.80	0.67	All	All
Bolts, dowels, drift pins, and lag screws†	1.00	0.67	1.00	0.67	A	All
	1.00	0.67	1.00	0.67	B	0°
	1.00	0.67	0.40	0.27	B	90°
	1.00	0.67	0.40	0.27	C	All
Nails, spikes, and wood screws						
Lateral loads	1.00	0.67	0.80	0.67	All	All
Withdrawal loads	1.00	0.67	0.40	0.40	All	90°

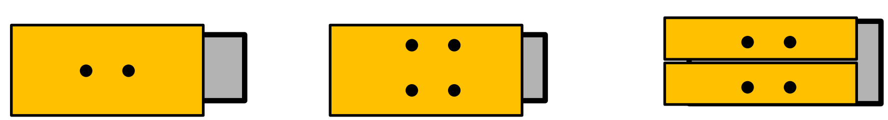
Legend:

A = a single fastener or single row parallel to grain with steel splice plates

B = a single row parallel to grain with wood splice plates, two rows parallel to grain not more than 127 mm apart with a common wood splice plate, or multiple rows with separate wood or steel splice plates for each row

C = all other arrangements

B

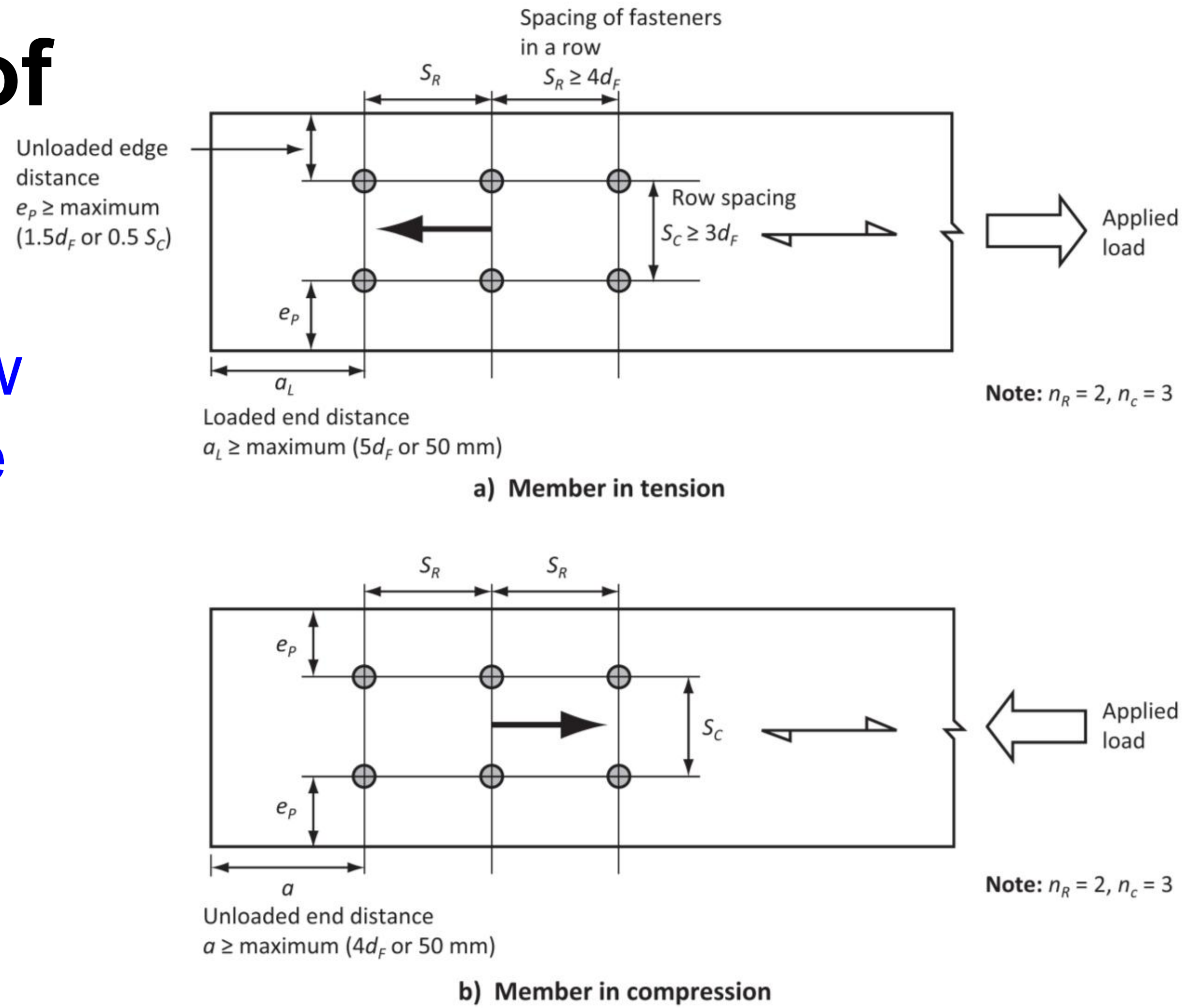


WSW WSW WSW

Geometry of bolt group:

- Row spacing
- Spacing in row
- Edge distance
- End distance

etc



Legend:

S_R = spacing of fasteners in row

S_C = row spacing

a = unloaded end distance

a_L = loaded edge distance

e_p = unloaded edge distance

Corrections:

S_R = spacing parallel to grain

S_C = spacing perpendicular to grain

a_L = loaded end distance

Figure 12.4.3.1

Placement of bolts and dowels in a connection loaded parallel to grain

Spacing requirements for bolts and dowels in CLT

12.4.3.4 Placement of fasteners in panel edge of CLT

In a group of fasteners installed in panel edge of CLT, the minimum spacings measured from centres of fasteners shall be as follows (see Figure 12.4.3.3):

(a) spacing of fasteners in row, S_R: four bolt or dowel diameters; Spacing in load direction

(b) row spacing, S_C: three bolt or dowel diameters; Spacing perp to load direction

(c) loaded end distance, a_L: five bolt or dowel diameters or 50 mm, whichever is greater;

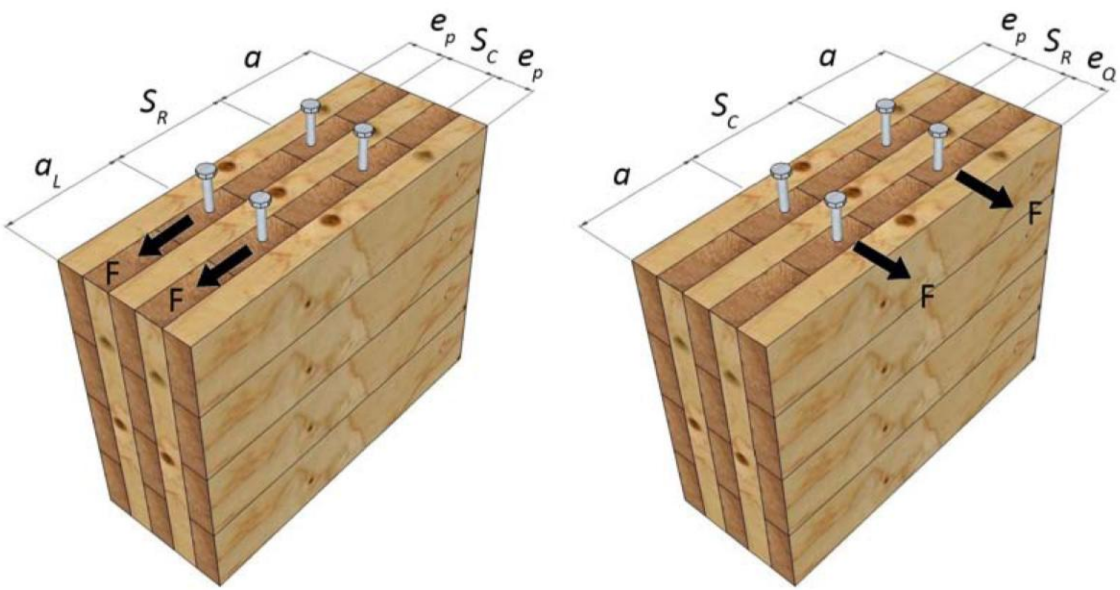
(d) unloaded end distance, a: four bolt or dowel diameters or 50 mm, whichever is greater;

(e) loaded edge distance, e_Q: five bolt or dowel diameters; and

(f) unloaded edge distance, e_P: 1.5 bolt or dowel diameters or half the row spacing, whichever is greater.

End distance – parallel to grain of face layer [(c), (d)]

Edge distance – perp to grain of face layer [(e), (f)]



Note: fastener can be in any layers

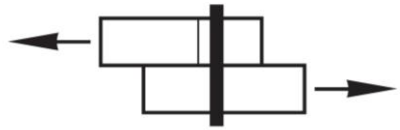
Lag screws - Lateral strength

Lag screw

12.6.6.1.2

The unit lateral strength resistances, p_u or q_u, shall be taken as the smallest value determined as follows:

(a) $f_1 d_F t_1$



(f) $f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}}$

where

d_F = lag screw diameter, mm

f_2 = embedment strength of main member, MPa

= $50G\,(1–0.01 d_F)\,J_x$ for parallel-to-grain loading

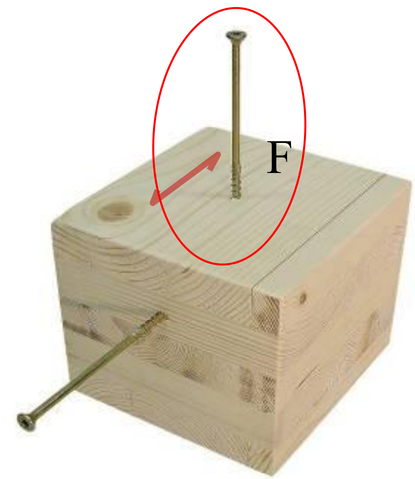
= $22G\,(1–0.01 d_F)$ for perpendicular-to-grain loading

where

G = mean relative density (Table A.12.1)

J_x = 0.90 for CLT

= 1.0 in all other cases



J_x (0.9) introduced for bolt, dowel, lag screw, nail and wood screw for parallel to grain only

Lag screws - Lateral strength

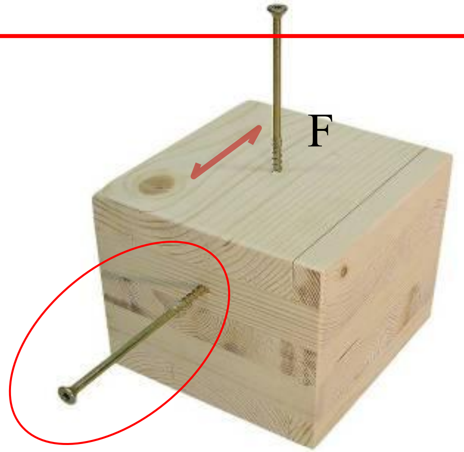
12.6.6.2 End grain

The lateral resistance of lag screws inserted parallel to grain in the end grain of the main member shall be not greater than two-thirds of the lateral side grain resistance for perpendicular to grain if wood side plates are used. If steel side plates are used, the lateral resistance shall be not greater than one-half of the lateral side grain resistance for perpendicular-to-grain loading in the main member.

12.6.6.3 Panel edge of CLT

The lateral resistance of lag screws inserted in panel edge of CLT shall be not greater than two-thirds of the lateral resistance for perpendicular to grain of laminations in the panel face if wood side plates are used, or one-half if steel side plates are used.

Basically treated as inserted through end grain (conservative)



Other dowel-type fasteners

- Nails, spikes and wood screws
  - No differentiation between parallel and perpendicular to grain loading
  - Embedment property provided for plywood and OSB
  - Geometry requirements differ

Nail – Lateral Resistance

12.9.4 Lateral resistance

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)

n_F = number of fasteners in the connection

n_S = number of shear planes per nail or spike

$J_F = J_E \ J_A \ J_B \ J_D$

EYM

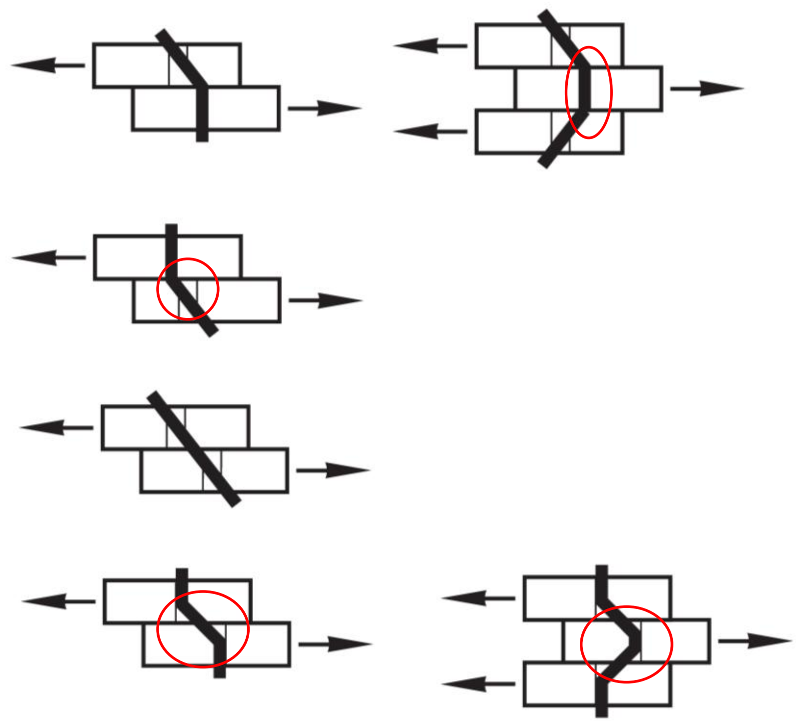
Similar to bolt and lag screw but use of f_3 instead of f_2 for modes (d), (f) and (g) to reflect the 'additional' resistance provided by formation of plastic hinge in main member

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



Embedment Strength

t_1 = head-side member thickness for two-member connections, mm

= minimum side plate thickness for three-member connections, mm (Clause 12.9.2.2)

d_F = nail or spike diameter, mm

f_2 = embedment strength of main member, MPa

$= 50\ G\ (1 - 0.01 d_F)\ J_x$

where

G = mean relative density

Note: *Mean relative density values for wood members are provided in Table A.12.1.*

J_x = 0.9 for CLT

= 1.0 in all other cases

Embedment Strength

t_2 = length of penetration into point-side member for two-member connections, mm
= centre member thickness for three-member connections, mm (Clause 12.9.2.2)

f_3 = embedment strength of main member where failure is fastener yielding, MPa
$= 110\, G^{1.8}\, (1-0.01 d_F)\, J_x$

where
J_x = 0.9 for CLT
= 1.0 in all other cases

f_y = nail or spike yield strength, MPa
$= 50\, (16 - d_F)$

f_1 = embedment strength of side member, MPa

For lumber and CLT:

$f_1 = 50\, G\, (1- 0.01 d_F)\, J_x$

where
J_x = 0.9 for CLT
= 1.0 in all other cases

For structural panels:

$f_1 = 104\, G\, (1- 0.1 d_F)$

where
G = 0.49 for DFP
= 0.42 for CSP and OSB

$f_3 > f_2$ for same G and d_F

Also embedment strength for steel side member

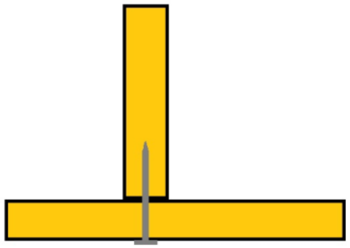
Modification factors – J_E, J_A, J_B, J_D

J_E = end grain factor
= 0.67 for nailing into end grain
= 1.0 in all other cases

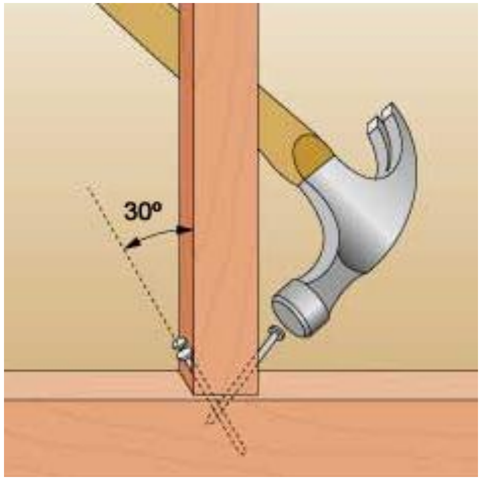
J_A = toe-nailing factor
= 0.83 for toe-nailing, where toe-nails are started at approximately one-third of the nail length from the end of the piece and driven at an angle of about 30° to the grain of the member
= 1.00 for cases other than toe-nailing

J_B = nail clinching factor
= 1.6 for nail clinching on the far side in a two-member connection
= 1.0 if not clinched or in three-or-more member connections

J_D = factor for diaphragm and shearwall construction
= 1.3 for nails and spikes used in diaphragm and shearwall construction
= 1.0 in all other cases



End grain, J_E



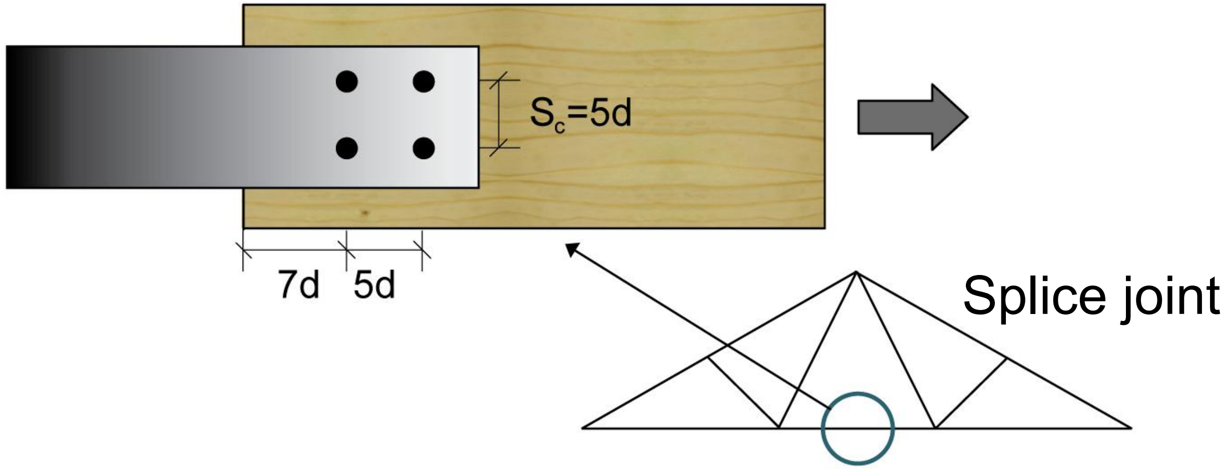
Toe nail, J_A

Nail clinching, J_B

Exercise 1

Steel-to-Wood Bolted Connection

Steel-Wood-Steel Connection



- D-fir glulam, 20f-EX, 130x190 mm
- Steel-wood-steel connection, 6.35 mm steel side plates
- ¾" bolts, 7d end distance, 5d bolt spacing, 5d row spacing
- (Assume dry, no treatment and dead and live load combination)

What is the factored resistance ?

Determine the Yielding Capacity

Evaluate the yielding resistance (Clause 12.4.4.3)

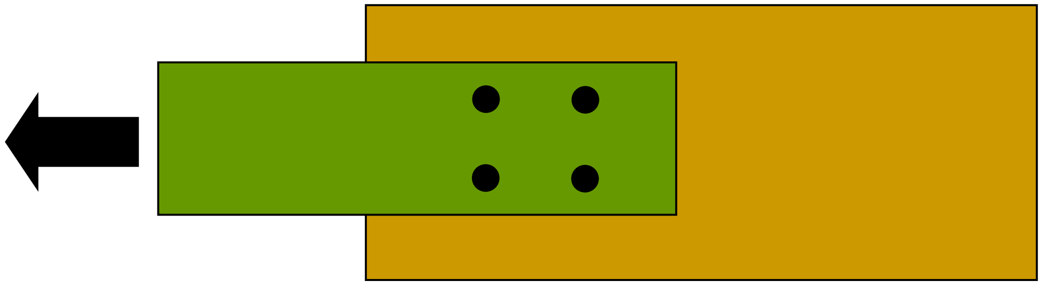
f_1 = 3.0 · (0.67/0.8) · 450 MPa = **1131 MPa** (side member)

t_1 = 6.35 mm

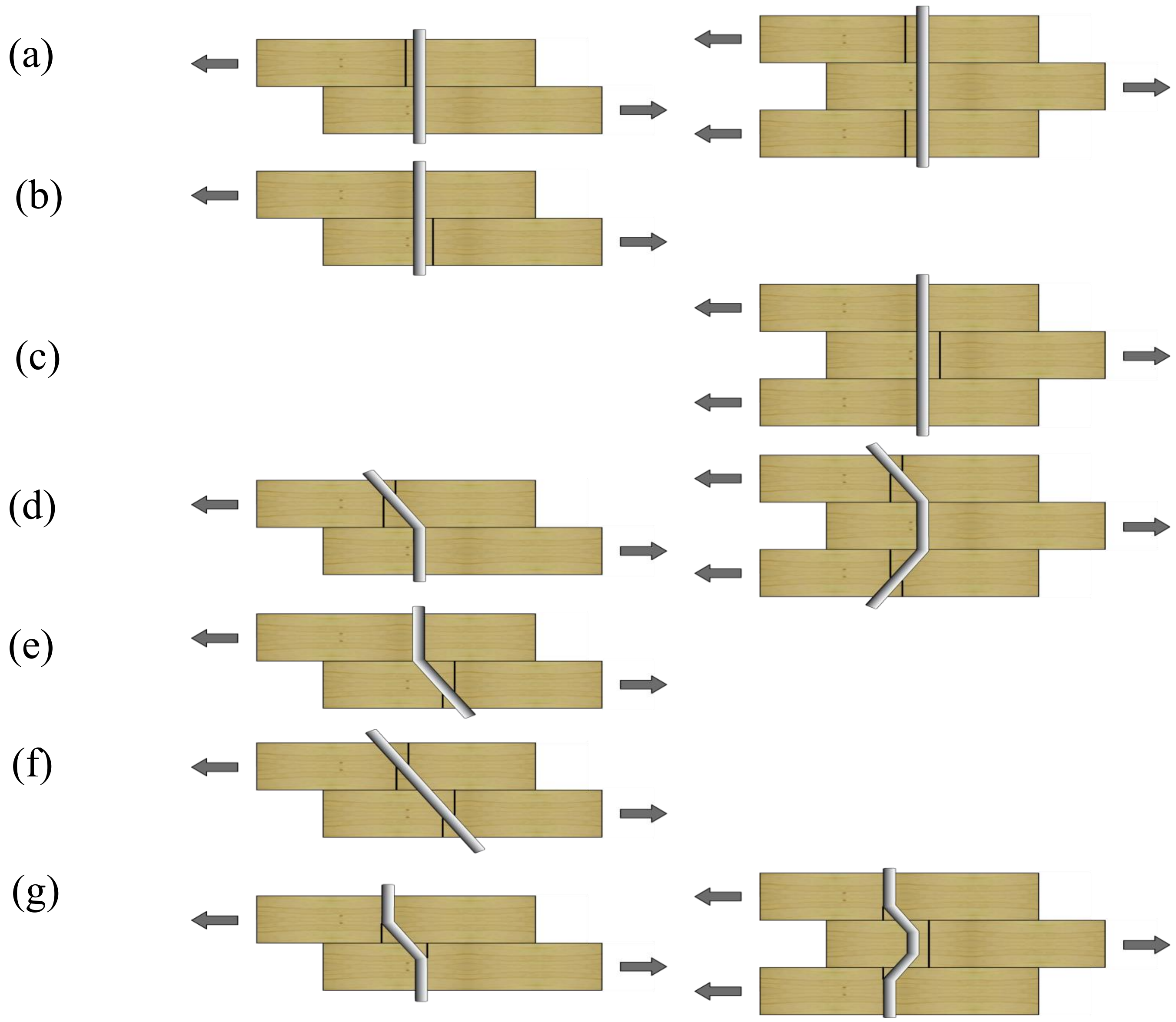
f_2 = 50 G (1-0.01d) $K_D \cdot K_{SF} \cdot K_T$ = 50 · 0.49 (1- 0.01 x 19.1) x 1 x 1 x 1

= **19.8 MPa**

t_2 = 130 mm (main member)



Unit Lateral Load Resistance (n_u)



Evaluate Yielding Capacity

$$n_u = f_1 d_f t_1$$

n_u = **<u>137.1</u>** kN/shear plane - (a)

$$n_u = 0.5 \cdot f_2 d_f t_2$$

n_u = **<u>24.6</u>** kN/shear plane - (c) **GOVERNS**

$$n_u = f_1 d_f^2 \left(\sqrt{\frac{f_2 f_y}{6 (f_1 + f_2) f_1}} + \frac{t_1}{5d} \right)$$

n_u = **<u>41.4</u>** kN/shear plane - (d)

$$n_u = f_1 d_f^2 \sqrt{\frac{2 f_2 f_y}{3 (f_1 + f_2) f_1}}$$

n_u = **<u>27.9</u>** kN/shear plane - (g)

Evaluate Yielding Capacity

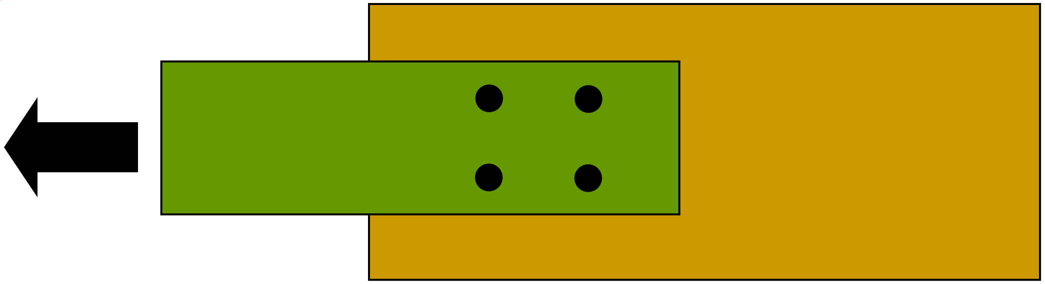
n_u = **24.6** kN/shear plane - (c) GOVERNS

$N_r = \phi \, n_u \, n_s \, n_f$

$= 0.8 \cdot 24.6 \text{ kN}/n_s \cdot 2 \cdot 2 \cdot 2$

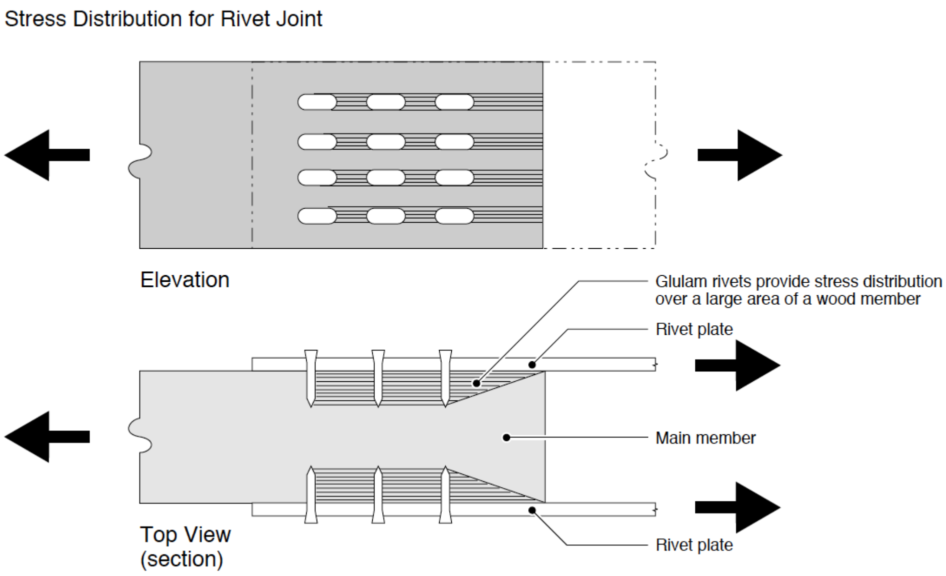
Yielding Capacity

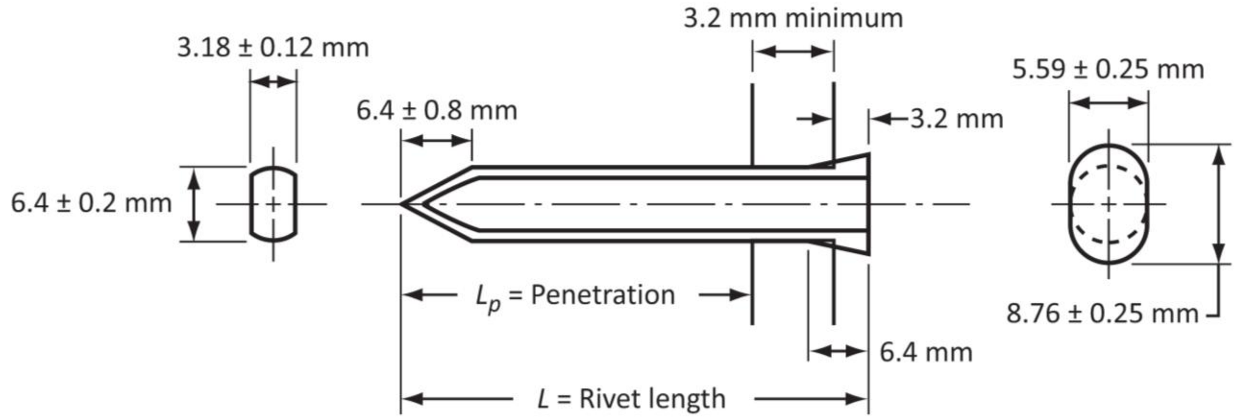
$N_r = 157$ kN

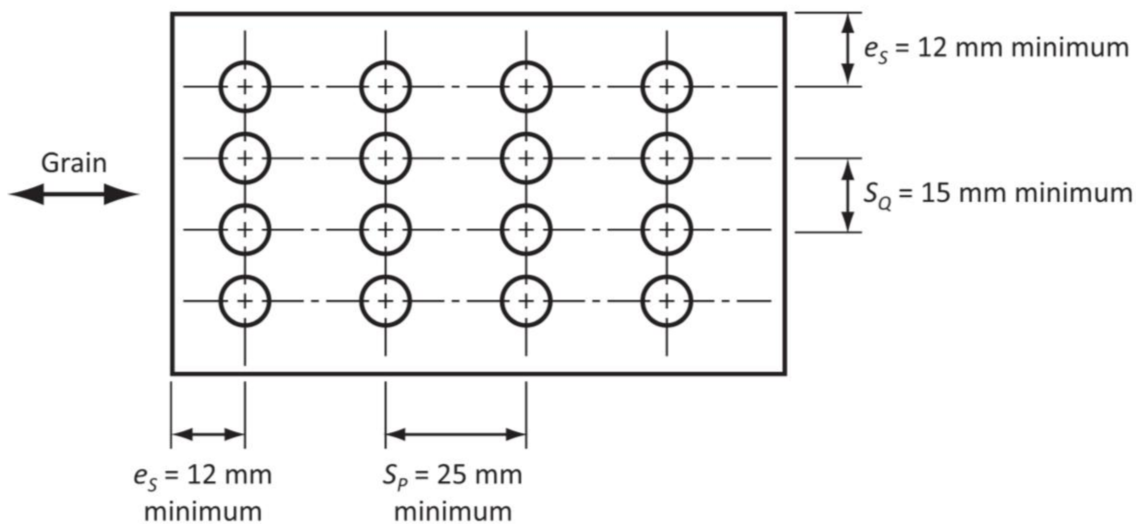


Timber Rivets

- Developed in Canada in the 1960's – known as glulam rivets at the time
- Advantages over bolts and shear plates:
  - Simple field fabrication
  - Permits greater load transfer per area
  - No member strength reduction due to net area effect







Notes:

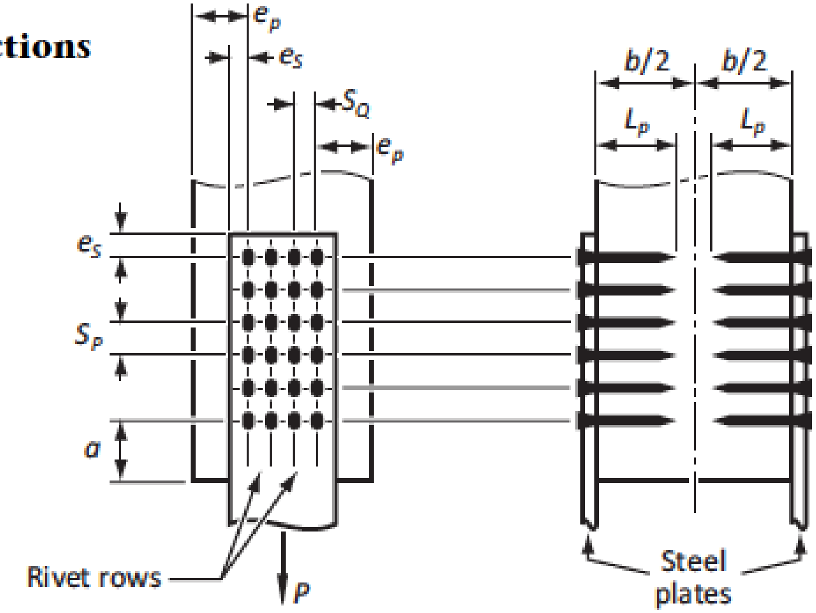
(1) *Hole diameter:* 6.4 mm *minimum to* 7.0 mm *maximum.*

(2) *Tolerance in location of holes:* 3 mm *maximum in any direction.*

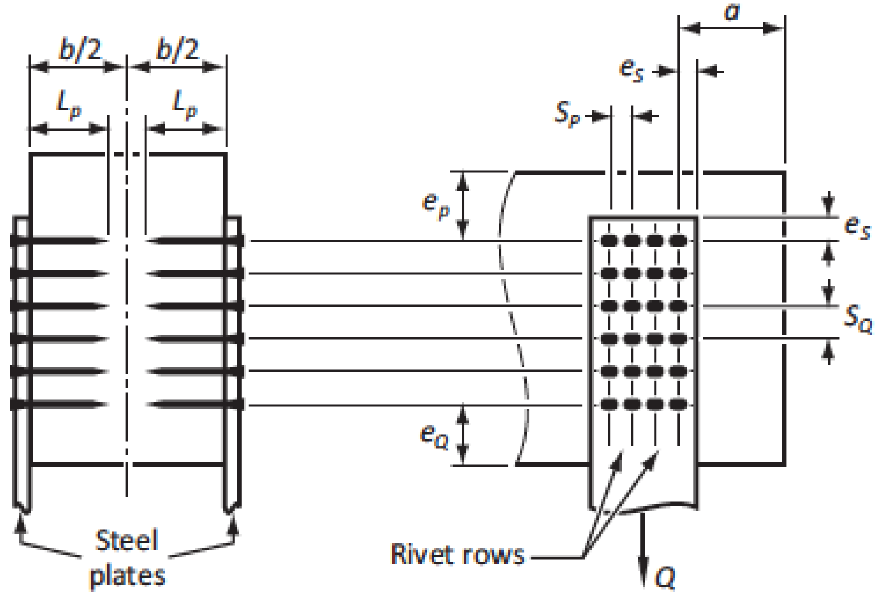
(3) *Orient wide face of rivets parallel to grain, regardless of plate orientation.*

Figure 12.7.1.7
End and edge distances for timber rivet connections

a = end distance in wood
e_S = end distance in steel
e_P = unloaded edge distance in wood
e_Q = loaded edge distance in wood
S_P = fastener spacing parallel to grain
S_Q = fastener spacing perp to grain



Load parallel to grain



Load perpendicular to grain

Timber Rivet - Lateral resistance

Factored capacity parallel to grain

12.7.2.2

For loading parallel to grain, the factored lateral strength resistance, P_r, of the joint shall be taken as follows:

$$P_r = \phi P_u H$$

where

ϕ = 0.6

P_u = lateral resistance parallel to grain, kN (Clause 12.7.2.3)

H = material factor

= 1.00 for Douglas Fir-Larch glulam

= 0.80 for Spruce-Lodgepole Pine-Jack Pine glulam

= 0.50 for Douglas Fir-Larch sawn timber

= 0.45 for Hem-Fir sawn timber

= 0.40 for Spruce-Pine-Fir sawn timber

= 0.35 for Northern Species sawn timber

Note: failure modes can be either ductile (rivet) or brittle (wood), depending on diameter, spacing and end distance.

Timber Rivet - Lateral resistance

Unit strength - Parallel-to-grain

12.7.2.3

The unit capacity per rivet joint parallel to grain, P_u, shall be calculated as the lesser of P_y or P_w, as follows:

$$P_y = (1.09 L_p^{0.32} n_R n_C)\, J_Y\, (K_{SF}\, K_T) \text{ for rivet capacity}$$

$$P_w = p_w\, (K_D\, K_{SF}\, K_T) \text{ for wood capacity}$$

where

L_p = length of penetration, mm (Figure 12.7.1.1)
= (overall rivet length) – (plate thickness) – 3.2

n_R = number of rows of rivets parallel to direction of load

n_C = number of rivets per row

J_Y = side plate factor
= 1.00 for a side plate thickness of 6.4 mm and more
= 0.90 for a side plate thickness of 4.7 mm or more but less than 6.4 mm
= 0.80 for a side plate thickness of 3.2 mm or more but less than 4.7 mm

p_w = lateral resistance parallel to grain, kN (Table 12.7.2.3), using wood member thickness for the member dimension in Table 12.7.2.3 for connections with steel plates on opposite sides and using twice the wood member thickness for the member dimension in Table 12.7.2.3 for connections having only one plate

Note: *As an alternative, p_w may be calculated in accordance with Clause A.12.7.2.3.1.*

Timber Rivet - Lateral resistance

Factored capacity perpendicular to grain

12.7.2.4

For loading perpendicular to grain, the factored lateral strength resistance, Q_r, of the joint shall be taken as follows:

$Q_r = \phi Q_u H$

ϕ = 0.6

P_u = lateral resistance parallel to grain, kN (Clause 12.7.2.3)

H = material factor (Clause 12.7.2.2)

12.7.2.5

The unit capacity per rivet joint perpendicular to grain, Q_u, shall be calculated as the lesser of Q_y or Q_w, as follows:

$Q_y = (0.62 L_p^{0.32} n_R n_C)\ J_Y\ (K_{SF} K_T)$ for rivet capacity

$Q_w = (q_w L_p^{0.8} C_t)(K_D\ K_{SF}\ K_T)$ for wood capacity

where L_p, n_R, n_C, J_Y are as specified in Clause 12.7.2.3, q_w, kN, is determined from Table 12.7.2.5A and C_t is determined from Table 12.7.2.5B.

Note: *As an alternative, q_w and C_t may be calculated in accordance with Clause A.12.7.2.3.2.*

Table 12.7.2.3
Values of p_w, kN, parallel to grain for timber rivet joints 40 mm rivets — Spacing: S_p = 25 mm; S_Q = 25 mm

Member dimension, mm*	Rivets per row, n_C	Number of rows, n_R									
		2	4	6	8	10	12	14	16	18	20
80	2	24	56	88	125	160	195	225	260	290	330
	4	35	74	110	155	200	240	270	310	350	390
	6	46	92	135	185	240	280	320	360	410	460
	8	58	108	160	215	270	320	370	410	460	520
	10	68	125	180	245	310	370	410	460	510	580
	12	76	140	205	270	340	400	450	510	570	640
	14	84	155	220	300	370	440	490	560	630	710
	16	88	170	245	320	410	480	530	600	680	770
	18	98	185	270	350	430	510	570	640	730	810
	20	104	205	280	370	460	540	600	680	770	850
130	2	31	62	72	88	112	130	150	180	235	290
	4	44	80	92	108	135	160	180	215	260	310

Note: *As an alternative, p_w may be calculated in accordance with Clause A.12.7.2.3.1.*

Table 12.7.2.5B
Values of factor C_t

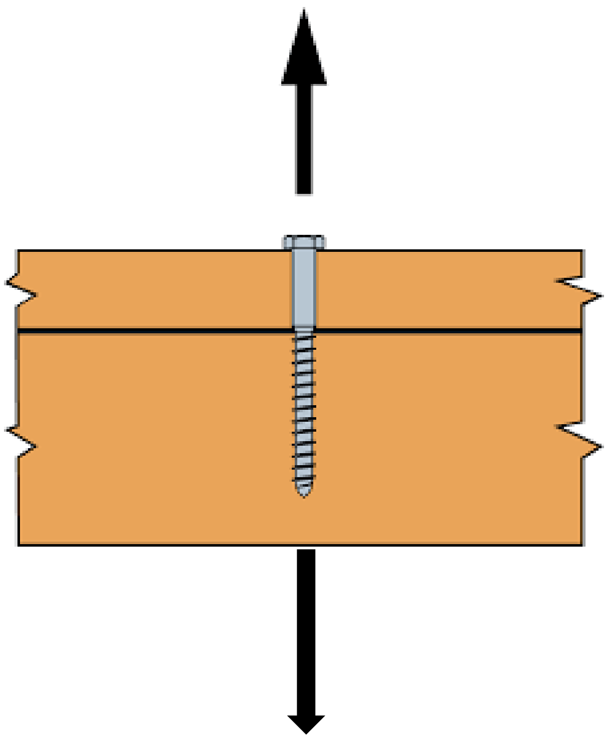
$e_p/[(n_c\text{-}1)s_Q]$	C_t	$e_p/[(n_c\text{-}1)s_Q]$	C_t
0.1	5.76	3.2	0.79
0.2	3.19	3.6	0.77
0.3	2.36	4.0	0.76
0.4	2.00	5.0	0.72
0.5	1.77	6.0	0.70
0.6	1.61	7.0	0.68
0.7	1.47	8.0	0.66
0.8	1.36	9.0	0.64
0.9	1.28	10.0	0.63
1.0	1.20	12.0	0.61
1.2	1.10	14.0	0.59
1.4	1.02	16.0	0.57
1.6	0.96	18.0	0.56
1.8	0.92	20.0	0.55
2.0	0.89	25.0	0.53
2.4	0.85	30.0	0.51
2.8	0.81		

General method for lateral resistance of timber rivets.

- Model is given in A12.7.2.3 as the general design approach.
- Provisions in 12.7.2.2 to 12.7.2.5 are simplified approach.

Withdrawal resistance design

- Permitted for lag screws and wood screws, timber rivets and nails
- Timber rivets and nails – short-term loading only
- For wood screws, need to check head pull-through and tensile strength of fastener



Timber rivets

12.7.3 Withdrawal resistance

Long-term withdrawal loading not permitted

12.7.3.1

Timber rivets loaded in withdrawal shall only be permitted for dry service conditions for short-term and standard-term load durations.

12.7.3.2

The factored withdrawal resistance from the side grain, P_{rw}, of a timber rivet joint shall be taken as follows. P_{rw} shall be greater than or equal to the effect of the factored loads

$$P_{rw} = \phi Y_w L_p n_R n_C$$

where

ϕ = 0.6

$Y_w = y_w(K_{SF} K_T)$

where

y_w = withdrawal resistance per millimetre of penetration, N/mm

= 13 for glulam

= 7 for sawn timber

and L_p, n_R, and n_C are as specified in Clause 12.7.2.3.

Lag screws

12.6.5 Withdrawal resistance

12.6.5.1

The factored withdrawal resistance, P_{rw}, of a group of lag screws in a connection shall be equal to the effect of the factored loads, as follows:

$$P_{rw} = \phi Y_w L_t n_F J_E$$

L_t = Threaded length

where

ϕ = 0.6

$Y_w = y_w (K_D K_{SF} K_T)$

where

y_w = basic withdrawal resistance per millimetre of threaded shank penetration into main member, N/mm

$= 59\, d_F^{0.82}\, G^{1.77}\, J_X$

Unit withdrawal strength is a function of diameter and SG of wood

where

d_F = nominal lag screw diameter, mm

G = mean relative density of main member (Table A.12.1)

J_X = 0.9 for CLT

= 1.0 in all other cases

J_E = end grain factor for lag screws

= 0.75 in end grain

= 0.67 in panel edge of CLT

= 1.00 in all other cases

J_x introduced for lag screw, nail and wood screw

Wood screws

12.11.5.2 Withdrawal resistance of main member

For a two-member connection connected with wood screws, the factored withdrawal resistance, P_{rw}, in N, of the main member shall be taken as follows:

$$P_{rw} = \phi Y_w L_{pt} n_F$$

L_{pt} = Threaded length

where

$\phi = 0.6$

$$Y_w = y_w (K_D K_T K_{SF})$$

where

y_w = basic withdrawal resistance per millimetre of threaded shank penetration in main member, N/mm

$= 59 d_F^{0.82} G^{1.77} J_x$

where

d_F = nominal wood screw diameter, mm (Table 12.11.1)

G = mean relative density of main member (Table A.12.1)

J_x = 0.9 for CLT

= 1.0 in all other cases

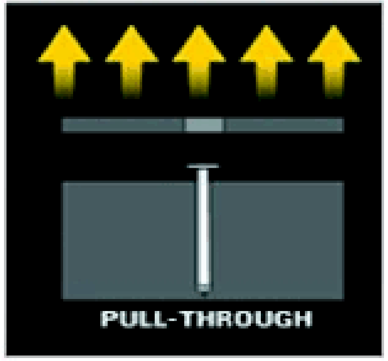
Wood screws

12.11.5.3 Head pull-through resistance of side member

For connections with steel side plates, the factored head pull-through resistance, P_{pt}, in N, shall be taken as follows:

$$P_{pt} = 1.5\phi\, t_1 d_w f_u n_F$$

For connections with lumber, glulam, CLT, or structural panel side plates, the factored head pull-through resistance shall be taken as follows:



$$P_{pt} = 65\phi\, t_1 n_F K_D$$

where

ϕ = 0.4

t_1 = thickness of side plate, mm

d_w = diameter of screw head, mm

f_u = specified minimum tensile strength of steel, MPa

Note: *The specified minimum tensile strength of steel, f_u, is given in the relevant material standards, e.g., for*

(a) ASTM A36/A36M steel, f_u = 400 MPa;

(b) CSA G40.21 steel, Grades 300W and 350W, f_u = 450 MPa; and

(c) cold-formed light gauge steel, Grade SS 230, f_u = 310 MPa.

n_F = number of wood screws in the connection

Nails

12.9.5.1
Nails and spikes loaded in withdrawal may be used only for wind or earthquake loading.

12.9.5.2
The factored withdrawal resistance of the nail or spike connection, P_{rw}, shall be greater than or equal to the effect of the factored loads, as follows:

$$P_{rw} = \phi Y_w L_p n_F J_A J_B$$

L_p = Length in main member

where

ϕ = 0.6

$Y_w = y_w (K_{SF} K_T)$

where

y_w = withdrawal resistance per millimetre of penetration into main member, N/mm

$= 16.4 d_F^{0.82} G^{2.2} J_x$

where

d_F = nail diameter, mm

G = mean relative density (Table A.12.1)

J_x = 0.9 for CLT

= 1.0 in all other cases

L_p = length of penetration into main member, mm

n_F = number of fasteners in the connection

J_A = toe-nailing factor

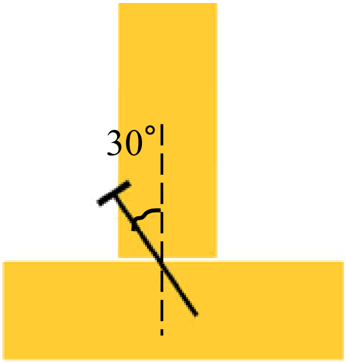
= 0.67 for toe-nailing

= 1.00 for cases other than toe-nailing

J_B = nail-clinching factor

= 1.6 for nail-clinching on the far side of a two-member connection

= 1.0 if not clinched or in three-or-more member connections



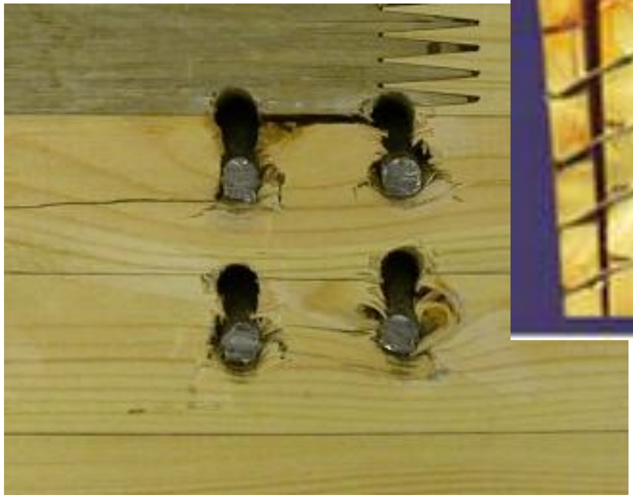
Toe Nailing

Nail clinching

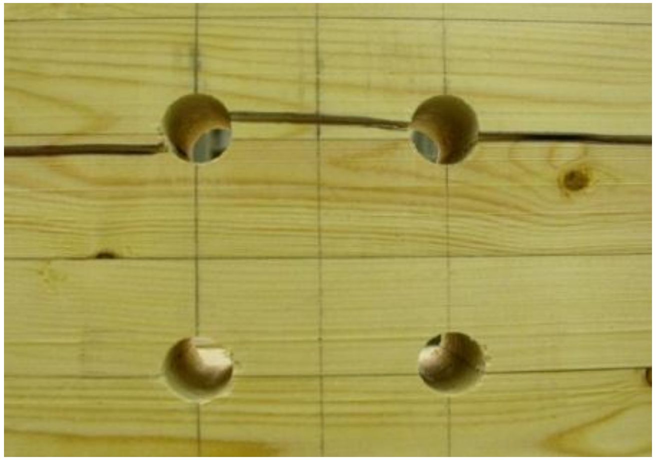
Design of connections II

- Design of connections that fail in a brittle manner (bolt and steel dowel)

Response of timber connection to lateral load



- Ductile failure
  - Fastener bending
  - Wood crushing (embedment)
- Brittle failure
  - Wood splitting



Factors influencing failure modes

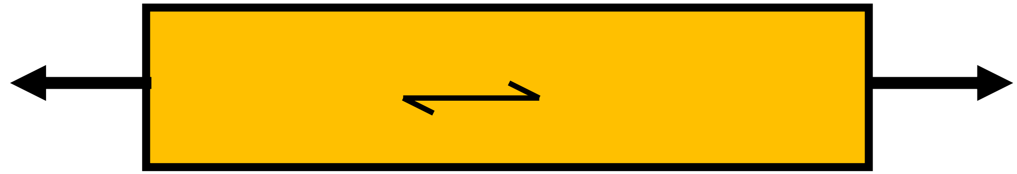
- Fastener
  - Slender ratio (bearing length / diameter)
  - Stiffness and yield strength
- Wood
  - Embedment, tension perp, shear and splitting resistance
- Geometry / Orientation
  - Row spacing and end distance
  - Direction of loading

Bolted connections under lateral load (12.4.4.2)

$N_r \geq N_f$ **(Yield failure)**

$P_r \geq P_f$

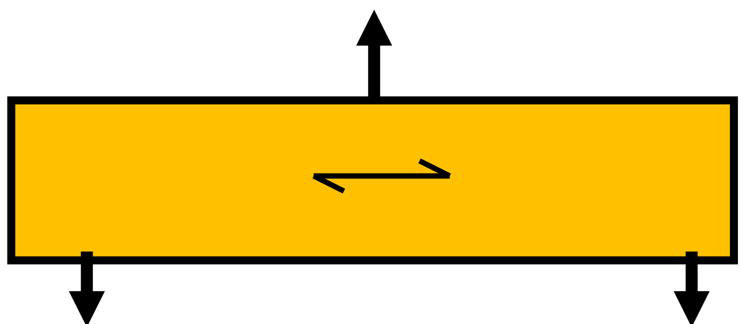
(Brittle)



parallel

$Q_r \geq Q_f$

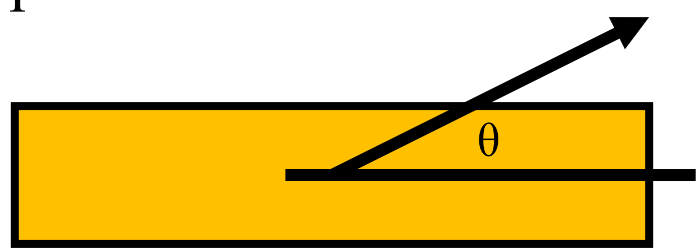
(Brittle)



perpendicular

$$N_f \leq \frac{P_r Q_r}{P_r \sin^2\theta + Q_r \cos^2\theta}$$

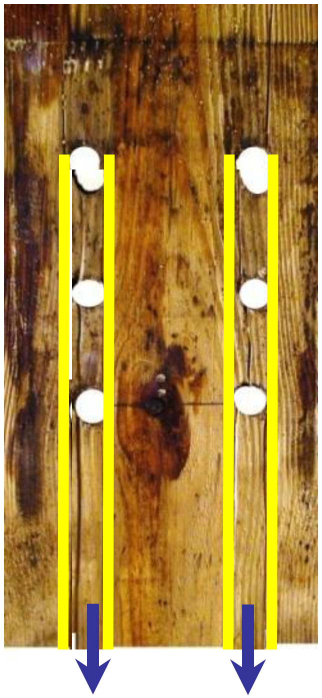
(Brittle)



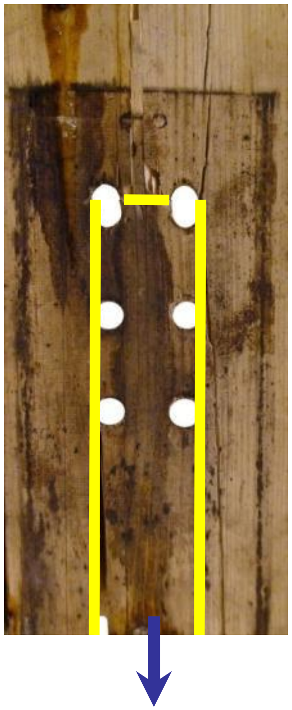
Bearing – wood embedment

Source: Quenneville

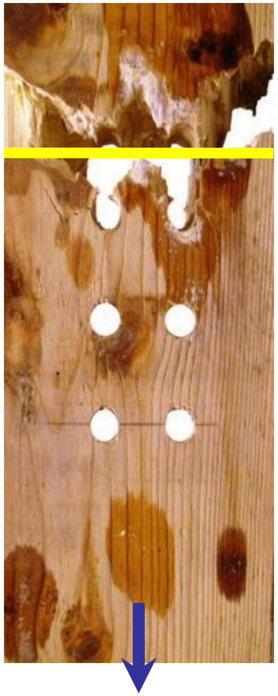
Brittle failure modes for bolts and steel dowels



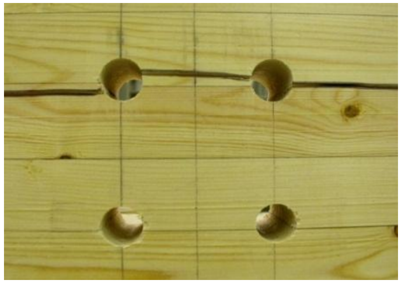
Row shear



Group tear out

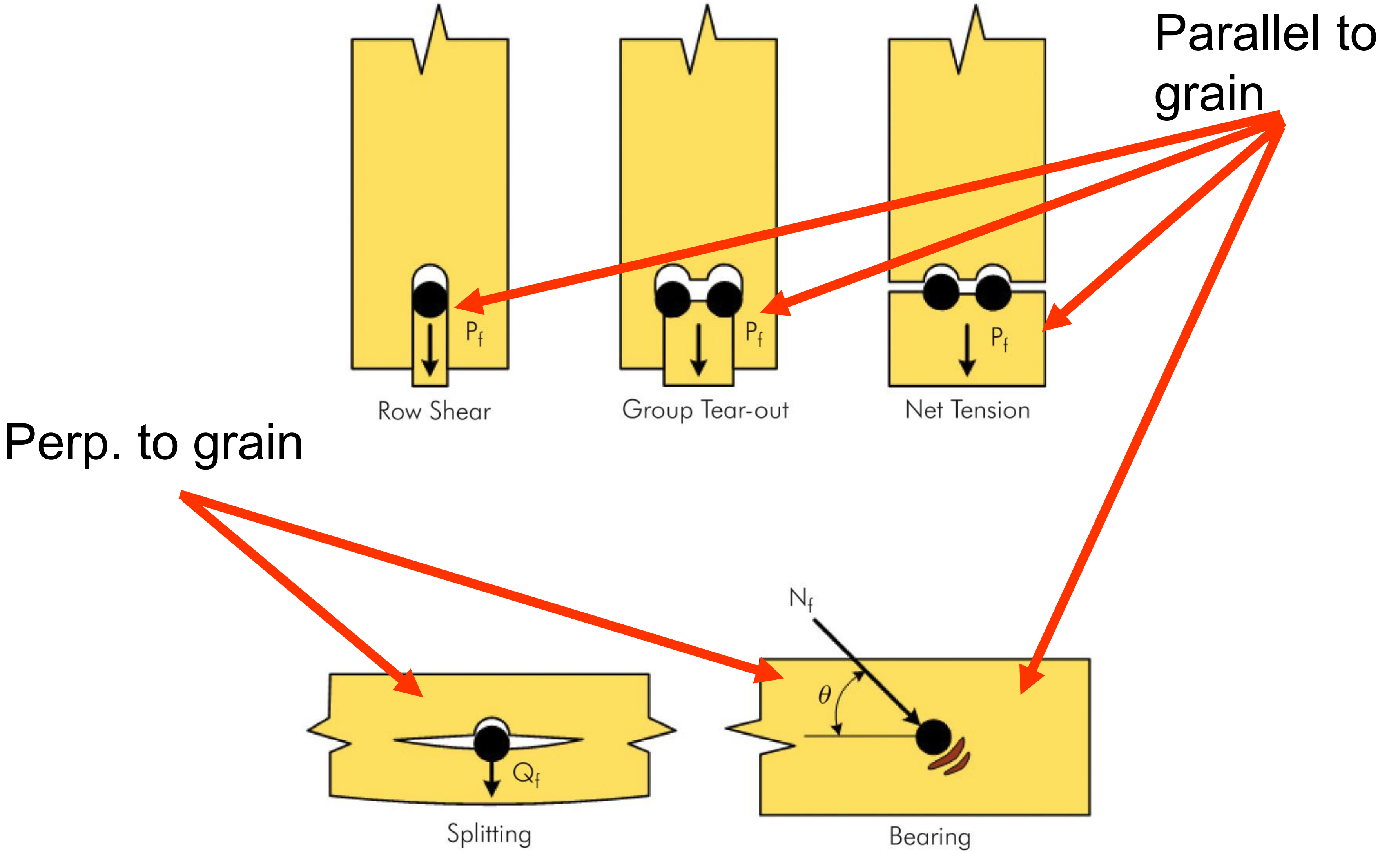


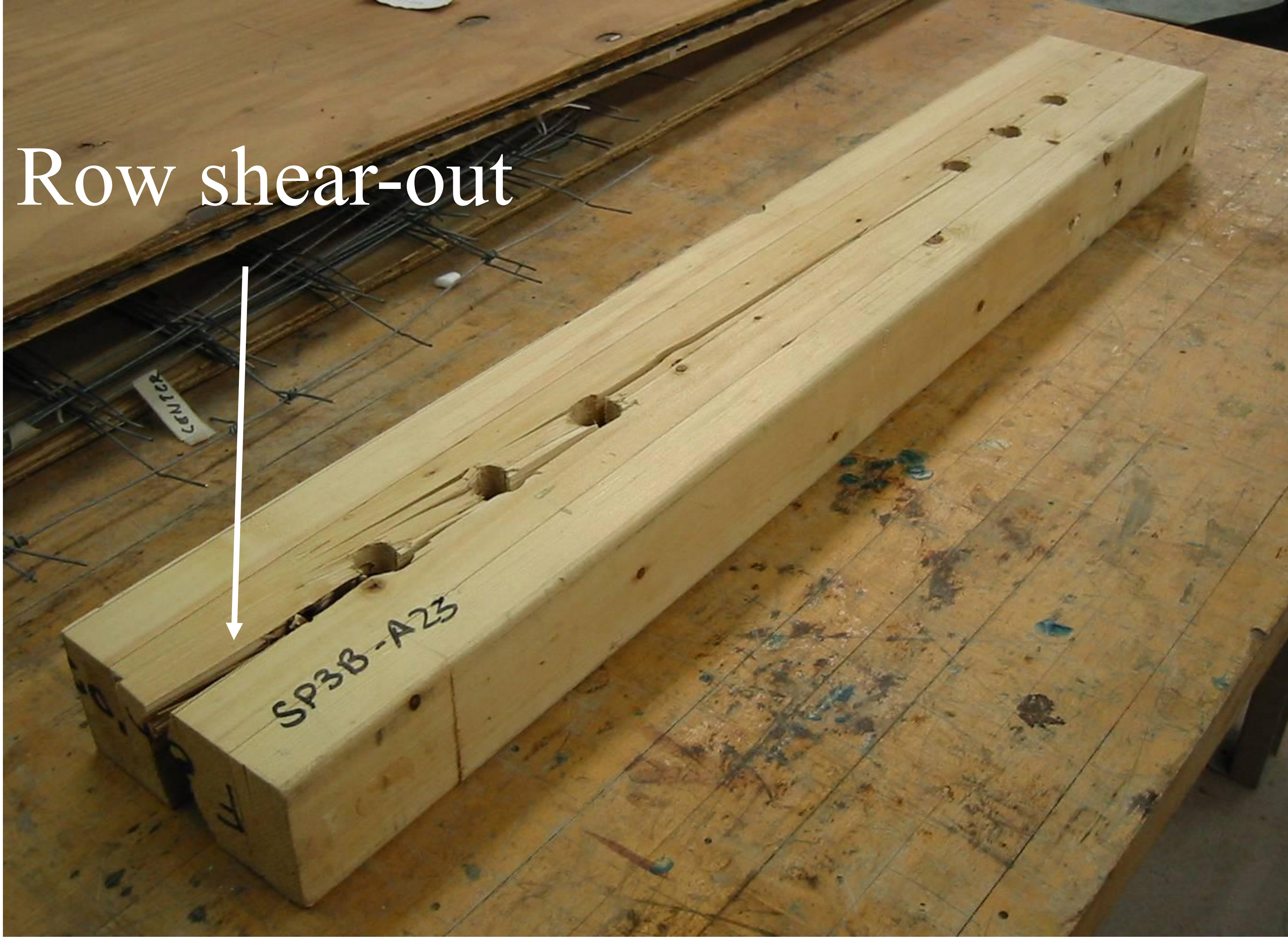
Net tension



Splitting

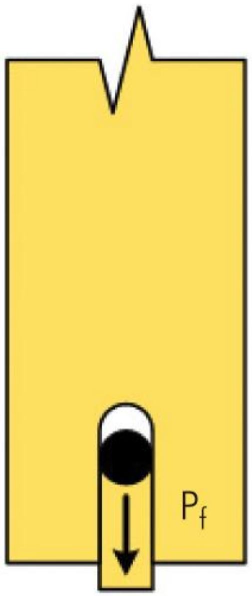
Failure modes in bolted connections

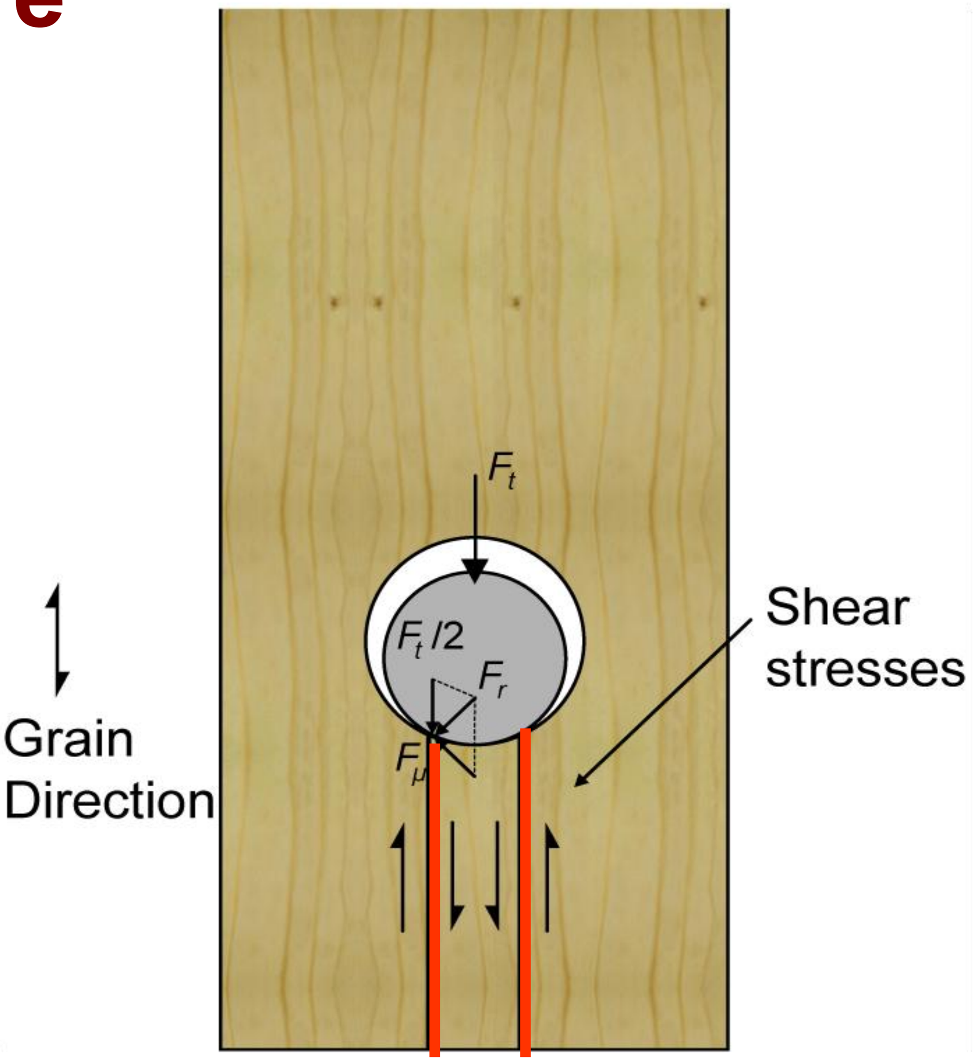




Source: Quenneville

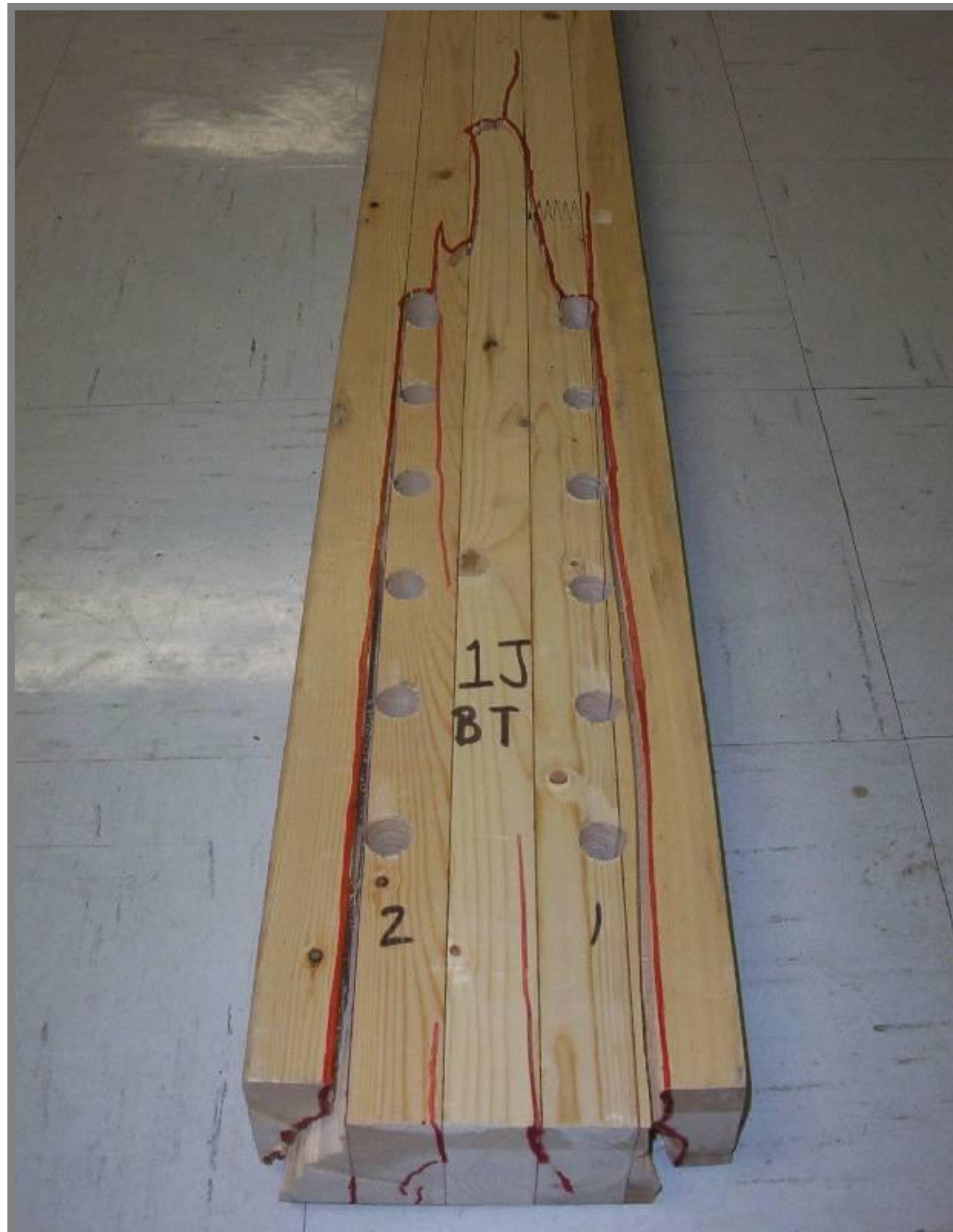
Row Shear Failure





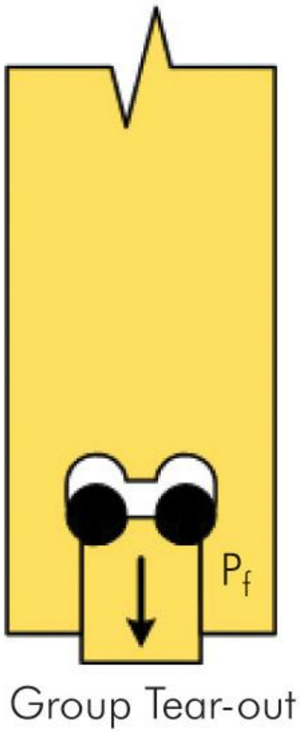
Row shear is dependent on shear strength //

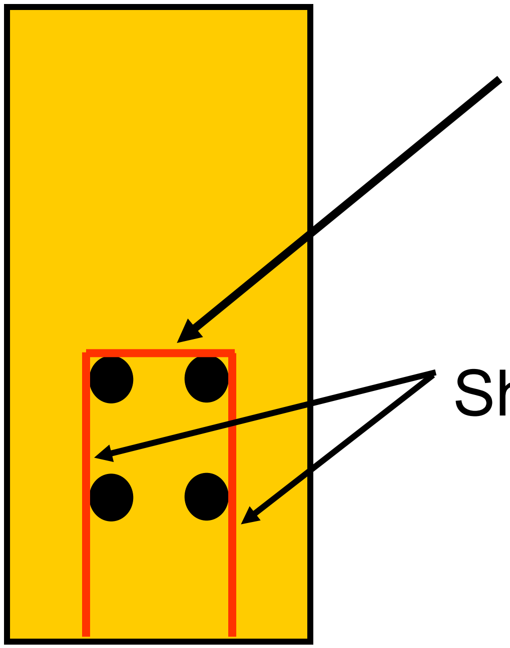
Group Tear Out



Source: Quenneville

Group Tear-Out Resistance





Group Tear-out = Tension // + Shear //

Splitting

Source: Quenneville

12.4.4.2 Requirements

Connections shall be designed in accordance with the following requirements:

(a) $N_f \leq N_r$

where

N_f = factored load on the connection

N_r = factored lateral yielding resistance (Clause 12.4.4.3)

(b) $P_f \leq P_r$

where

P_f = factored load parallel to grain

P_r = factored resistance parallel to grain

= the lesser of PR_{rT}, PG_{rT}, or TN_{rT}

where

PR_{rT} = factored row shear resistance (Clause 12.4.4.4)

PG_{rT} = factored group tear-out resistance (Clause 12.4.4.5)

TN_{rT} = factored net tension resistance (Clause 12.4.4.6)

(c) $Q_f \leq Q_r$

where

Q_f = factored load perpendicular to grain

Q_r = QS_{rT}

where

QS_{rT} = factored splitting resistance (Clause 12.4.4.7)

(d) for loading at an angle to grain, θ:

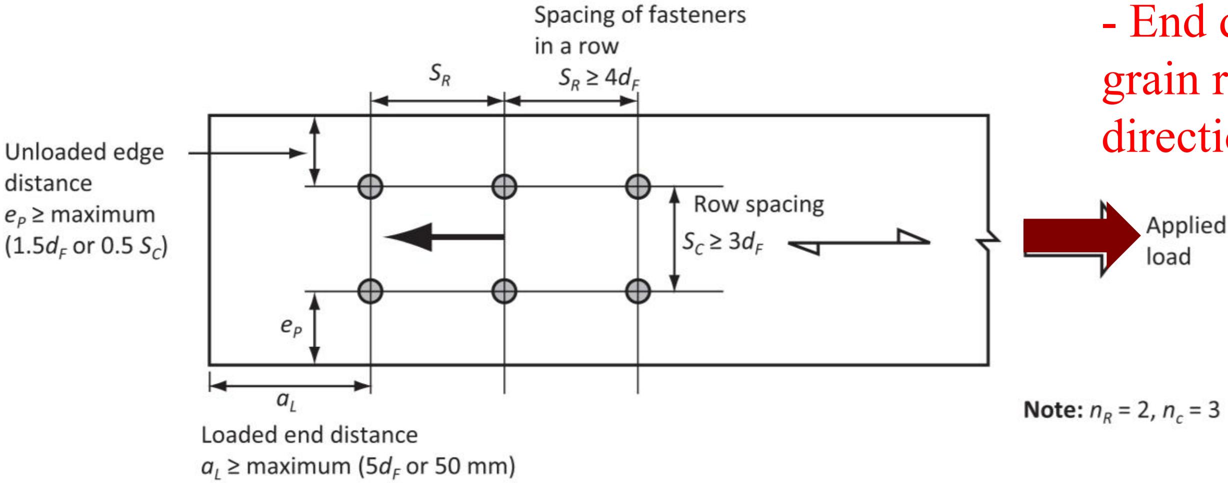
$$N_f \leq \frac{P_r \; Q_r}{P_r \sin^2 \theta + Q_r \cos^2 \theta}$$

where

θ = angle between the applied load and the grain

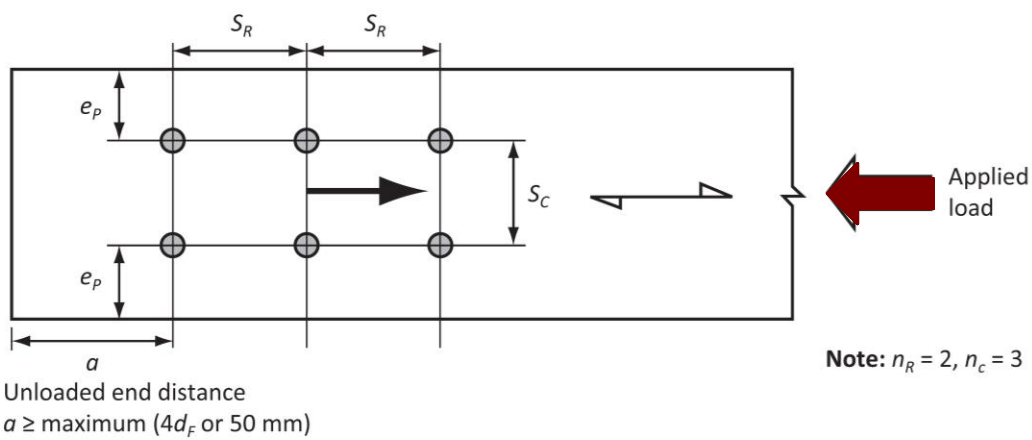
Figure 12.4.3.1
Placement of bolts and dowels in a connection loaded parallel to grain

- Row is in direction of loading
- End distance is in direction of grain regardless of loading direction



a) **Member in tension**

Load applied by wood



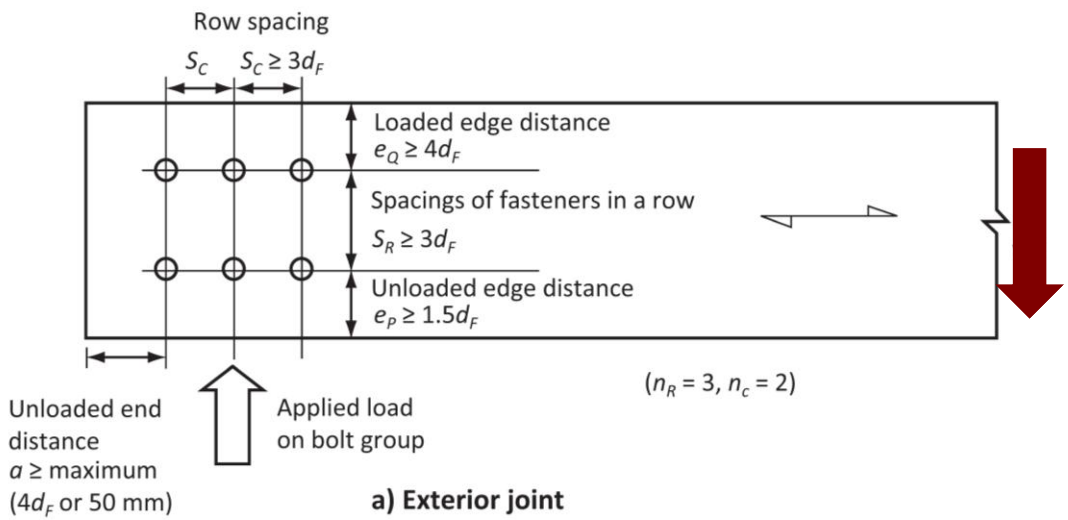
b) **Member in compression**

Legend:

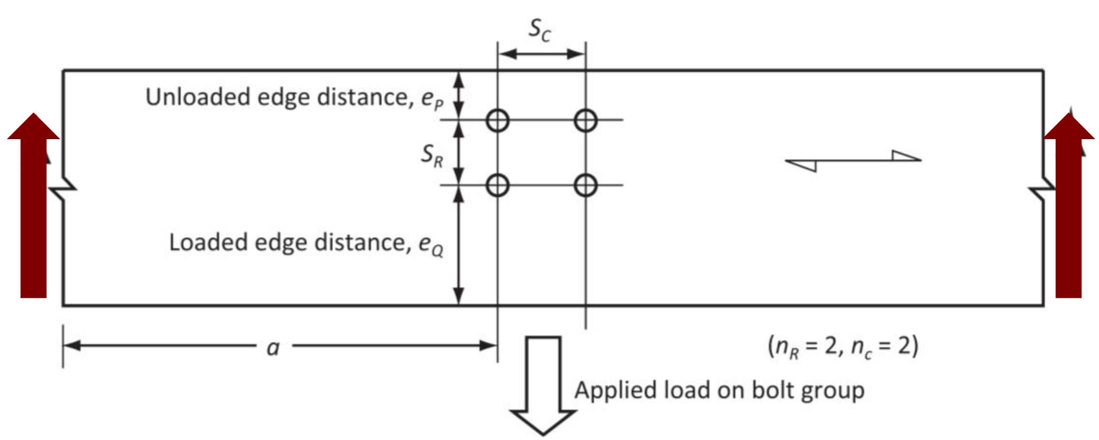
~~S_R = spacing of fasteners in row~~
~~S_C = row spacing~~
a = unloaded end distance
~~a_L = loaded edge distance~~
e_P = unloaded edge distance

Corrections:
S_R = spacing parallel to grain
S_C = spacing perpendicular to grain
a_L = loaded end distance

Figure 12.4.3.2
Placement of bolts and dowels in a connection loaded perpendicular to grain



a) Exterior joint



b) Interior joint

Legend:

S_R = spacing of fasteners in row

S_C = row spacing

a = unloaded end distance

e_Q = loaded edge distance

e_p = unloaded edge distance

Corrections:

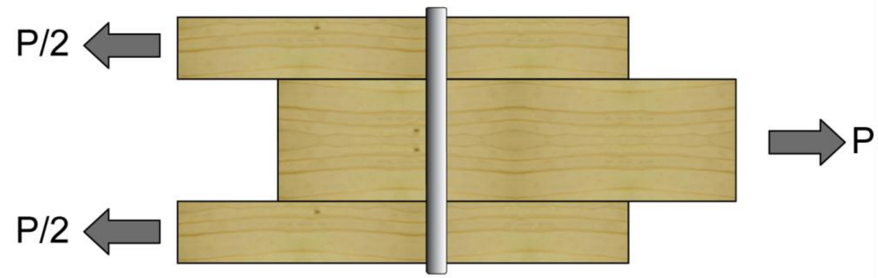
S_C = spacing parallel to grain

S_R = spacing perpendicular to grain

Row Shear Resistance (12.4.4.4)

Total RS resistance for **All Members** resisting the load

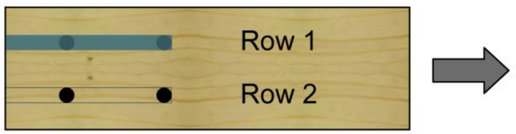
$$PR_{r\ T} = \Sigma\ (PR_{r\ i})$$



RS of All members

= No. of members X RS/member

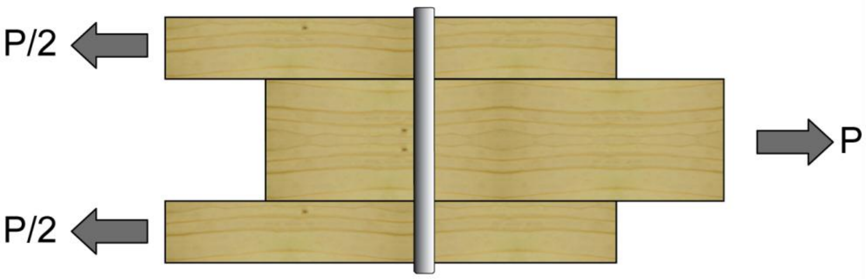
RS/member = RS/row x No. of rows

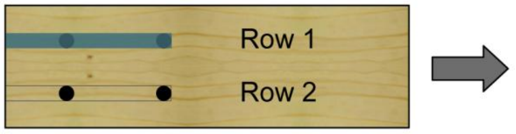


Row Shear Resistance (12.4.4.4)

Total RS resistance for **<u>All Members</u>** resisting the load

$$PR_{r\,T} = \Sigma\,(PR_{r\,i})$$





RS Per Member

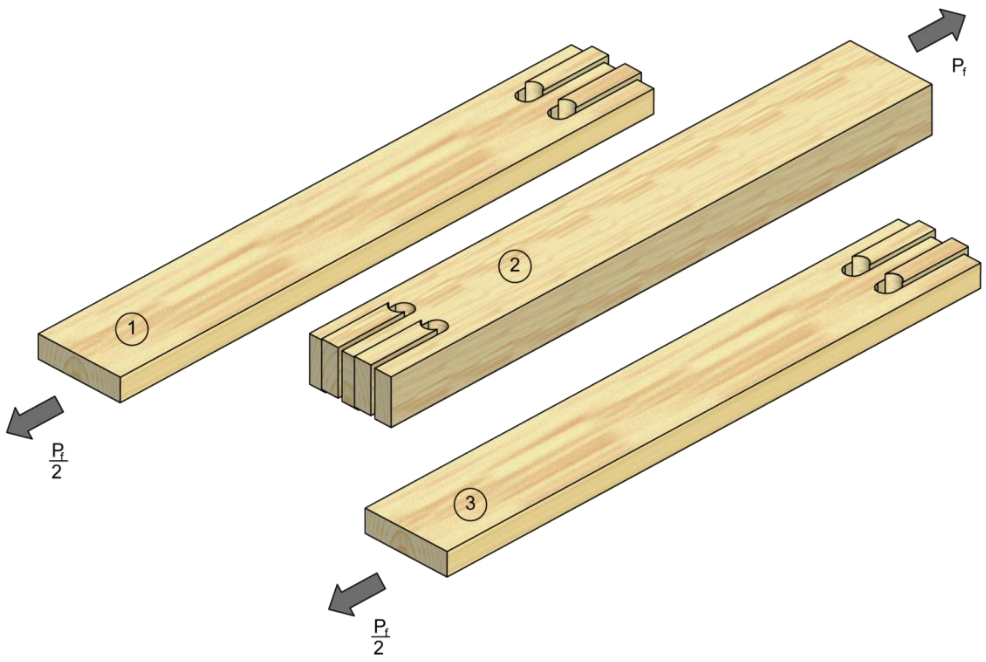
$$PR_{r\,i} = \phi_w\, PR_{ij\,min}\, n_R\, (K_D\, K_{SF}\, K_T)$$

Per row

No. of rows

Row Shear Resistance: (PR_{ij})/Member

$$PR_{ij\ min} = \min(PR_{i1}, PR_{i2}, \ldots PR_{in_R})$$



Row Shear Resistance

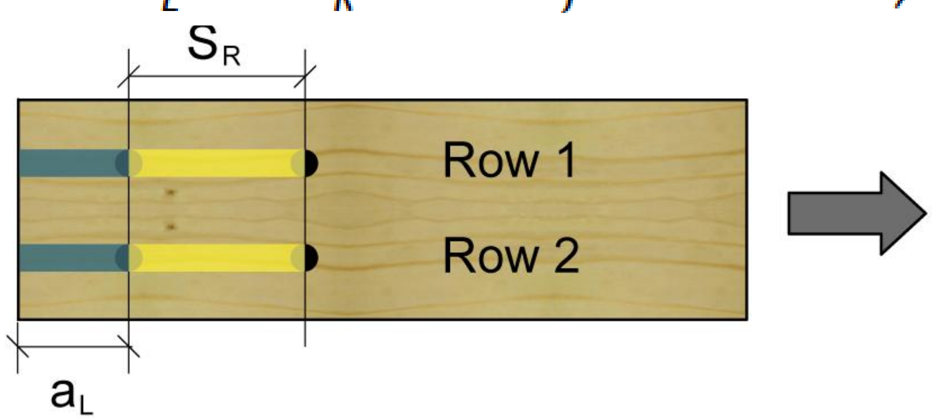
- Row shear resistance/row (j) per Member (i)

$$PR_{r\,i} = 1.2\, f_v\, K_{ls}\, t\, n_C\, a_{Cri}$$

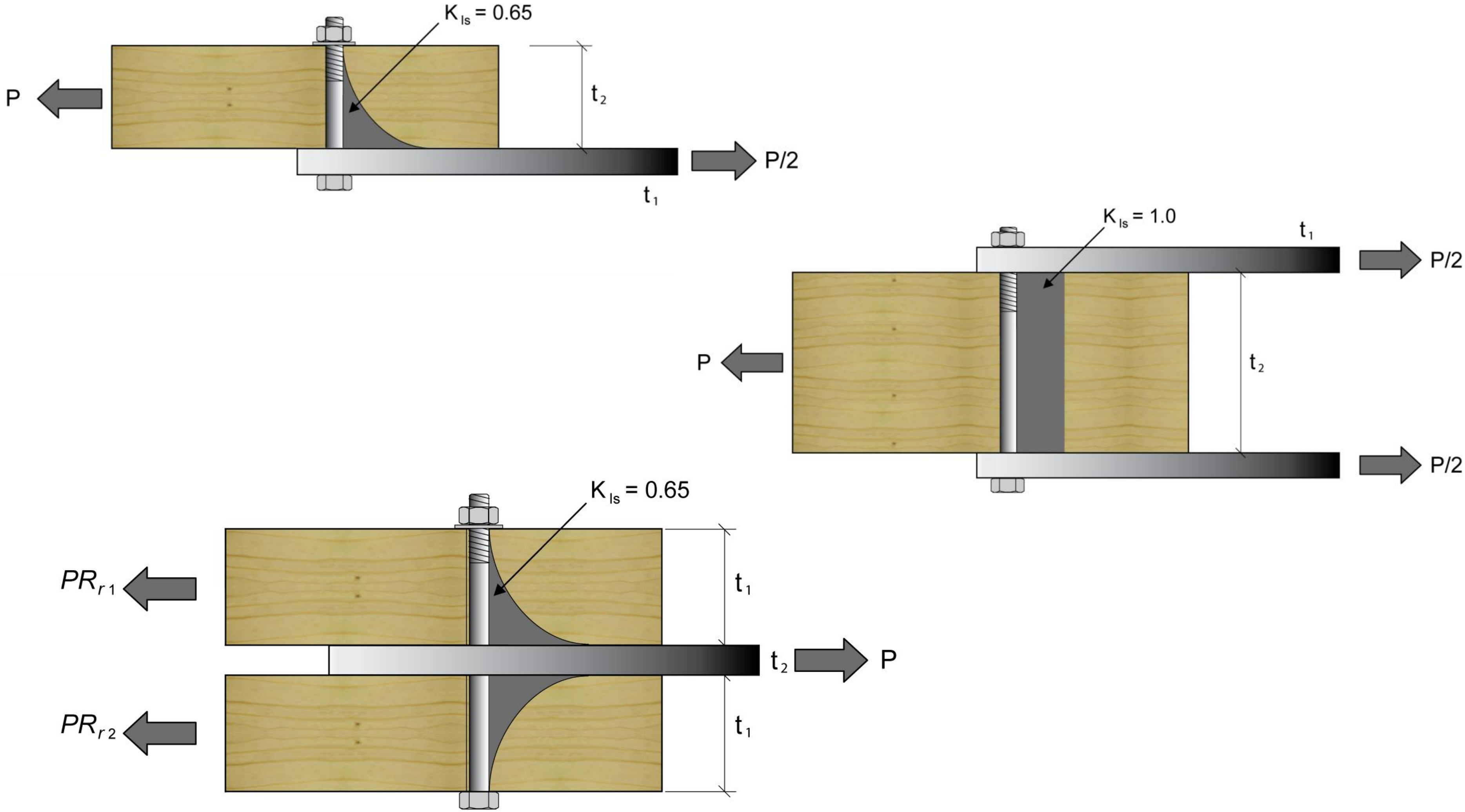
f_v = shear strength
t = thickness
n_C = no. of fasteners

K_{ls} = factor for member loaded surfaces
= 0.65 for side member
= 1 for internal member

$a_{cr\,i}$ = minimum of a_L and S_R for row j of member i, mm



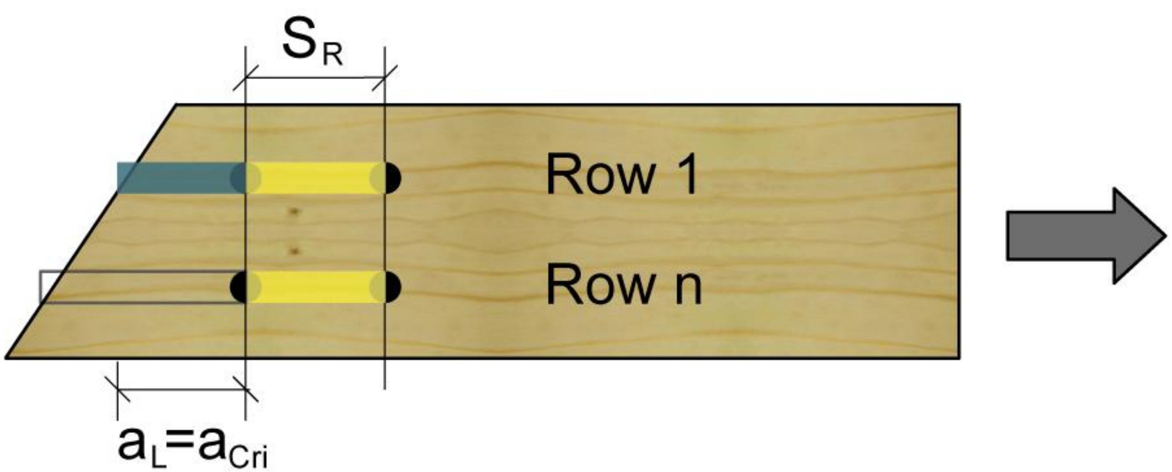
What is K_{ls}?
Effective thickness factor



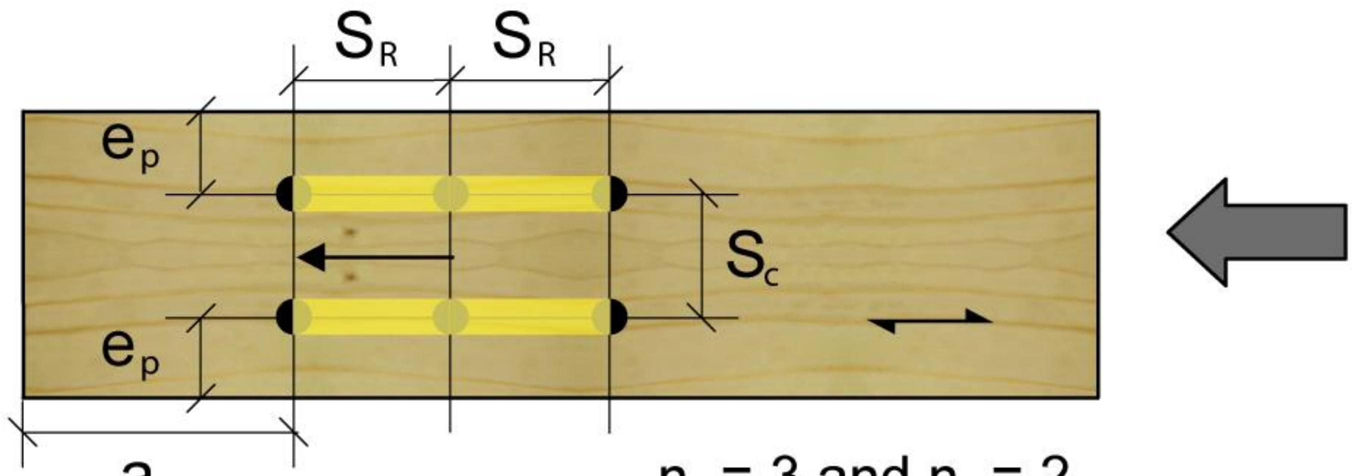
Row Shear Resistance

- Row shear resistance/row (j) per Member (i)

$$PR_{r\,i} = 1.2\, f_v\, K_{Is}\, t\, n'_C\, a_{Cri}$$



Row Shear Resistance



$n_c = 3$ and $n_R = 2$

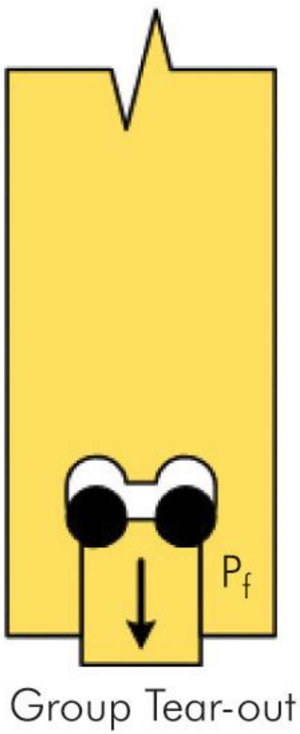
(b) Member in copmression

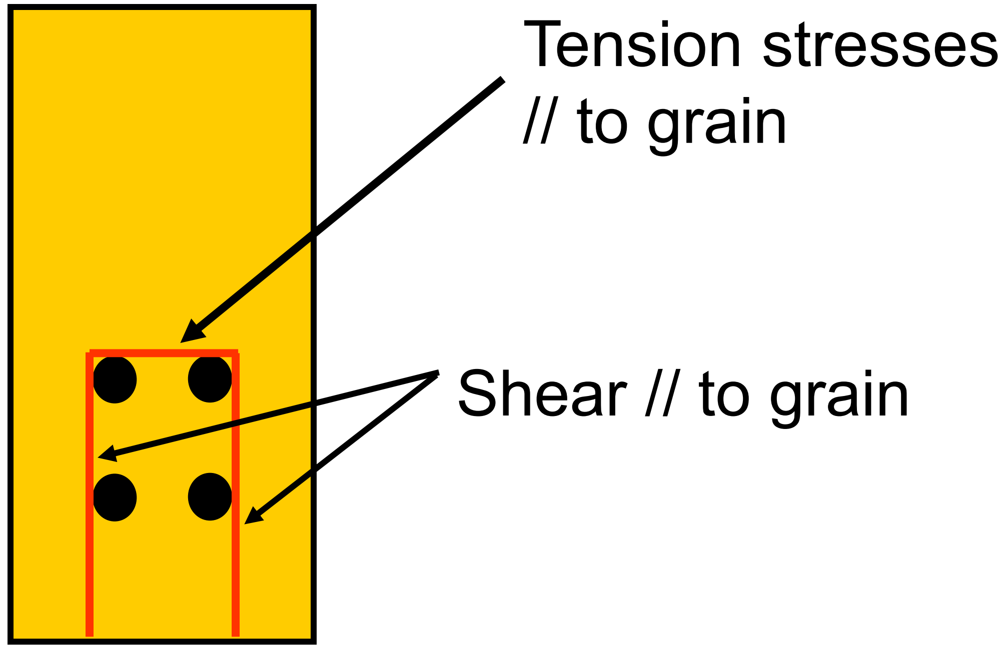
unloaded end distance

a≥ maximum (4 d_F or 50 mm)

Row shear could also occur in cases where the member is loaded in compression – $a_{cr\ i} = S_R$

Group Tear-out Resistance (12.4.4.5)

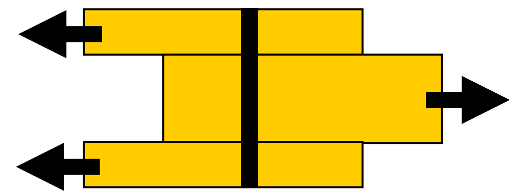




Tension Stress // + Shear = Group Tear-out

Group Tear-out Resistance

$$PG_{r\ T} = \Sigma\ (PG_{r\ i})$$

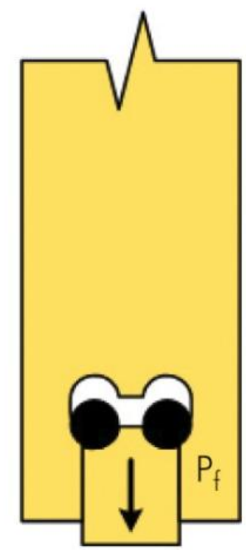


Group Tear out per Member (i): 2 components:

Row Shear

$$PG_{r\ i} = \phi_w \left[(0.5\ (PR_{i\ 1} + PR_{i\ nR}) + (f_t \cdot A_{PGi}\ (K_D\ K_{SF}\ K_T))\right]$$

Net Tension



Group Tear-out Equation

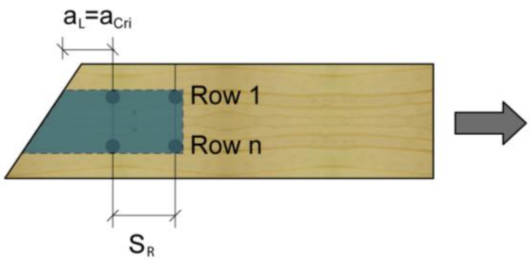
Shear component in Member (i):

$$PR_{i1} = 1.2\, f_v\, (K_D\, K_{Sv}\, K_T)\, K_{Is}\, t\, n_C\, a_{cr\,1}$$

$$PR_{inR} = 1.2\, f_v\, (K_D\, K_{Sv}\, K_T)\, K_{Is}\, t\, n_C\, a_{cr\,nR}$$

Tension component in Member (i):

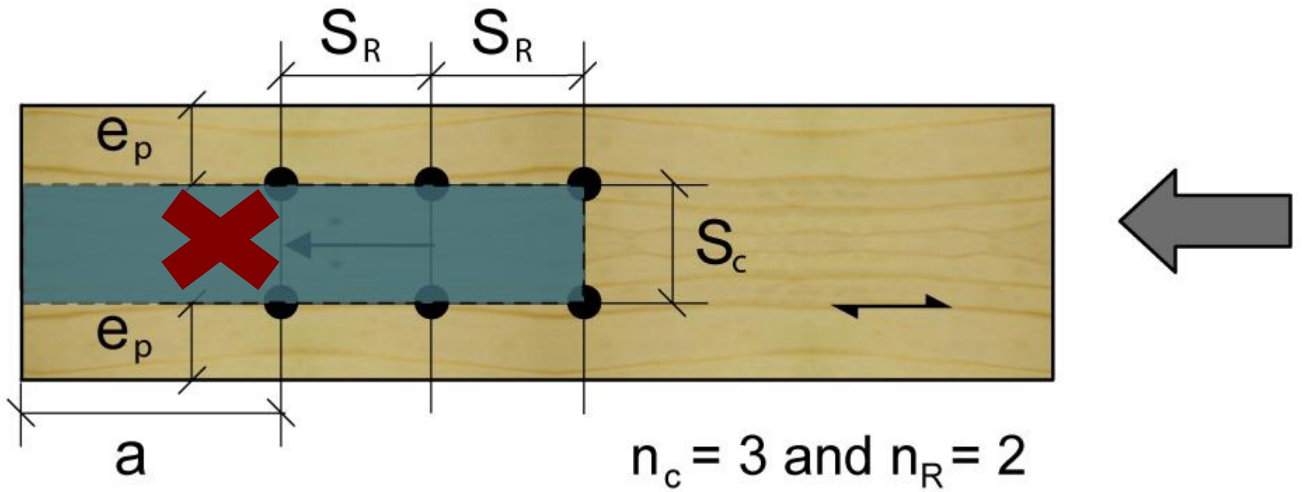
$$f_t\, (K_D\, K_{St}\, K_T)\, A_{PGi}$$



A_{PGi} = critical perpendicular net area between rows 1 and n_R of member i, mm^2

i.e. the area between the outer 2 rows excluding all bolt holes

Group Tear-out Equation



unloaded end distance

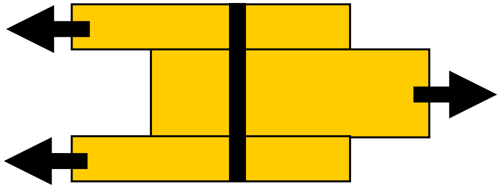
a≥ maximum (4 d_F or 50mm)

(b) Member in copmression

GT failure mode **is not possible** if the member is in compression!

Net Tension Resistance (12.4.4.6)

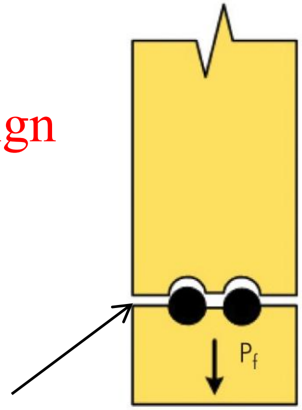
$$T_{Nr\ T} = \Sigma\ T_{Nr\ i}$$



Net Tension per <u>Member (i):</u>

$$T_{Nr\ i} = \phi\ f_t\ A_n\ (K_D\ K_{SF}\ K_T)$$

Tension member design



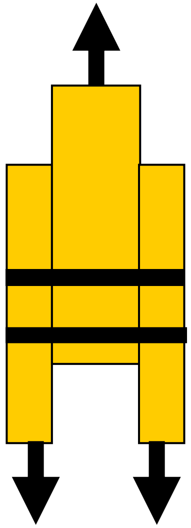
where
f_t = specified tensile strength of wood member
A_n = net cross sectional area (not less than 75% of gross area)

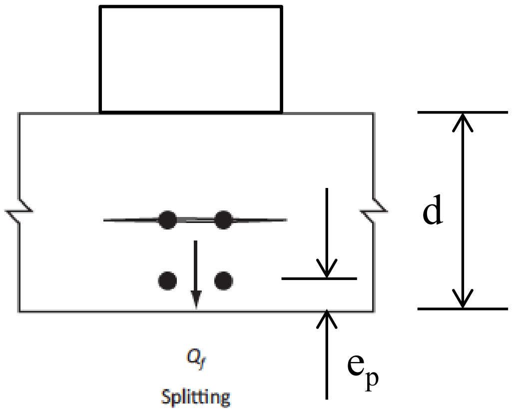
Perp to Grain Splitting Resistance (12.4.4.7)

$$QS_{r\,T} = \Sigma\, QS_{ri}$$

For member i,

$$QS_{ri} = \phi_w\, QS_i\, (K_D\, K_{SF}\, K_T)$$





$$QS_i = 14\, t \sqrt{\frac{d_e}{1 - \frac{d_e}{d}}}$$

where

t = thickness of member, mm

d_e = effective depth of member, mm

= $d - e_p$

where

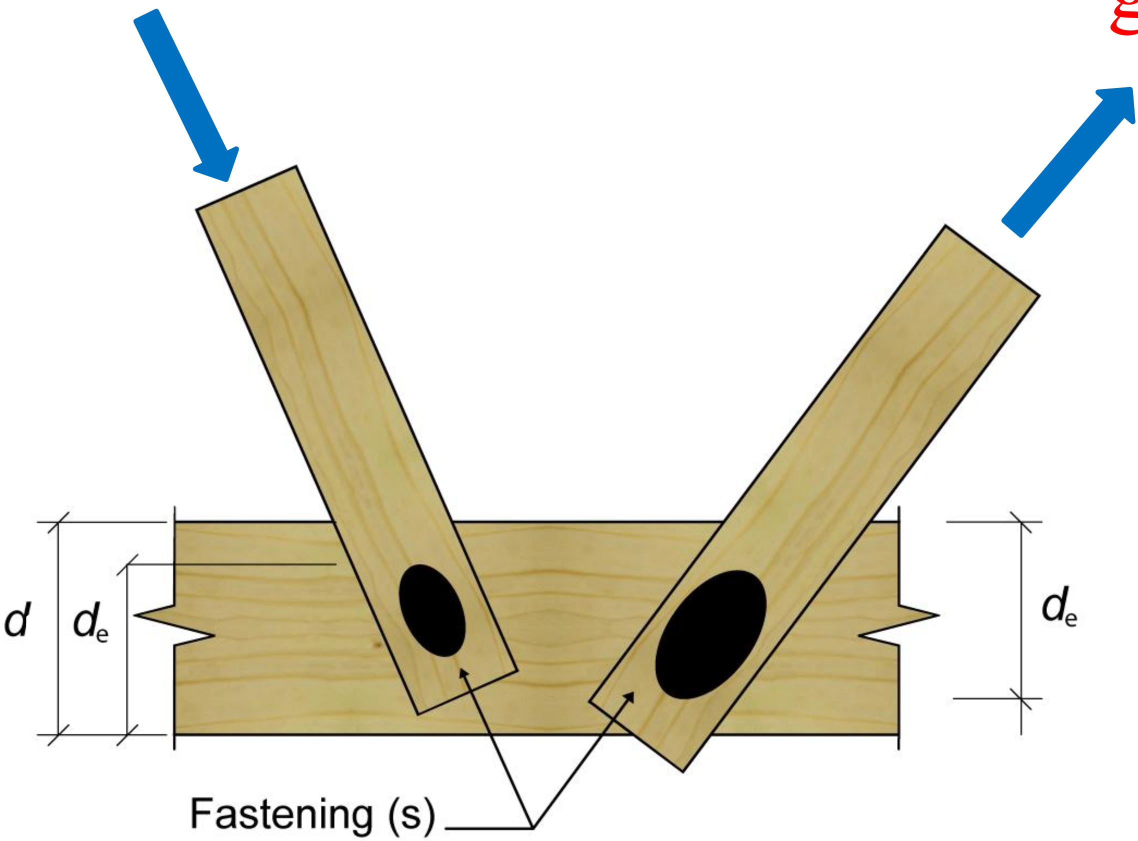
d = depth of member, mm

e_p = unloaded edge distance, mm

Member shear check based on effective depth (12.2.1.5)

$$V_{r\ neti} = \phi \ \frac{2}{3} f_v d_e t$$

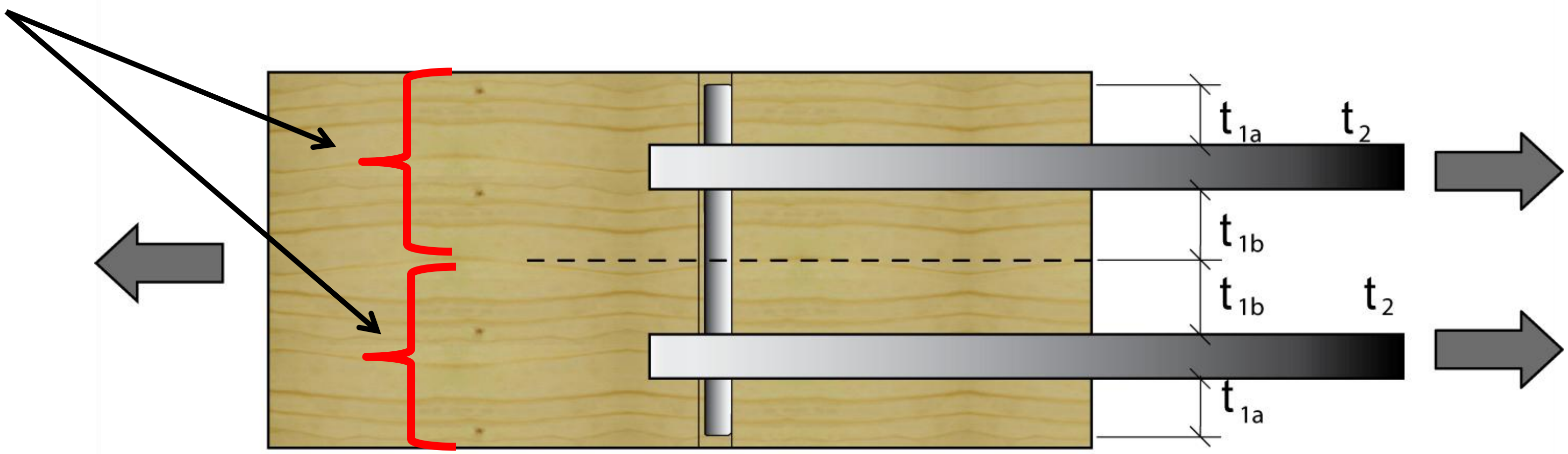
Minimum of $QS_{r\ T}$ or $V_{r\ neti}$ governs



d_e = effective depth (distance measured perp to axis of member from extremity of fastener group to loaded edge of member)

Multiple Members (12.4.1.3)

2 @ 3 members joint



$$t = \min(t_{1a}, t_{1b})$$

For a joint with 4 or more members, the resistance shall be determined assuming that each member is a part of a series of 3-member joint

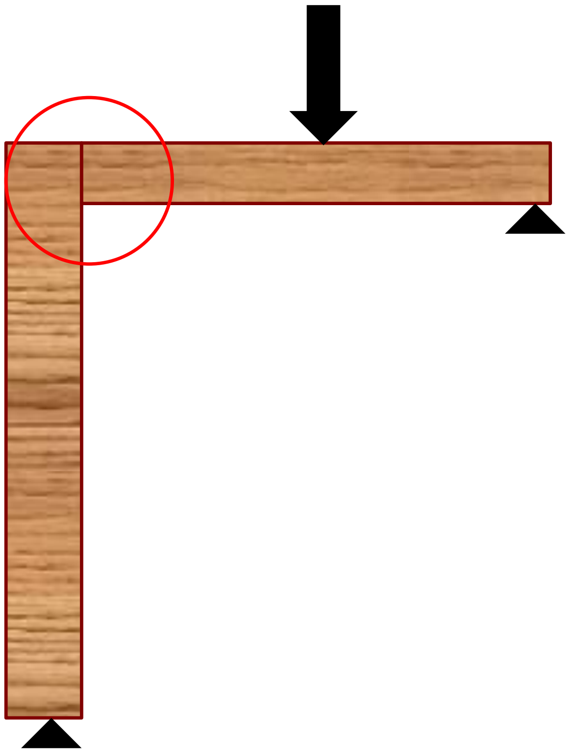
Design: Available Information

- Member sizes

- Species and grade

- Force to be transferred

- Condition factors (i.e., K_{SF}, K_D & K_T)



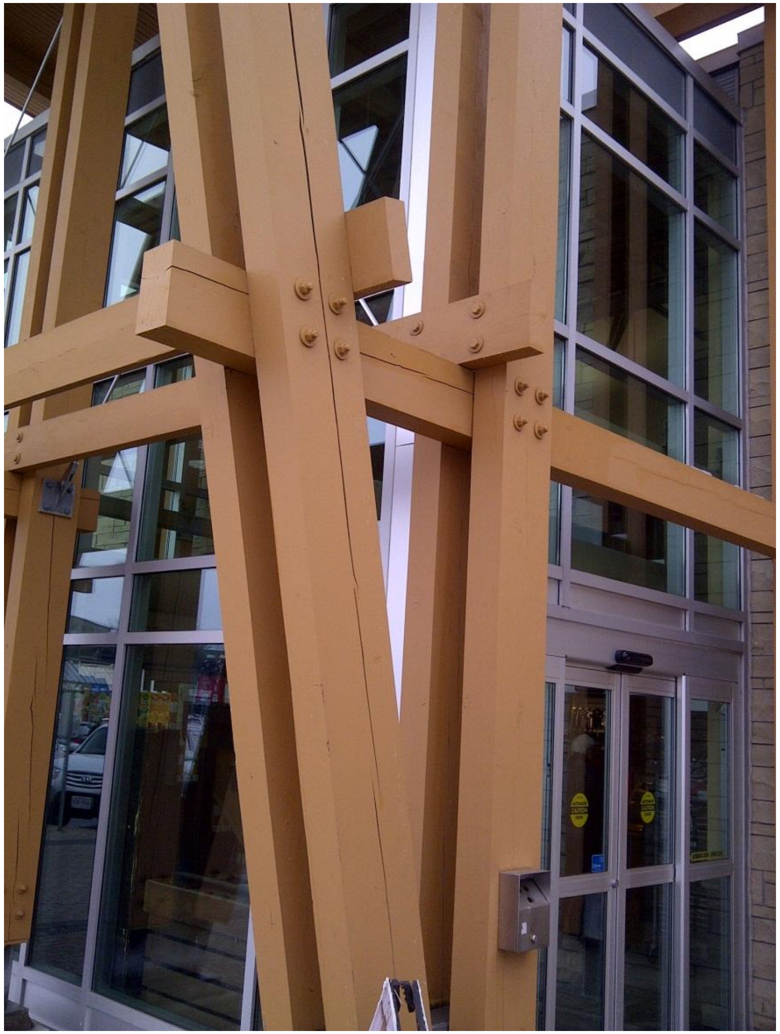
General design procedure for bolt and dowel connections

1. Choose bolt size and grade
2. Calculate the embedment resistance (f_1 and f_2)
3. Choose no. of bolts per row & no. of rows (n_c) & specify spacings & end distances
4. Calculate the yield resistance for the possible failure modes – minimum governs – adjust if necessary

General design procedure for bolt and dowel connections

5. Check brittle resistances (e.g. row shear, group tear-out, net tension resistances, and if appropriate, splitting)
6. Modify bolt spacings if governing resistance is inadequate ($a_L = S_R$ for optimum spacing)

Jointing members in one connection could be loaded at different grain angle



General design procedure for bolt and dowel connections

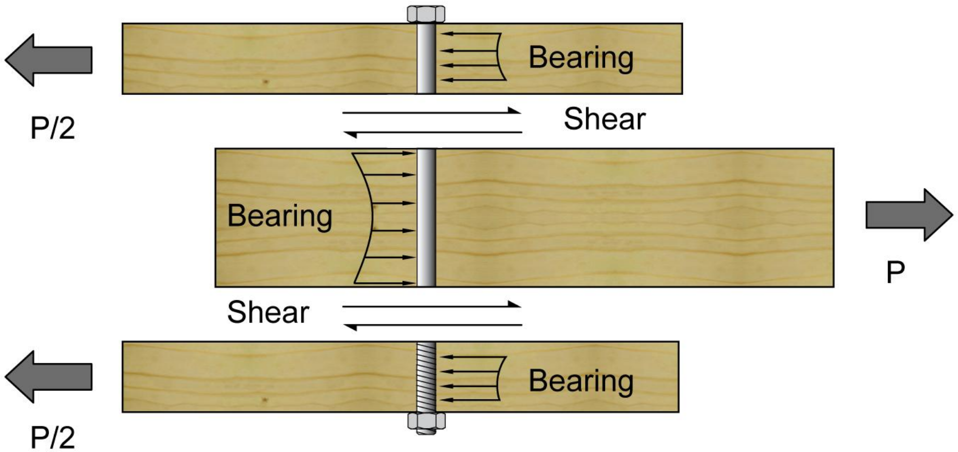
If still inadequate;

- Change bolt size (bigger is not necessarily better as volume of wood required increases)
- Try adding a shear plane
- Change member size to allow for more bolts
- Change member grade

Structural Efficiency: Load Carrying Capacity of Timber Connections

Generally, nails or dowels into pre-drilled holes through steel plates, are the most efficient connectors

End-grain connectors, where the load transfer is through direct compression, are <u>the most efficient connectors</u>



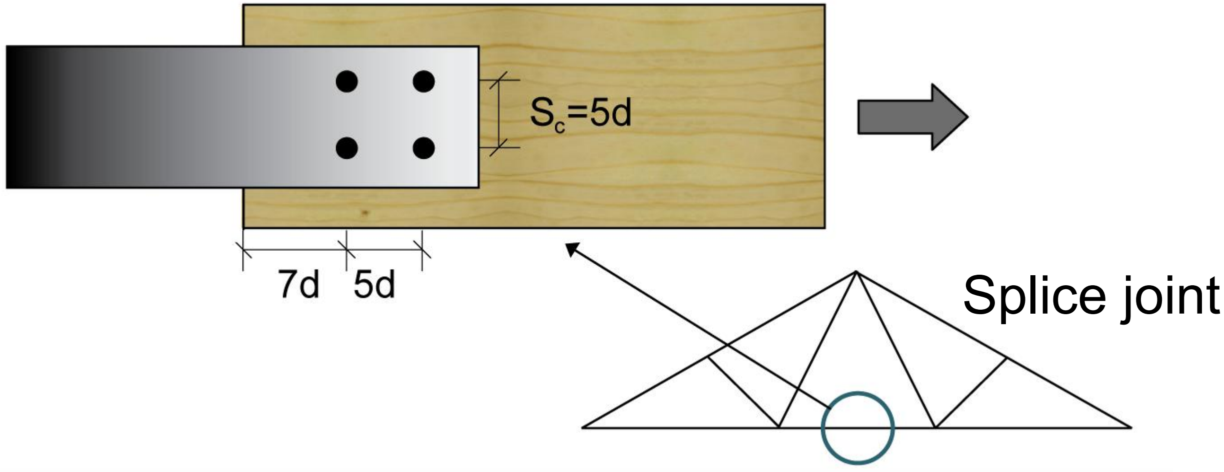
Note:

The choice of fastener will depend on the available space for the connection, aesthetics and other factors

Exercise 1

Steel-to-Wood Bolted Connection

Steel-Wood-Steel Connection



- D-fir glulam, 20f-EX, 130x190 mm
- Steel-wood-steel connection, 6.35 mm steel side plates
- ¾" bolts, 7d end distance, 5d bolt spacing, 5d row spacing
- (Assume dry, no treatment and dead and live load combination)

What is the factored resistance ?

Yielding Capacity

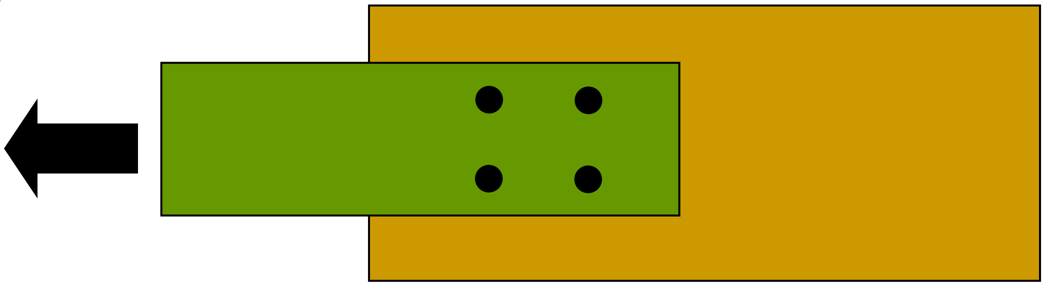
n_u = **24.6** kN/shear plane - (c) GOVERNS

$$N_r = \phi \, n_u \, n_s \, n_f$$

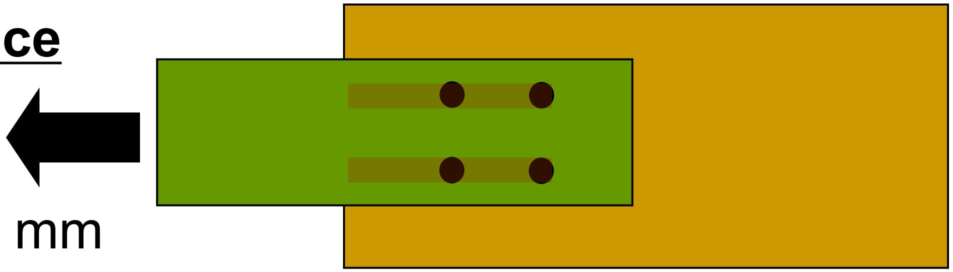
$$= 0.8 \cdot 24.6 \text{ kN}/n_s \cdot 2 \cdot 2 \cdot 2$$

Yielding Capacity

N_r = 157 kN



Brittle Resistance



Evaluate the ROW SHEAR resistance

f_v = 2.0 MPa (Table 7.3)

$a_{cr\,i} = \min(a_L, S_R) = \min(134, 95) = 95$ mm

$K_{ls} = 1$

$n_c = 2$

$1.2\, f_v\, K_{ls}\, t\, n_c\, a_{cr\,i} = 1.2 \cdot 2 \cdot 1 \cdot 130 \cdot 2 \cdot 95 =$ **59.3 kN/row**

$PR_{r\,i} = 0.7 \cdot 59.3 \cdot 2 \cdot 1 \cdot 1 \cdot 1 =$ **83.0 kN**

Res. per Wood Member

Since there is one wood member, **$PR_{r\,T}$ = 83.0 kN**

Brittle Resistance

Evaluate the GROUP TEAR-OUT resistance

f_t = 20.4 MPa Specified net tension – **Table 7.3**

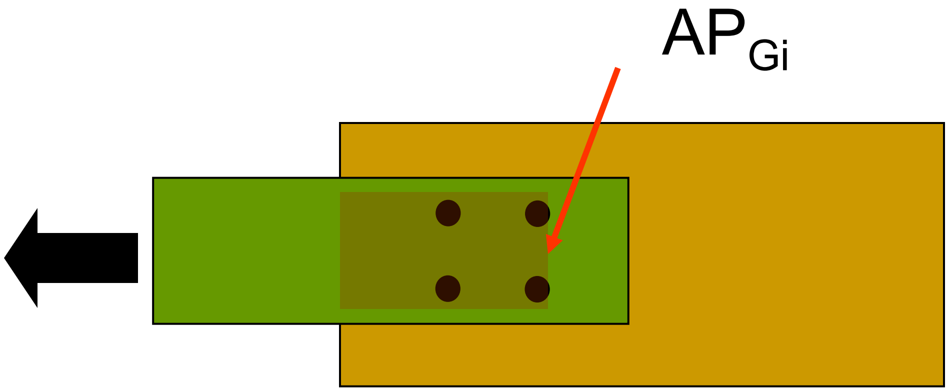
$A_{PGi} = (S_c - d_f) \cdot t = 76.4 \cdot 130 = 9932 \text{ mm}^2$

$PG_{r\,i} = 0.7 \cdot [(59.3 + 59.3)/2 + [(20.4 \cdot 1 \cdot 1 \cdot 1) \cdot 9932]$

$= \mathbf{183.3\ kN}$

With one wood member,

$PG_{r\,T}$ = 183.3 kN



Brittle Resistance

Evaluate the NET TENSION resistance (Cl 7.5.11)

$$T_r = \phi\, F_{tn}\, A_n = \phi\, (f_{tn}\, K_D\, K_H\, K_{SF}\, K_T)\, A_n$$

$$= 0.9 \cdot (20.4 \cdot 1 \cdot 1 \cdot 1 \cdot 1) \cdot (130 \cdot (190 - 2 \cdot (19.1+2)))$$

$\underline{T_r = 359.7\ kN}$

Connection Capacity

D-fir glulam, 20f-EX, 130x190 mm

Steel-wood-steel connection, 6.35 mm steel side plates

¾" bolts, 7d end distance, 5d bolt spacing, 5d row spacing

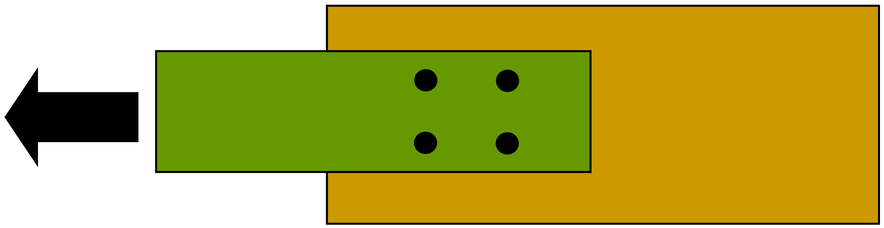
(assume dry, no treatment and dead and live load combination)

Nr = 157 kN Yielding Resistance

$PR_{r\,T}$ = 83.0 kN ROW SHEAR GOVERNS

$PG_{r\,T}$ = 183.3 kN Group tear-out

T_r = 359.7 kN

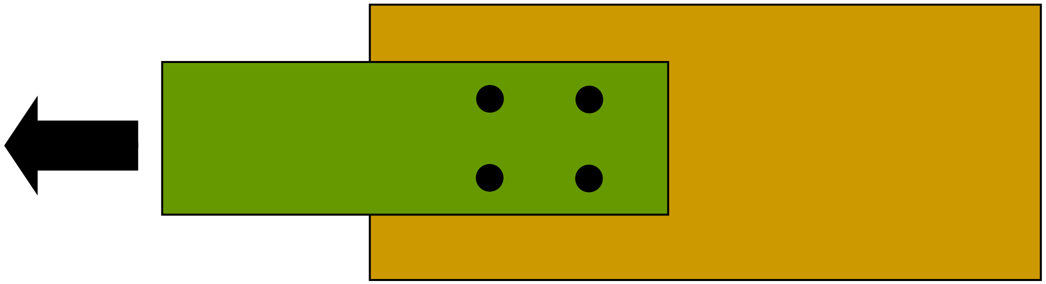


Changing Connection Design

- The maximum ductile resistance using **4 bolts** is **N_r = 157 kN**

- To force ductile behaviour, it is necessary to **increase** the bolt spacing in the row (a_L and S_R) and row spacing (S_C)

- By doing so, the row shear and group tear out resistances will increase, ultimately reaching yielding resistance (remains constant)



End
Lecture #6

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- Some of the pictures and drawings are provided by Dr. Pierre Quenneville, Mohammad Mohammad, and Dr. Jasmine B.W. McFadden