Ministry of Economics
Republic of Latvia

Apmācību semināru cikls

«Moderno koka konstrukciju projektēšana un ugunsaizsardzība»

ID Nr. EM 2021/42

Rīga, 2021

Ministry of Economics
Republic of Latvia

Training seminar / Apmācību seminārs

Modern Timber Structures

Modernas koka konstrukcijas

October 19, 2021, Riga

Andrew Lawrence and Ishan Abeysekera (United Kingdom)

Technical guidance

ETIQUETTE

- Stay focused and be with us
- Keep an eye on the clock
- Event is recorded, help us maintain recording quality
- Use your real name, rename if necessary
- Register by writing your full name in chat
- Choose any preferred language
- Ask questions in chat and monitor it
- Mute / stop webcam
- Ensure good connection

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Agenda

09:00 – 10:00	**Registration**
10:00 – 11:30	Introduction Timber properties + EC5 Modern engineered timber materials
11:30 – 12:00	**Coffee break**
12:00 – 13:30	Timber construction systems Timber connections Designing with a brittle material
13:30 – 14:00	**Lunch break**
14:00 – 15:30	CLT Floor dynamics – the new EC5 method Specification + common errors (Q&A)

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- We have interpreted building codes, research papers, industrial documents and so on, with a view to conveying current design practices. The material presented is only intended to provide guidance; the final design responsibility for any design lies with the structural engineer or designer.

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Agenda / 10:00 - 11:30

- Introduction
- Timber properties + EC5
- Modern engineered timber materials

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Timber Properties & EC5
Section No.1/ Sadaļa Nr.1.

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Tree Types

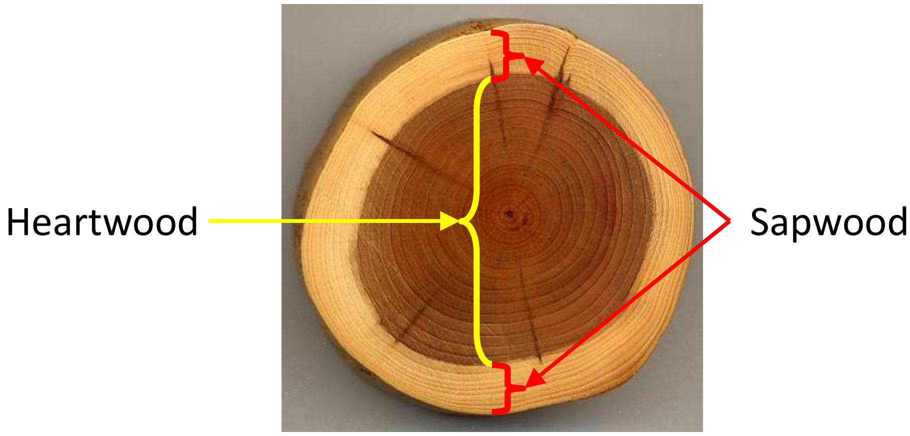
Softwood trees

- Spruce
- Pine
- Fir

Hardwood trees

- Ash
- Oak
- Teak

Tree Trunk



"File:Taxus wood.jpg" by MPF is licensed under CC BY-SA 3.0

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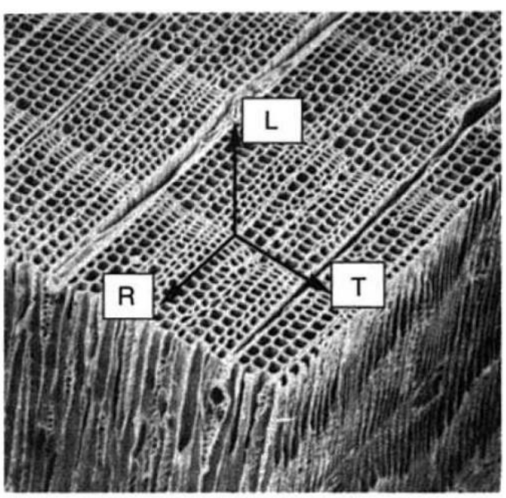
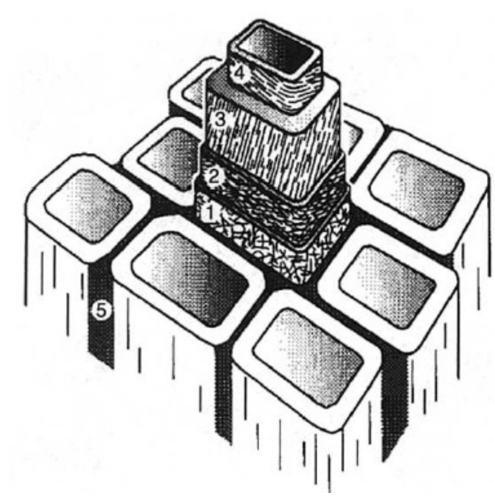
Microstructure of Timber



Thelandersson, S., Larsen, H. J., 2003, Timber Engineering, Wiley, England



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Designing With a Natural Material

cf. steel

- variable
- anisotropic

thus, different philosophy

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Knots

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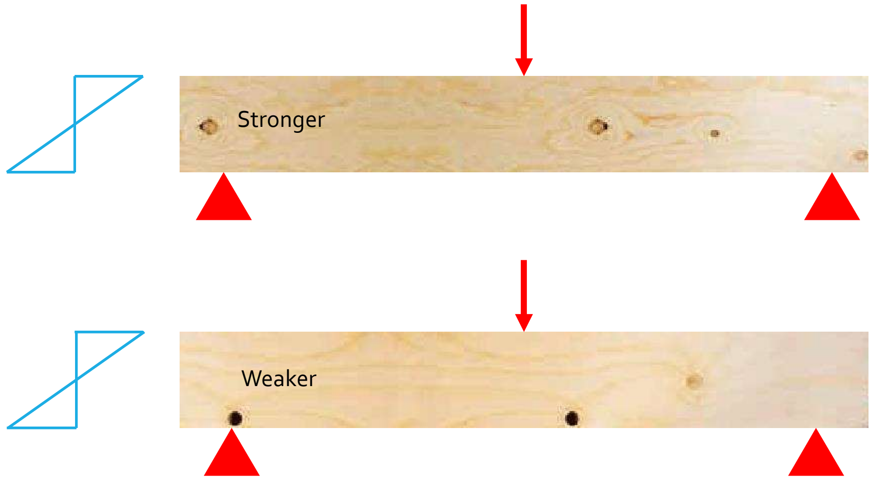
Knots

Knots have a greater effect on bending strength when they are closer to the edge



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Glulam Strengths in EN14080

Property	Symbol	GL 24h [N/mm²]
Bending strength	$f_{m,g,k}$	24
Tensile strength	$f_{t,0,g,k}$	19.2
	$f_{t,90,g,k}$	0.5
Compression strength	$f_{c,0,g,k}$	24
	$f_{c,90,g,k}$	2.5
Shear strength (shear and torsion)	$f_{v,g,k}$	3.5
Rolling shear strength	$f_{r,g,k}$	1.2
Modulus of elasticity	$E_{0,g,mean}$	11500
	$E_{0,g,05}$	9600
	$E_{90,g,mean}$	300
	$E_{90,g,05}$	250
Shear modulus	$G_{g,mean}$	650
	$G_{g,05}$	540
Rolling shear modulus	$G_{r,g,mean}$	65
	$G_{r,g,05}$	54
Density	$\rho_{g,k}$	385
	$\rho_{g,mean}$	420

BS EN 14080

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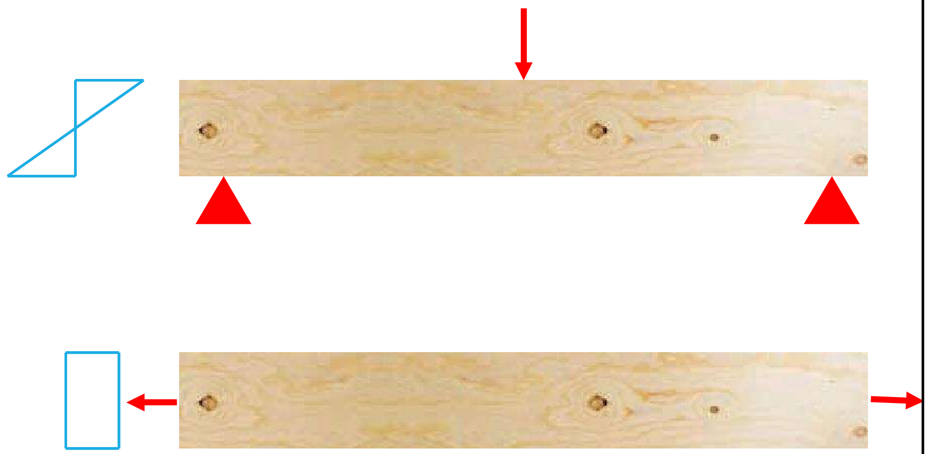
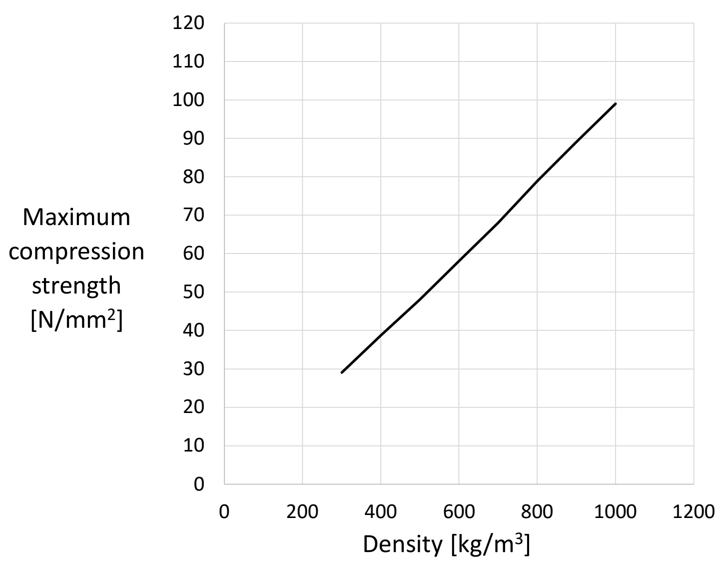
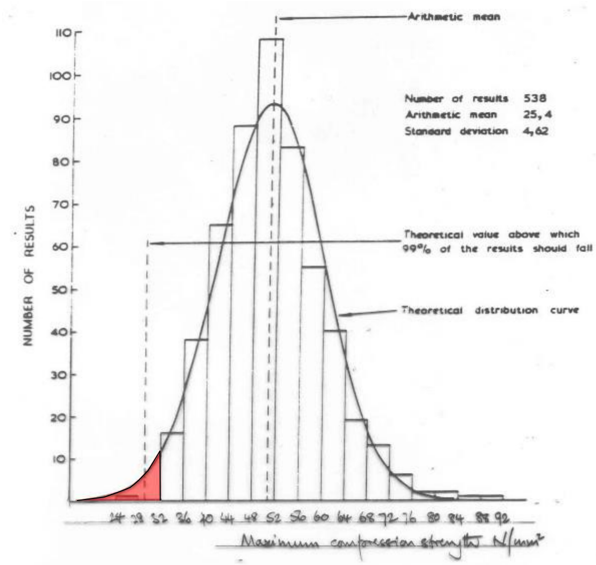
Why is tension strength weaker than bending strength parallel to grain?



Strength vs. Density Relationship



Strength Variability



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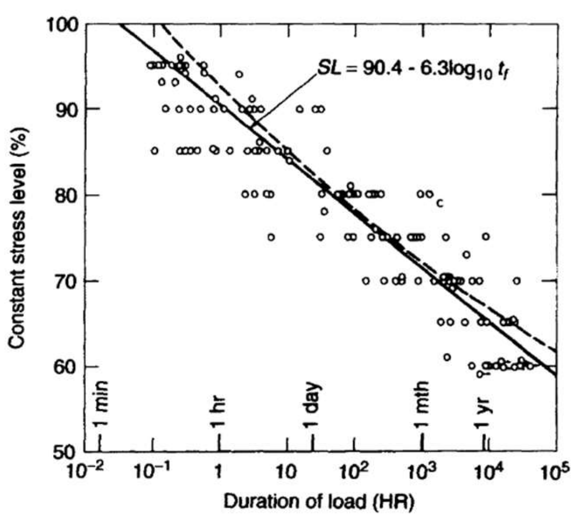
Strength vs. Load Duration

Strength of timber reduces with load duration



Thelandersson, S., Larsen, H. J., 2003, Timber Engineering, Wiley, England

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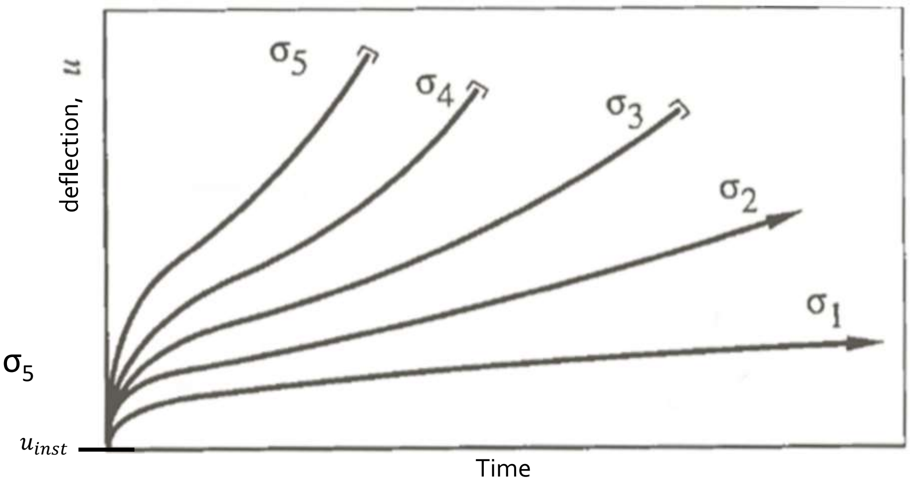
Strength vs. Load Duration

Long-term behaviour of timber for different stress levels where:

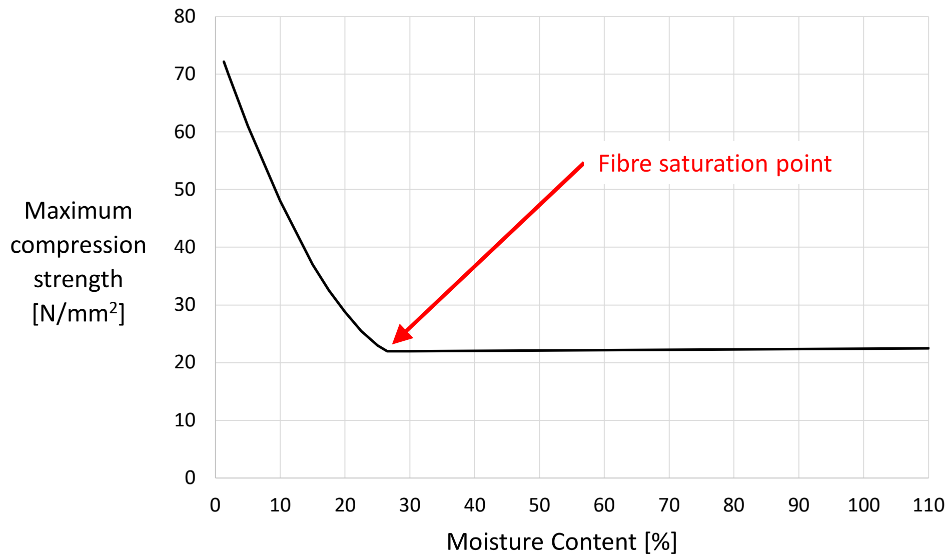
$\sigma_1 < \sigma_2 < \sigma_3 < \sigma_4 < \sigma_5$



STRUCTURAL TIMBER EDUCATION PROGRAMME. Timber Engineering STEP 1: Basis of design, material properties, structural components and joints. Report A19 Creep.

What's happening to this roof?

Strength vs Moisture Content



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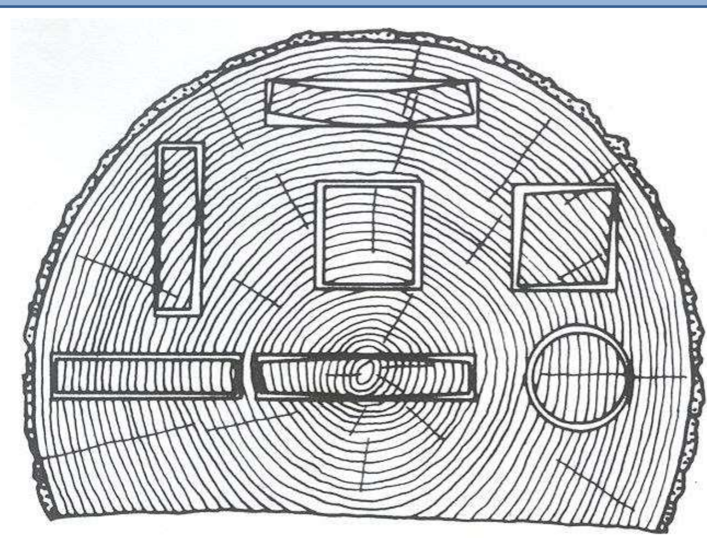
Shrinkage & Seasonal Movement



Thelandersson, S., Larsen, H. J., 2003, Timber Engineering, Wiley, England

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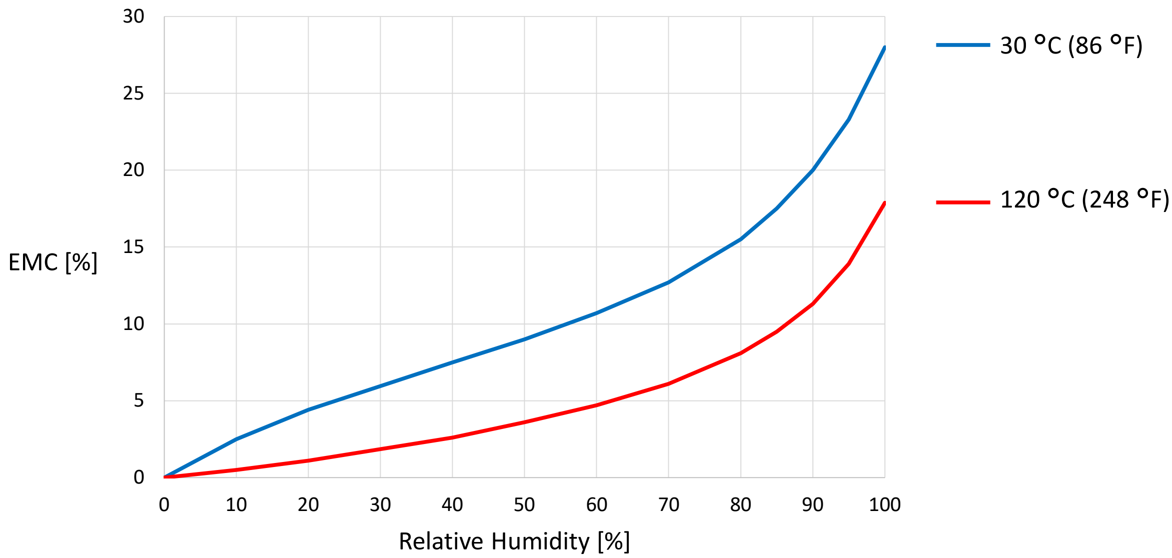
Softwood shrinkage

- 0.25% for every 1% change in EMC (perpendicular to grain)
- 0.0025% for every 1% change in EMC (Parallel to grain)

Why has the column split?

Equilibrium Moisture Content (EMC)



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Why doesn't the roof rot?

https://commons.wikimedia.org/wiki/File:Kryta_p%C5%82ywalnia_w_Ko%C5%9Bcianiee.jpg

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Durability

- Always keep timber dry.
- Keep out of the driving rain.
- Timber inside the building envelope is Class 1 in EN 1995-1-1

Timber is Brittle

- Loads will not redistribute .
- So timber is not as forgiving as well designed concrete (where failure is governed with steel yielding) or steel. Both of which are ductile.

Member Design to EN 1995

Design strength

Property	Symbol	GL 24h [N/mm²]
Bending strength	$f_{m,g,k}$	24
Tensile strength	$f_{t,0,g,k}$	19.2
	$f_{t,90,g,k}$	0.5
Compression strength	$f_{c,0,g,k}$	24
	$f_{c,90,g,k}$	2.5
Shear strength (shear and torsion)	$f_{v,g,k}$	3.5

BS EN 14080

$$f_{m,d} = \frac{k_{mod}\, k_h\, k_{crit}\, k_{sys}\, f_{m,k}}{\gamma_m}$$

Design strength

Characteristic strength

Material factor

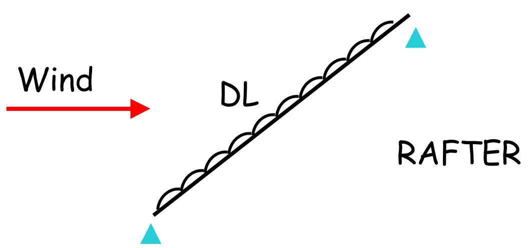
	Partial factor
Fundamental combinations	
Solid timber	1.3
Glued laminated timber	1.25
LVL, plywood, OSB	1.2
Particleboards	1.3
Fibreboards, hard	1.3
Fibreboards, medium	1.3
Fibreboards, MDF	1.3
Fibreboards, soft	1.3
Connections	1.3
Punched metal plate fasteners	1.25
Accidental combinations	1.0

BS EN 1995 (NA) – Table 2.3

$$f_{m,d} = \frac{k_{mod}\, k_h\, k_{crit}\, k_{sys}\, f_{m,k}}{\gamma_m}$$

Load duration

Strength varies with load duration, so largest load doesn't necessarily govern



Duration (EC5 NA)	Loadcase (EC0)
Permanent	1.35 DL
Long Term (< 10 yrs)	1.35 DL + 1.5 Storage
Med-term (< 6 mths)	1.35 DL + 1.5 Imposed
Short-term (< 1 week)	1.35 DL + 1.5 Snow
Instantaneous	1.35 DL + 1.5 Wind

Load duration

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	More than 10 years	Self-weight
Long-term	6 months – 10 years	Storage
Medium-term	1 week – 6 months	Imposed floor load
Short-term	Less than 1 week	Snow, wind
Instantaneous		Wind, accidental load

k_{mod} factor

$$f_{m,d} = \frac{k_{mod}\, k_h\, k_{crit}\, k_{sys}\, f_{m,k}}{\gamma_m}$$

Material	Service class	Load-duration class				
		Permanent	Long-term	Medium-term	Short-term	Instantaneous
Solid timber	1	0.60	0.70	0.80	0.90	1.10
	2	0.60	0.70	0.80	0.90	1.10
	3	0.50	0.55	0.65	0.70	0.90

Longer duration & wetter = weaker

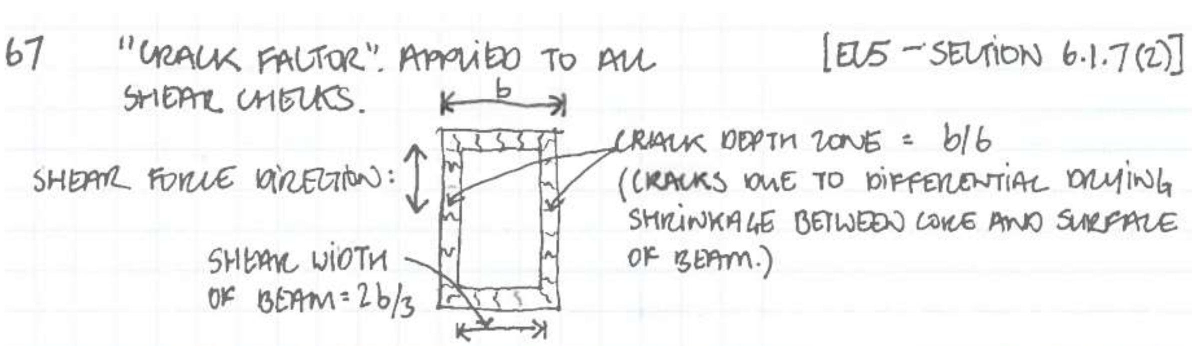
Shortest duration load in a given combination decides what the load duration of that combination is and what k_{mod} value we use.

k factors

$$f_{m,d} = \frac{k_{mod}\, k_h\, k_{crit}\, k_{sys}\, f_{m,k}}{\gamma_m}$$

Factor	
k_h	Size (bending or tension)
k_{crit}	Lateral Torsional Buckling (LTB)
$k_{c,y}$	Axial buckling (compression)
k_{sys}	Load sharing
k_{cr}	Allowance for fissures

k_{cr} crack factor



Combined Stresses – Biaxial Bending

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

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Combined Stresses – Axial Loading and Bending

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

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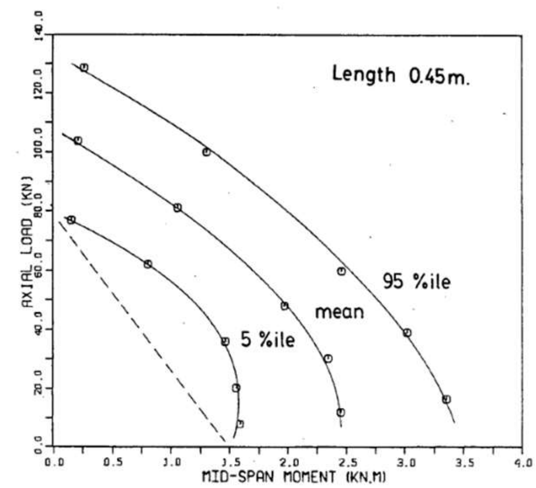
Combined Stresses – Axial Loading and Bending



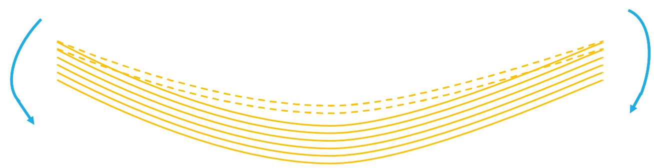
(a) 38 x 89mm

Buchanan, A, H (1984): *Strength Model and Design Methods for Bending and Axial Load Interaction in Timber Members*. Thesis submitted in partial fulfilment of the requirements for the degree of doctor of philosophy. University of British Columbia.

Elastic Modulus

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

Additional checks for curved glulams…



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The Timber Wave

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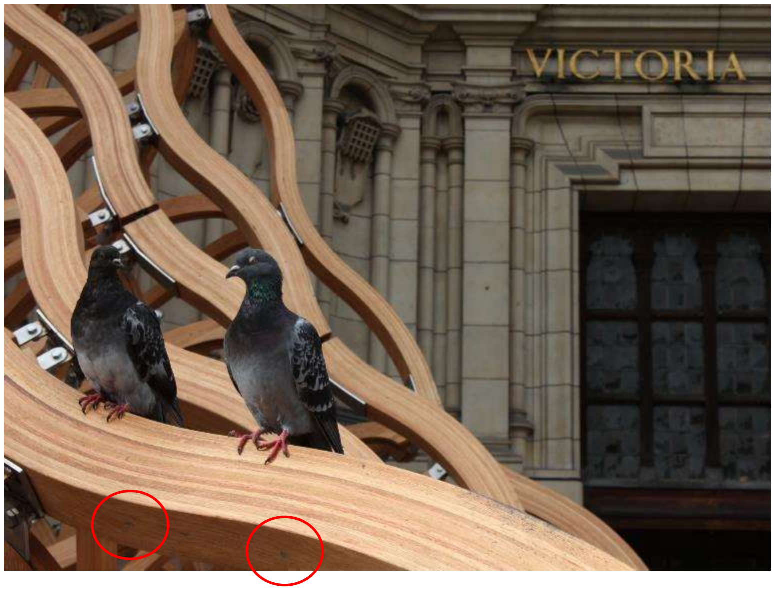
The Timber Wave



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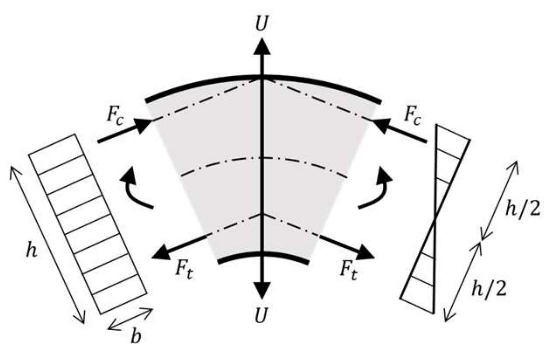
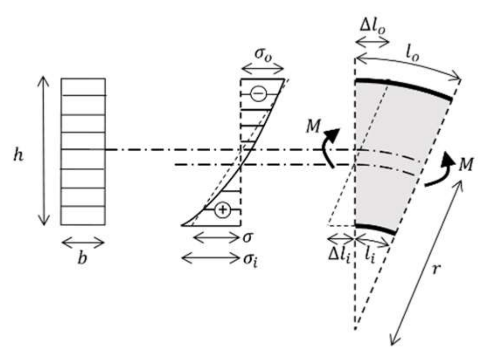
Curved glulam





- Tension perpendicular
- Correction for non-linear stress distribution
- Account for residual stress in laminates

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Serviceability

Instantaneous Deflection

- E_{mean} for serviceability
- Don't forget shear deflection

Creep

- Due to self-weight + ‘quasi-permanent’ variable load (Ψ_2 in EC0)
- For domestic floors $\Psi_2 = 0.3$, for roofs $\Psi_2 = 0$ (check this in the national annex!)
- $w_{creep} = w_{inst} \times k_{def}$

Material	Service class		
	1	2	3
Solid timber	0.60	0.80	2.00
Glue Laminated timber	0.60	0.80	2.00

Deflection Limits

- Refer to National Annex for deflection limits
- $w_{fin} = w_{inst} + w_{creep}$

Dynamics

But remember that vibration usually governs

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Modern Engineered Timber Materials
Section No.2/ Sadaļa Nr.2.

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Softwood

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Harvesting

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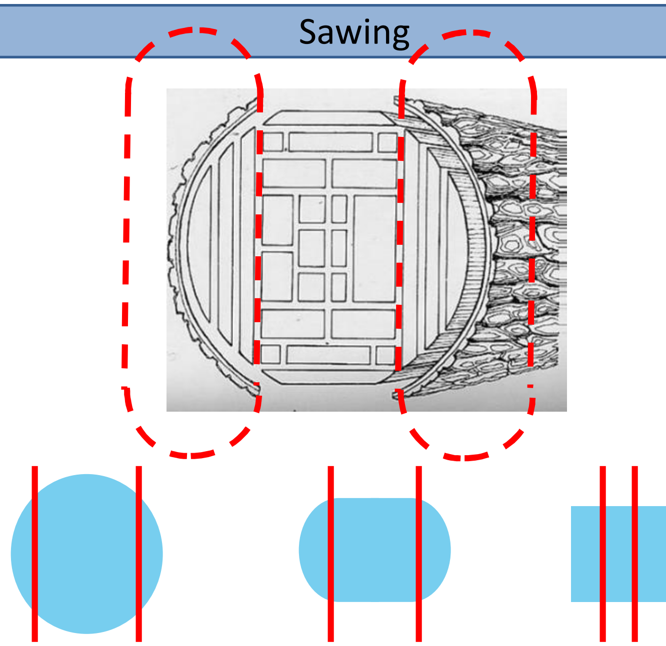
Sawing



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Drying

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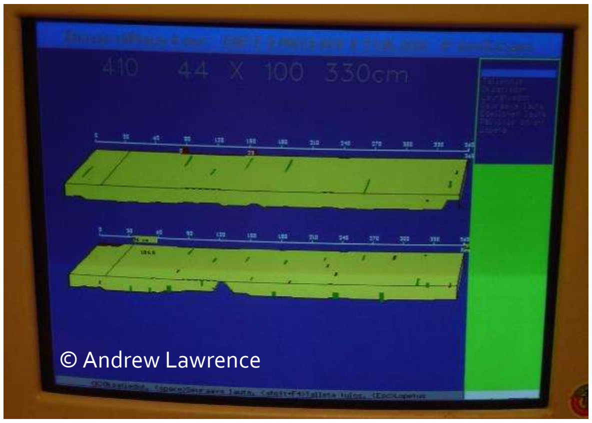
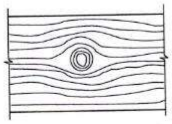
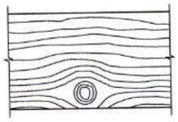
Grading



High grade





Low grade

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Storage

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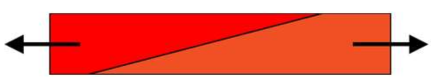
Waterproof glues

Finger joints



Gluing

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Clamp overnight

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Planing

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Block gluing

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Transport limitations

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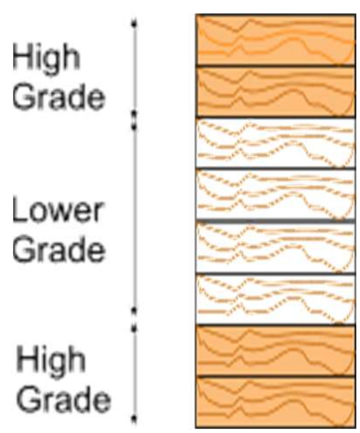
Combined grade glulam



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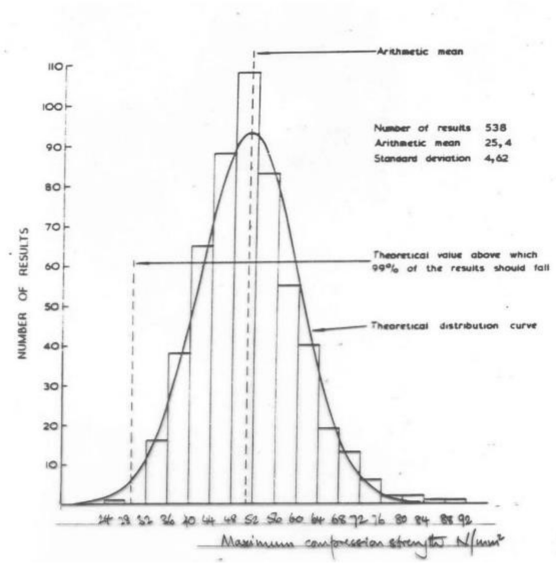
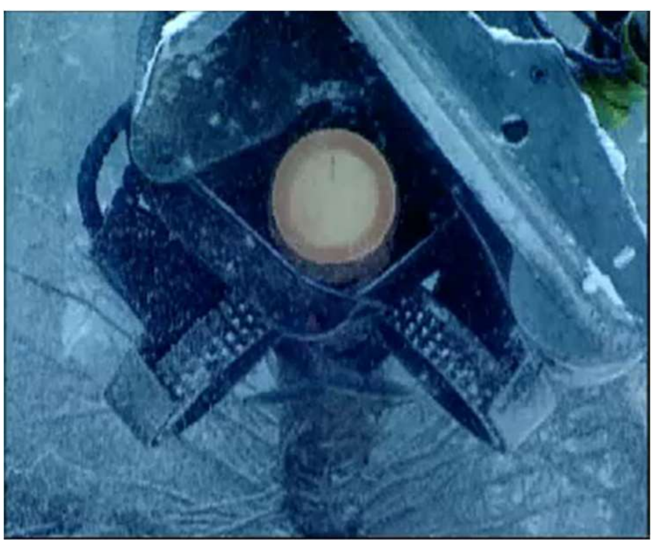
Glulam is stronger

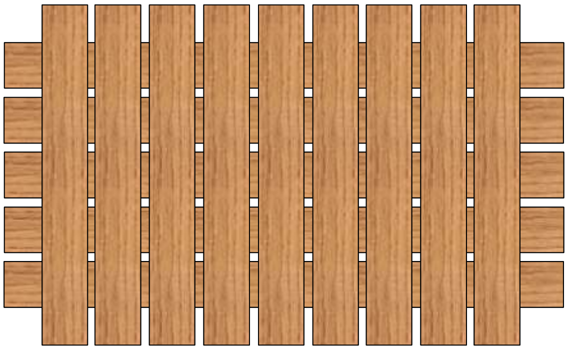


Laminated Veneer Lumber (LVL)



Video reproduced courtesy of Metsawood

Cross-Laminated Timber (CLT)



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Timber concrete composites

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Summary

	Maximum size	Use
Glulam	260 x 2400 x 40m	Beams, columns
CLT	12m x 3m x 300	Slabs, walls
LVL	100 x 2000 x 20m	Roofs, stressed skin floors, beams, columns

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Coffee break / 11:30 - 12:00

70

Agenda / 12:00 - 13:30

- Timber Construction Systems
- Connections
- Designing with a brittle material

Ministry of Economics
Republic of Latvia

Training seminar / Apmācību seminārs

Timber Construction Systems
Section No.3/ Sadaļa Nr.3.

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Engineered Timber Buildings

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Stadthaus, London

Structural Engineer: Techniker

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Forte Tower, Melbourne

Image supplied courtesy of Lendlease

Sky Believe in Better Building, London

Sky Believe in Better Building

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Sky Believe in Better Building

© Simon Kennedy

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Sky Health & Fitness

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How tall can we build in timber?

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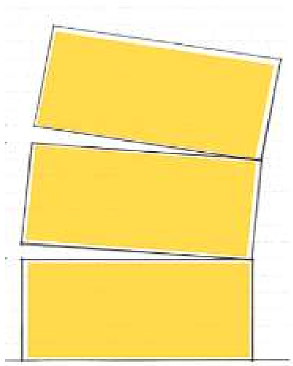
Forte Tower, Melbourne



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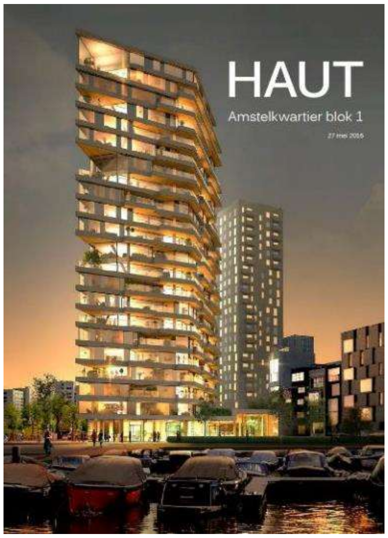
Haut, Amsterdam



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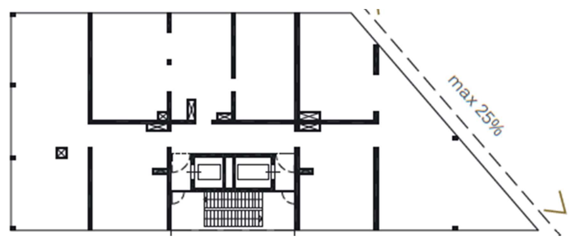
Haut, Amsterdam



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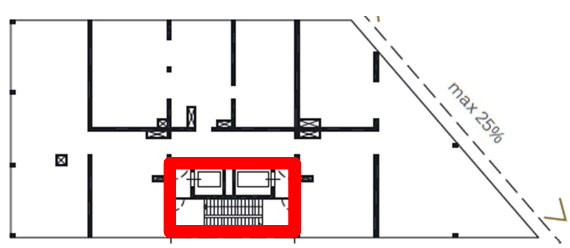
Haut, Amsterdam



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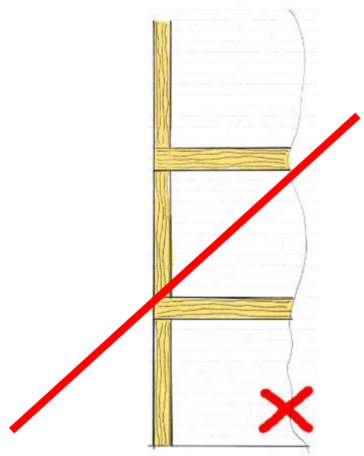
Concrete Core With Timber Vertical Structure

- Concrete and timber have different stiffnesses and different rates of creep.
- Hence there is a risk that in time under vertical load the timber structure deflects (axially) more than the core.
- This should be check using staged construction calculation procedure.
- Differential axial shortening most noticeable at the top of the building.

Avoid cross grain forces for taller buildings.



Brock Commons / Fast & Epp

© naturallywood.com / Brudder
Structural Engineer: Fast + Epp

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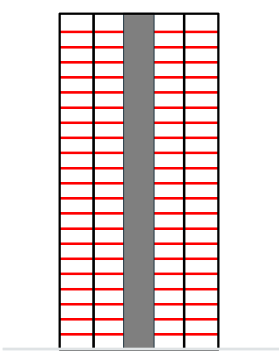
Sky Central



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Sky Central

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Sky Central

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Timber Connections
Section No.4/ Sadaļa Nr.4

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Connections didn't change for 1000 years

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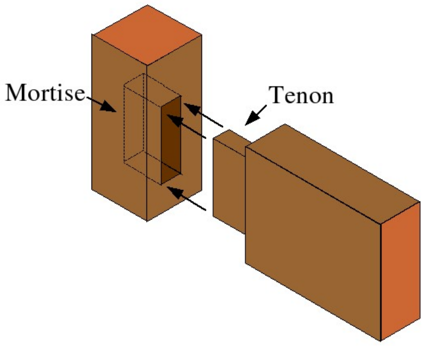
Mortise & Tenon



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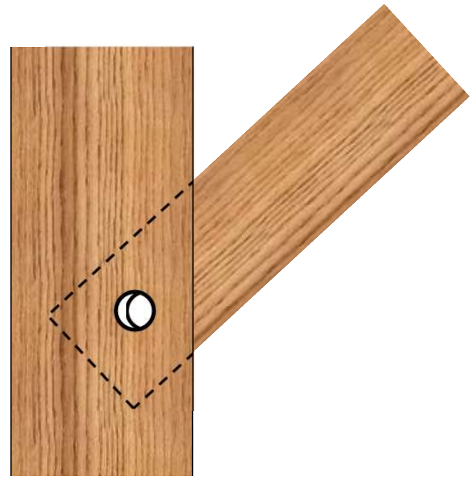

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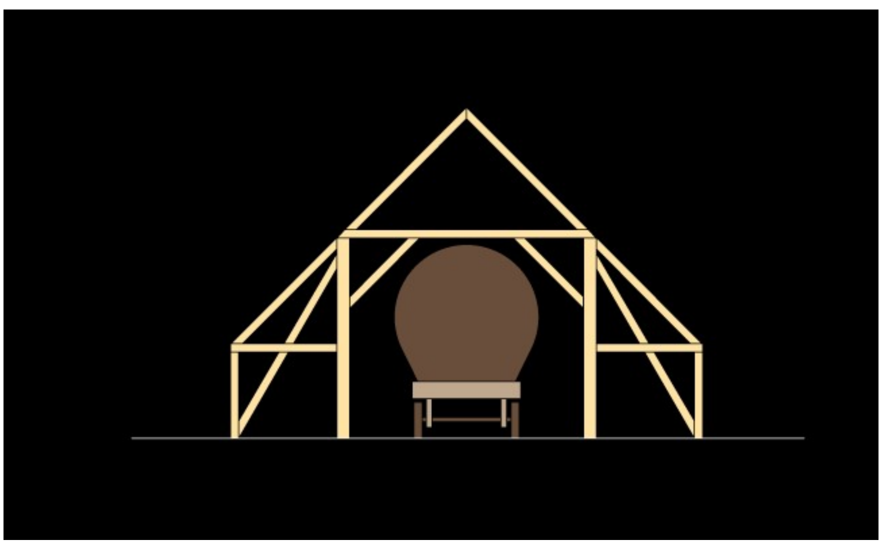
Compression only bracing



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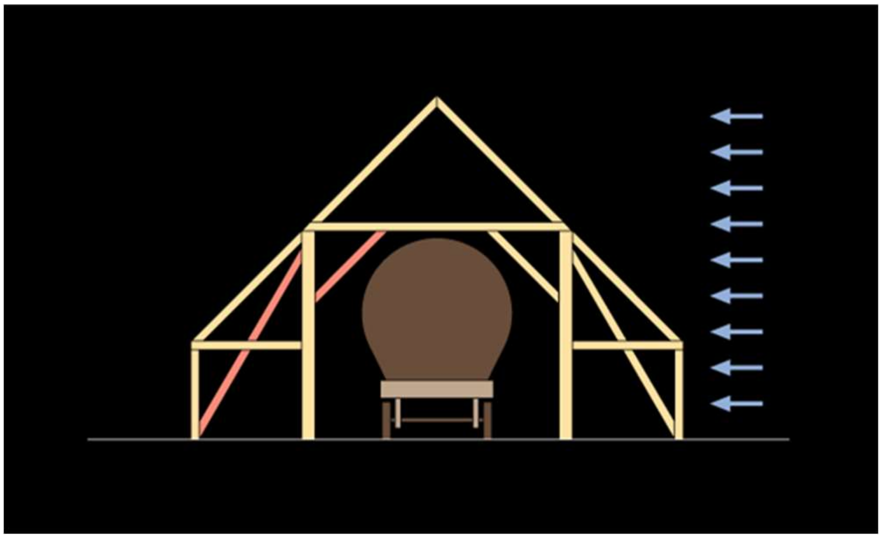
Compression only bracing



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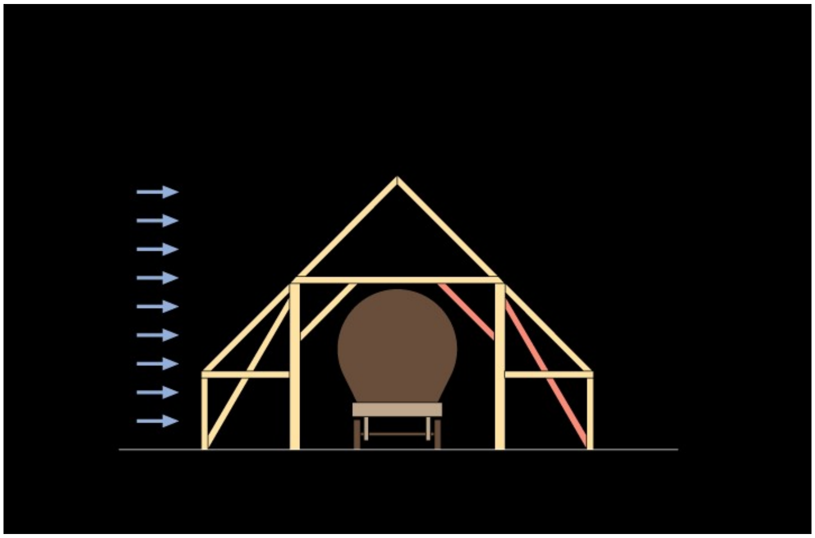
Compression only bracing



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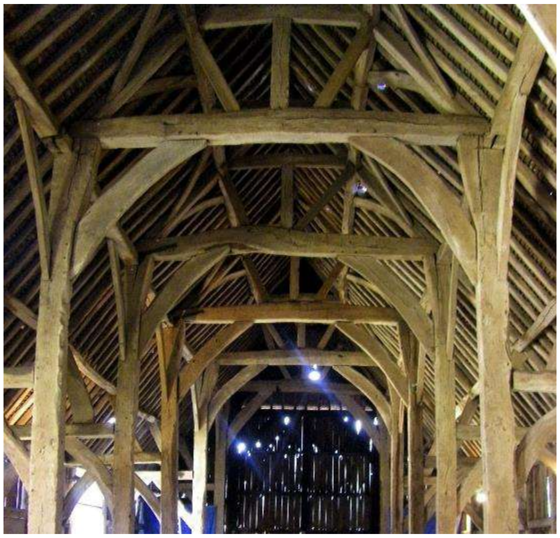
Traditional joints

- No tensile strength
- Labour intensive
- Weaken the member



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Modern timber connections

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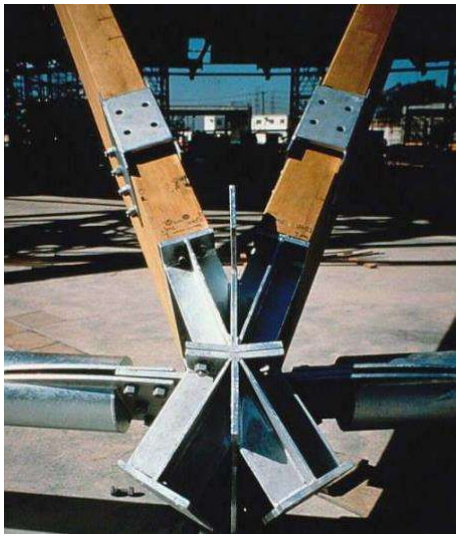
End bearing



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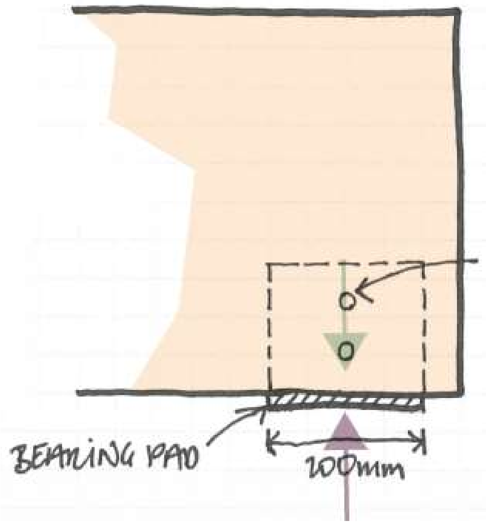
End bearing



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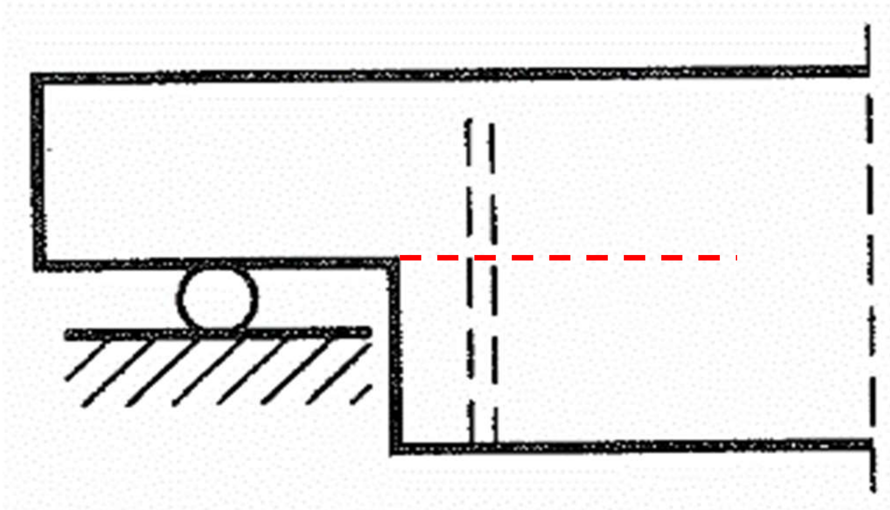
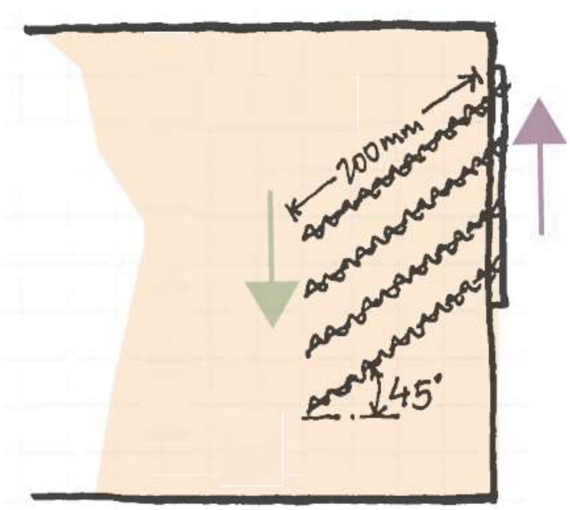
Effect of having a notch

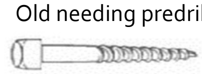


Screws in tension

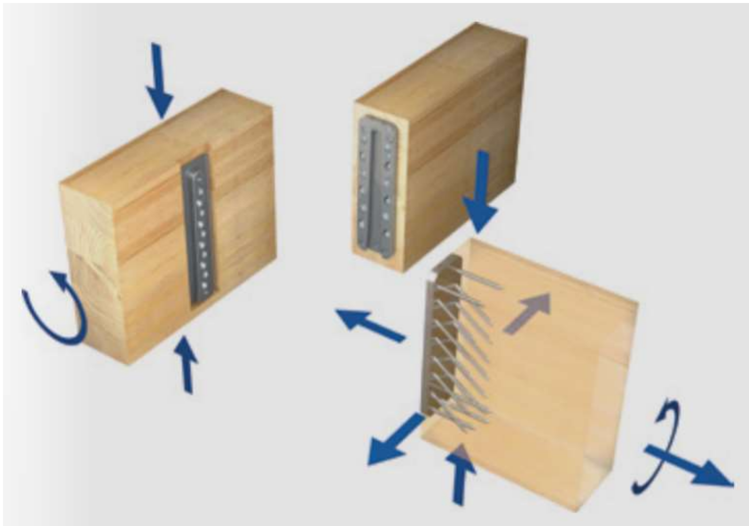


New self-tapping

Old needing predrilling



Screws in tension with steel plate



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Opposing screws in tension

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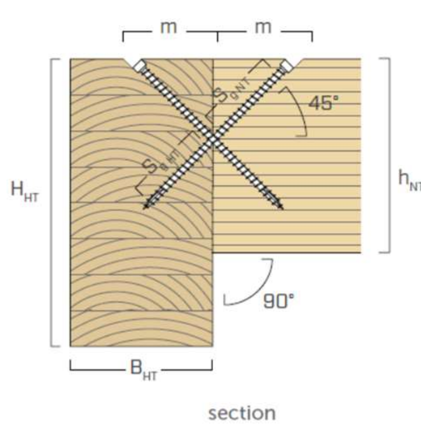

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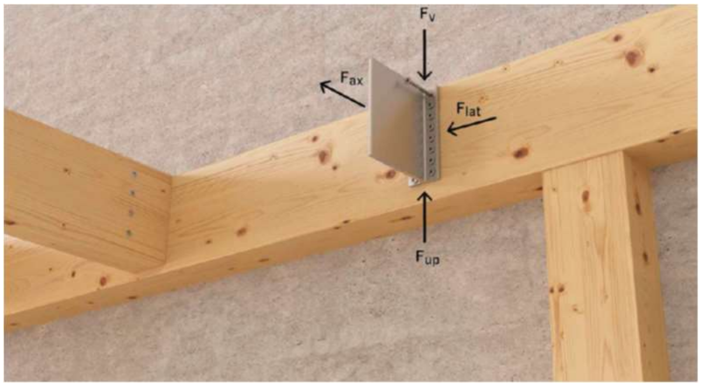
Fasteners in shear



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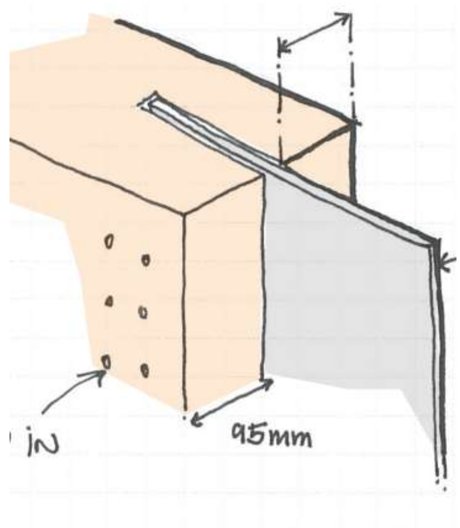

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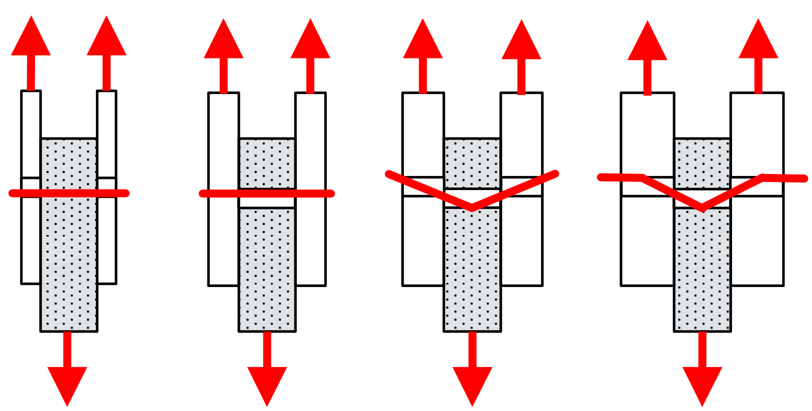
Failure modes

- Failure mode depends on relative proportions



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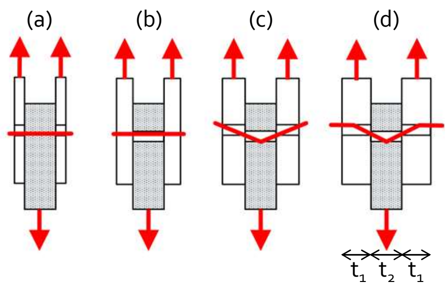
Johanssen formulae…

- Weakest mode governs



ratio of embedment strengths

$$F_{v,Rk} = \text{MIN}\begin{cases} f_{h,1,k} t_1 d \text{ (pure crushing)} & \text{(a)} \\ 0{,}5 f_{h,2,k} t_2 d & \text{(b)} \\ 1{,}05 \dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d \quad t_1^2}} - \beta\right] & \text{(c)} \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d} & \text{(d)} \end{cases}$$

crushing/embedment strength

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Embedment strength… (empirical formulae)

Nails (predrilled) $f_{h,k} = 0.082\,(1 - 0.01d)\rho_k$

Bolts $f_{h,0,k} = 0.082\,(1 - 0.01d)\rho_k$

$$f_{h,90,k} = \frac{f_{h,0,k}}{1.35+0.015d}$$

In small fasteners capacity dominated by T-perp, thus independent of load direction

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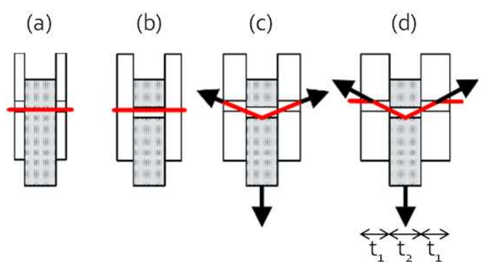
Bolts & screws are stronger due to rope effect…



$$F_{v,Rk} = \text{MIN}\begin{cases} f_{h,1,k} t_1 d & \text{(a)}\\ 0,5 f_{h,2,k} t_2 d & \text{(b)}\\ 1,05 \frac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d\, t_1^2}}-\beta\right]+\frac{F_{ax,Rk}}{4} & \text{(c)}\\ 1,15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d}+\frac{F_{ax,Rk}}{4} & \text{(d)}\end{cases}$$

Contribution limited to limit slip before failure

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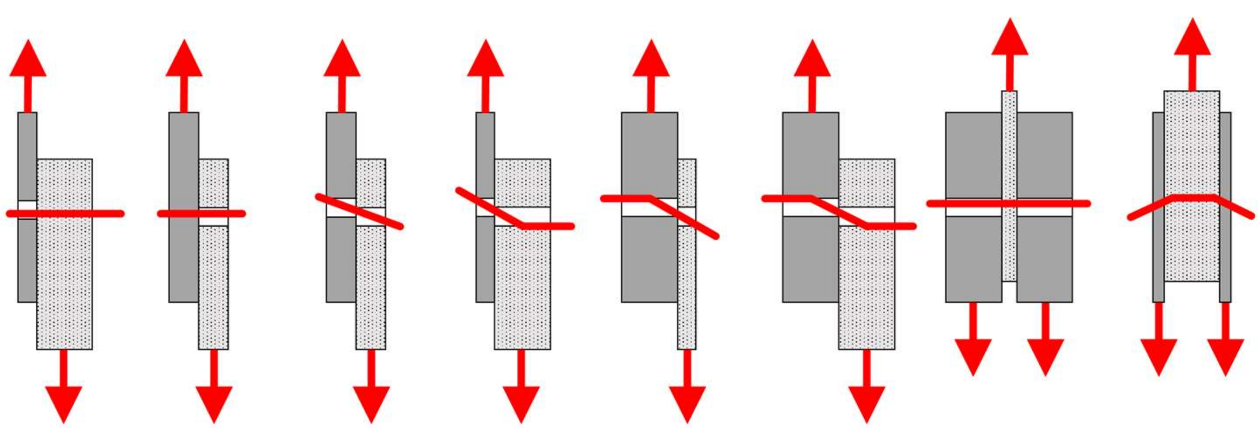
Other configurations…

- EC5 includes formulae for each mode



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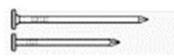
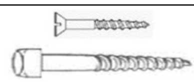
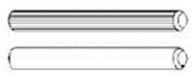
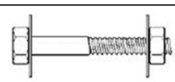
'Dowel type' fasteners in shear compared…

	Typical diameters [mm]	Typical design shear capacities [kN]
	2.5 - 3.5	0.5 - 1
	6 - 12	2 - 10
	10 - 20	8 - 20
		

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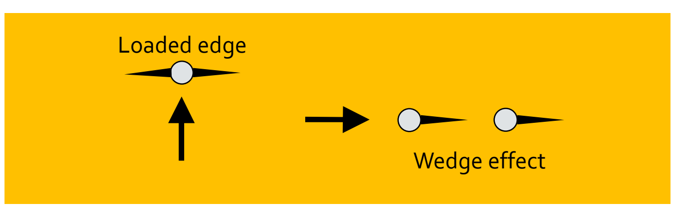
Spacing rules

- edge & spacing rules from testing
- predrilling reduces splitting tendency & allows closer spacings



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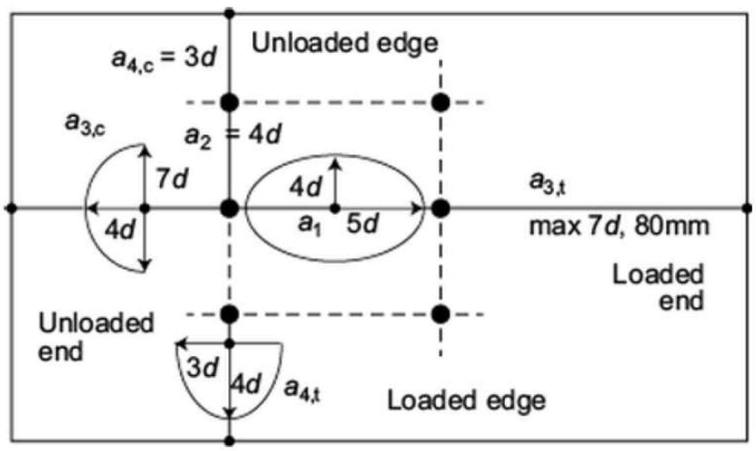
Spacing rules

Bolts with d > 6mm



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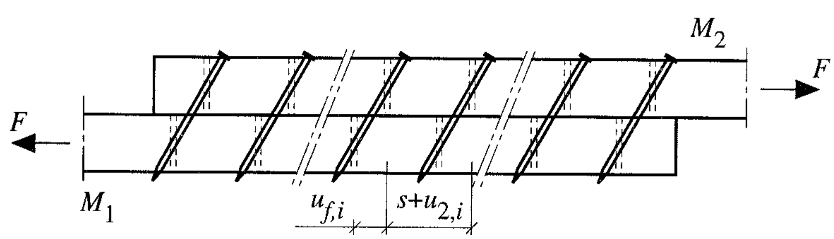
Group effects 1) Reduction for fixings in line parallel to grain



Hence n_{ef} factor (10 - 40% reduction)

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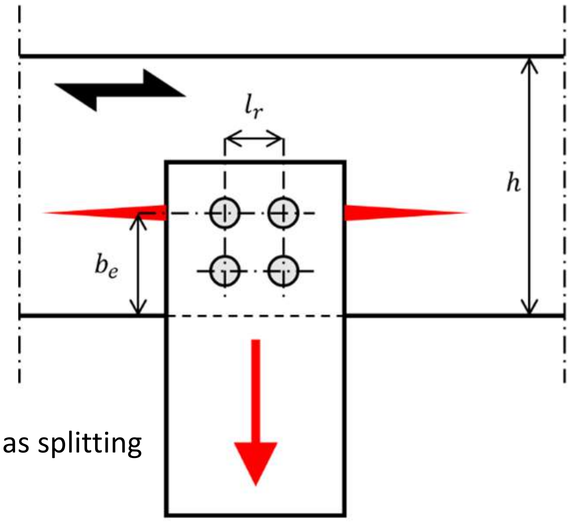
2) Tension perpendicular on group of fixings



Also known as splitting

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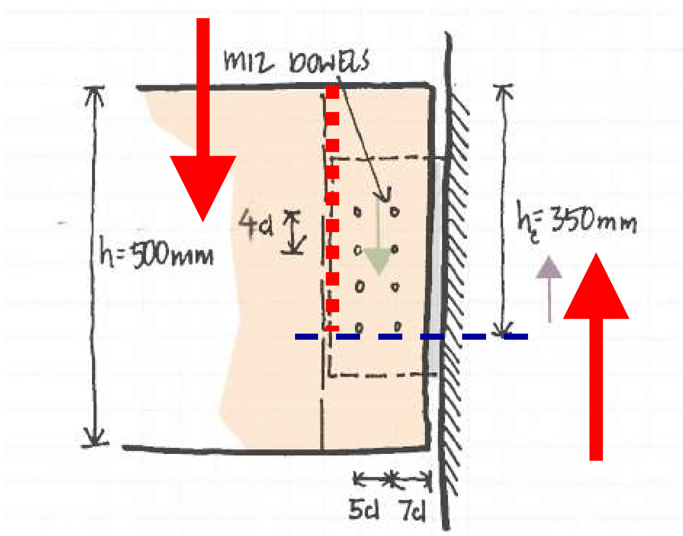
3) Reduced section in shear



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4) Shrinkage and therefore limitations on connection size; splitting due to restraint by steel plate

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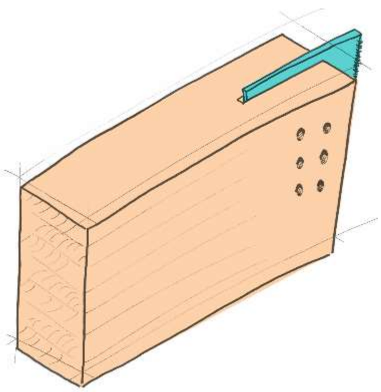
Summary of checks required on any connection



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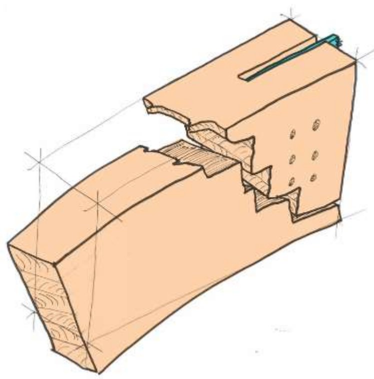

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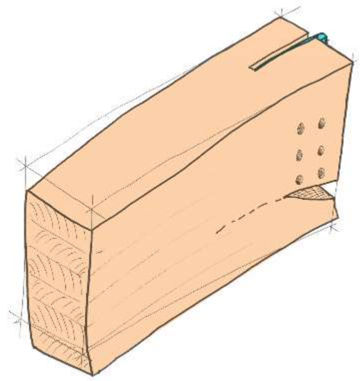

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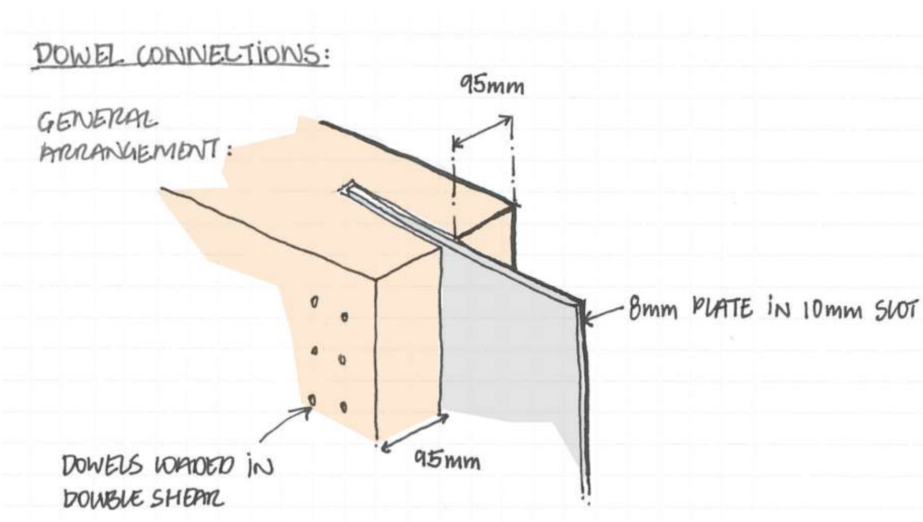
Worked examples

Sample calc. – dowels in shear with eccentricity



Sample calc. – dowels in shear with eccentricity

CHARACTERISTIC CAPACITY FOR M12 DOWELS:

PARALLEL TO GRAIN = $F_{0,k}$ = 21 kN

PERPENDICULAR TO GRAIN = $F_{90,k}$ = 14 kN … From IStructE Manual

DESIGN STRENGTHS FOR M12 DOWELS:

PARALLEL TO GRAIN:

$$F_{0,d} = n_{ef} \times k_{mod} \times \frac{F_{0,k}}{\gamma_m} = 0{\cdot}66 \times 0{\cdot}8 \times \frac{21}{1{\cdot}3} = \boxed{8\ kN}$$

PERPENDICULAR TO GRAIN:

$$F_{90,d} = n_{ef} \times k_{mod} \times \frac{F_{90,k}}{\gamma_m} = 0{\cdot}66 \times 0{\cdot}8 \times \frac{14}{1{\cdot}3} = \boxed{5\ kN}$$

NOTE: THE INCLUSION OF n_{ef} IS DEEMED TO BE CONSERVATIVE IF THE DOWEL IS LOADED PERPENDICULAR TO THE GRAIN. HOWEVER FOR MOST CONNECTIONS THERE WILL BE A COMPONENT OF FORCE NOT PERPENDICULAR TO GRAIN. E.G. TO THE PRESENCE OF SECONDARY MOMENTS.

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Sample calc. – shear in section with reduced depth

CHECK DOWEL GROUP CAPACITY IN SHEAR:

(CONNECTION DOESN'T ATTRACT MOMENT. OTHERWISE CHECK FOR EXTRA FORCES ON DOWELS.)

SHEAR CAPACITY = $F_{90,d}$ × NUMBER OF BOLTS = 5 kN × 8

= 40 kN

CHECK SHEAR STRENGTH OF BEAM SECTION LOCAL TO CONNECTION:

$$\text{SHEAR STRENGTH} = \frac{f_{v,d} \times (b \times h_e)}{1{\cdot}5} = \frac{0{\cdot}67 \times (200 \times 350)}{1{\cdot}5}$$

$$= \boxed{31\ kN}$$ GOVERNS IN THIS CASE

NOTE: DIVIDE BY 1.5 DUE TO PARABOLIC STRESS DISTRIBUTION IN SECTION

MUST ALSO CHECK SPLITTING LOCAL TO CONNECTION (SEE EC5)

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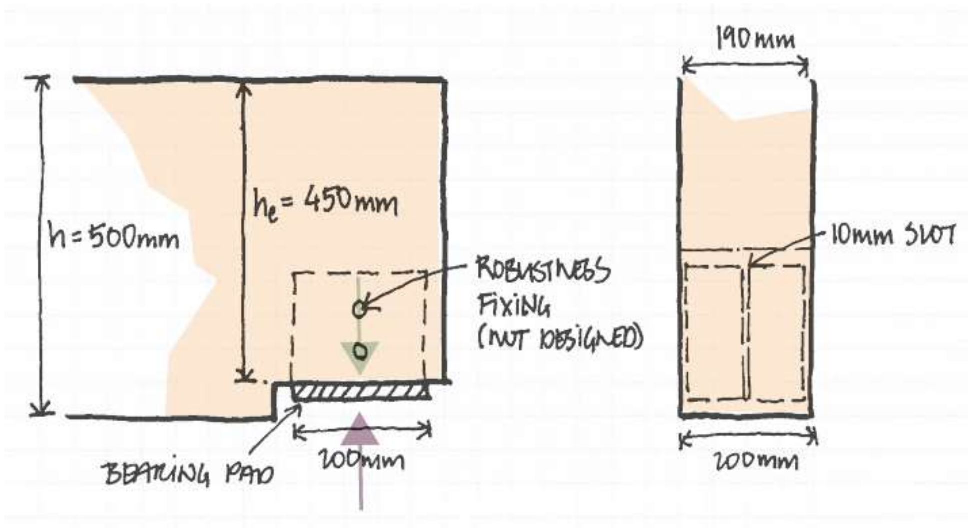
ARUP 122

Sample calc. – bearing connection



Sample calc. – bearing connection

CHECK BEARING CAPACITY:

BEARING CAPACITY $= f_{c,90,d} \times A_{PAD} = 2.5 \times (200 \times 180)$

$= 90\ kN$

CHECK STRENGTH OF BEAM SECTION:

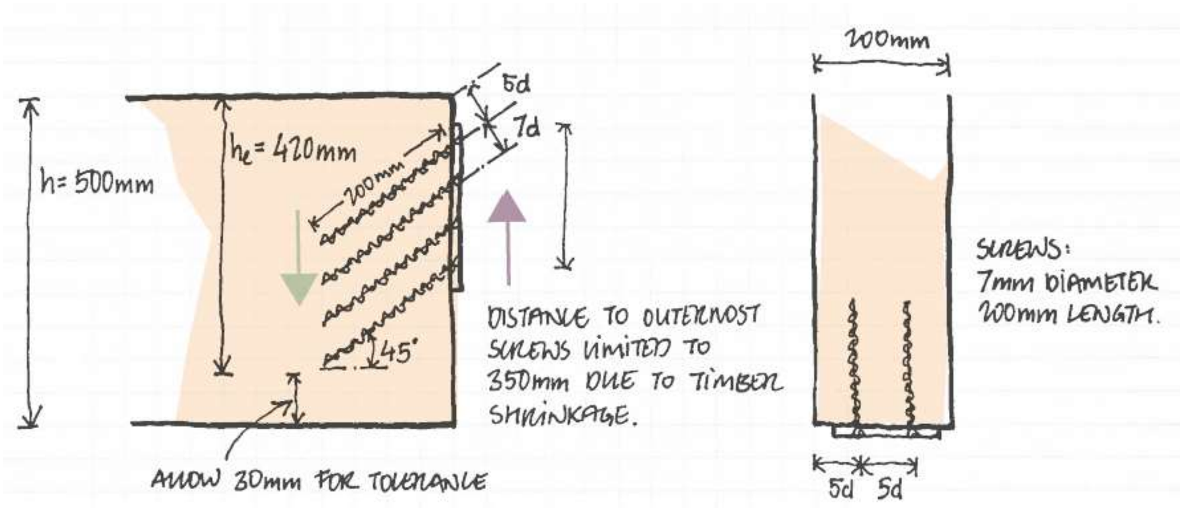
[REDUCTION IN $f_{v,d}$ DUE TO PRESENCE OF NOTCH. IN THIS EXAMPLE $h_{ef}/h = 450/500 = 0.9$. FIND THE REDUCTION FACTOR, k_v, FROM EC5]

SHEAR STRENGTH $= \frac{k_v \times f_{v,d} \times (b \times h_e)}{1.5} = \frac{0.7 \times 0.67 \times (200 \times 450)}{1.5}$

$= 28\ kN$ GOVERNS IN THIS CASE

NOTE: DIVIDE BY 1.5 DUE TO PARABOLIC STRESS DISTRIBUTION IN SECTION.

Sample calc. – screwed connection



Sample calc. – screwed connection

CHARACTERISTIC WITHDRAWAL CAPACITY FOR A FULLY THREADED 7mm DIAMETER, 200mm LONG SCREWS:

From ETA eg Rothoblass

1 SCREW $= 11\ N/mm^2 \times d \times l_{ef}$

$= 11\ N/mm^2 \times 7mm \times 200mm$

$= 15.4\ kN$

NOTE: THIS VALUE IS LIMITED TO 13kN, FOR A 7mm DIAMETER SCREW, DUE TO THE TENSILE AND HEAD TEAR OFF CAPACITY.

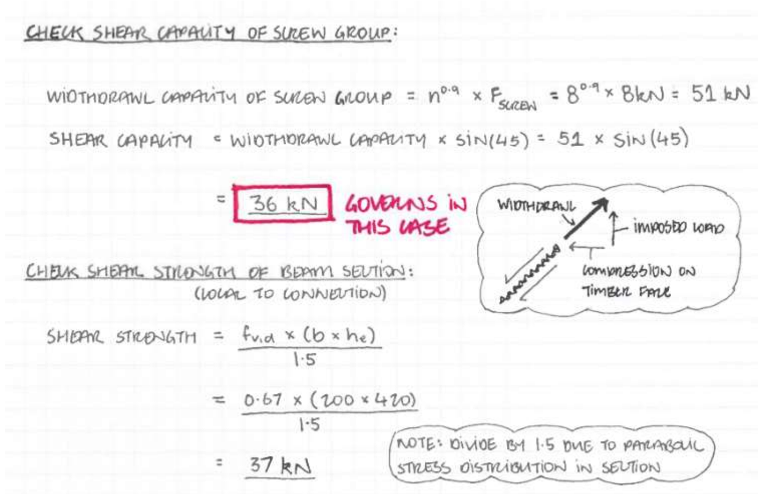
From ETA eg Rothoblass

DESIGN WITHDRAWAL CAPACITY:

$F_{screw} = k_{mod} \times \frac{13\ kN}{1.3} = 8\ kN$

n SCREWS $= n^{0.9} \times F_{screw}$

Sample calc. – screwed connection



MUST ALSO CHECK SPLITTING LOCAL TO CONNECTION (SEE EC5)

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Connections are the weak points

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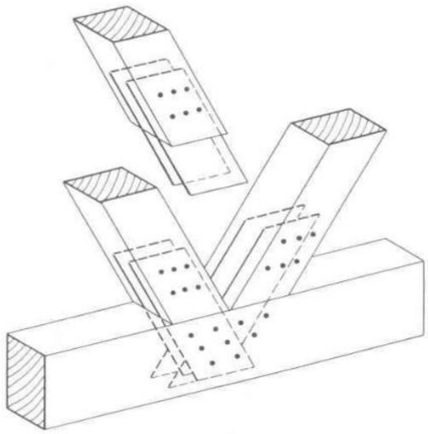
Connections are the weak points



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Connections are the weak points

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Avoid local eccentricities

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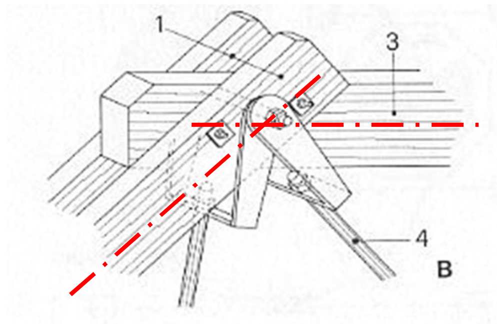

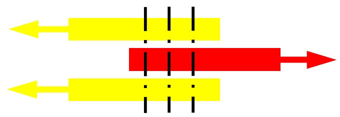

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The connections determine the size of the members

- Same Engineer responsible for both members & connections
- Check connections at concept stage
- Avoid secondary forces due to eccentricity

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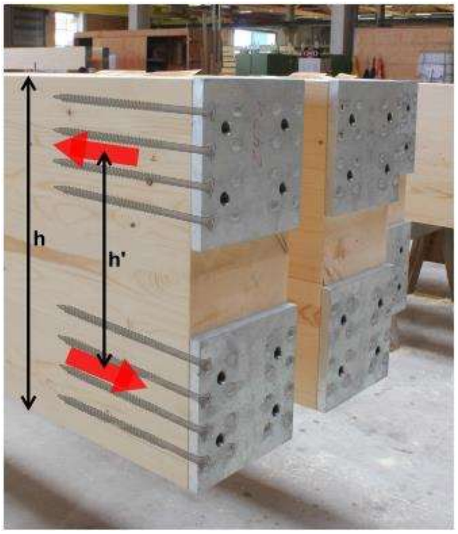
Avoid moment connections



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Connection stiffness

- Oversized holes for tolerance
- Crushing of timber (see clause in EC5)
- Overall 1-2mm slip means most connections behave as pinned

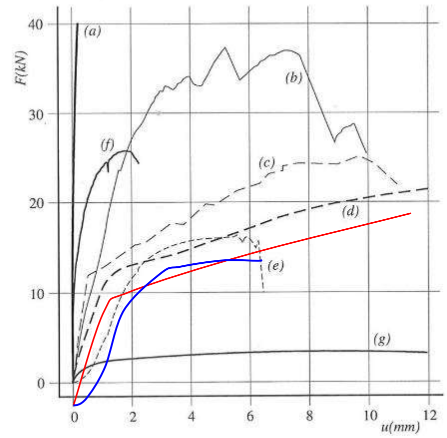

Figure 3 Experimental load-slip curves for joints in tension parallel to the grain: (a) glued joints ($12{,}5\ 10^3\ mm^2$), (b) split-ring (100 mm), (c) double sided toothed-plate (62 mm) (Hirashima, 1990), (d) dowel (14 mm), (e) bolt (14 mm), (f) punched plate ($10^4\ mm^2$), (g) nail (4,4 mm).

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Cost

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Cost

Minimum cost = Minimise the number of connections & keep them as simple as possible

Longer beams

© Wiehag

National Automotive Innovation Centre, Warwick

© Cullinan Studio/ Hayes Davidson

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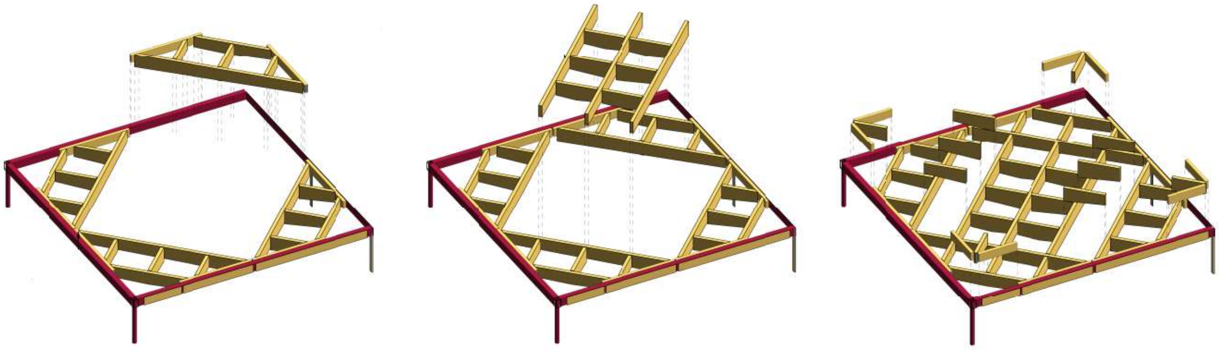
1-way spanning



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Rules for connections

- Use **end bearing** wherever possible
- **Avoid** secondary forces from **eccentricity**
- Allow for shrinkage
- Avoid **moment** connections
- **Minimise** number of connections
- **Check connections early** in design

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Ministry of Economics
Republic of Latvia

Training seminar / Apmācību seminārs

Designing with a Brittle Material
Section No.5/ Sadaļa Nr.5

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Introduction

- What are the risks when designing in a brittle material?
- Can you suggest one safe way to design using a brittle material?

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Determinate 3-pinned arch

© Paul Raftery/VIEW

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Indeterminate Spaceframe

© Hufton+Crow

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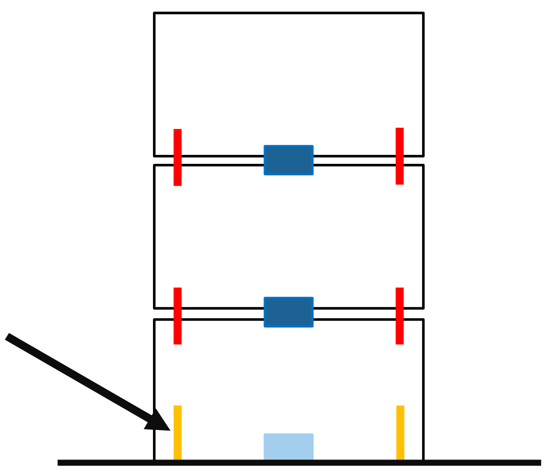
Timber Shear Wall Connections

© Rothoblaas



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Timber Shear Wall Connections

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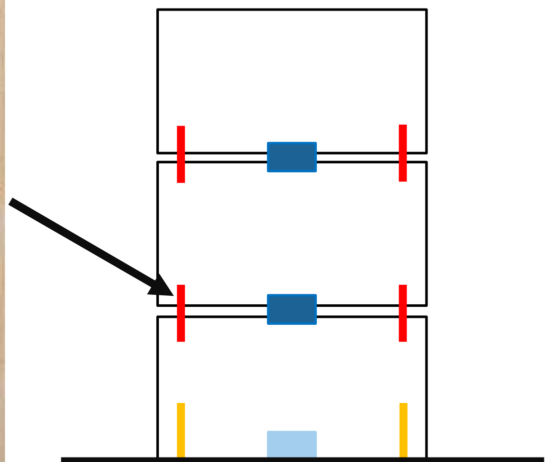

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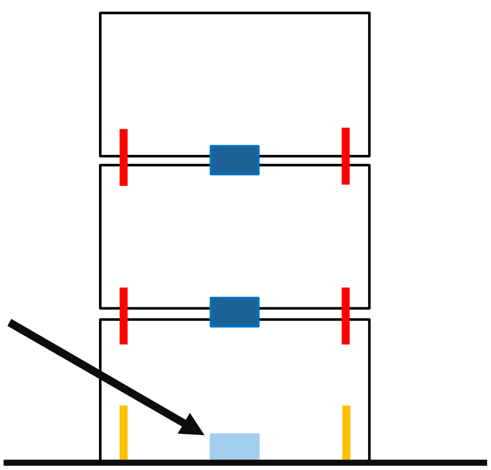
Timber Shear Wall Connections

© Rothoblaas



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Timber Shear Wall Connections

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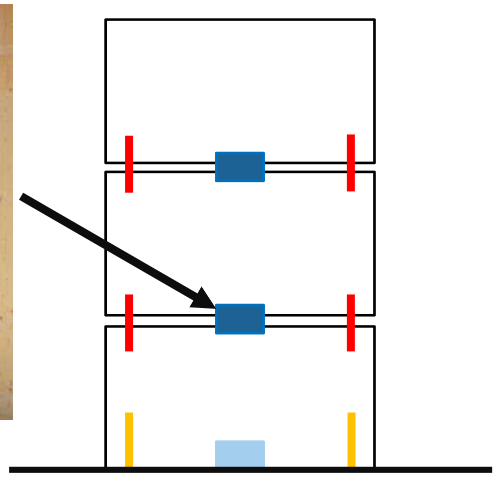

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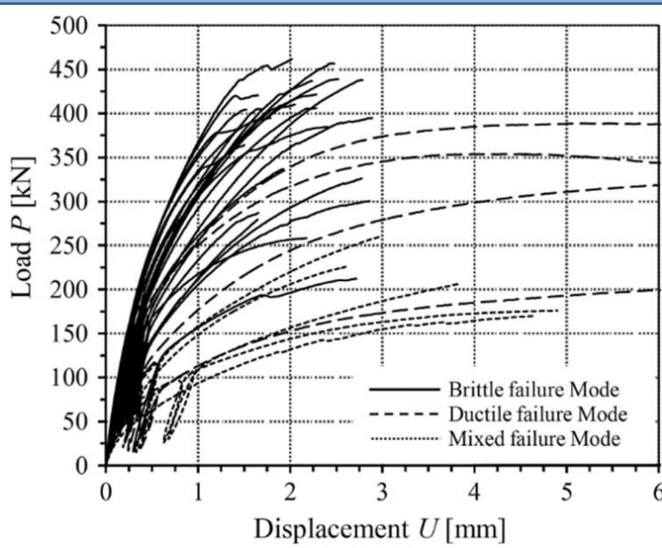
Force Displacement Curve for Timber Connections



Zarnani, P., 2013, *Load-Carrying Capacity and Failure Mode Analysis of Timber Rivet Connections.*

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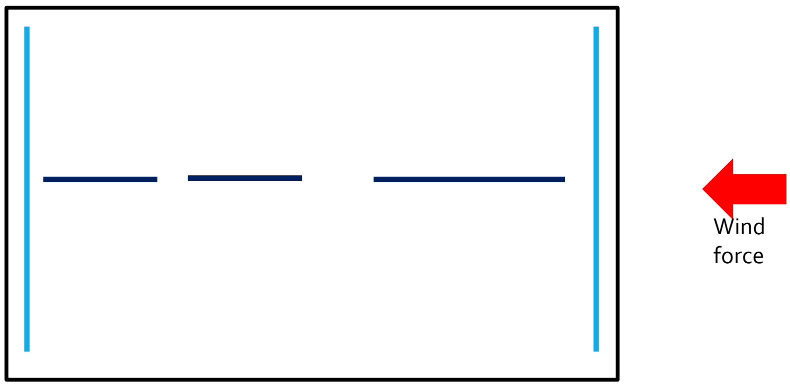
CLT Shear Wall System



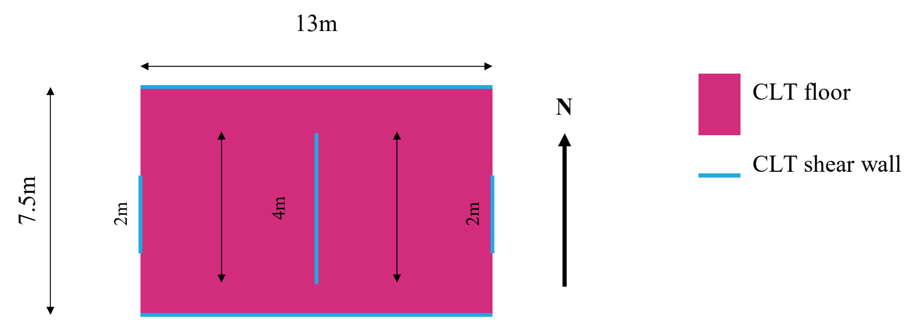
How is load distributed in an indeterminate structure?

- $f_{W,i} = \frac{A_i}{\sum A}$ i.e. Based on shear stiffness?
- $f_{W,i} = \frac{I_i}{\sum I}$ i.e. Based on bending stiffness?

- Actually load distributed based on relative ‘total lateral stiffnesses’.

Illustrative Example A – 3 walls, 1 storey

- Central N-S shear wall 100mm thick, all others 200mm thick.



Plan view (3m tall)

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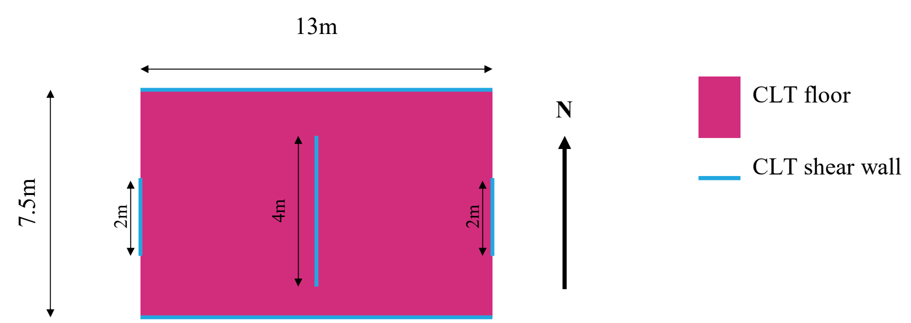
Illustrative Example A – 3 walls, 1 storey

- Design wind in N-S direction is 50 kN:



Plan view (3m tall)

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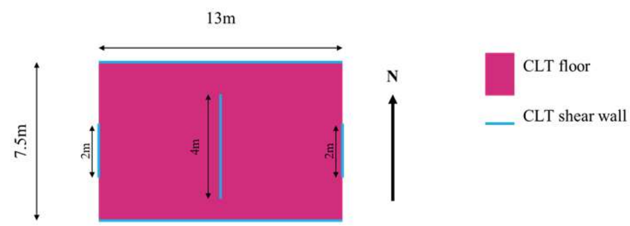
Illustrative Example A – 3 walls, 1 storey

- Initially assume forces distributed according to shear areas:
  - All 3 N-S walls have same shear area, so each takes 16.7kN.
- Uplift will occur at the base of the shear walls as there is no axial force, so tie-down connectors need to be added. Design these according to expected moment at base of walls:
  - All 3 N-S walls have the same moment: 16.7kN × 3m = 50kNm.
  - Assume a tie-down lever arm of 0.7 × wall length.
  - Tie-downs A & C designed for 35.7kN, B for 17.9kN.



Plan view (3m tall)

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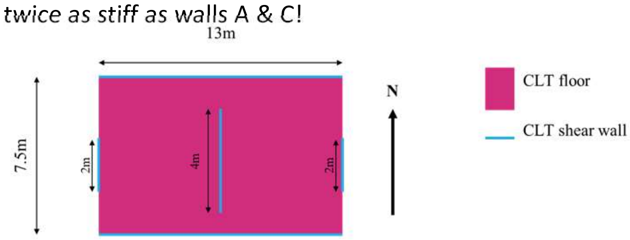
Illustrative Example A – 3 walls, 1 storey

- Tie-downs A & C designed for 35.7kN, B for 17.9kN
- Assume tie-down connector stiffness is proportional to strength
  - Tie-down connector B is ½ as stiff as tie-down connectors A & C.
- Assume that rocking stiffness actually governs, not shear stiffness as previously assumed, with flexibility mostly from the tie-down connectors:

$$K_{rocking} = K_{tie-dow\quad connector} \times \left(\frac{L}{h}\right)^2$$

- Wall B is actually *twice as stiff as walls* A & C!



Plan view (3m tall)

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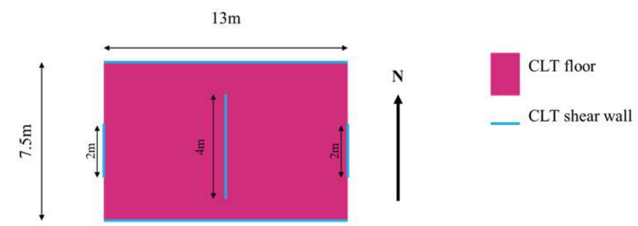
Illustrative Example A – 3 walls, 1 storey

- Wall B is actually twice as stiff as walls A & C!

Wall	Assumed Force	Actual Force	% Change
A	16.7kN	12.5kN	-25%
B	16.7kN	25kN	+50%
C	16.7kN	12.5kN	-25%

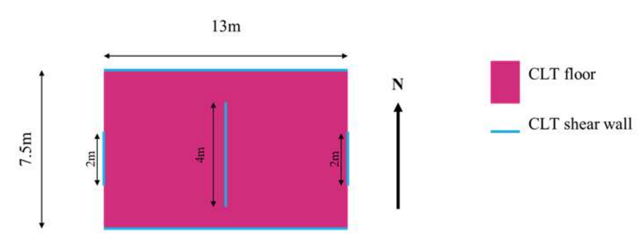
- This suggests that we have under designed the central wall by **50%!**



Plan view (3m tall)

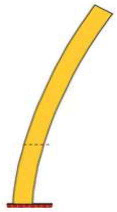
Illustrative Example A – 3 walls, 1 storey

- This suggests that we have under designed the central wall by **50%!**
- A more in-depth study was carried out including the stiffnesses of shear connectors and the shear stiffness of the wall, showing the central wall was only **28%** under-designed.
- If a possible 15% connection stiffness variability is also included, this increases to **38%** under-designed.

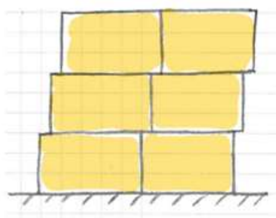


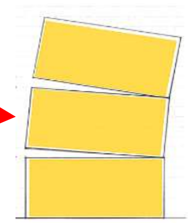
Plan view (3m tall)

Total Lateral Stiffness of an Individual Wall



- Bending
- Shear
- Sliding
- Rocking





$$\Delta_{Tot} = \Delta_{EI} + \Delta_{GA} + \Delta_{Rock} + \Delta_{Slide}$$

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Total Lateral Stiffness of an Individual Wall

$$K_{wall} = \frac{1}{\frac{1}{K_{EI}} + \frac{1}{K_{GA}} + \frac{1}{K_{Rock}} + \frac{1}{K_{Slide}}}$$

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Lateral Stiffness of an Individual Wall

$$K_{wall} = \frac{1}{\frac{1}{K_{EI}} + \frac{1}{\mathrm{K_{GA}}} + \frac{1}{\mathrm{K_{Connections}}}}$$

- No data available for the designer on connection stiffness and also variability of these connection stiffnesses.
- Hence wall stiffness is uncertain.

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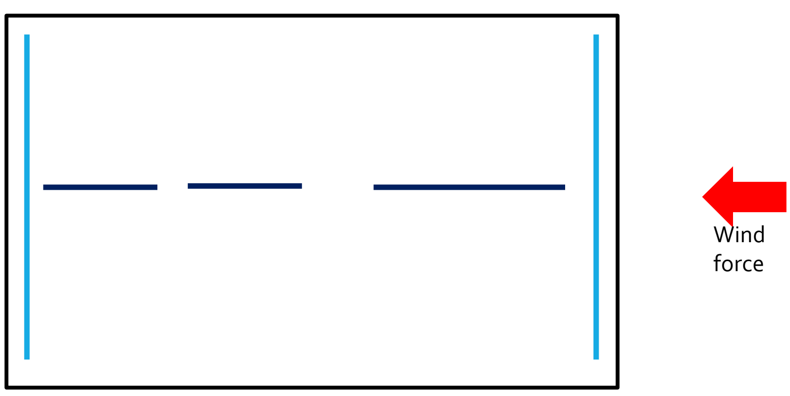
CLT Shear Wall System



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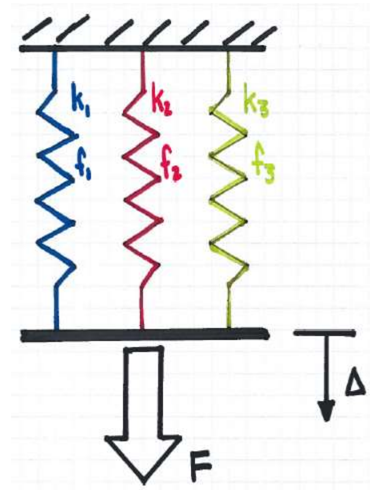
Indeterminate CLT system can be represented as springs

- Forces distributed amongst springs based on relative stiffness.
- Spring stiffnesses are uncertain (due to uncertain connection stiffness).
- Hence individual spring forces are uncertain.



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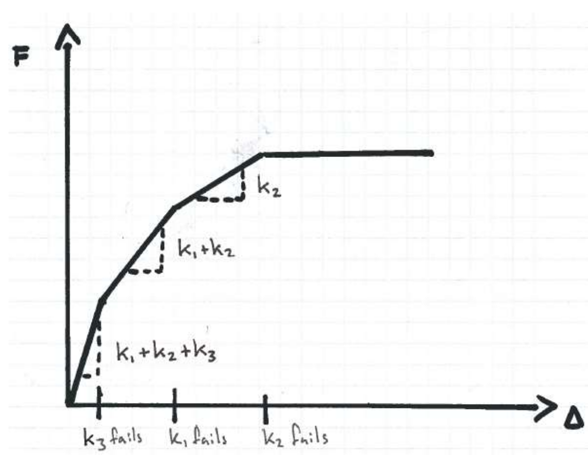
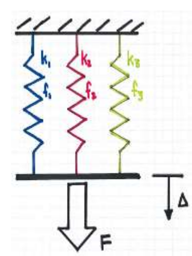
Let's assume springs behave in a **ductile** manner

Note: Behaviour the same for both force and displacement control.





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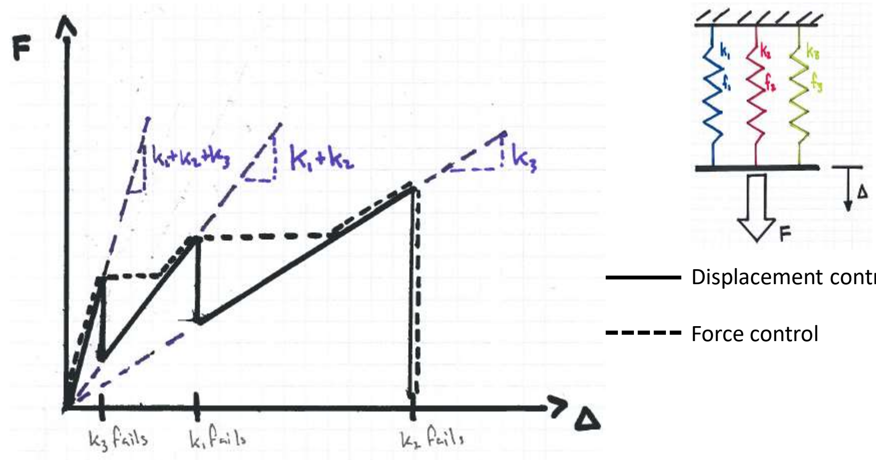
But what if springs behave in a **brittle** manner?

Note: Behaviour different for force control and displacement control !!!



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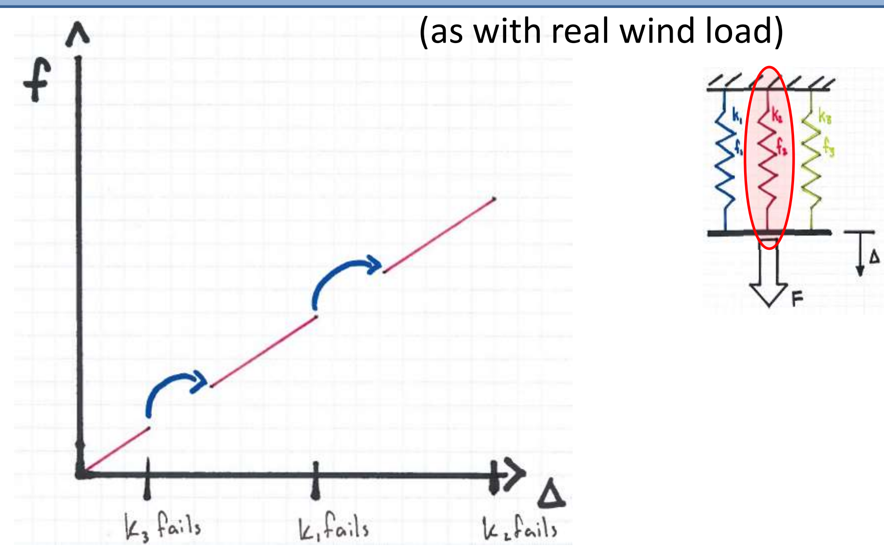
Forces on strongest spring in **brittle** system for force control

(as with real wind load)



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- For a brittle system following first failure no guarantee that new higher force levels in remaining brittle springs will be within their capacity.

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- Hence point of first failure is likely to be point of collapse for brittle system.

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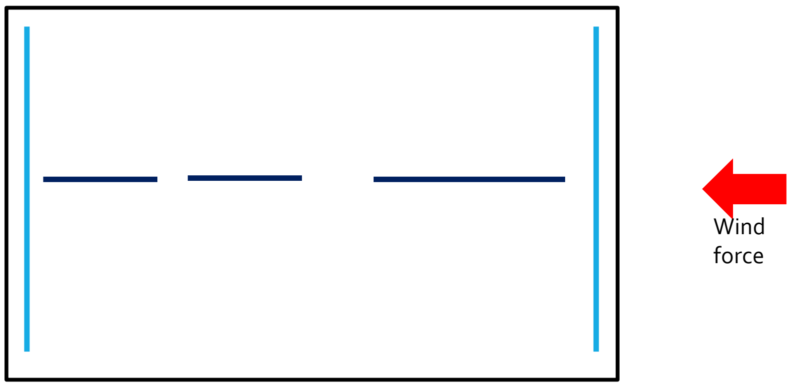
CLT Shear Wall System

- Assume 3 shear walls of equal strength f but uncertain slightly different stiffnesses.



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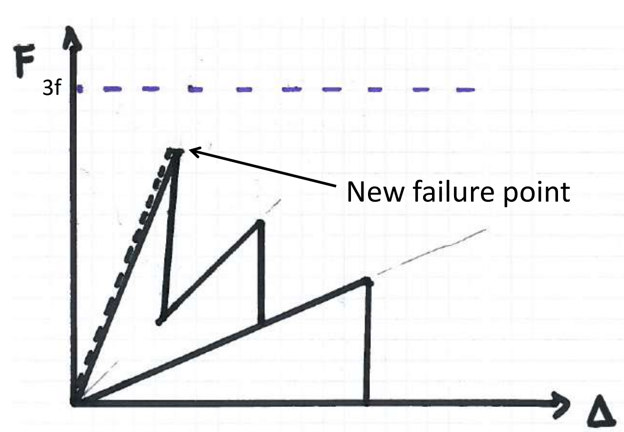
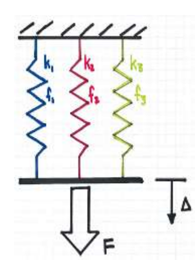
Global Behaviour of Brittle System





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Solutions: Sledgehammer?

- Check all possible force cases for each wall: Monte Carlo?
- For each wall upper bound and lower bound stiffness.
    - Several stiffness states in-between.
    - Consider all possible stiffness states of the system.
- Verify each wall for forces that give suitable probability of failure.

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Solutions: Ductile Design?

Eg: Design such that failure occurs in a ductile mode:

- Force failure modes that cause plastic hinge.
- Overstrength the timber.
- Cross grain screws.

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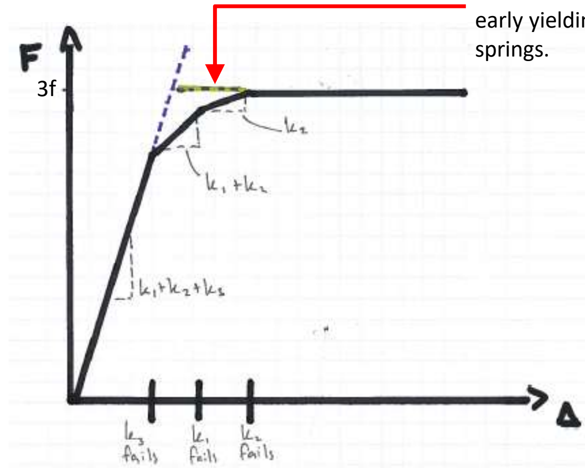
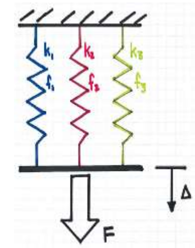
Solutions: Ductile Design

Additional deformation due to early yielding of 2 springs.





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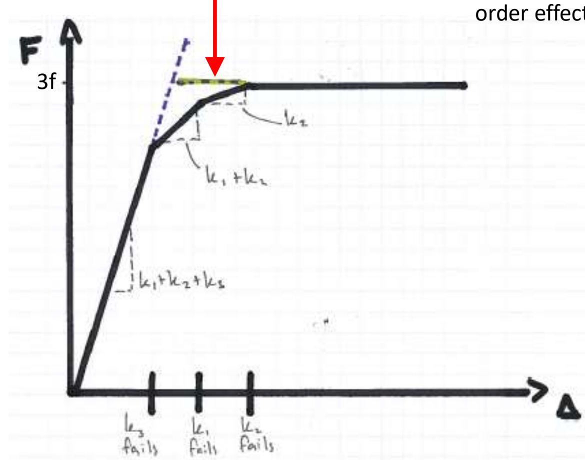
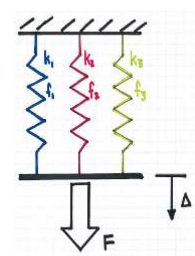
Solutions: Ductile Design

Issue if structure is sensitive to second order effects.





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What should be in EC5?

- The code should remind the designer that timber is brittle.
- Given the difficulty of determining the load path in an indeterminate system the code should encourage the use of ductile connections.

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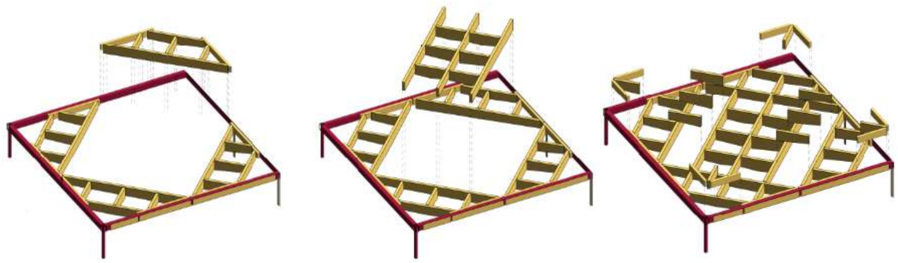
1-way spanning



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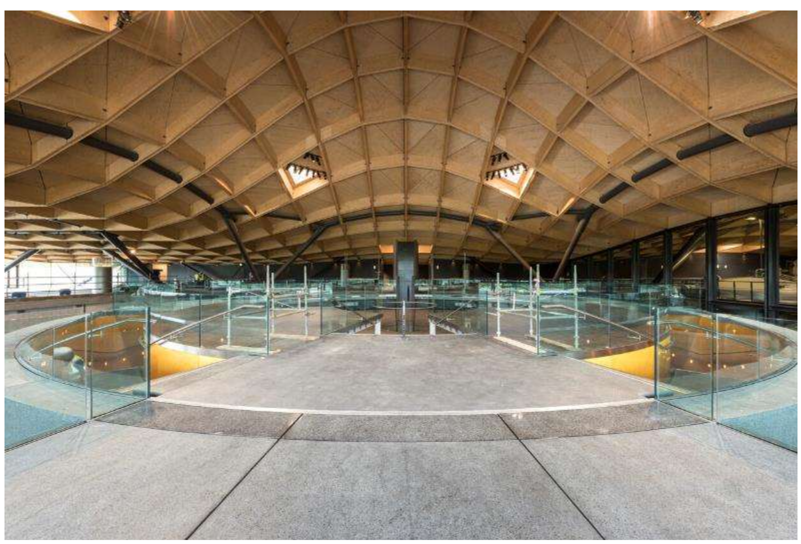
Macallan Distillery



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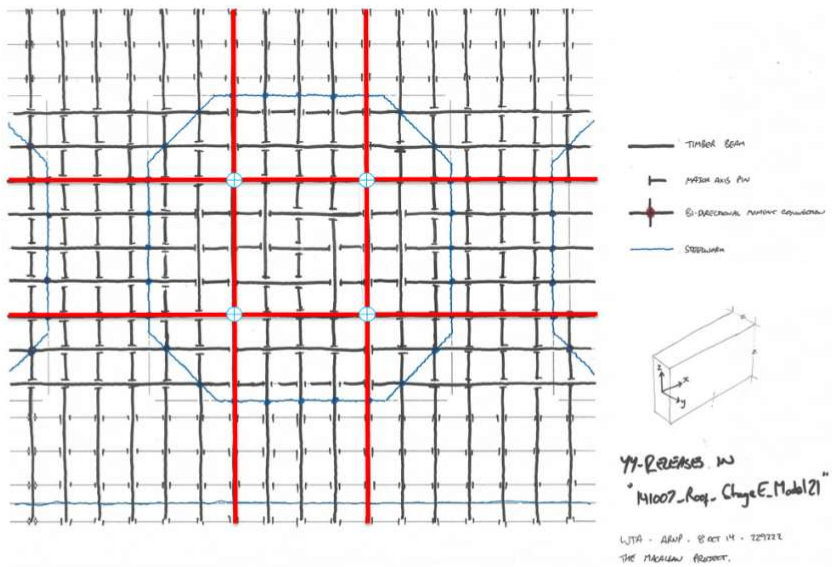
1-way spanning



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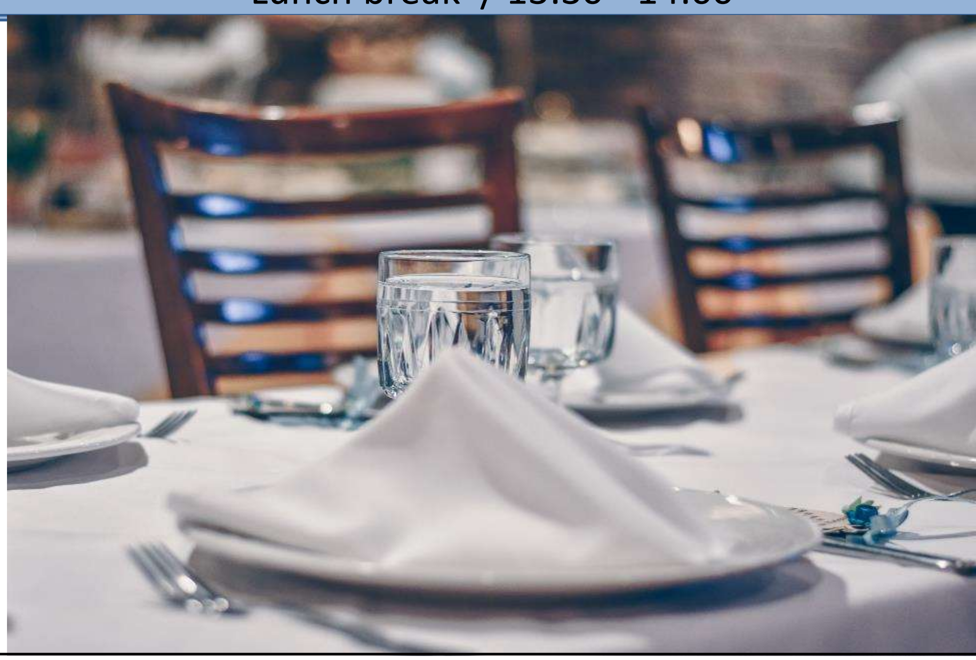
Lunch break / 13:30 - 14:00



180

Agenda 14:00-15:30

- CLT
- Floor Dynamics
- Specification + Common errors
- Questions and answers

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Ministry of Economics
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Training seminar / Apmācību seminārs

CLT
Section No.6/ Sadaļa Nr.6

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

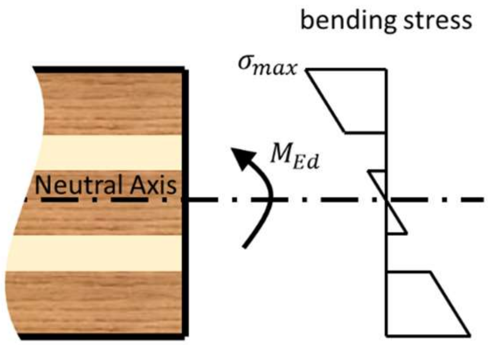
CLT: Cross-Laminated Timber

What is it?

- Made by gluing timber planks together with direction of grain alternating across layers
- Usually out of C24 softwood

https://commons.wikimedia.org/wiki

Out-of-plane Bending



$$\sigma_{max,d} = \frac{M_{Ed}y}{I}$$

$$\sigma_{max,d} \leq f_{m,d}$$

- M: Moment
- I: Second moment of area
- σ: Design stress value at outermost fibre
- y: Distance to neutral axis from edge (depth/2)

- I value can conservatively be calculated neglecting cross layers since $E_0/E_{90} = 33$

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Things to think about

Advantages

- Many available manufacturers
- CNC machined to plan dimensions, slots, unusual geometries
- Easy connection. Self-tapping screws
- Long planks can help increase construction speed

Disadvantages:

- Cost
- Limited availability of visual-grade panels
- Fire

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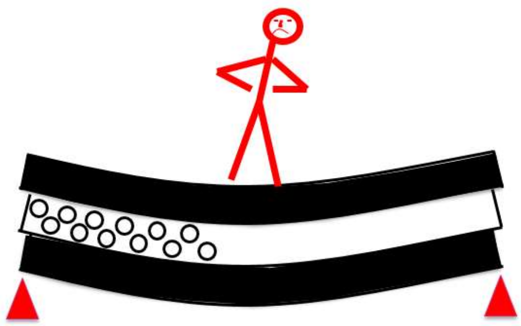
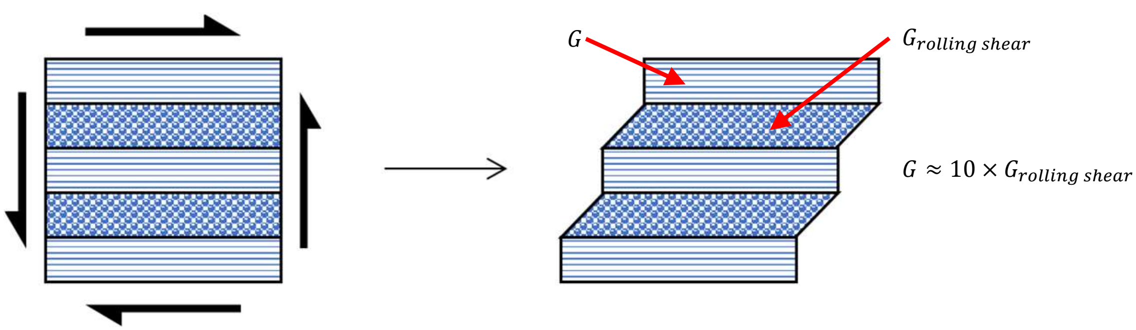
Out-of-plane Shear

- Rolling shear governs



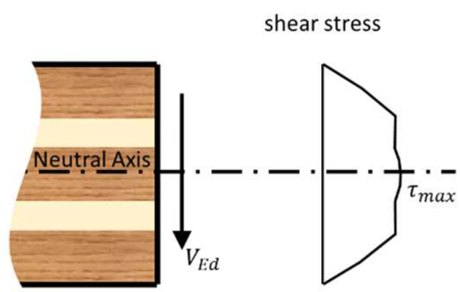
Out-of-plane Shear

- Rolling shear governs



Out-of-plane shear

- Maximum shear tends to $1.5V/bd$ as number of layers increase as shear stress diagram tends to a parabolic shape. (Because the cross layers have a very low stiffness the shear stress can't increase through these layers).
- But it's the rolling shear stress which governs.



$$\tau_{v,Ed} = \frac{1.5V_{Ed}}{bd}$$

$$\tau_{v,Ed} \leq f_{v,90,d}$$

- V: Shear force
- b: Breadth of section
- d: Depth of section
- τ: Shear stress
- $f_{v,90,d}$: Design rolling shear strength

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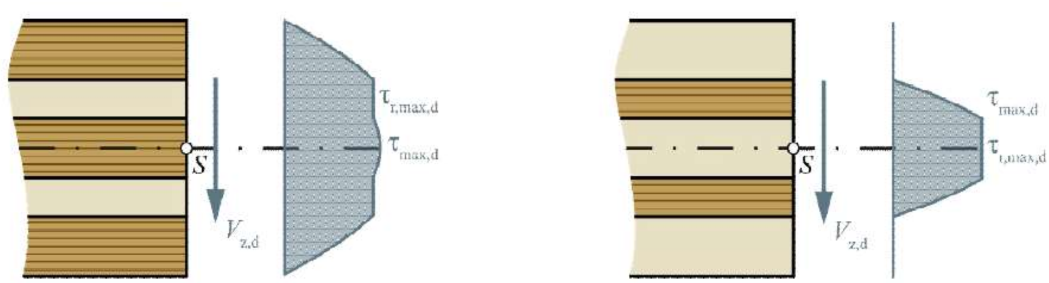
Out-of-plane shear



Harris, R., Ringhofer, A., Schickhofer, G., 2013, *COST Action FP1004 with TU Graz*, European Conference on Cross Laminated Timber (CLT)

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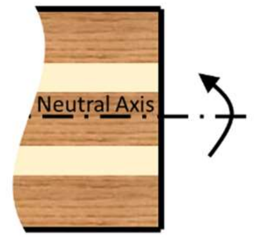
Out-of-plane Deflection For Bending in Strong Direction

- Rolling shear
- Use A, I, and mean E_0 and G_0 of parallel to span layers only.
- Bending deflection: $E_0 I$
- Shear deflection: $\kappa G_0 A$
- κ is a correction for the rolling shear of the cross layers and can be conservatively taken as 0.2-0.3



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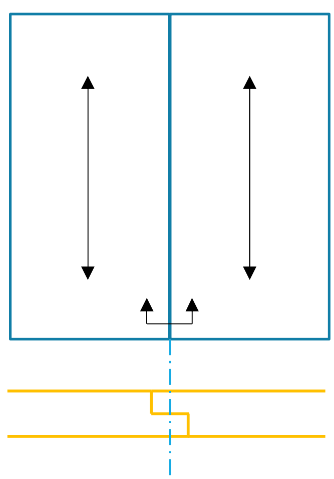
CLT only one-way spanning due to connections!



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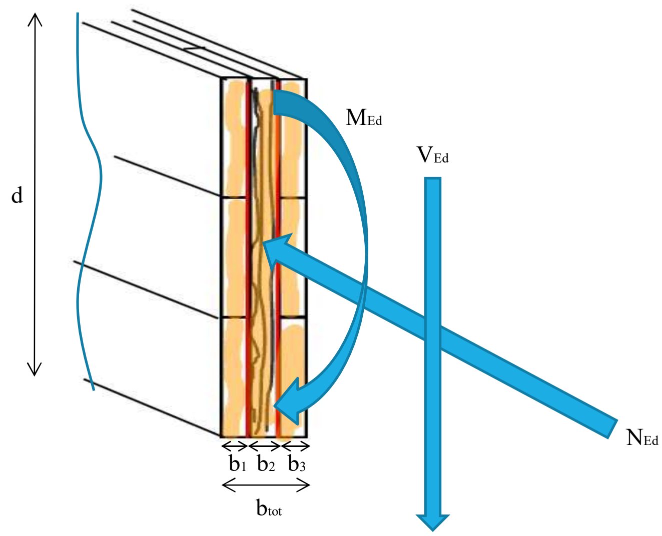
ARUP 192

In-plane Loading



$$A_{net} = (b_1 + b_3)d$$

$$I_{net} = \frac{(b_1 + b_3)d^3}{12}$$

Axial Loading

- For short stocky members axial capacity based on useful layers (parallel to span layers)

Axial Loading

Check as if CLT wall is a slender column as per EC5 taking b=1.

The only difference is that relative slenderness is calculated as follows to include shear flexibility:

Where:

$$\lambda = \sqrt{\frac{A_{net,} f_{c,0,k}}{n_{cr}}}$$

$$n_{cr} = \frac{E_{0,0.05} I_{net} \pi^2}{{l_{ef}}^2 + \frac{E_{0,0.05} I_{net}}{G_{0,0.05} A_{net}}}$$

- A_{net}: Area of CLT with grain parallel to load
- $f_{c,0,k}$: Characteristic strength of area of CLT with grain parallel to load
- λ: Relative slenderness
- $E_{0,0.05}$: Characteristic young's modulus of CLT layers with grain parallel to load
- $G_{0,0.05}$: Characteristic shear modulus of CLT layers with grain parallel to load
- I_{net}: Second moment of area of net section
- A_{net}: Area CLT layers with grain parallel to load
- n_{cr}: Ideal elastic buckling load

Bending

- Combine with any axial stress due to axial loads

$$\sigma_{max,d} = \frac{M_{Ed} y}{I_{net}}$$

$$\sigma_{max,d} \leq min \begin{pmatrix} f_{c,0,d} \\ f_{t,0,d} \end{pmatrix}$$

- $\sigma_{max,d}$: Maximum design bending and axial stress
- M_{Ed}: Design moment
- y: Distance from neutral axis to extreme fibre
- I_{net}: Second moment of area of net section
- $f_{t,0,d}$: Design tensile strength of area of CLT with grain parallel to load
- $f_{c,0,d}$: Design compressive strength of area of CLT with grain parallel to load

In-plane shear

- In-plane shear of CLT is highly complex for simplified verification methods refer manufacturer European technical approval.

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Training seminar / Apmācību seminārs

Floor Dynamics – the New EC5 Method
Section No.7/ Sadaļa Nr.7

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Why?

- Competition
- Wood is lighter
- Too conservative = too expensive
- Non –conservative will scare people off timber
- **Floor vibration generally governs floor depth for large multi storey buildings.**
- **This lecture will touch on the updates to the next version of EN 1995-1-1**

Background reading

Things to think about

Loading:
- Amplitude
- Frequency
- Duration

→

Structural characteristics:
- Frequency
- Modal mass
- Stiffness
- Mode shape
- Damping

→

Response
- Acceleration
- Velocity

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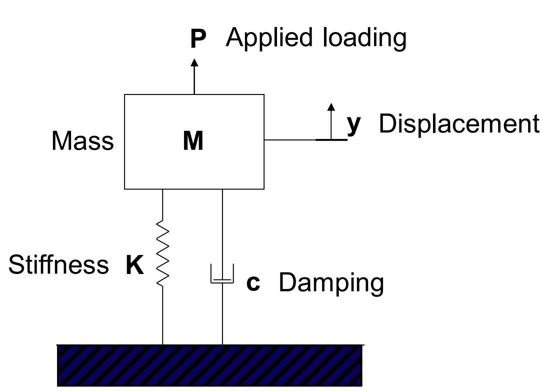
ARUP 201

Structural Characteristics of Single Degree of Freedom system



$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

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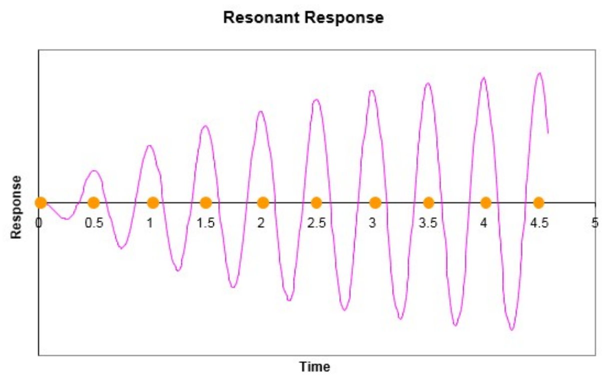
ARUP 202

Resonant Response

- Resonant build up occurs when the forcing frequency matches natural frequency



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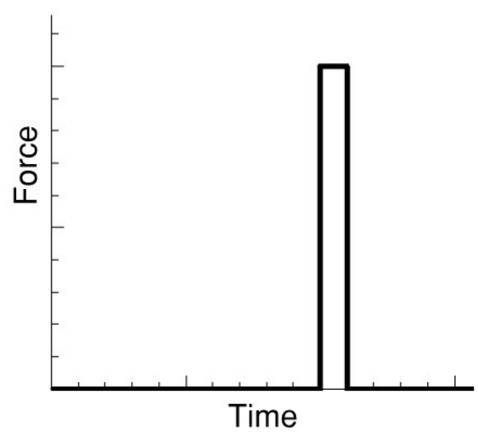
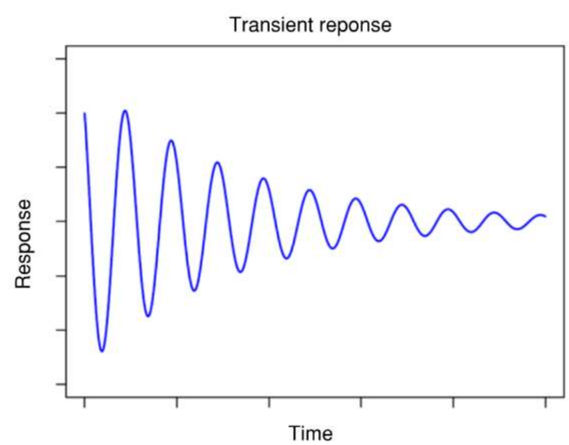
Transient Response





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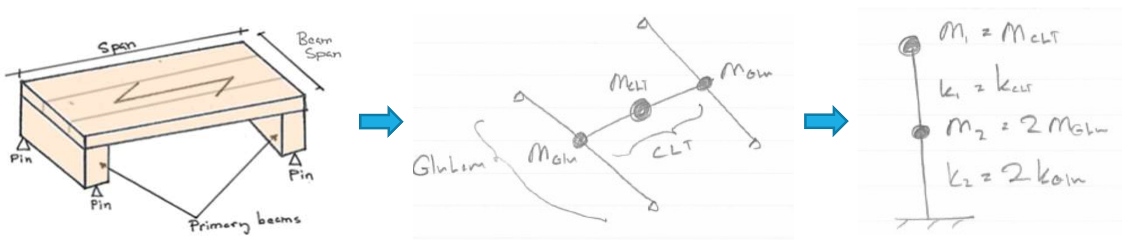
Multi Degree of Freedom Systems

- Floors have more than 1 degree of freedom.



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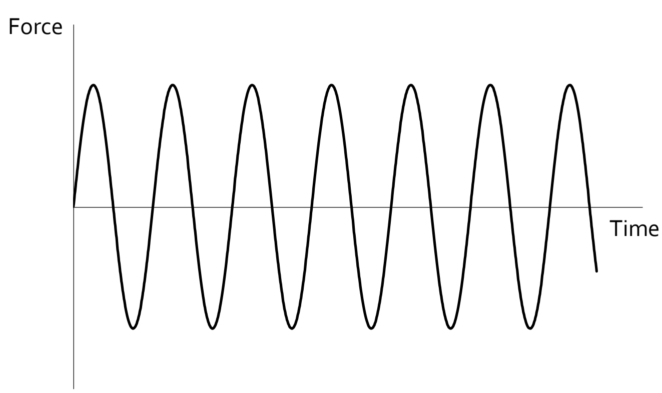
ARUP 205

Harmonic Loading



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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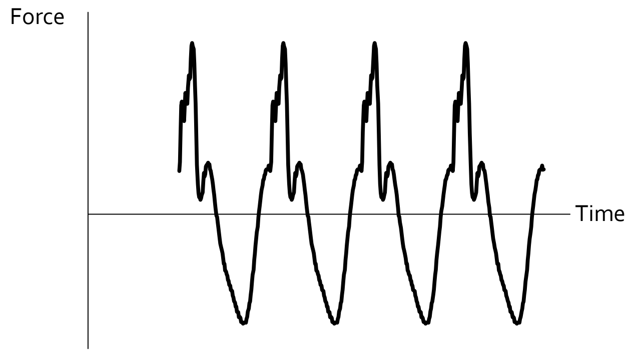
ARUP 206

Periodic Loading From a Real Footfall

- Several frequencies
- Can be broken down in to harmonic components



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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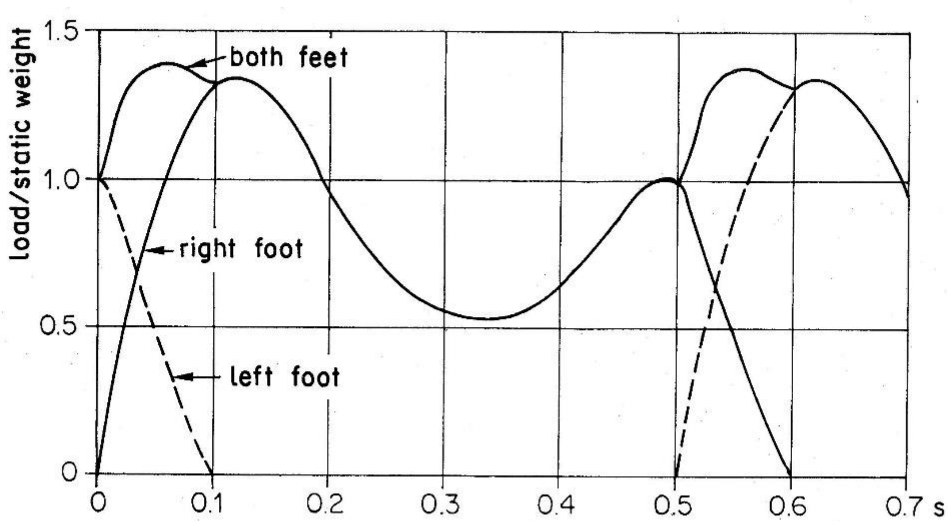
ARUP 207

People walk between 1.5 – 2.5 Hz



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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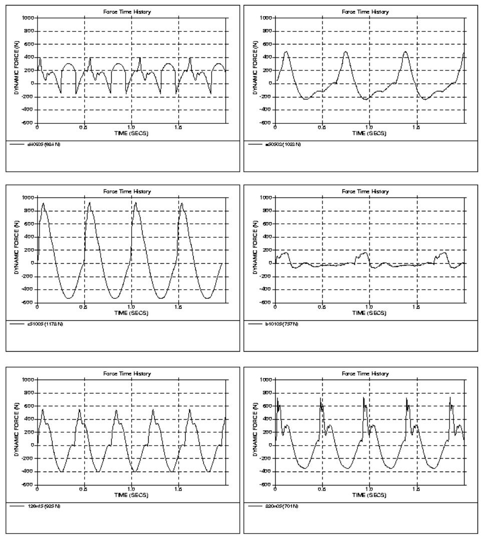
ARUP 208

Force varies with:

- Individual
- Footwear
- Speed



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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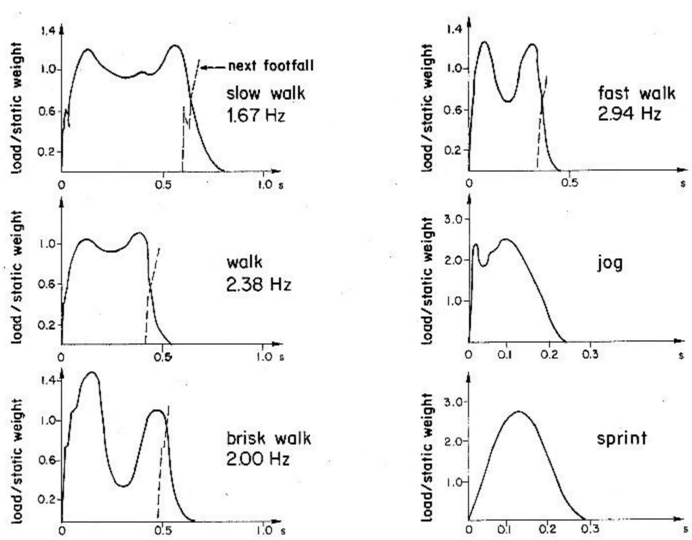
ARUP 209

Force varies with:

- Speed



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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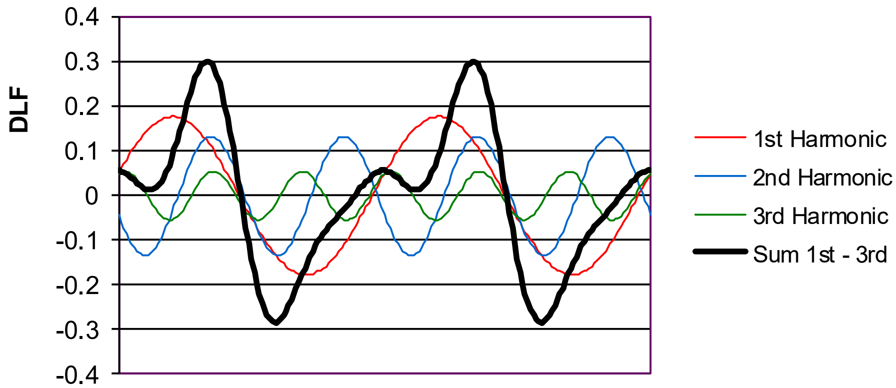
ARUP 210

First 3 Harmonics from a Footfall Walk



$$DLF = \frac{amplitude\ of\ harmonic\ component}{static\ weight\ of\ walker}$$

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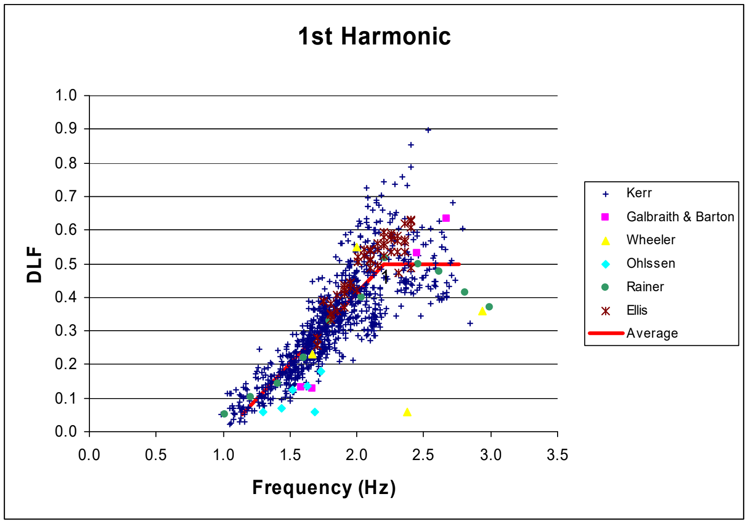
ARUP 211

Amplitude of 1st harmonic compared with walking frequency



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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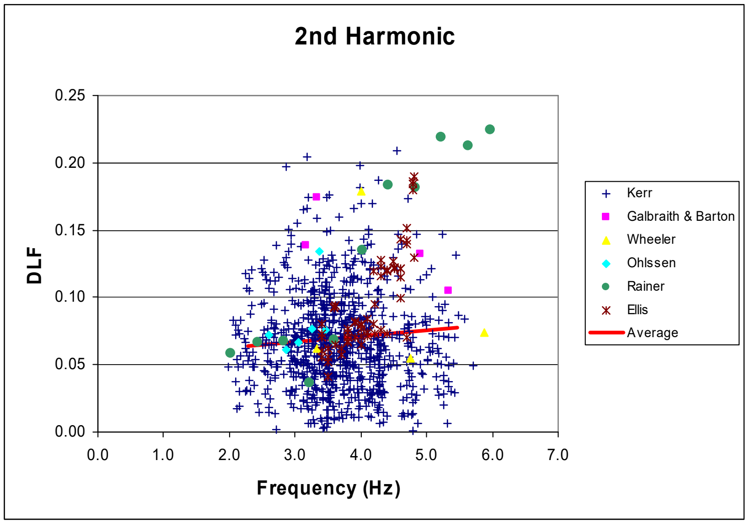
ARUP 212

Amplitude of 2nd harmonic compared with double the walking frequency



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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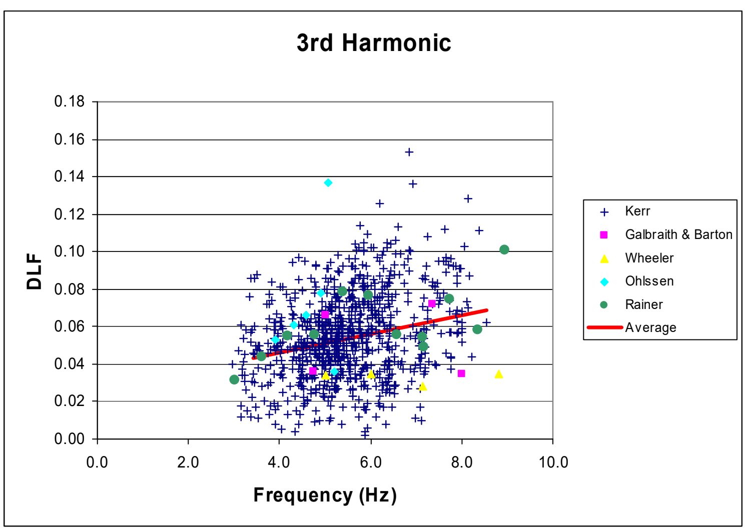
ARUP 213

Amplitude of 3rd harmonic compared with triple the walking frequency



Willford, M. R., Young, P., 2006, *A Design Guide for Footfall Induced Vibrations of Structures*, The Concrete Society

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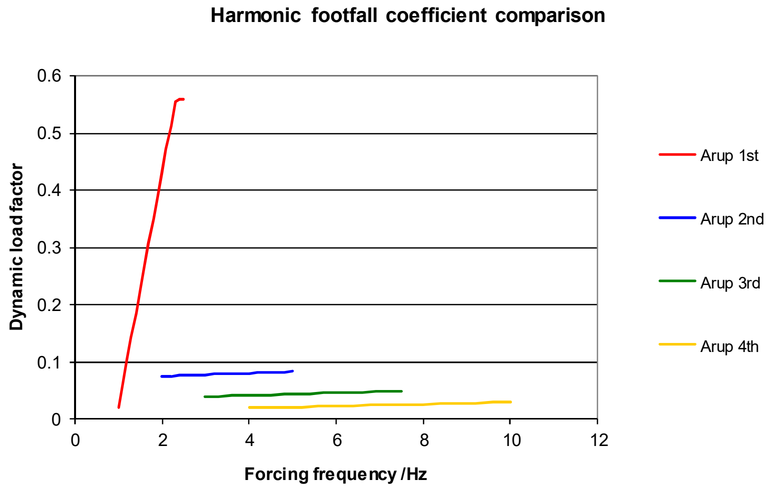
ARUP 214

Harmonic Force vs. Frequency



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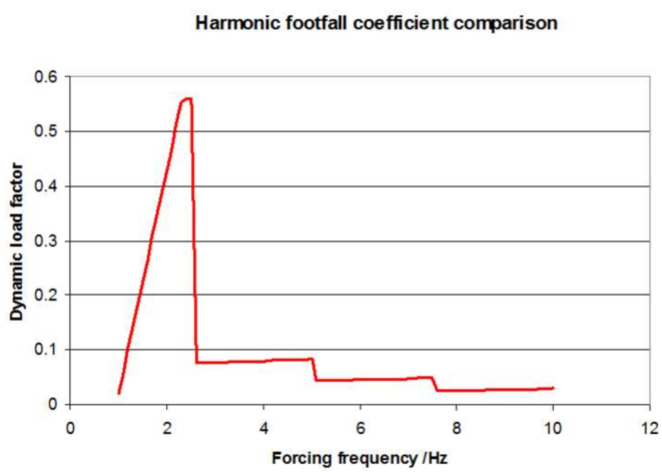
ARUP 215

Harmonic Force vs. Frequency



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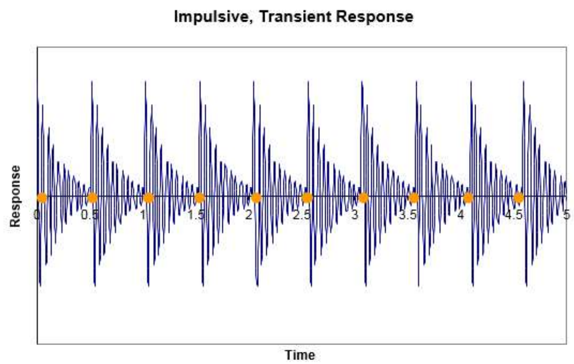
ARUP 216

Impulsive Loading

- When floor frequency is higher than the 4th harmonic of walk.
- The floor sees a chain of impulsive loading between which response dies off.
- Majority of domestic timber floors

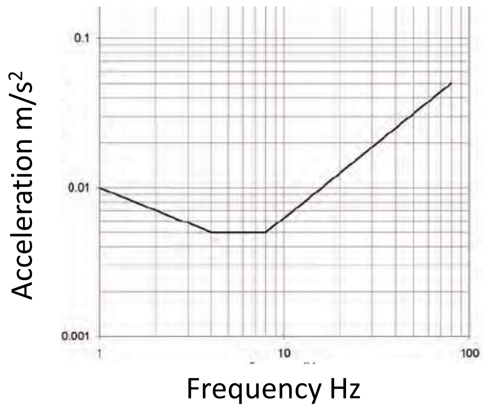


Acceptable Response

- What we feel is acceleration.
- Response factor, $R = \frac{Response}{Minimum\ percievable\ repsonse}$

Acceptable Response

- BS 6472 – perceptible levels of acceleration



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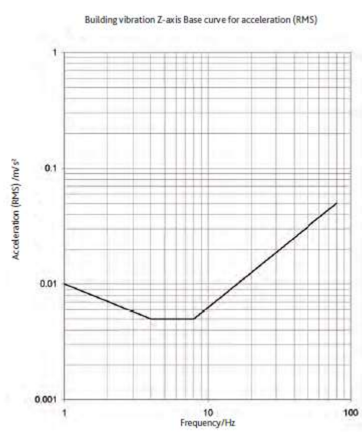
ARUP 219

Acceptable Response

- BS 6472 – perceptible levels of acceleration (also CCIP)
- For $4\ Hz \leq f_n \leq 8\ Hz$:
  - $R = \frac{a_{rms}}{5m\ \ /s^2}$
- For $f_n > 8\ Hz$:
  - $R = \frac{V_{rms}}{0.1mm/s}$



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ARUP 220

Acceptable Response

- Perception is subjective.

Depends on:

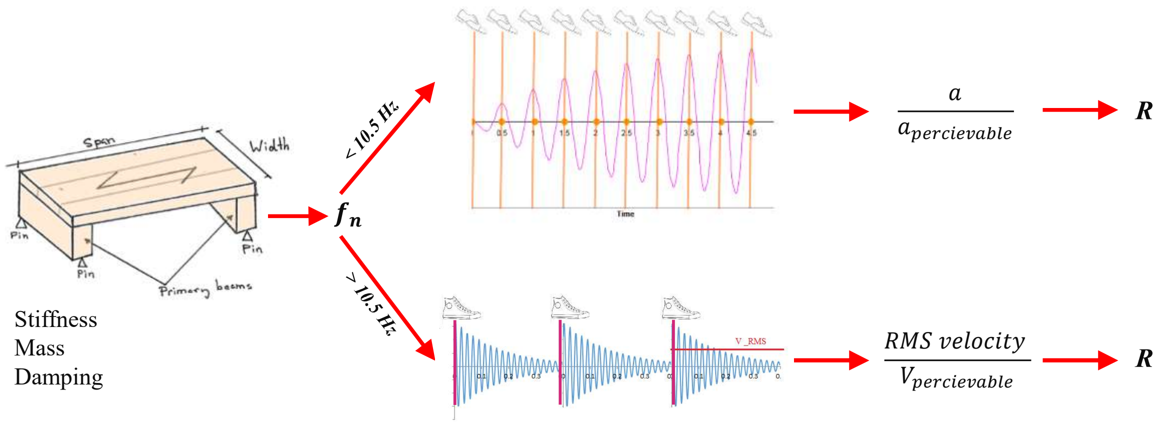
- Person
- Use of floor
- Look of floor
- How often floor is excited

Recommended Limits Based on Testing

- Standard offices: $R \leq 8$
- High end office: $R \leq 4$

How does a footfall analysis work?

Floor system | Natural frequency | Loading & Response | Perception level



How does this all apply to a simple floor?

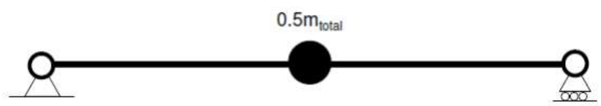
- Stiffness (EI, $Length$, etc)
  - EI: Material and floor thickness
  - Span: One way vs two way (stiffness)
  - Span: length
  - End condition: Pinned vs fixed
- Mass
  - Dead Load + (0.1 * Live Load)
- Damping: Connection detail, false floor, partitions etc.

How do we model a CLT floor?

How do we go from this......

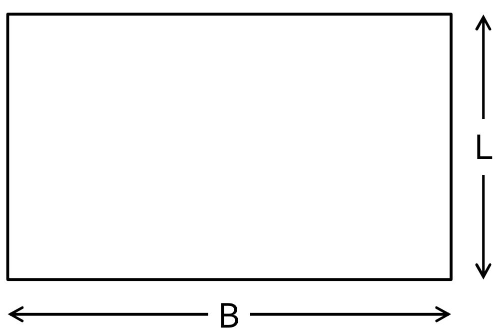
......to this?

Reality



Accurate idealisation

Analysis of a simple floor – Pin supported at edges



$$f_1 = k_{e,2} \frac{\pi}{2L^2} \sqrt{\frac{(EI)_L}{m}}$$

$$k_{e,2} = \sqrt{1 + \left(\frac{L}{B}\right)^4 \frac{(EI)_T}{(EI)_L}}$$

Analysis of a simple floor – Resonant calculation

(If frequency less than 4 x walking frequency)

$$a_{rms} = \frac{k_{res} \mu F_h}{\sqrt{2}\, 2\, \zeta\, M^*}$$

$$R = \frac{a_{rms}}{5\ mm/s^2}$$

k_{res}	is a factor to account for higher modes of vibration, as calculated with.
μ	is the resonant build up factor, which may be conservatively taken as μ = 1
F_h	is the vertical dynamic force caused by the assumed weight of a walking person and should be taken as F_h = 50 N.
ζ	is the modal damping ratio.
M^*	is the modal mass as given in in kg.

$$k_{res} = max \begin{cases} 0{,}192 \left(\frac{B}{L}\right)\left(\frac{(EI)_L}{(EI)_T}\right)^{0{,}25} \\ 1{,}0 \end{cases}$$

$$M^* = \frac{m\, L\, B}{4}$$

Analysis of a simple floor – Transient calculation

(should be checked for floors of all frequencies)

1. $I_m = \frac{42 f_w^{1.43}}{f_1^{1.3}}$ Ns
2. $v_{1,peak} = 0.7 \frac{I_m}{(M^*+70)}$ m/s
3. $v_{tot,peak} = k_{imp} v_{1,peak}$ m/s
4. $k_{imp} = max \begin{cases} 0{,}48 \left(\frac{B}{L}\right)\left(\frac{(EI)_L}{(EI)_T}\right)^{0{,}25} \\ 1{,}0 \end{cases}$
5. $v_{rms} = v_{tot,peak}(0{,}65 - 0{,}01 f_1)(1{,}22 - 11{,}0\zeta)\eta$

$$\eta = 1{,}35 - 0{,}4k_{imp}, when\ 1{,}0 \le k_{imp} \le 1{,}9\ else\ \eta = 0{,}59 \quad (for\ joisted\ floors)$$

$$\eta = 1{,}35 - 0{,}4k_{imp}, when\ 1{,}0 \le k_{imp} \le 1{,}7\ else\ \eta = 0{,}67 \quad (for\ all\ other\ floors)$$

The new EN 1995-1-1

- Will have hand methods for simple floors.
- Will have an Annex explaining how to use modal results from FE to carry out a multi modal analysis for more complex floors.

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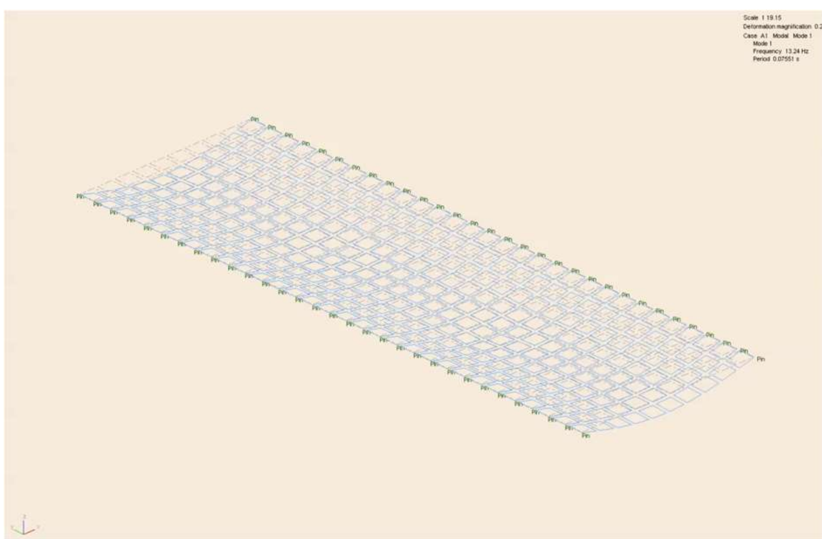
ARUP 229

CLT on Pinned Supports

- 3m span
- f = 13.24 Hz



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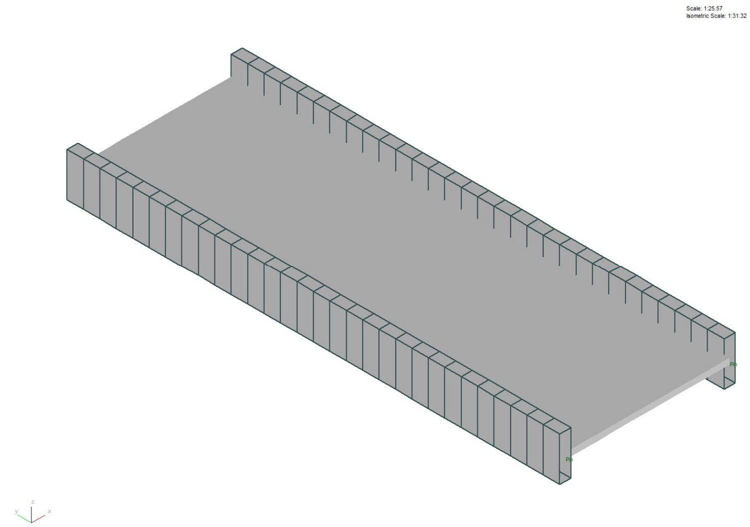
ARUP 230

CLT on Beams

- 3m CLT span
- 9m beam span



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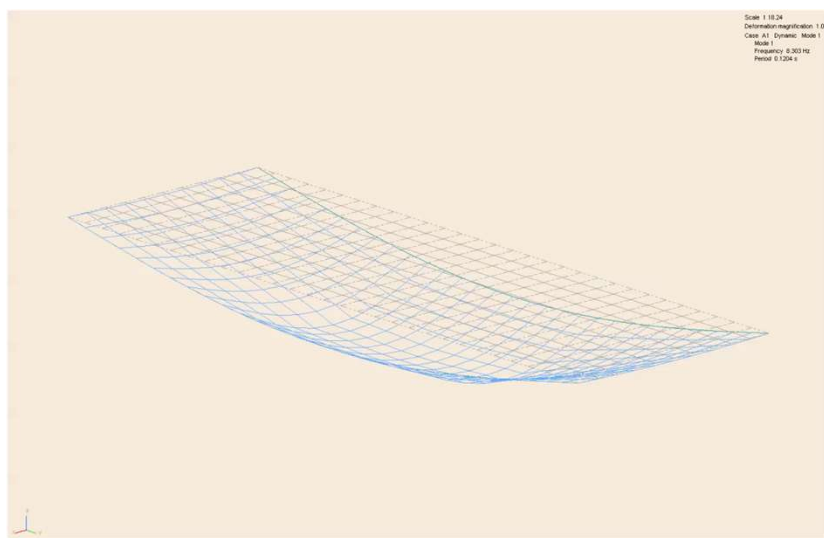
ARUP 231

CLT on Beams

- 3m CLT span
- 9m beam span
- f = 8.5 Hz



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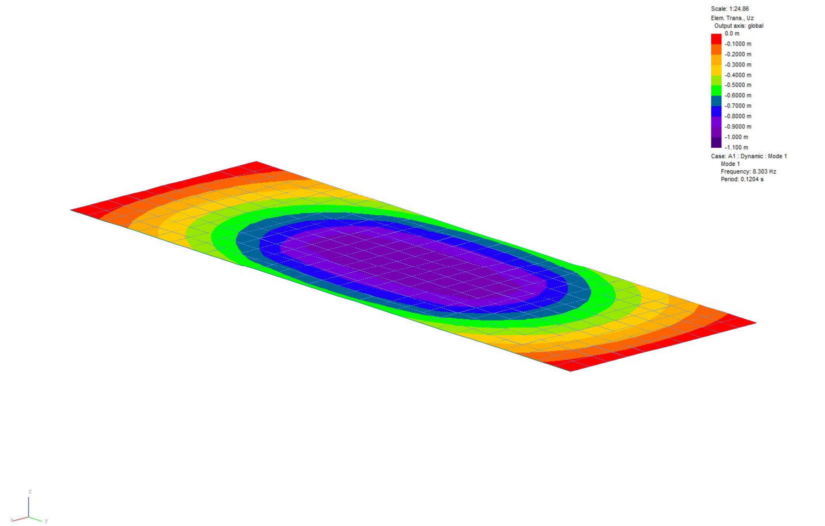
ARUP 232

CLT on Beams

- 3m CLT span
- 9m beam span
- f = 8.3 Hz



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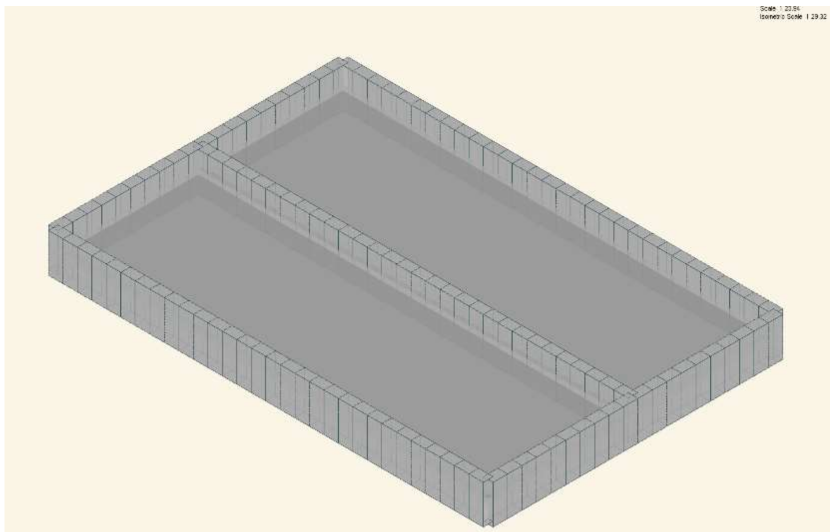
ARUP 233

CLT on Beams Double Span

- 3m CLT span
- 200x800 beams
- 9m beam span



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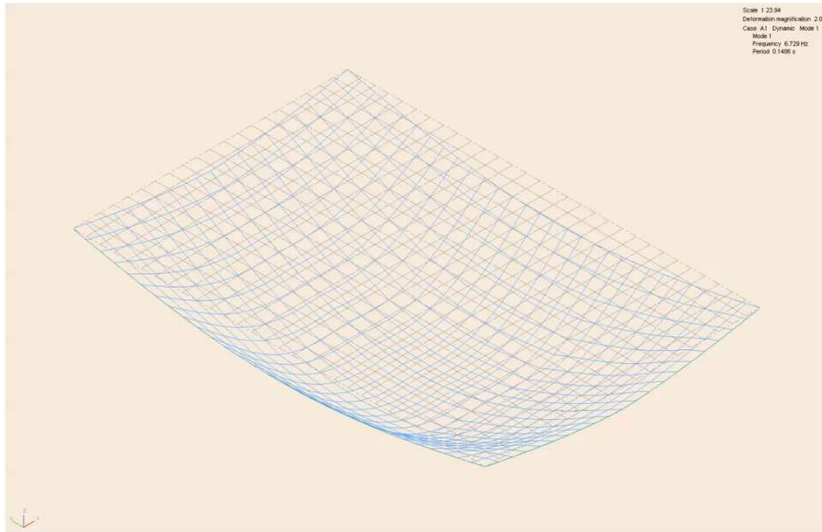
ARUP 234

CLT on Beams Double Span

- 3m CLT span
- 200x800 beams
- 9m beam span
- f = 6.7 Hz



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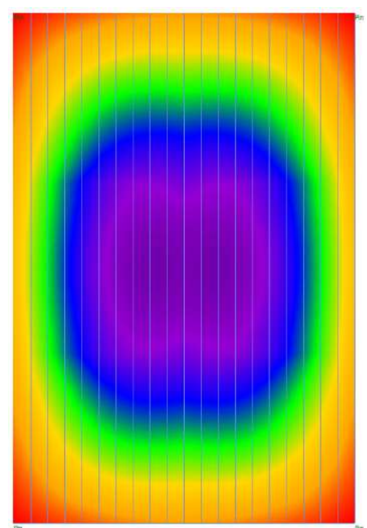
ARUP 235

CLT on Beams Double Span

- 3m CLT span
- 9m beam span
- f = 6.7 Hz



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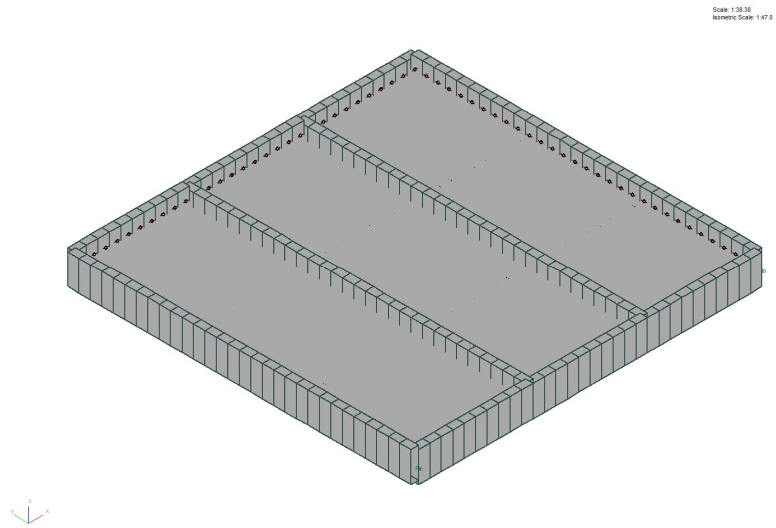
ARUP 236

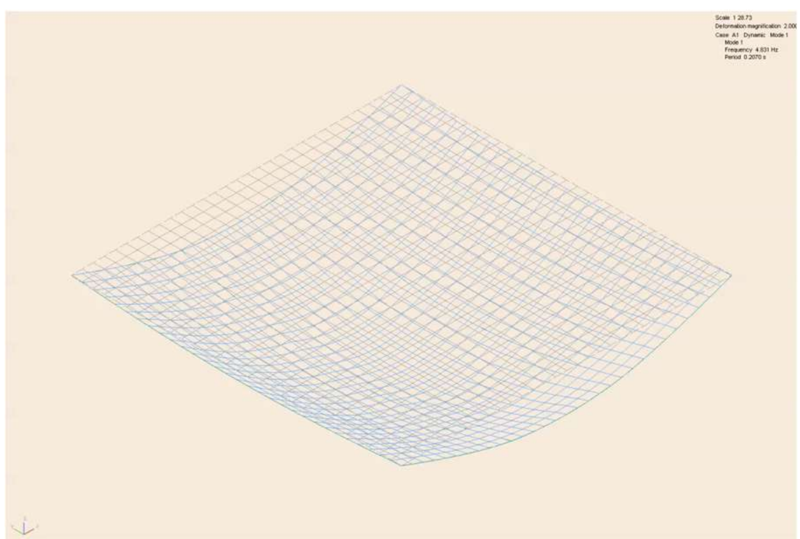
CLT on Beams Multiple Span

- 3m CLT span
- 200x800 beams
- 9m beam span



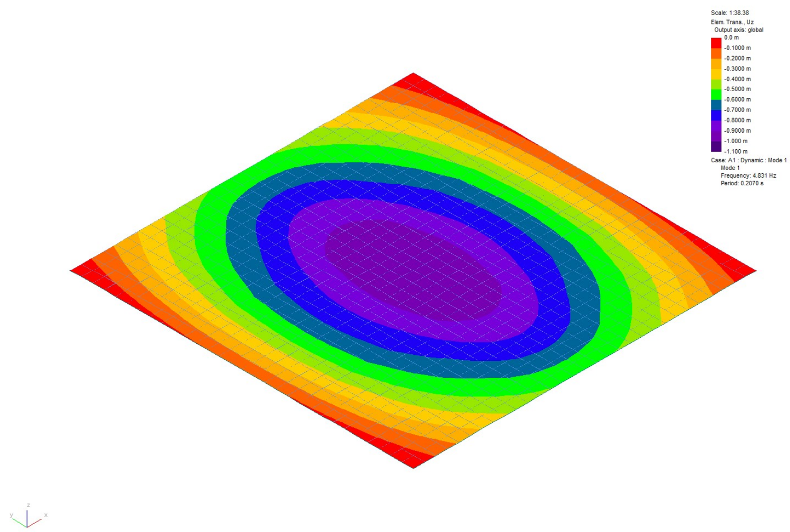
CLT on Beams Multiple Span

- 3m CLT span
- 200x800 beams
- 9m beam span
- f = 4.8 Hz



CLT on Beams Multiple Span

- 3m CLT span
- 200x800 beams
- 9m beam span
- f = 4.8 Hz



Beam flexibility has a considerable effect on dynamic behaviour of floors

- 13 Hz → 5 Hz
- Codes should explicitly state that effect of beams should be taken into account.

Footfall Measurement

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ARUP 241

Ministry of Economics
Republic of Latvia

Training seminar / Apmācību seminārs

Specification & Common Errors
Section No.8/ Sadaļa Nr.

Andrew Lawrence and Ishan Abeysekera (Arup,United Kingdom)

Concept Stage

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Durability – protect structure from rain + always provide 2 lines of defence

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Fire – determines how much wood can be left exposed

© Simon Kennedy

Dynamics – governs floor design

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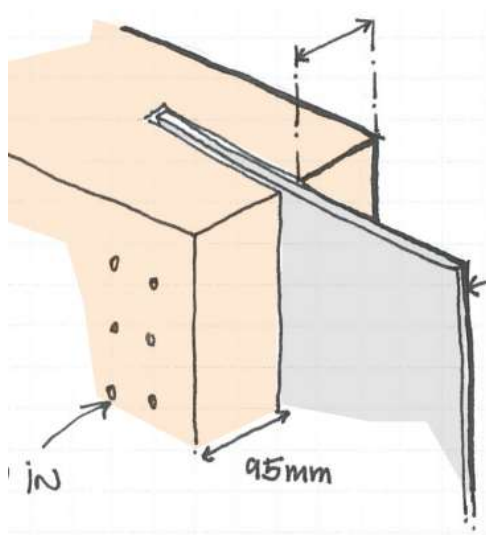
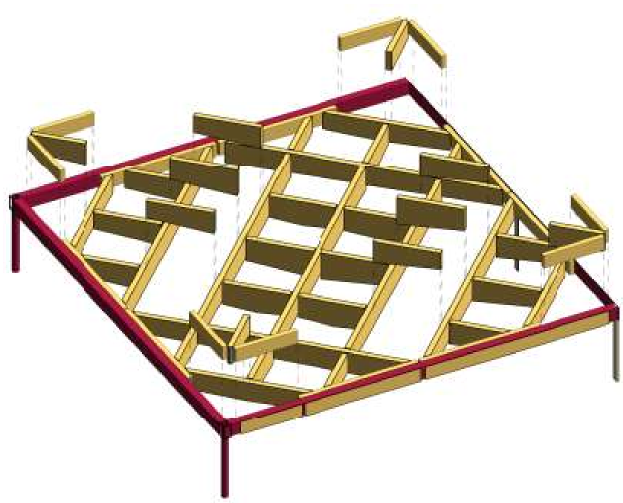
Connections – govern size of members



Brittle material – make structures determinate



Detail Design Stage

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Species – use spruce for economy

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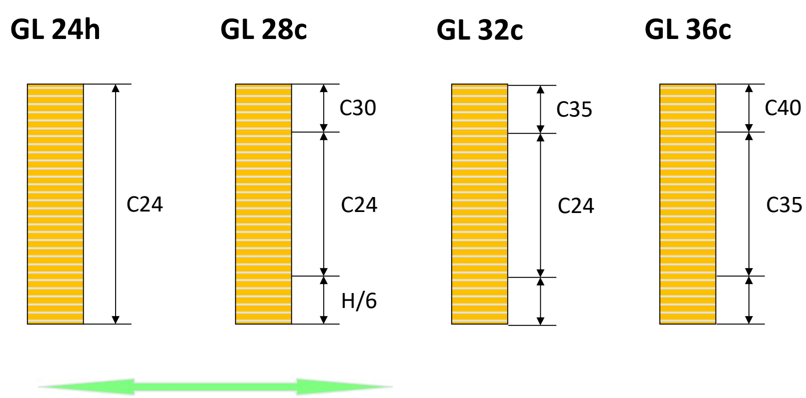
Use standard grades & standard sizes



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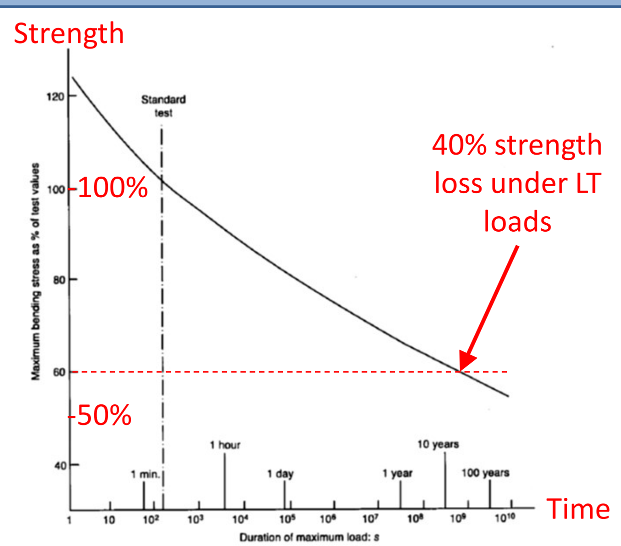
ARUP 251

Load duration – check all governing loadcases



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Design connections to accommodate shrinkage

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Construction Stage

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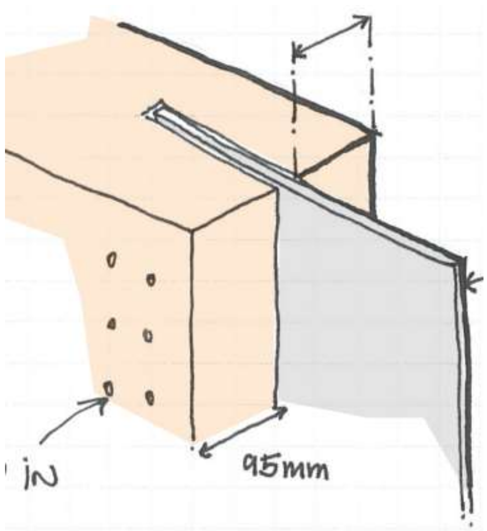
ARUP 254

Same Engineer must be responsible for member & connection design



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Source sustainably

FSC©

PEFC™

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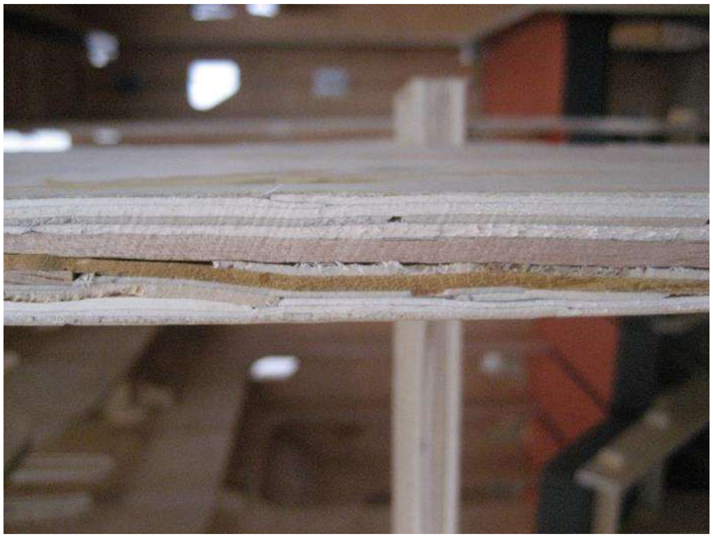
Gluing is highly workmanship dependent – avoid site gluing & use reliable suppliers



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ARUP 257

Don't overspecify the appearance

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ARUP 258

Avoid structural damage during construction

- Ensure builder has a moisture control plan
- Provide falls to flat roofs
- Seal end grain at connections
- Where possible ensure the wood can breathe in the finished building to allow construction moisture to escape

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Avoid appearance damage during construction

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Specify key tolerances

- Supports
- Surfaces requiring contact in bearing?
- Exposed architectural work

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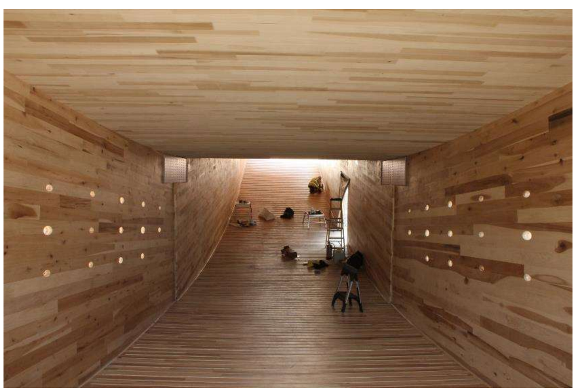

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Question and answer session

Ministry of Economics
Republic of Latvia

Thank you for attention!

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