CLT Wood Design Seminar

MONCTON, December 1st 2015

Design of CLT wall and floor to resist lateral and gravity loads

Roberto Tomasi*

UNITRENTO

In cooperation with

TIMBERTECH

Startup of the University of Trento

* Harrison McCain Visiting Professor at UNB

Layout presentation

1. Introduction

2. Cross laminated timber, the product and the constructive system

3. Comparison between the CSAO86 and Eurocode 5

4. Design example

UNIVERSITÀ DI TRENTO 50 ANNI 1962 - 2012

16.000 students
600 professors and researcher

Department of
CIVIL, ENVIRONMENTAL AND MECHANICAL ENGINEERING

Course in **Timber Engineering** in Master degree in Building Engineering/Architecture and Civil Engineering

Department of
CIVIL, ENVIRONMENTAL AND MECHANICAL ENGINEERING

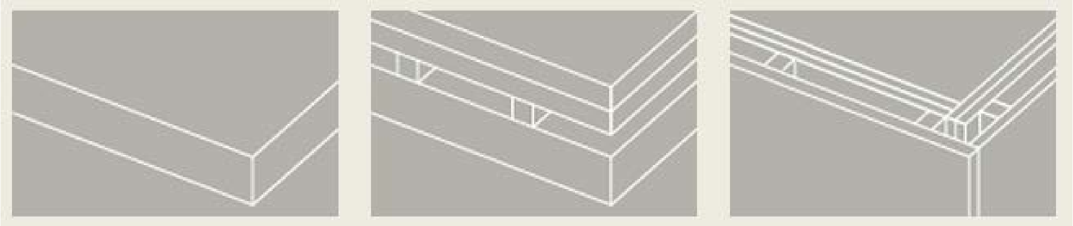
TRENTO TIMBER
RESEARCH GROUP

EXPERTISE

- **Seismic design of new and existing timber structures**
- **Technology transfer (software, courses, industrial research)**

Catalogo dataholz
Sistemi costruttivi

dataholz.com
Servizio di Holzforschung Austria



promo_legno risponde
Consulenza tecnica

In collaborazione con
TIMBERTECH
SOLUTIONS FOR TIMBER ENGINEERING

promo_legno risponde
- Tecnologia e prodotti
- Fisica tecnica
- Statica e calcolo
- Antisismica
- Certificazione

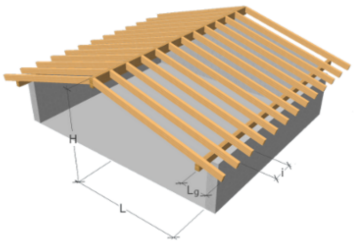
Calcolo Coperture
Tool Online

Predimensionamento coperture

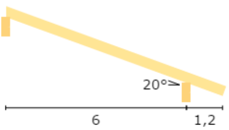
Geometria | Stratigrafia | Neve | Vento | Risultati

Geometria della copertura

Luce	L	6	[m]
Gronda	L_g	1,2	[m]
Pendenza	α	20	[°]
Interasse	i	0,7	[m]
Altezza colmo	H	5	[m]



Geometria travi secondarie



Legno Lamellare

Il legno lamellare incollato è costituito da almeno tre tavole o lamelle essiccate e incollate tra loro con le fibre parallele. Prima di essere incollate, le lamelle vengono classificate secondo la resistenza in modo visivo o meccanico e piallate. Il legno lamellare incollato è particolarmente adatto per componenti da costruzione soggetti a carichi elevati e con una luce molto ampia, oltre che per esigenze elevate di stabilità della forma e di estetica.

Indietro — Avanti

Materiale travi secondarie

Materiale	Legno lamellare di conifera
Classe di resistenza	GL 24h

Department of
CIVIL, ENVIRONMENTAL AND MECHANICAL ENGINEERING

TRENTO TIMBER
RESEARCH GROUP

EXPERTISE

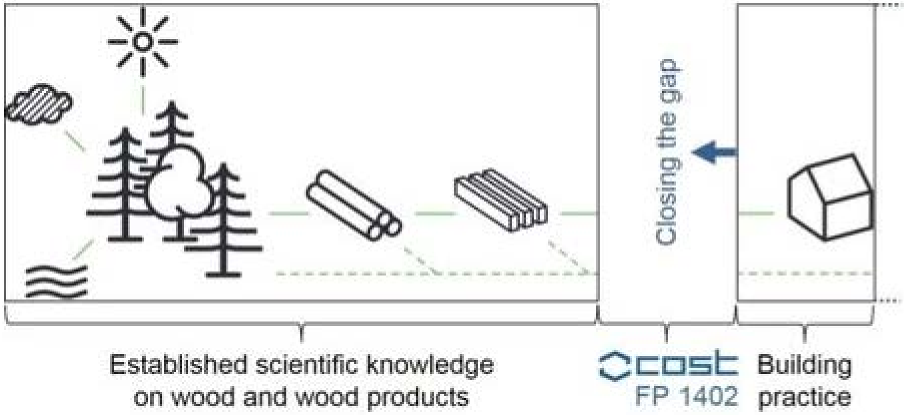

- **Seismic design of new and existing timber structures**
- **Technology transfer (software, courses, industrial research)**

CLT Training Course
In the frame of COST Action FP1004

April 15th-17th 2014, Trento, Italy

85 People 28 Countries

COST Action FP1402 "Basis of Structural Timber Design" - from research to standards



Layout presentation

1. Introduction
2. Cross laminated timber, the product and the constructive system
3. Comparison between the CSAO86 and Eurocode 5
4. Design example

Cross Laminated Timber (CLT) is a …

is the generic term for a class of thick or massive engineered wood plate products suitable for applications like wall and floor slabs.

First production in Austria and in Germany

‘90

First Approval in Austria

1998

0,5 M m^3/year

2013

2015

THE PRODUCTION IS PROJECTED TO 0,6÷1 M mc

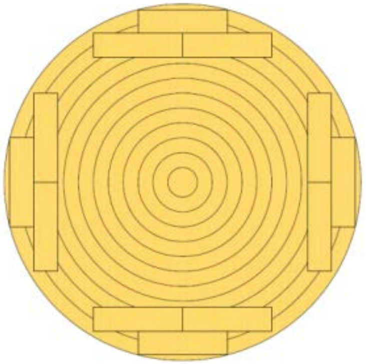
- XLAM (Italy)
- BSP Brettsperrholz (Germany-Austria)
- CLT (Cross Laminated Timber)

▪2013 more than 35 production sites worldwide

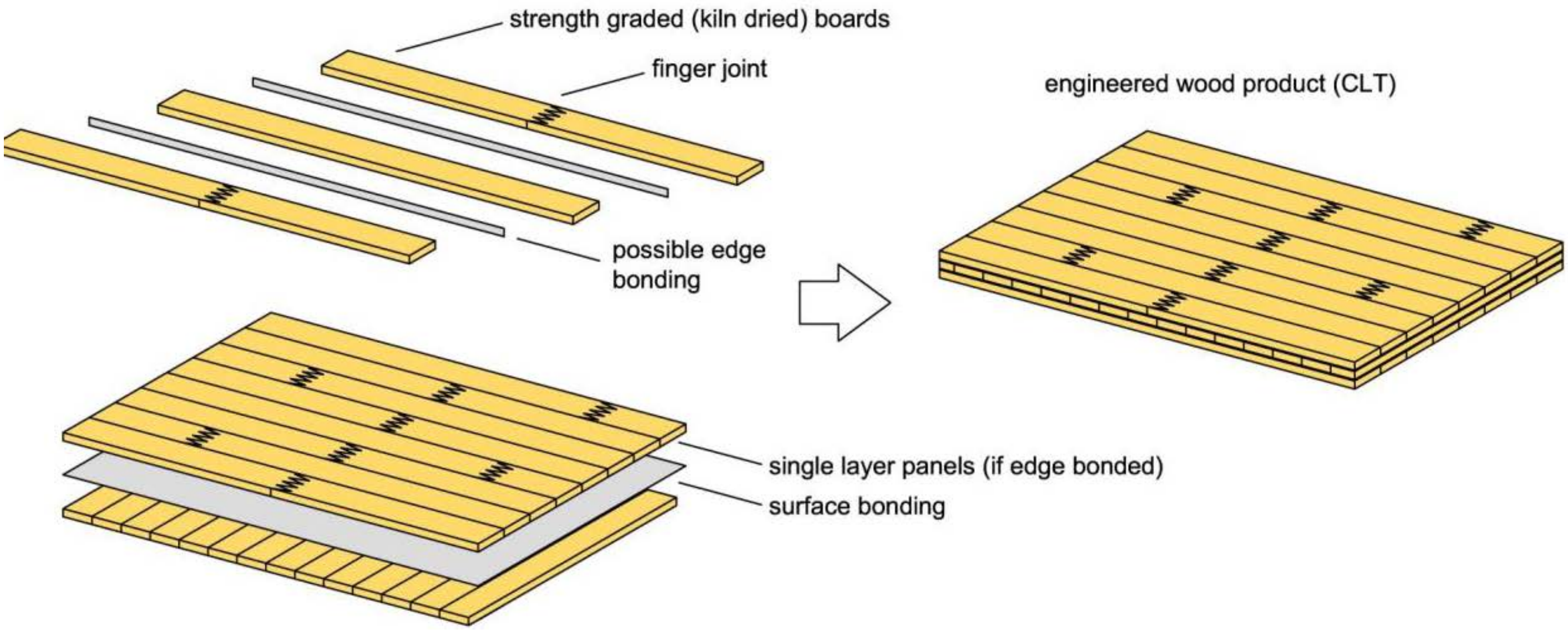
▪currently roughly 95 % of the production volume in Central Europe

▪Austria (~ 63 %)
▪Germany (~ 26 %)
▪Switzerland (~ 6 %)

CLT production process



- missing market for the side-boards from sawmilling
- in wood technology good mechanical properties of side boards.



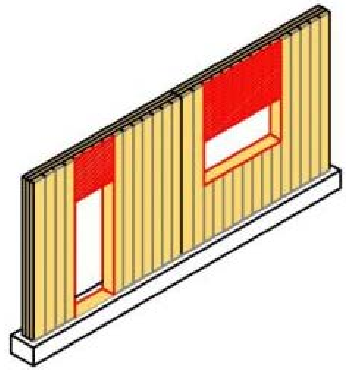
- Lumber pieces in some layers are arranged orthogonal to pieces in other layers,
- cross-reinforced in all directions like plywood

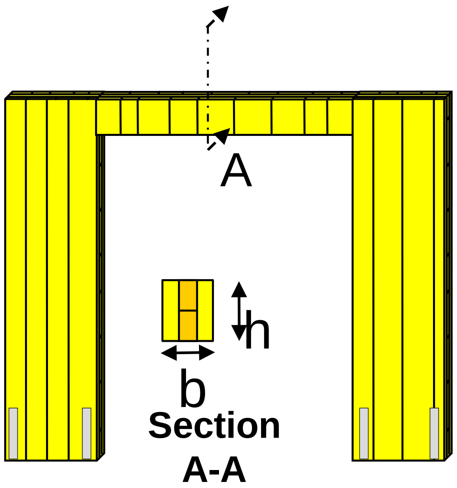
Cross Laminated Timber (CLT) is a …

- used for **large-sized wall and floor elements (2D) product** for load bearing in-plane and out-of-plane

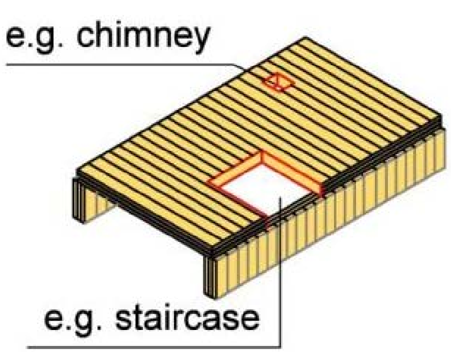
WALL

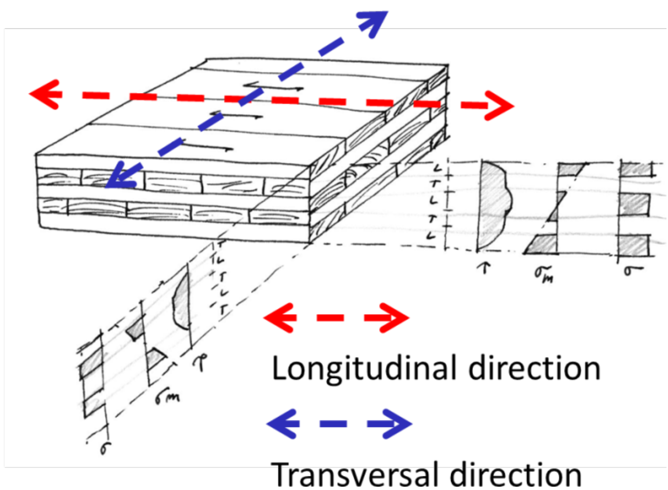
with openings





FLOOR





Modified from Brandner (2014)

MID- AND HIGH-RISE CLT MULTI-STOREY BUILDINGS

7 storeys

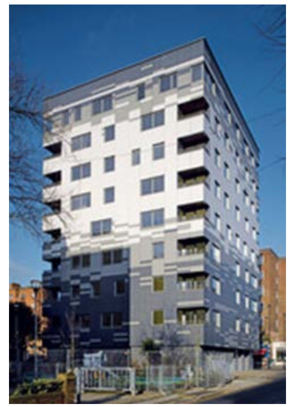
SOFIE project 2007

7+1 storeys

Växjö 2009

8+1 storeys

London 2008



8 storeys

London 2011

8 storeys

Ås 2013

9+1 storeys

Milan 2012

9+1 storeys

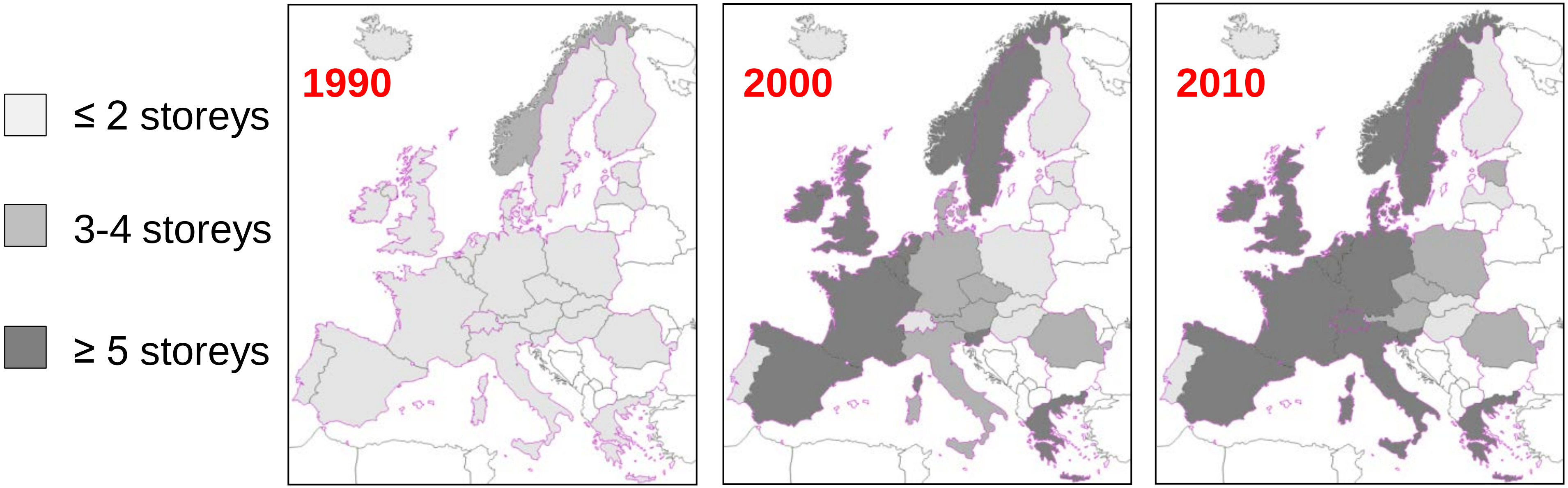
Melbourne 2012

12+1 storeys

Quebec 2016

FIRE SAFETY IN MULTI-STOREY TIMBER BUILDINGS

Fire safety is widely considered as one of the most significant obstacles for increasing the use of wood in construction. Most fire regulations in Europe have traditionally been very **prescriptive** and based on experience from large city fires. World-wide, several research projects on the fire behaviour of timber structures have been conducted over the past decades, aimed at providing basic data and information on the safe use of timber. **Novel fire design concepts** and **models** have been developed, based on extensive testing and modelling.



From: Birgit Östman, Bo Källsner, National building regulations in relation to multi-storey wooden buildings in Europe, School of Technology and Design, Reports, No. 60, Växjö University Sweden 2011

SEISMIC RESEARCH IN MULTI-STOREY TIMBER BUILDINGS

SOFIE- PROJECT (2007)

LEAD USER
CECCOTTI
CNR-IVALSA
TREES AND TIMBER INSTITUTE

SERIES PROJECT (2012-2013)

SEVENTH FRAMEWORK PROGRAMME

UNITRENTO

LEAD USER
TOMASI-PIAZZA

UNIVERSITY OF TRENTO

USER TEAM

ACCESS PROVIDER

TU Graz

UNIVERSITY OF GRAZ
SCHICKHOFER-FLATSCHER

CLT

UNIVERSITY OF MINHO
LOURENCO - BRANCO

LOGHOUSE

LNEC

PLATFORM

SEISMIC RESEARCH IN MULTI-STOREY TIMBER BUILDINGS

SOFIE- PROJECT (2007)

SERIES PROJECT (2012-2013)

PSEUDO-DYNAMIC TEST

U-TRENTO LABORATORY 2006

SHAKE TABLE TEST

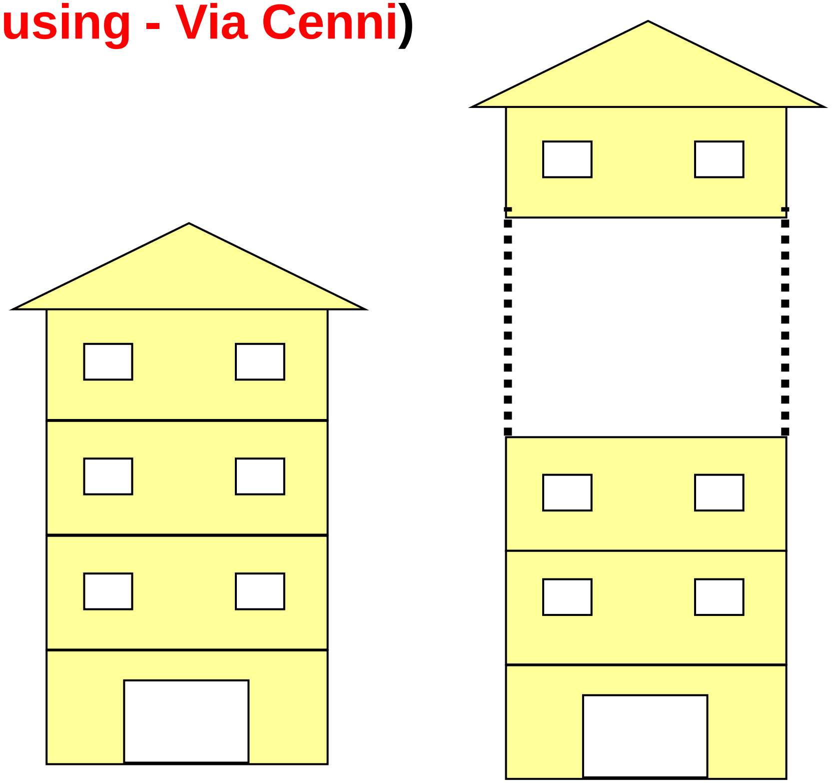
LNEC LABORATORY 2013

PGA 0,50g

SEISMIC SAFETY IN MULTI-STOREY TIMBER BUILDINGS

≥ 4 STOREY TIMBER BUILDING

TECHNICAL APPROVAL FROM HIGH COUNCIL OF PUBLIC WORKS (ART.52) del Consiglio Superiore dei ll.pp. per n.4 edifici (9 piani) a Milano (Social housing - Via Cenni)



From December 2011 this law was abolished

Consiglio Superiore dei Lavori Pubblici – Servizio Tecnico Centrale

Ministero delle Infrastrutture e dei Trasporti

PRESIDENZA DEL CONSIGLIO SUPERIORE DEI LAVORI PUBBLICI

SERVIZIO TECNICO CENTRALE

DICHIARAZIONE DI IDONEITA' TECNICA

ai sensi dell'art.52 del DPR 380/2001 (art.1 Legge n.64/1974)

Progetto di n.4 edifici in Via Cenni (Milano)
con struttura portante in Pannelli base legno
società Polaris Investment Italia SGR spa

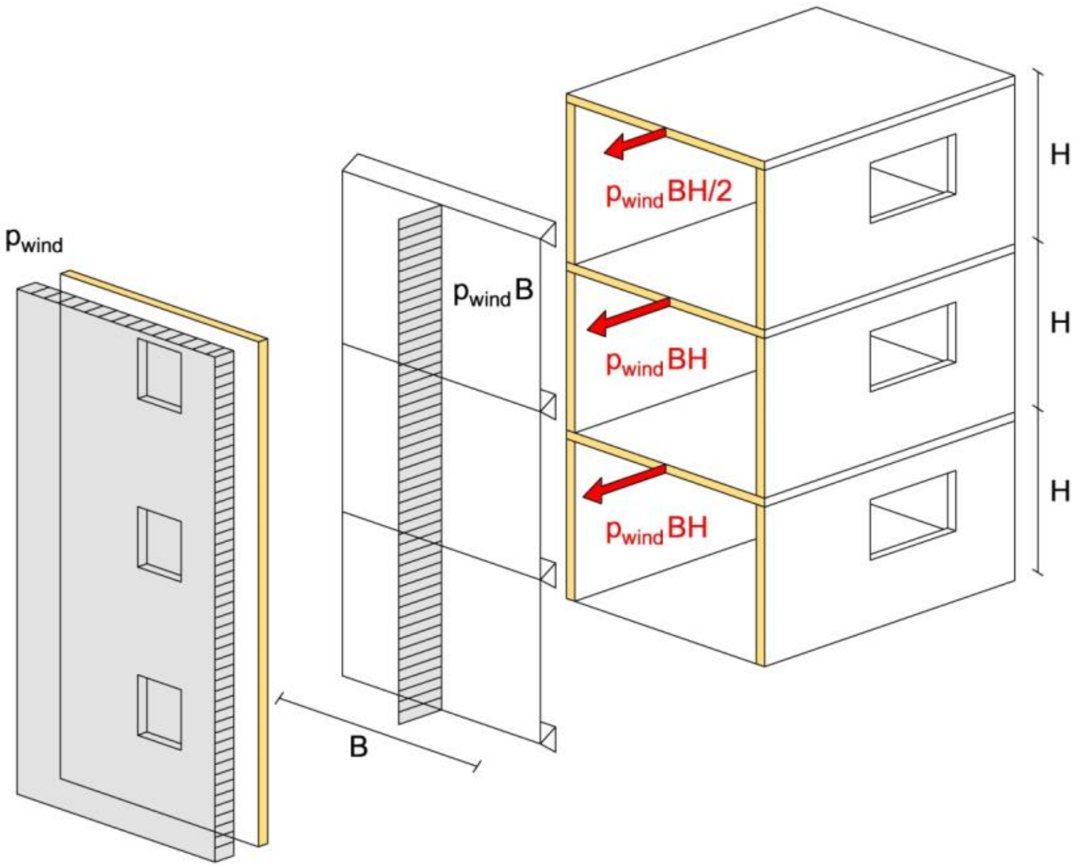
Organismo di Certificazione ed Ispezione sui Prodotti da Costruzione ai sensi dell'art.8 del D.P.R. 246/93 (notifica Commissione Europea n. 0969)
Organismo di Benestare Tecnico Europeo (membro EOTA) ai sensi dell'art.5 del D.P.R. n.246/93.
Organismo di Benestare Tecnico Nazionale ai sensi del D.M. 14.01.2008

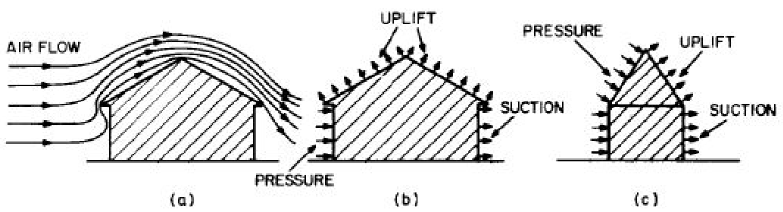
Via Nomentana 2 – 00161 Roma
Tel. 06.4412.4101, Fax 06.4412.7383
www.cslp.it

DESIGN TIMBER BUILDINGS FOR LATERAL LOADS

HORIZONTAL FORCES: WIND

Wind pressure on a building surface depends primarily on its **velocity** and of the shape and **slope** of the surface. They are picked up by surface members (**pressure or suction**) which transfer them to the lateral stability devices.



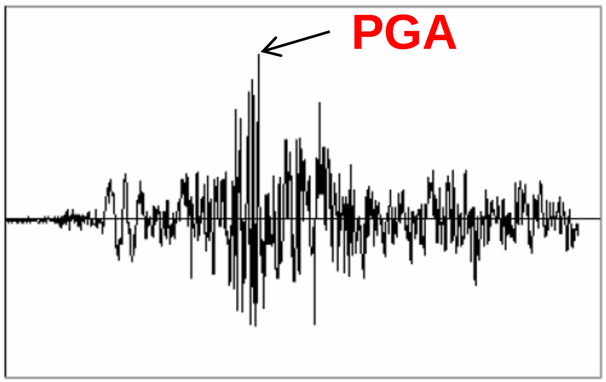


A primary role is played by roof and horizontal diaphragms which, beside resisting to vertical forces, transfer lateral forces to **shear walls**, cross brace or frames.

HORIZONTAL FORCES: EARTHQUAKE

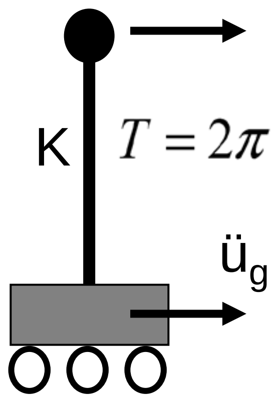
The **seismic loads** on the structures during an earthquake are due to **internal inertia** that results from **ground accelerations** to which the mass of the system is subjected. They depend on the intensity of the ground motion (**PGA**), on the dynamic properties of the buildings (e.g. mode shapes and **periods**), on the **mass** of the components of the buildings.

(accelerogram)



GROUND MOTION $\ddot{u}_g$

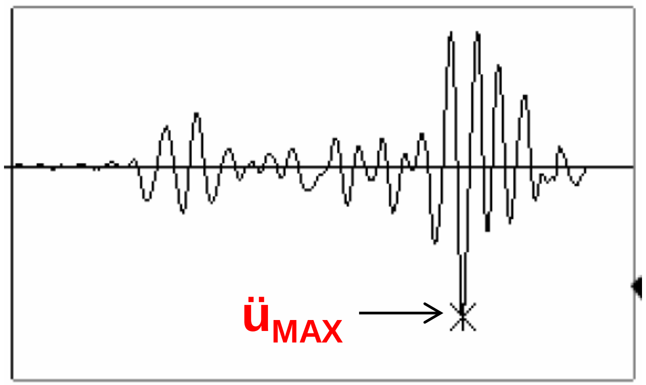
M $\ddot{u}$



$$T = 2\pi \cdot \sqrt{\frac{M}{K}}$$

1 SDOF

(acceleration response)

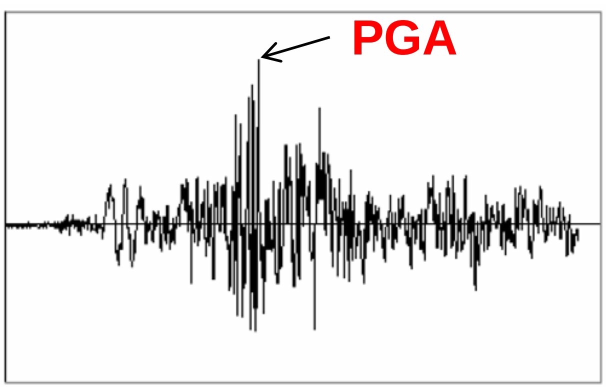


$M\ddot{u}_{MAX}$ MAXIMUM VALUE OF ELASTIC RESPONCE

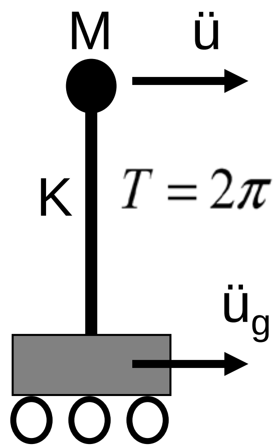
For convenience in design, an earthquake is translated into an **equivalent static load** acting horizontally on the structures

HORIZONTAL FORCES: EARTHQUAKE

The seismic loads on the structures during an earthquake are due to internal inertia that results from ground accelerations to which the mass of the system is subjected. They depend on the intensity of the ground motion (**PGA**), on the dynamic properties of the buildings (e.g. mode shapes and **periods**), on the **mass** of the components of the buildings.

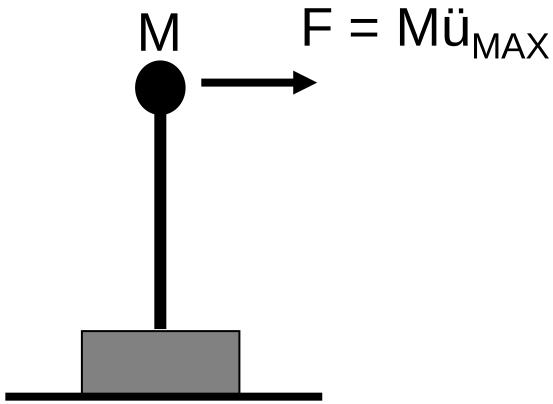


GROUND MOTION $\ddot{u}_g$



$$T = 2\pi \cdot \sqrt{\frac{M}{K}}$$

1 SDOF

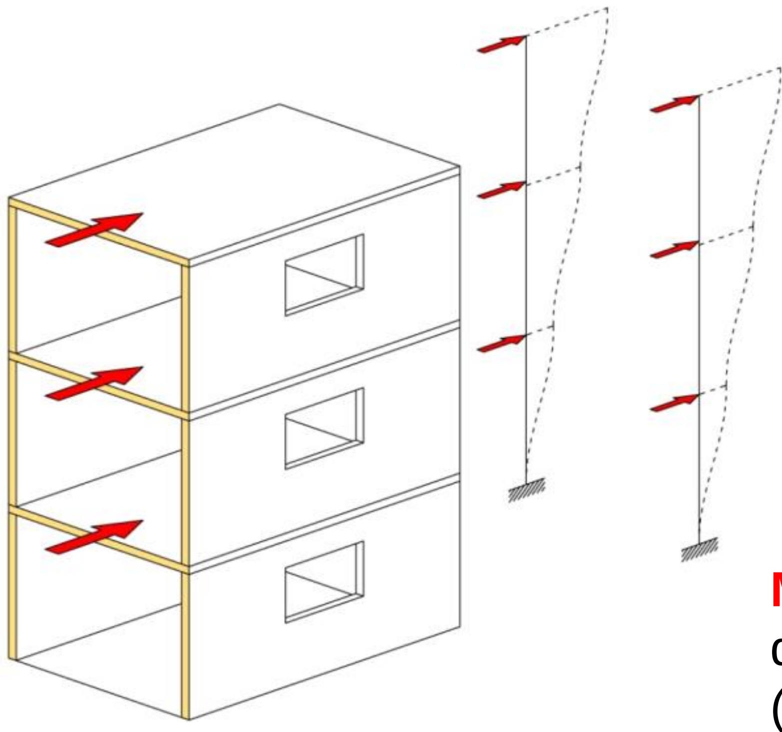


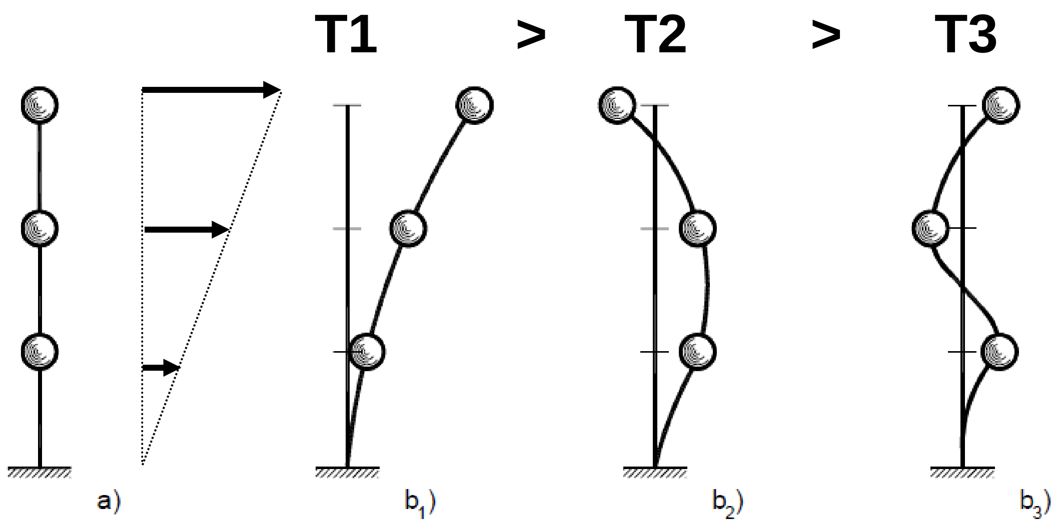
$M\ddot{u}_{MAX}$ MAXIMUM VALUE OF ELASTIC RESPONCE

For convenience in design, an earthquake is translated into an **equivalent static load** acting horizontally on the structures.

HORIZONTAL FORCES: EARTHQUAKE

The **seismic loads** have a slight different character than wind forces, as they are usually concentrated at high-mass areas (e.g. roof and floors), but they can similarly represented by a **series of forces acting laterally on the primary structure**.

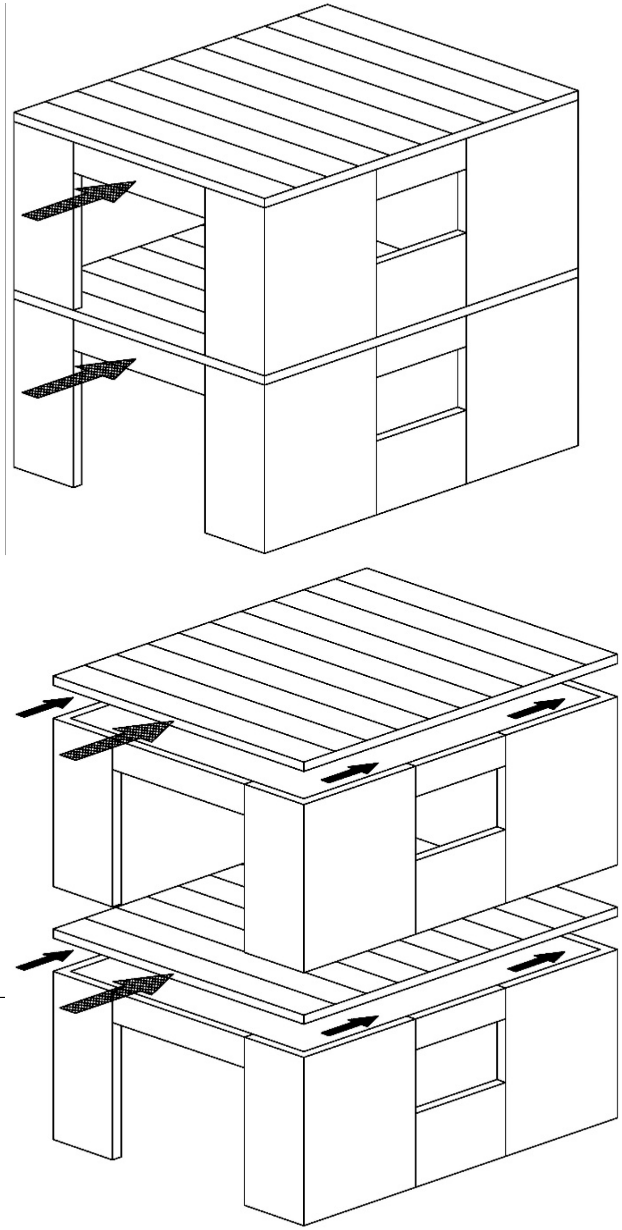




Multi-storey buildings can be modelled as a discrete **multi degree of freedom system** (MDOF), with the mass associated to the floors, and the structural vertical elements (e.g. shear walls) associated to the elastic reaction.

STRUCTURAL CONNECTION SYSTEM IN CLT BUILDINGS:
ROLE OF THE STRUCTURAL JOINTS

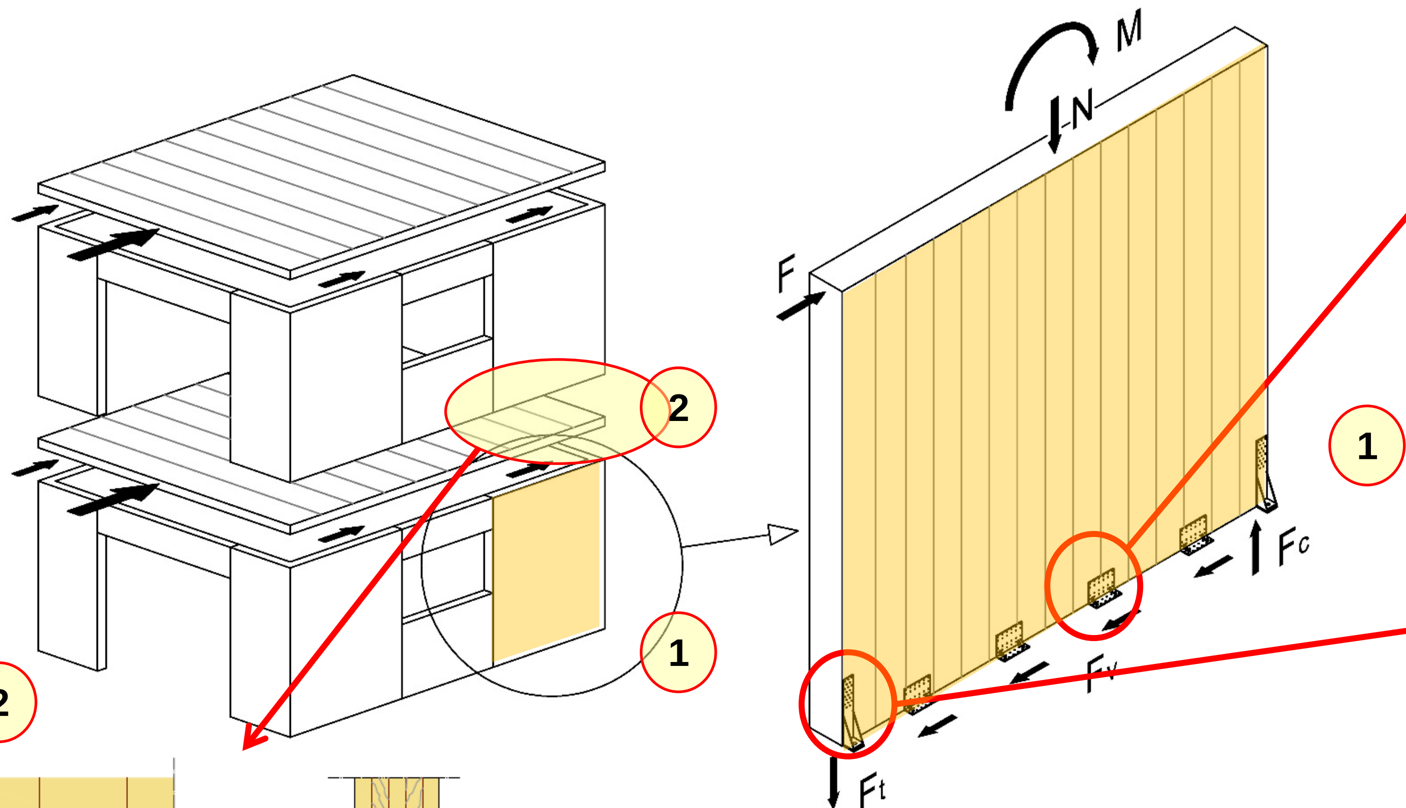
prefabricated "platform" system;



SEVENTH FRAMEWORK PROGRAMME
LNEC
UNITRENTO
TU Graz
SERIES

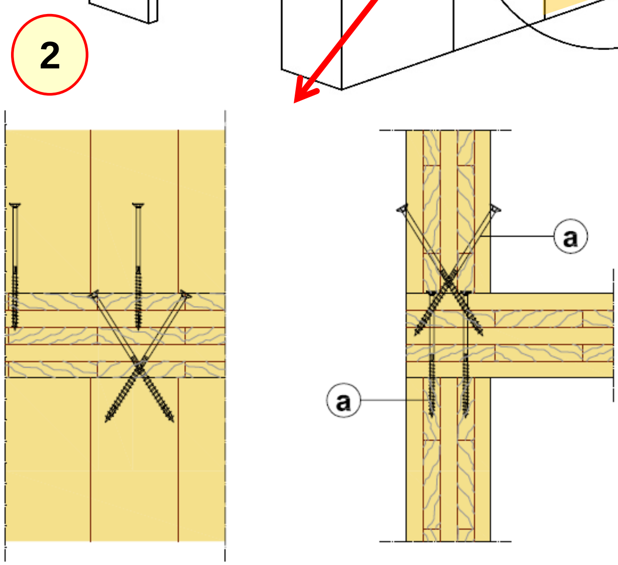
Seismic performance of multi-storey timber buildings

STRUCTURAL CONNECTION SYSTEM IN CLT BUILDINGS:
JOINT BETWEEN VERTICAL PANELS AND HORIZONTAL DIAPHRAGMS



ANGLE BRACKETS

HOLD-DOWN



Transmission of the horizontal forces:

- Between floor and wall;
- Between upper and lower wall;

IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: COMPONENTS FOR RESISTANCE TO LATERAL LOADS

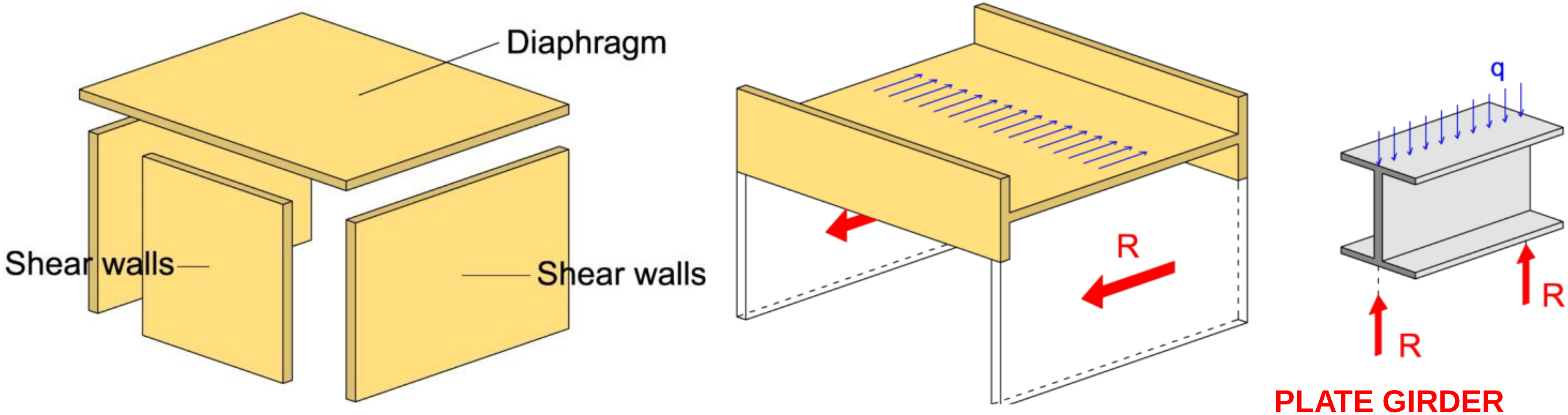
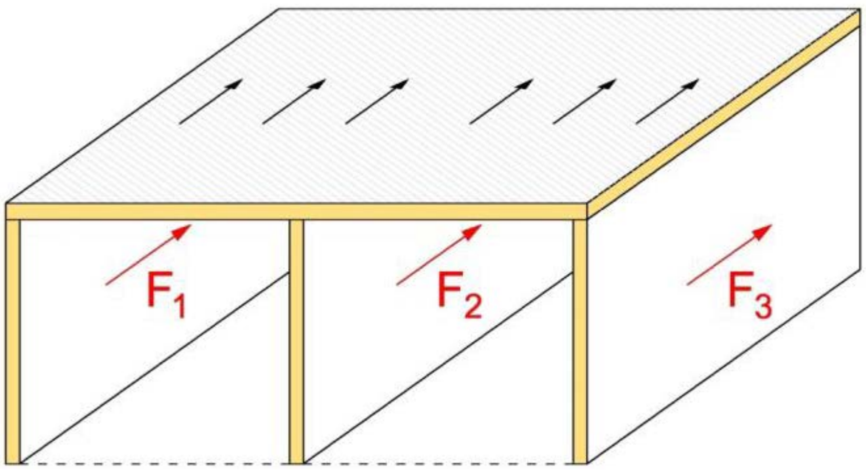


PLATE GIRDER MODEL ANALOGY

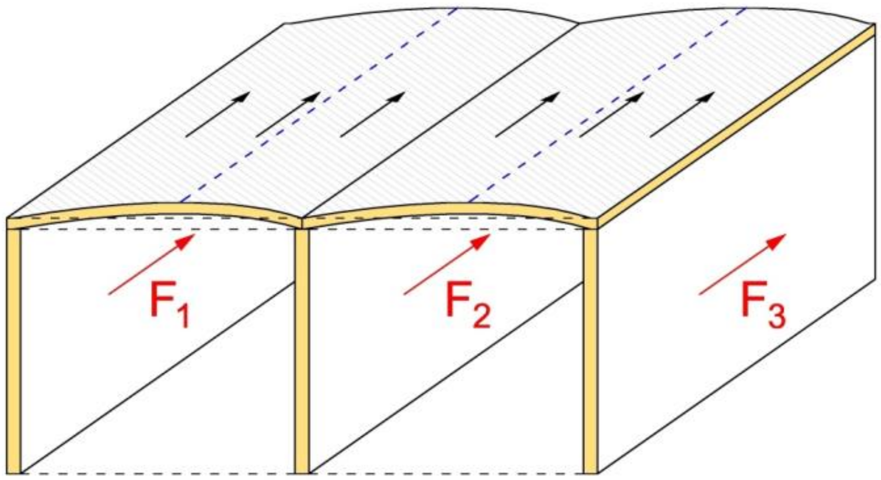
vertical plane: **shear walls**, braced frames, and moment resisting frames

horizontal plane **diaphragms** (floor and roof plans) of the building, or horizontal trusses

IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: ROLE OF THE DIAPHRAGMS STIFFNESS



RIGID DIAPHRAGM



FLEXIBLE DIAPHRAGM

STIFFNESS DEPENDS ON

- size of the diaphragms;
- type of material;
- relative stiffness betw. walls and diaphragms

The flexibility of the diaphragm, relative to the shear walls whose forces it is transmitting, also has a major influence on the nature and magnitude of those forces.

IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: ROLE OF THE DIAPHRAGMS STIFFNESS

LATERAL FORCE DISTRIBUTION RULES

DIAPHRAGM	WALLS TAKE LOADS
FLEXIBLE	according to **tributary areas**
RIGID	according to **the relative stiffness**
SEMI-RIGID	modelling the semi-rigid behavior with shell elements or diagonal struts elements

ALTERNATIVE ENVELOP APPROACH:

- the **highest forces** from **rigid** and **flexible** assumptions can be used as a conservative approach avoiding a numerical modelling of the diaphragms behavior

IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: DEFINITION OF RIGID DIAPHRAGMS ACCORDING TO STANDARD

ASCE 7 (2010) - ASCE 41-06

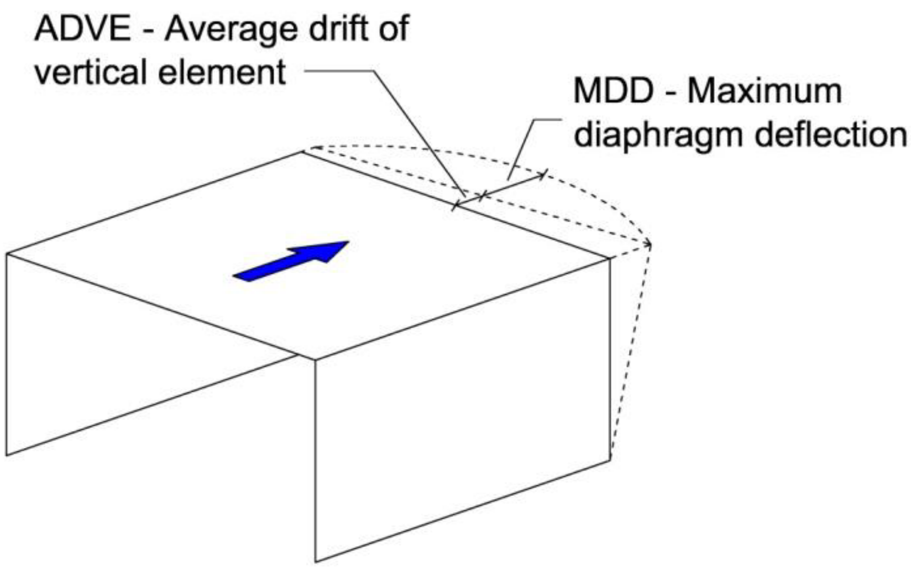
Displacement condition	DIAPHRAGM STIFFNESS
MDD > 2·ADVE	flexible
MDD < 0,5·ADVE	rigid
0,5·ADVE < MDD < 2·ADVE	semi-rigid

EN 1998 – Italian code 2008

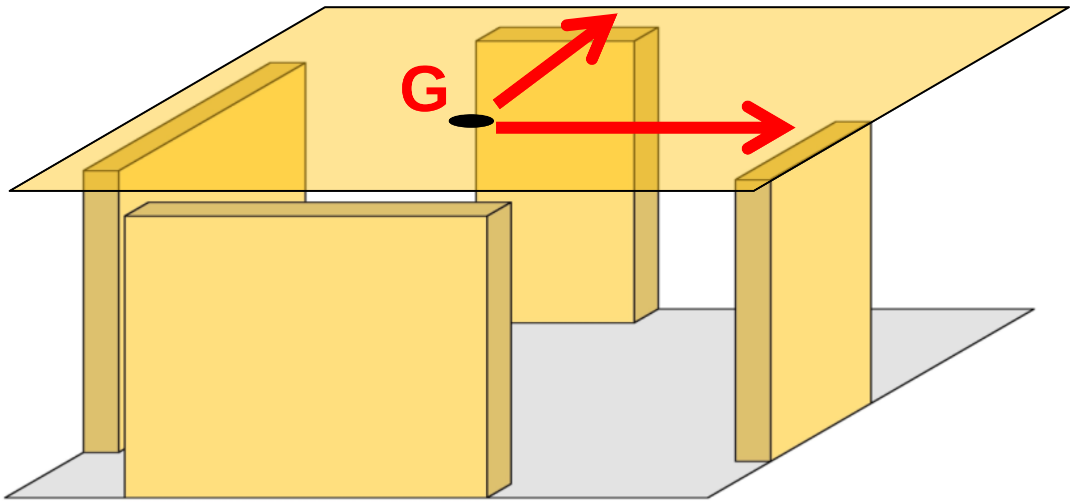
Displacement condition	DIAPHRAGM STIFFNESS
MDD < 0,1·ADVE	rigid

ASCE standards provide formulae for determining the deflections of diaphragms and shear walls constructed with wood technology

Eurocode 8 and Italian code 2008 give more severe rule for a rigid diaphragm



IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: LATERAL FORCE DISTRIBUTION IN A ONE-STOREY BUILDING



A ONE-STOREY BUILDING

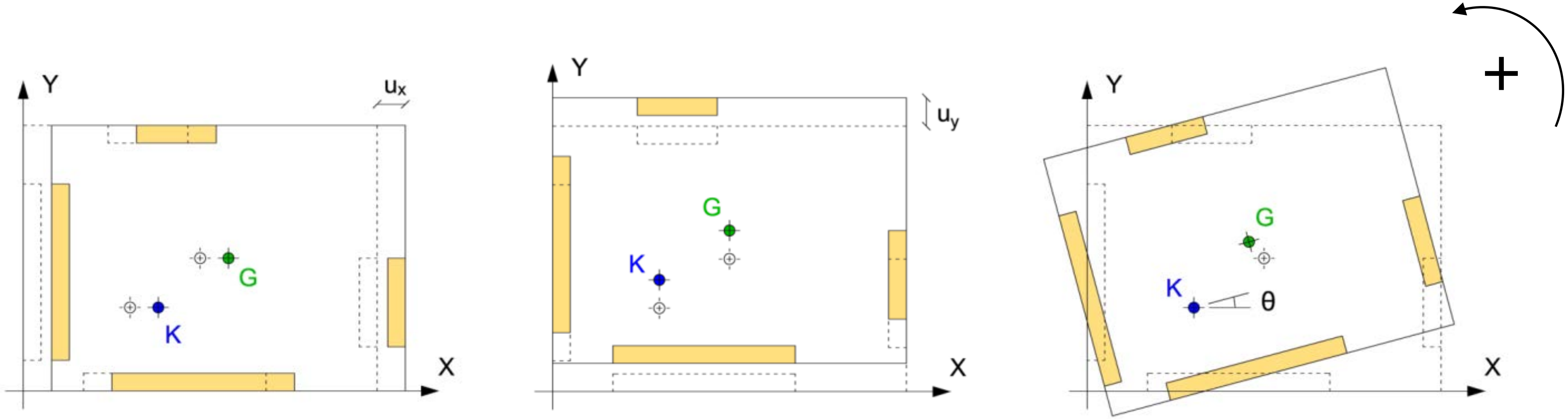
The horizontal diaphragm can be considered **infinitely rigid**

Mass is considered evenly distributed, therefore the center of mass can be located approximately in the geometric center of the floor (**centroid**)

Storey shear and Storey Torsional Moment is distributed to each wall **according to relative stiffnesses**

Determine location of **center of rigidity** and **center of force** to determine any eccentricity and the storey torsional moment

IN-PLANE DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: LATERAL FORCE DISTRIBUTION IN A ONE-STOREY BUILDING



If we take into account the contribution of the three DOF rigid body motion we obtain the lateral force distribution rules for an hyperstatic shear wall one-storey building

relative rigidity rule

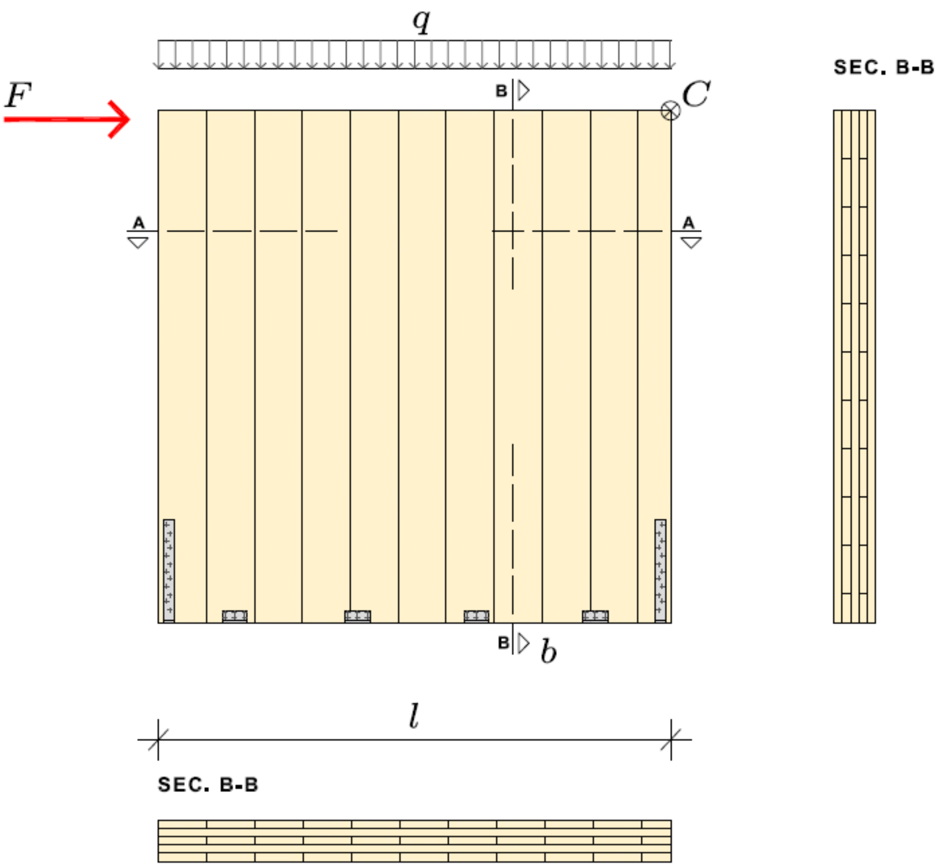
$$F_{x,i} = \frac{k_{x,i}}{\sum k_{x,i}} \cdot F_x - \frac{k_{x,i} \cdot y_i \cdot M_T}{k_{\theta,tot}}$$

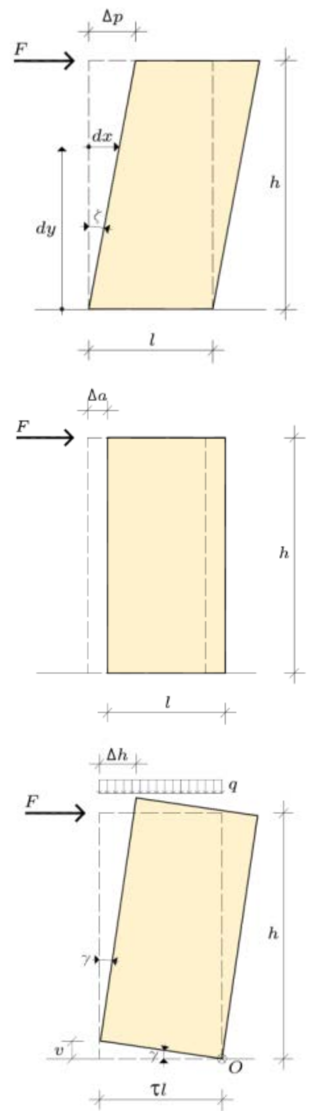
$$F_{y,i} = \frac{k_{y,i}}{\sum k_{y,i}} \cdot F_y + \frac{k_{y,i} \cdot x_i \cdot M_T}{k_{\theta,tot}}$$

UNITRENTO

ELASTIC HORIZONTAL DISPLACEMENT OF A TIMBER SHEAR WALL:
DEFORMATION CONTRIBUTION FOR A CLT- WALL

3 main contributions





1.Shear deformation

accounts for the **shear deformation** of the CLT panel (the bending deformation can be neglected for ratio h/l near to 1).

2.Rigid-body translation

accounts for the shear deformation of **angle-brackets or screws** used along the length of the wall

3.Rigid-body rotation

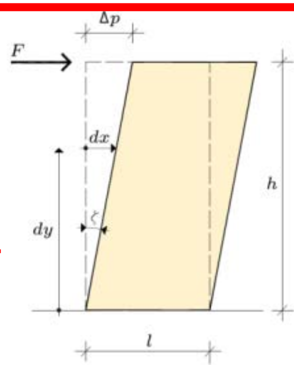
accounts for the tensile deformation of the **hold-downs** used at each corner of the wall

UNITRENTO

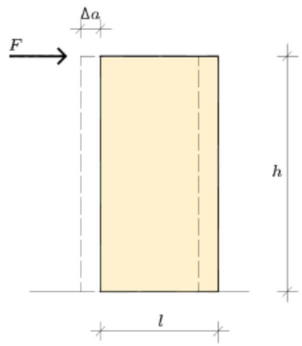
ELASTIC HORIZONTAL DISPLACEMENT OF A TIMBER SHEAR WALL:
DEFORMATION CONTRIBUTION FOR A CLT- WALL

$$\Delta = \boxed{\frac{F \cdot h}{l} \cdot \frac{1}{G_{CLT} \cdot t_{CLT}}} + \boxed{\frac{F \cdot i_a}{k_a \cdot l}} + \boxed{\left[\frac{h}{\tau \cdot l \cdot k_h} \cdot \left(\frac{F \cdot h}{\tau \cdot l} - \frac{q \cdot l}{2}\right)\right]}$$

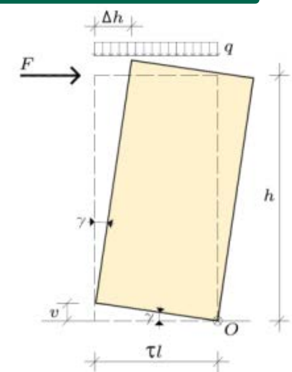
1. Δ_{CLT}



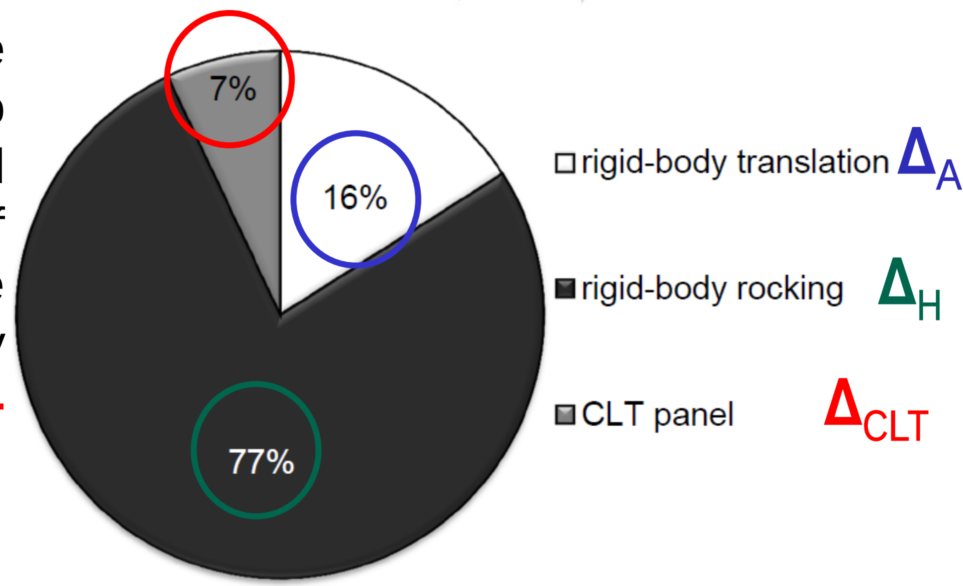
2. Δ_A



3. Δ_H



The average percentage of deformations due to **each single contribution**, with regards to a typical CLT wall configuration, are obtained considering the **most typical range** of mechanical and geometrical properties of the shear wall structural components commonly adopted in the European market. **The hold-down in tension case** is considered.



(Tomasi et al. 2015)

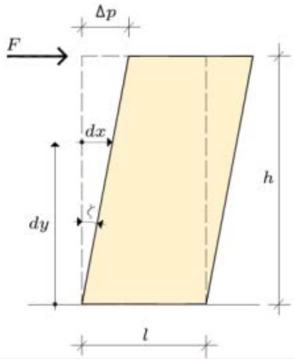
UNITRENTO

THE STIFFNESS OF A CLT SHEAR WALL:
SECOND REGIME -> HOLD-DOWN ACTIVATED

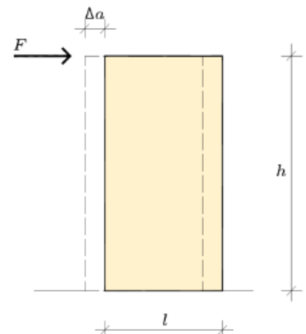
$$\Delta = \boxed{\frac{F}{K_{CLT}}} + \boxed{\frac{F}{K_A}} + \boxed{\frac{F}{K_H}} - \frac{N \cdot h}{\tau \cdot l \cdot k_h}$$

N is the half part of the vertical load

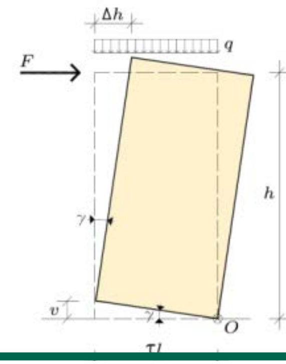
1. K_{CLT}



2. K_A



3. K_H



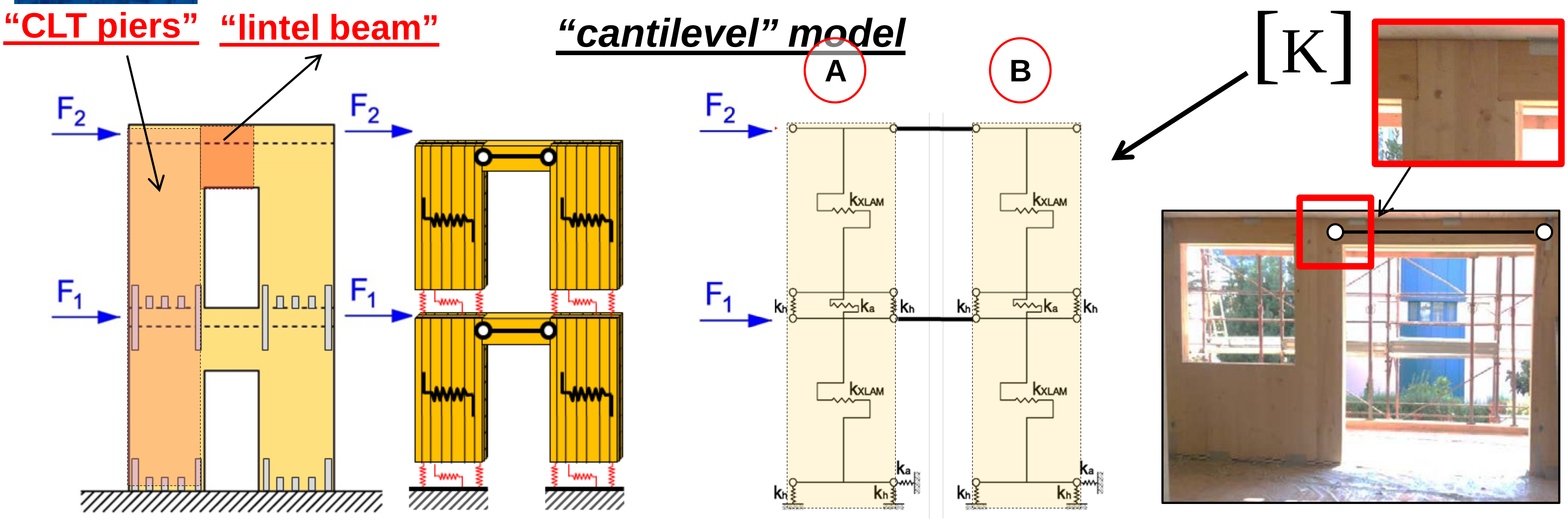
$$\boxed{K_{CLT} = \frac{G_{CLT} \cdot t_{CLT} \cdot l}{h}}$$

$$\boxed{K_A = \frac{k_a \cdot l}{i_a}}$$

$$\boxed{K_H = \frac{k_h \cdot \tau^2 \cdot l^2}{h^2}}$$

- The stiffness contribution due to “rocking” is **not linearly proportional** with the length of the wall;
- **Vertical load** reduces the horizontal displacement and is crucial to the **activation of the spring** associated to the hold-down (“rocking mechanism”)

DISTRIBUTION OF THE HORIZONTAL FORCES ON THE SHEAR WALLS: MODELS FOR MULTY-STOREY BUILDING



The distribution of the force between element is based on a **cantilever model.** The **lintel beams** are considered as **hinged beam** axially rigid, not able to transmit flexural moment between adjacent CLT piers.

Each "cantilever" is modelled considering all the displacement contribution previously discussed (shear, translation and rocking displacement).
The formulation of the **matrix of stiffness parameters** allow to determine the force distribution between CLT piers.

Layout presentation

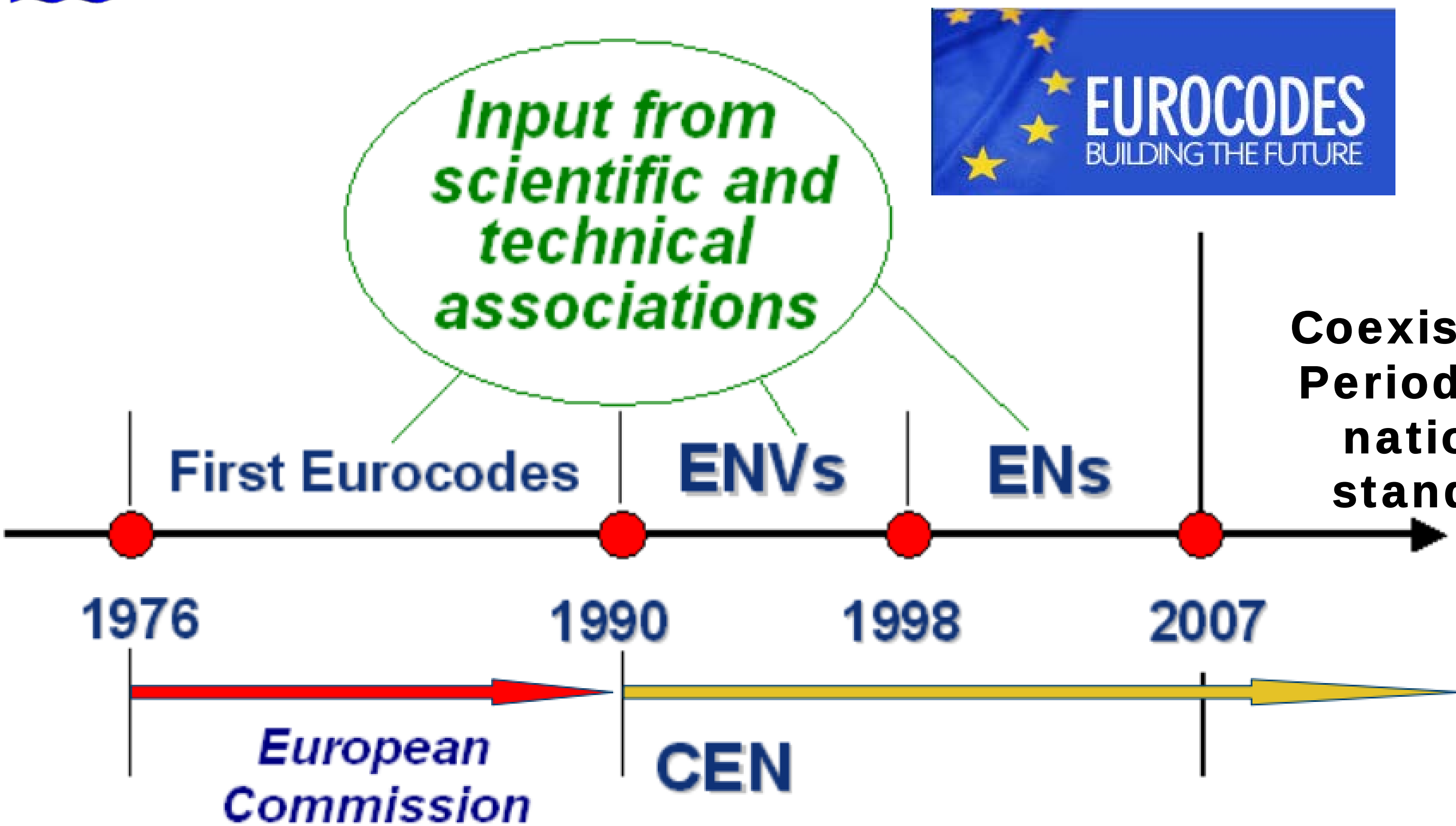
1. Introduction
2. Cross laminated timber, the product and the constructive system
3. Comparison between the CSAO86 and Eurocode 5
4. Design example

Background of the Eurocode programme

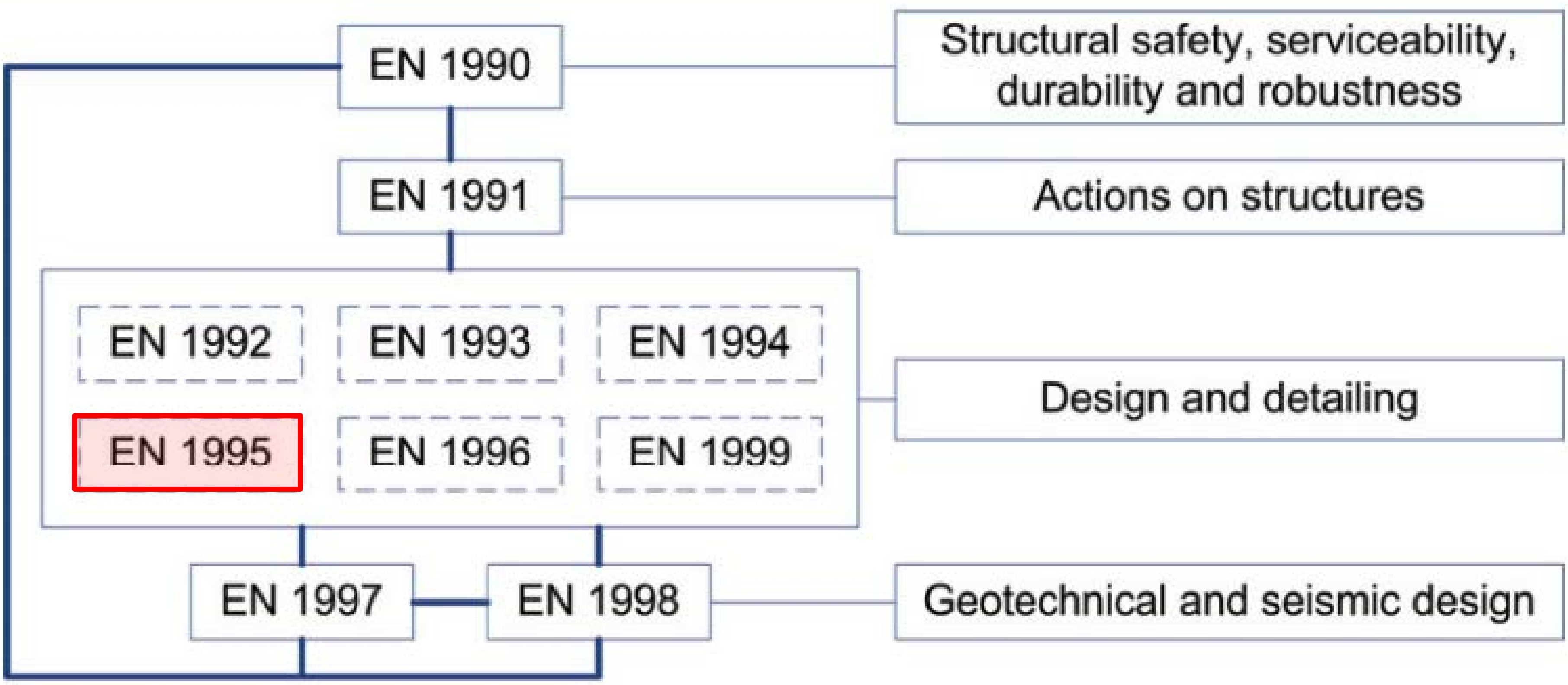
In **1975** the Commission of the European Community decided of an action programme in the field of construction. The objective was the elimination of technical obstacles to trade and the **harmonisation of technical specifications**.

Within this programme, the Commission took the initiative to establish a set of **harmonised technical rules for the design of construction** works which would serve, in a first stage as an alternative to the national rules, and ultimately would replace them.

Background of the Eurocode programme



Structure of the Eurocodes



10 Eurocodes – 58 Parts – 5320 pages

Structure of the Eurocodes

IMPLEMENTATION OF THE EN EUROCODES

EUROPEAN LEVEL

EN 199n-p

Main text

Normative Annexes

Informative Annexes

Choices Nationally Determined Parameters (NDPs)

Decisions

EN 1995 – 1 – 1

Timber Structures

General Rules

Buildings

MEMBER STATE

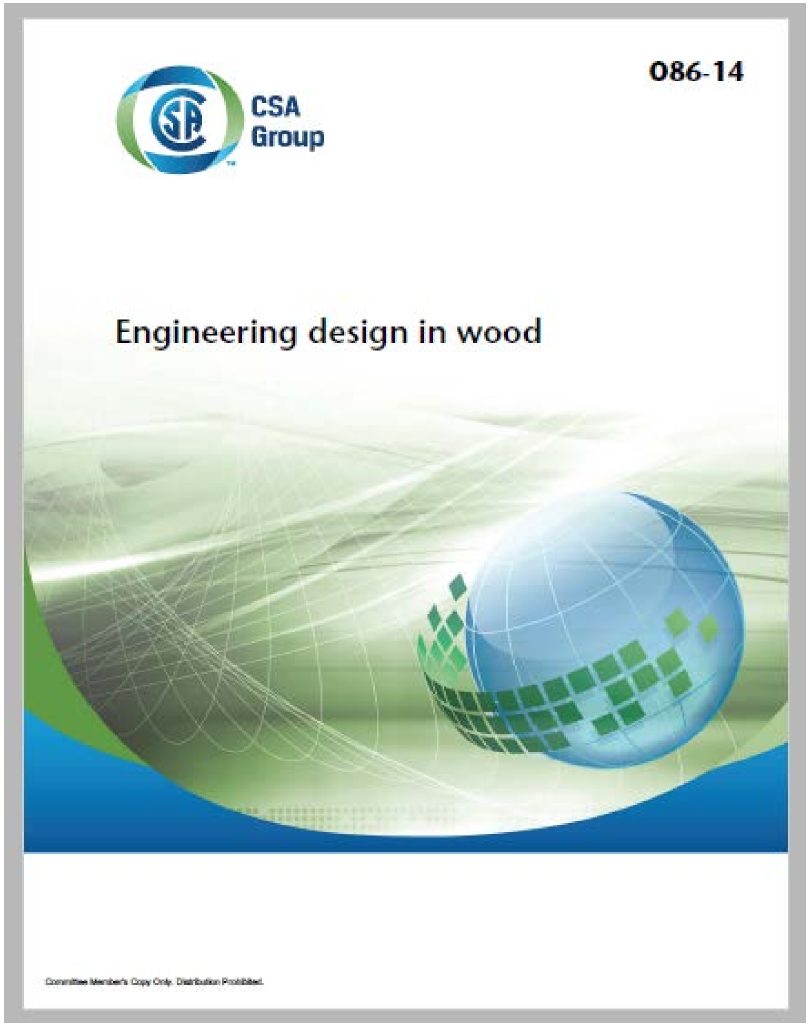
Transformation into a National Standard (« NS » EN 199n-p)

+

National Annex (National Standard)

Project specification

Design specification for timber elements and buildings



NORMA EUROPEA

Eurocodice 5
Progettazione delle strutture di legno
Parte 1-1: Regole generali - Regole comuni e regole per gli edifici

UNI EN 1995-1-1

FEBBRAIO 2005

Versione italiana dell'ottobre 2005

Eurocode 5
Design of timber structures
Part 1-1: General - Common rules and rules for buildings

La norma si applica alle strutture di legno (massiccio, laminato, ecc.) ed ai pannelli a base di legno, uniti mediante adesivi o collegamenti meccanici, e fornisce i criteri di progettazione strutturale compresa la durabilità.

EN 1995 – 1 – 1

+ Italian NAD

TESTO ITALIANO

La presente norma è la versione ufficiale in lingua italiana della norma europea EN 1995-1-1 (edizione novembre 2004).

La presente norma è la revisione della UNI ENV 1995-1-1:1995.

ICS 91.010.30; 91.080.20

UNI
Ente Nazionale Italiano di Unificazione
Via Battistotti Sassi, 11B
20133 Milano, Italia

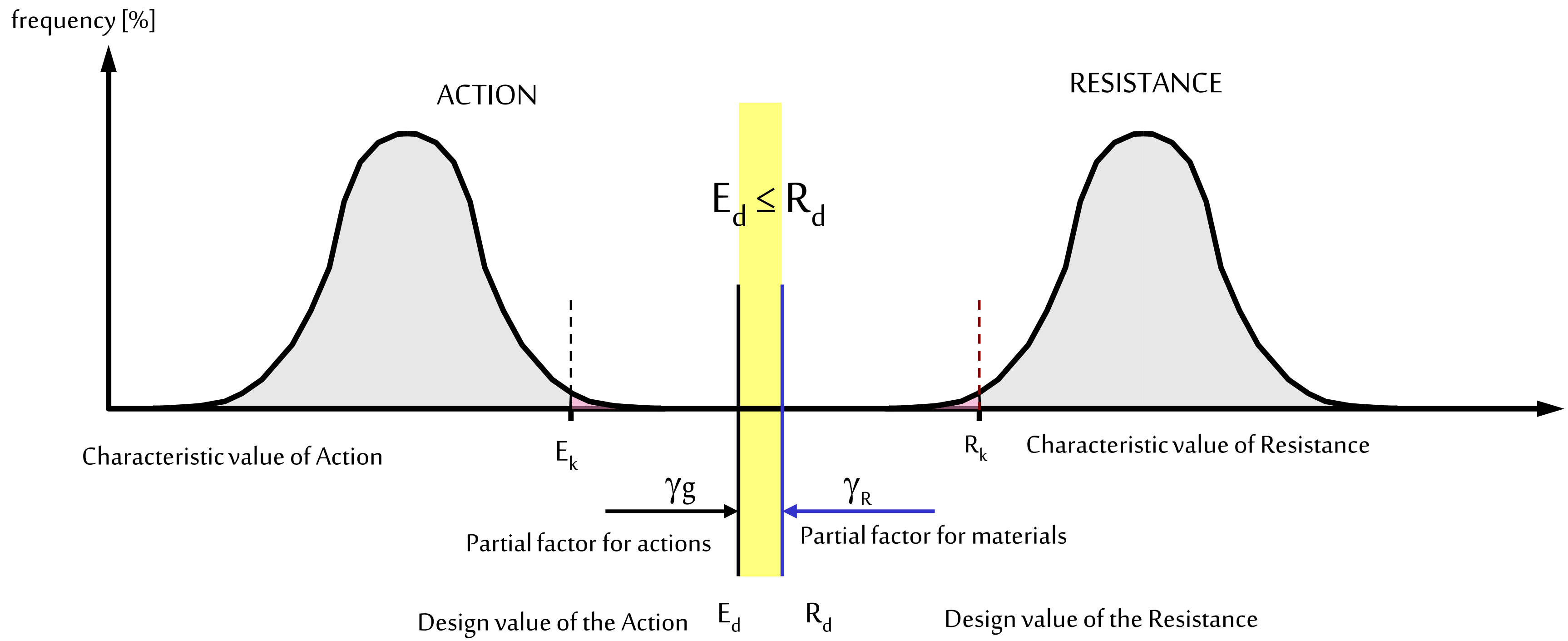
www.uni.com

Limit state Design Method

Both the Eurocode and the CSA are based on the **Limit States Design Method (LSD)**

Limit state design requires the structure to satisfy two principal criteria the **ultimate state (ULS) (equilibrium, internal failure, ground)** and the **serviceability state (SLS) (deformation, vibration, damage)**.

Limit state design – Partial Factor Method



ULS Value of Actions

RESISTANCE

$$R_d \geq E_d$$

ACTION

$$R_d = \phi R$$

- R is the nominal resistance;
- ϕ is the resistance factor;

$$R_d = {R_k}/{\gamma_m}$$

- R_k is the characteristic resistance;
- γ_m is the partial factor for material;

$$S_d = \alpha_D \mathrm{D} + \Psi\gamma\{\alpha_L L + \alpha_Q Q + \alpha_T T\}$$

- Ψ is the Combination Factor;
- γ is the Importance Factor;
- α_D is Dead Load Factor;
- α_L is Live Load Factor;
- α_Q is Earthquake Load Factor;
- α_T is Thermal Effect (Temperature) Load Factor;

$$E_d = \sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i\geq 1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{ki}$$

- G_k is the permanent action;
- $Q_{k,1}$ is the leading variable action;
- Q_k is an accompanying variable action;
- γ_G is the partial factor for permanent loading;
- γ_Q is the partial factor for variable loading
- Ψ_0 is the factor that converts a variable action into its combination rule;

ULS Value of Actions

CSA O86

verifications are carried out in terms of load carrying capacity :

$$V_F \leq V_r$$

$$X_r = \phi F_x K_z K_k$$

$F_x = f_x(K_D K_H K_{Sx} K_T)$, in which f_x is the specified strength associated to the action considered, *MPa*.

K_z is a parameter that takes in account the influence of the size of the member on its strength;

K_k is used to indicate all those parameters that depend on the geometrical properties of the member (Section modulus, Lateral stability factor, etc).

EN 1995 – 1 – 1

verifications are carried out in terms of stress:

$$\tau_d \leq f_{v,d}$$

$$X_d = \frac{k_{mod} X_k}{\gamma_m}$$

X_k is the characteristic value of a strength property;

γ_m is the partial factor for a material property;

k_{mod} is a modification factor taking into account the effect of the duration of load and moisture content.

Resistance according to Eurocode 5

The modification factor k_{mod} plays the same roule of the modification factors of K_S and K_D in the formulation proposed in the CSA Standards; some considerations:

- Load combination usually consists of different actions with different duration classes, the modification factor k_{mod} related to the variable action having the shortest duration
- It accounts both for the viscous behaviour of wood of the load as well as for the moisture content;

Italian Standard

k_{mod} values for different load-duration classes

	Permanent Action	Long- term Action	Medium-term Action	Short-term Action	Instantaneous Action
Service class 1	0.60	0.70	0.80	0.90	1.00

EN 1995-1-1
1.10

Italian standard

ULS factors	γ_M
Solid timber	1.5
Connections	1.5

Resistance of CLT elements

strength / stiffness graded of solid wood timber C24 (C16) acc. to **EN 14081** (bending), or solid timber panel acc. to **EN13353**

Characteristic value of a single layer of the cross laminated timber			layer of lamellas: timber layer based on boards according to EN 14081-1[a]	solid timber panel: timber layer based on panels according to EN 13353[b]
bending strength		$f_{m,lay,k}$	$k_{sys} \cdot f_{m,k}$	$f_{m,k}$
tension strength	parallel	$f_{t,0,lay,k}$	$k_{sys} \cdot f_{t,0,k}$	$f_{t,0,k}$
	perpendicular	$f_{t,90,lay,k}$	$k_{sys} \cdot f_{t,90,k}$	$f_{t,90,k}$
compression strength	parallel	$f_{c,0,lay,k}$	$k_{sys} \cdot f_{c,0,k}$	$f_{c,0,k}$
	perpendicular	$f_{c,90,lay,k}$	3,0	
shear strength cross plane	Shear	$f_{v,0,lay,k}$	3,5	
	Torsion	$f_{tor,lay,k}$	2,5	
shear strength in plane	Shear	$f_{v,lay,k}$	2,3	
	rolling shear strengt[h]	$f_{r,lay,k}$	0,7	
modulus of elasticity	parallel	$E_{0,lay,mean}$	$1{,}05 \cdot E_{0,mean}$	$E_{0,mean}$
	perpendicular	$E_{90,lay,mean}$	450	
shear modulus	parallel	$G_{0,lay,mean}$	$G_{0,mean}$	
rolling shear modulus [e]	perpendicular	$G_{r,lay,mean}$	65	

Ex: Austrian Annex of EC5 for CLT

SERVICE CLASS according to Eurocode 5

Inside the building, dry and warm	Outside the building but not directly exposed to rain and sun	Outside the building without protection, wet and cold

It is proposed, that the use of CLT in service class 3 is not allowed.

1

© Pictures: KLH

10-storey building
Melbourne (AUS) | 2012
CLT by KLH

2

© Pictures: BBS

Kindergarden
Innsbruck (AT) | 2008
CLT by Binderholz BS

3 *

© Pictures: AHEC

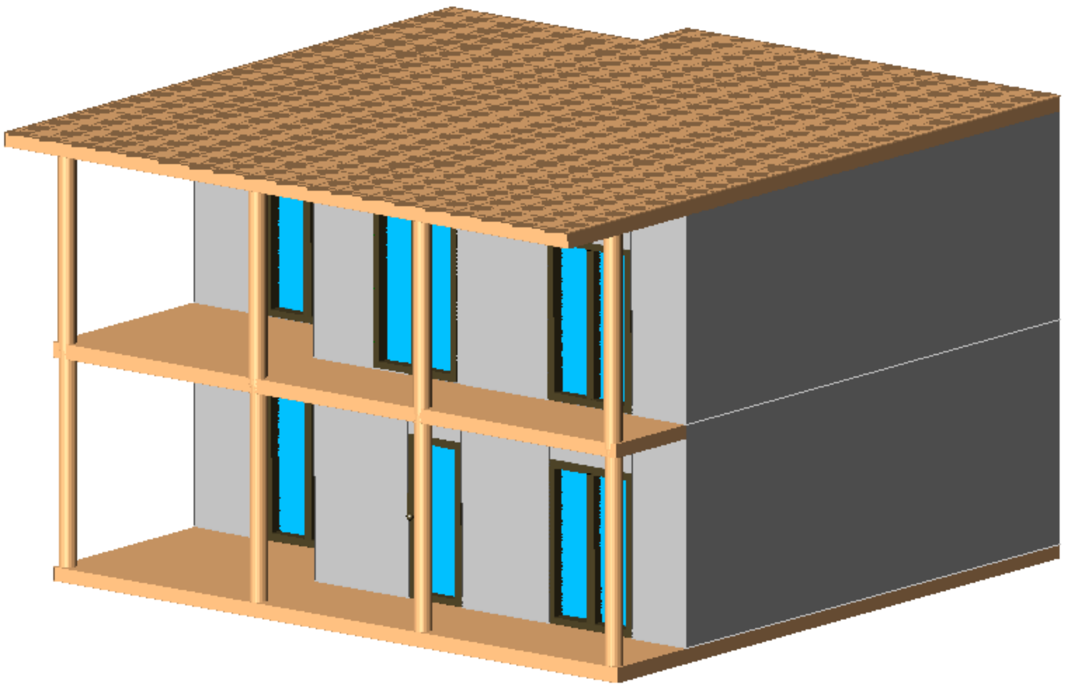
Endless Stair
London (GBR) | 2013
CLT by Imola Legno

* Temporary art installation

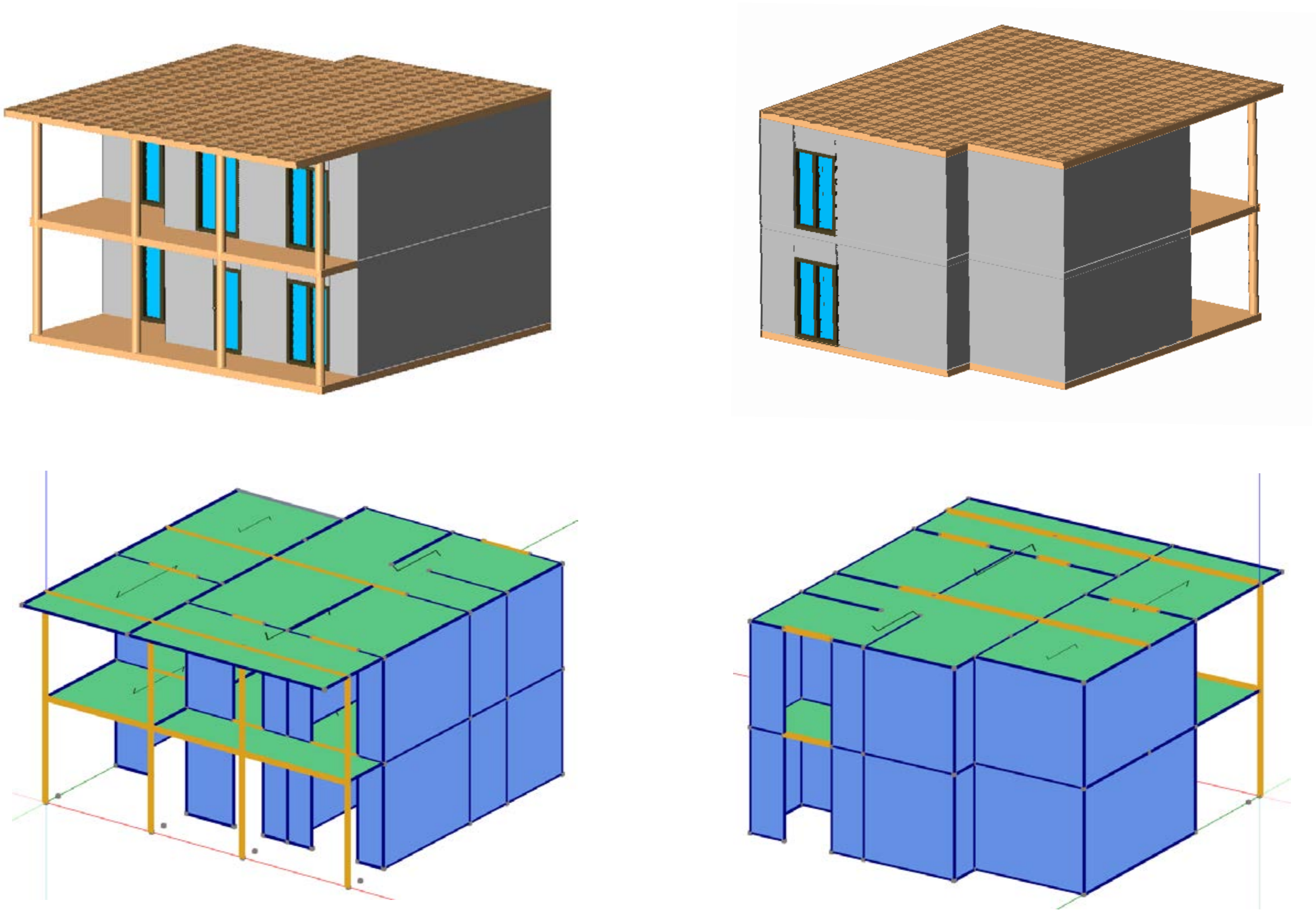
Layout presentation

1. Introduction
2. Cross laminated timber, the product and the constructive system
3. Comparison between the CSAO86 and Eurocode 5
4. Design example

THE BUILDING

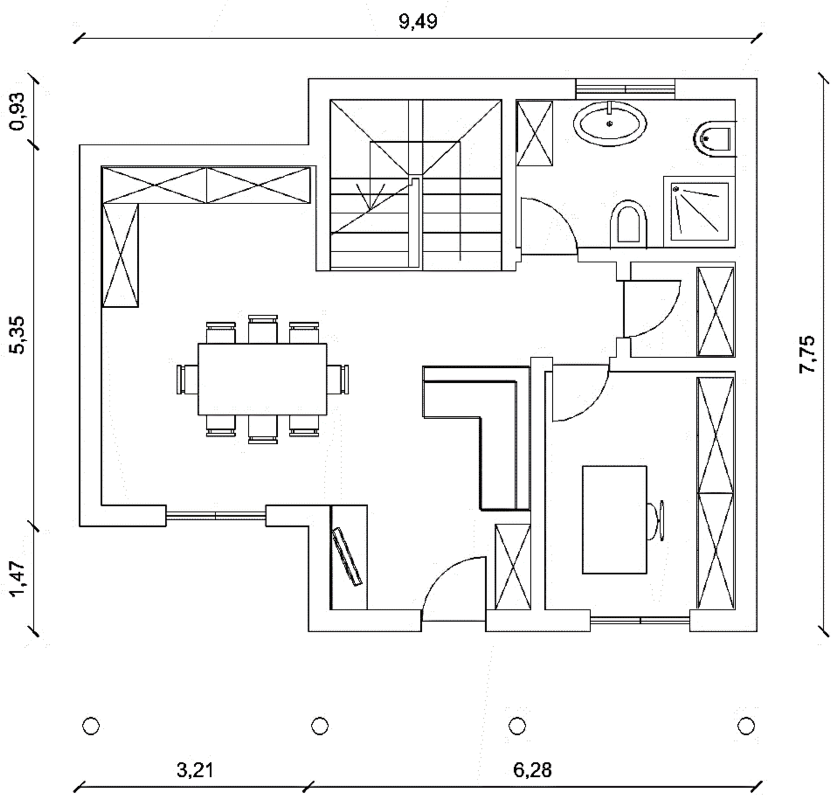


Two storey residential building

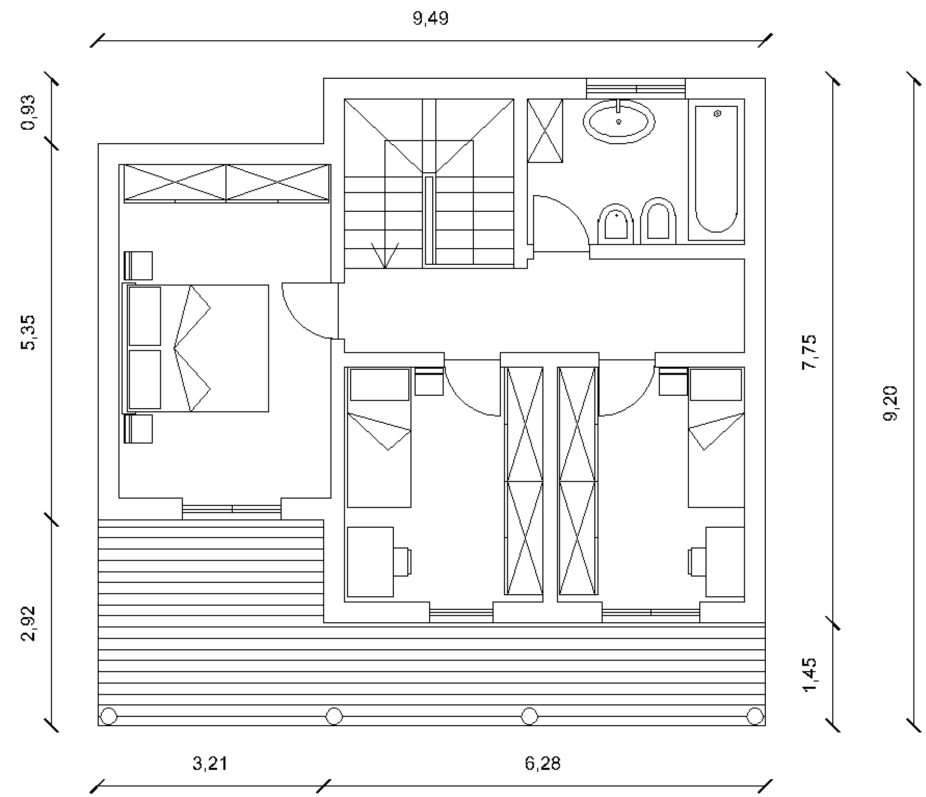


Two storey residential building

Ground floor



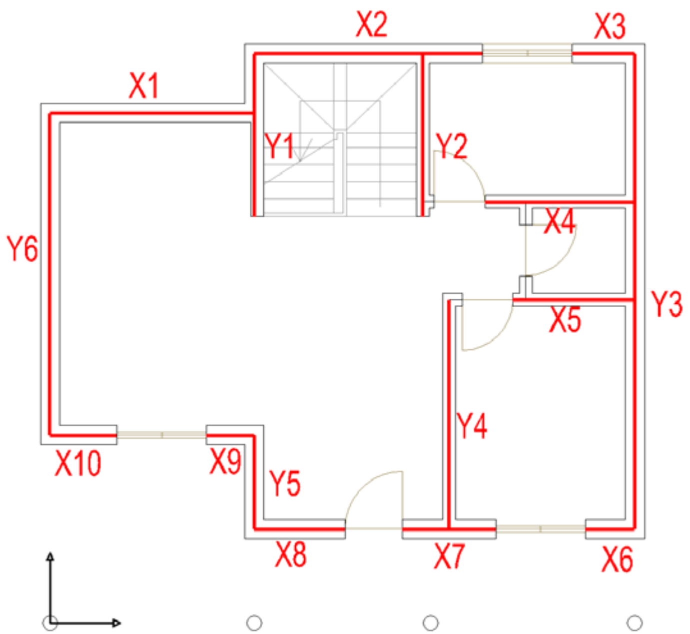
First floor



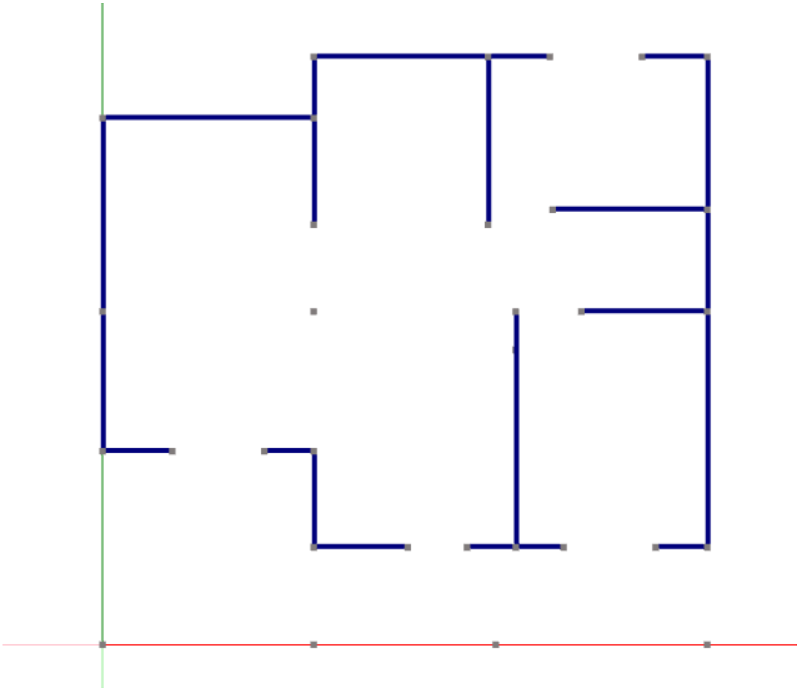
The pictures represent the plants of the ground and first floor.

Two storey residential building

Ground floor: bearing walls



Ground floor: model

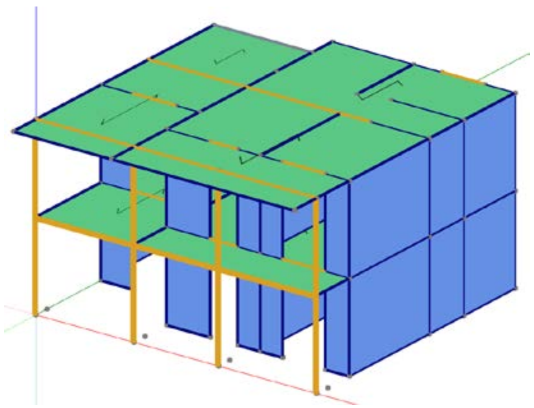


The pictures represent the timber load-bearing walls of the structure and the corrisponding model in Timber Tech Buildings

Two storey residential building

The building

The structure that we are going to analyse in this example is a two storey single-family residential building.



Loads

The loads acting on the structure are the following:

- $q_{live} = 2.00\ kN/m^2$
- $q_{snow} = 1.20\ kN/m^2$

Materials

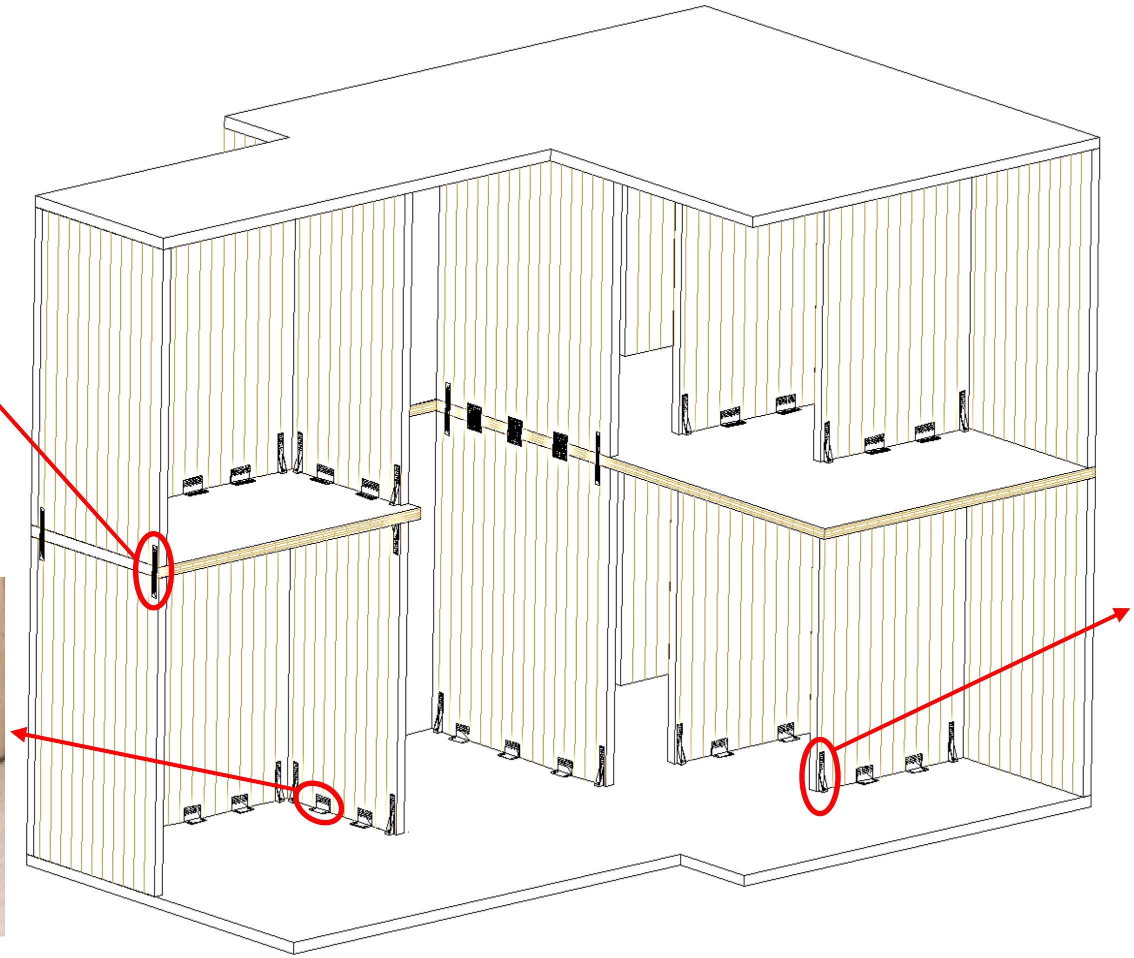
Walls and floors are built with CLT panels with the following characteristics:

- total thickness: 100 mm
- 5 layers (20 mm per layer)
- Timber grade C24

Tension and shear connections

The connections used in the project are shown in the pictures below.

Tie-down

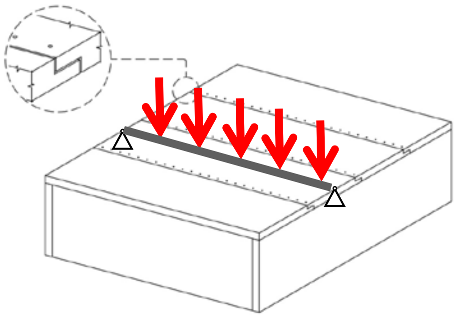


Angle bracket

Hold-down

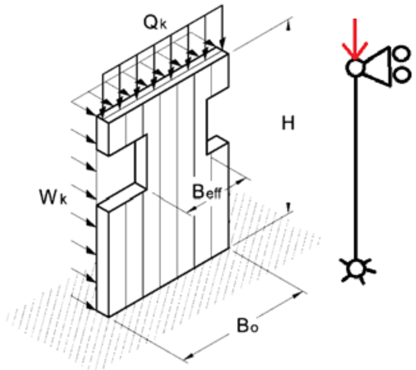
STRUCTURE VERIFICATION

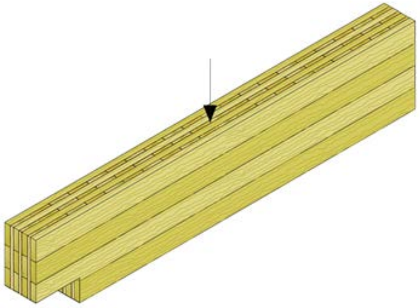
Design example



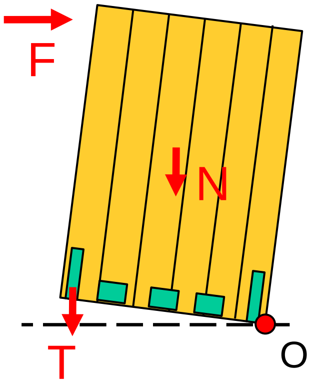
In this example we will check the structure for:

- Vertical loads:
  - CLT floors (bending, shear and compression perpendicular to the grain)
  - CLT walls (instability)
  - CLT lintels (bending, shear)



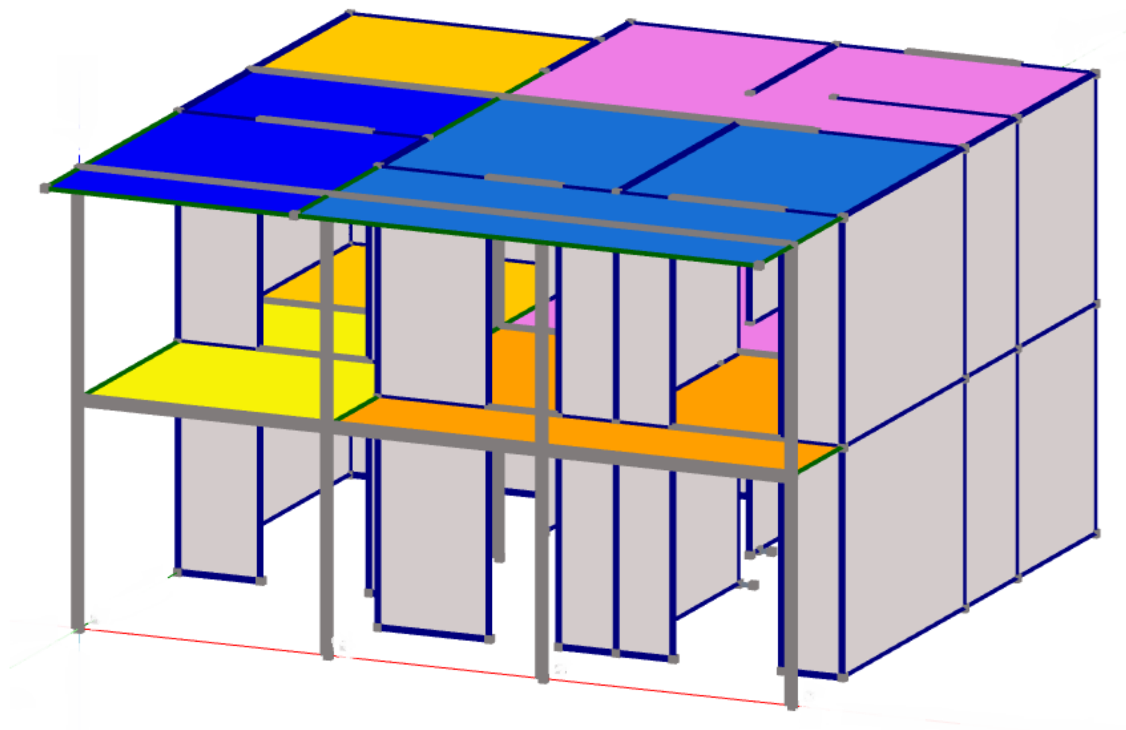


- Horizontal loads:
  - CLT walls (shear)
  - Connections (hold-down, angle brackets)



… following Eurocode 5 and ETA of the products

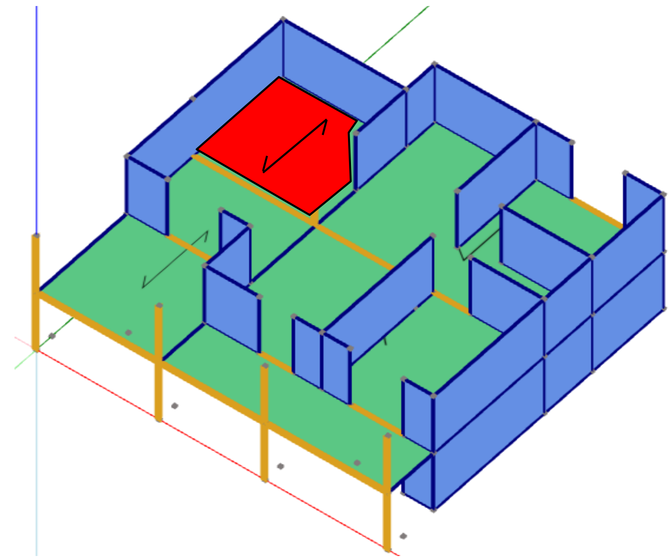
DESIGN FOR VERTICAL LOADS: FLOORS



Design for vertical loads: floors

The single span floor that we analyse is highlighted in red and presents the following characteristics:

- span l = 2.94 m
- thickness t = 100 mm
- 5-layers CLT (20 mm per layer)
- Timber grade C24



The loads acting on the floor are:

$g_{1,k}$ =	0.60	kN / m^2	Characteristic value of the self-weight
$g_{2,k}$ =	2.00	kN / m^2	Characteristic value of the permanent action
q_k =	2.00	kN / m^2	Characteristic value of the variable action

Design for vertical loads: floors

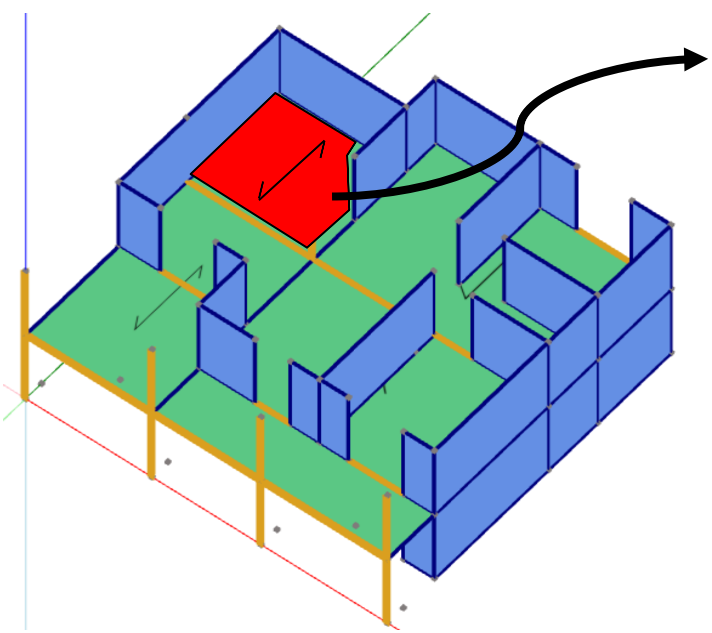
The design load (ULS combination) per square meter is

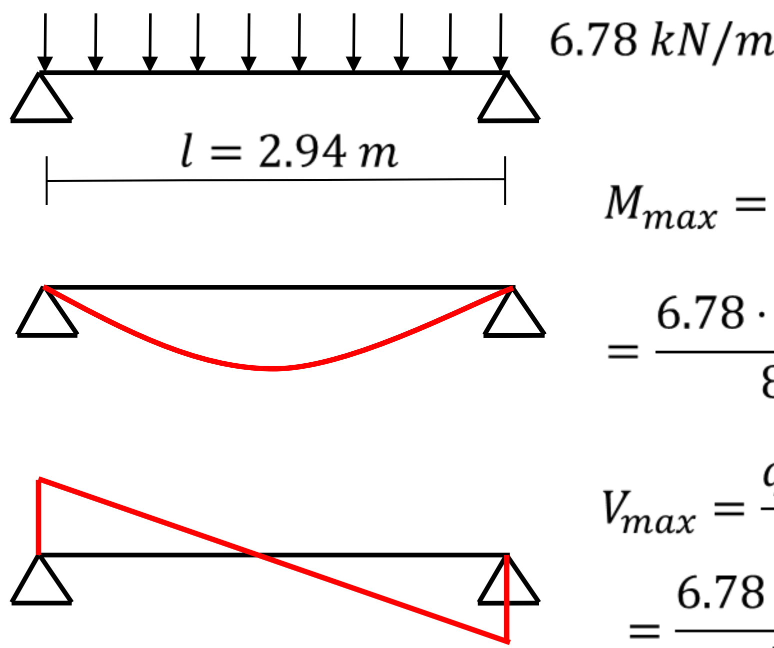
$$q_d = 1.3 \cdot g_1 + 1.5 \cdot g_2 + 1.5 \cdot q = 1.3 \cdot 0.6 + 1.5 \cdot 2 + 1.5 \cdot 2 = 6.78\ kN/m^2$$

The design load per meter acting on the floor is

$$Q_d = q_d \cdot b = 6.78 \cdot 1 = 6.78\ kN/m$$

The maximum bending moment and the shear force can be calculated as





$$M_{max} = \frac{q_d \cdot l^2}{8} =$$

$$= \frac{6.78 \cdot 2.94^2}{8} = 7.33\ kNm$$

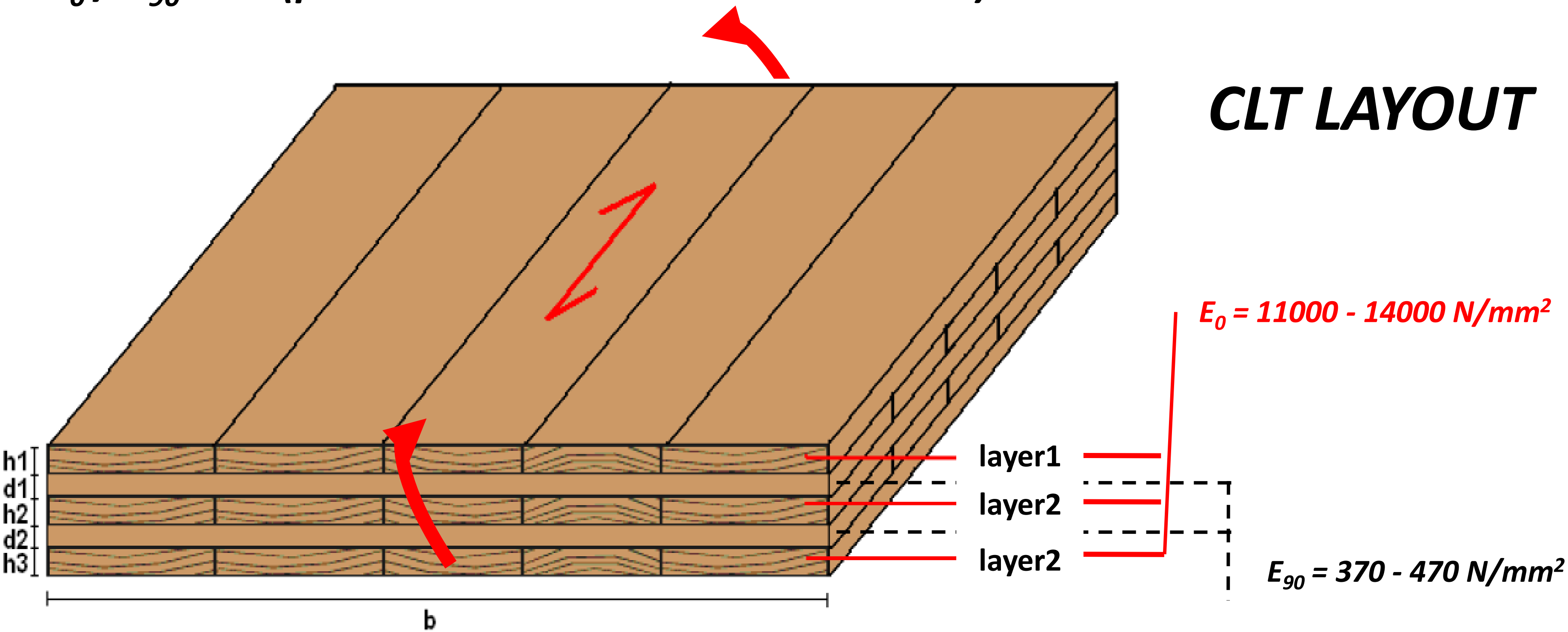
$$V_{max} = \frac{q_d \cdot l}{2} =$$

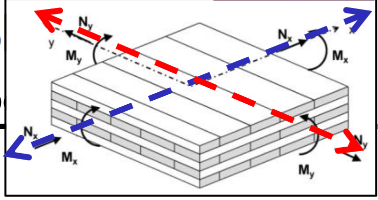
$$= \frac{6.78 \cdot 2.94}{2} = 9.97\ kN$$

LOADS OUT OF PLANE EULER-BERNOULLI

- *Cross laminated board assembled with an angle of 90°*
- *Different Elastic Modulus of the orthogonal layers along the to direction*
- *Glue line assumed infinitely rigid*
- *E_0 / E_{90} ~ 30 (possible cracks between the boards)*

CLT LAYOUT



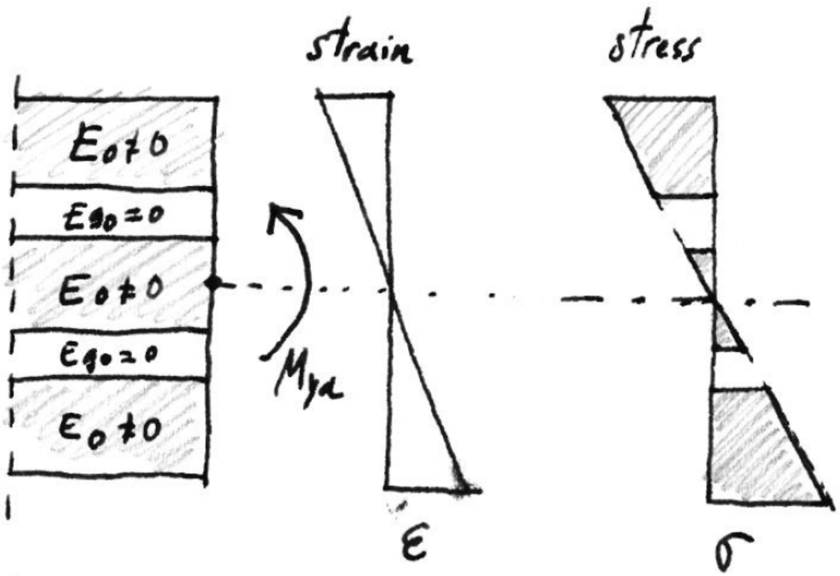


LOADS OUT OF PLANE EULER-BERNOULLI

Shear Rigidity of the glued line and transversal layers assumed infinite

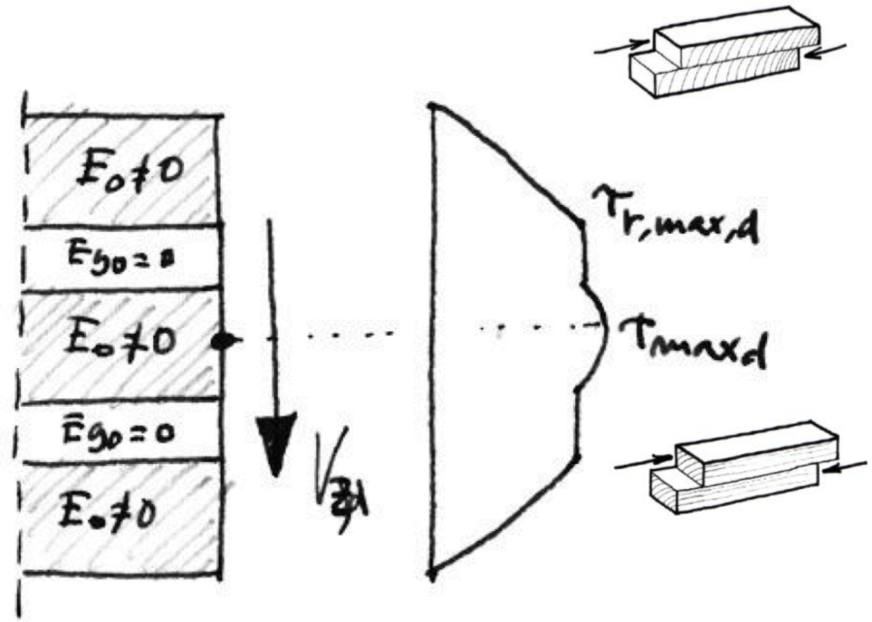
Bernoulli's hypothesis: the cross sections remain plane even during deformation

BENDING STRESS



The image part with relationship ID rId5 was not found in the file.

SHEAR STRESS

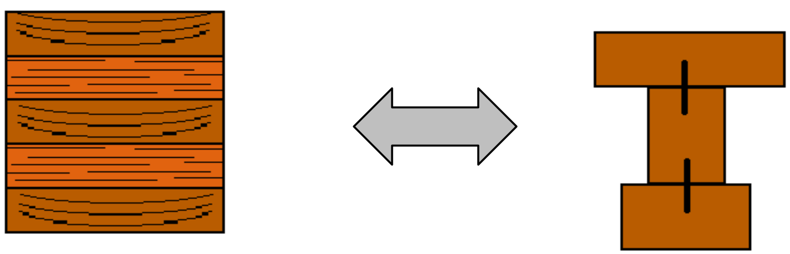


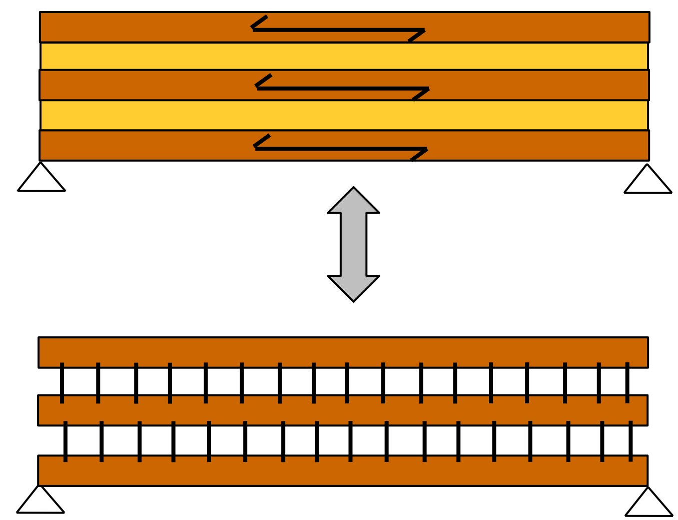
$$\tau(z_0) = \frac{V_{z,d} \cdot \int_{A_0} E(z) \cdot z \cdot dA}{K_{CLT} \cdot b(z_0)}$$

Design for vertical loads: floors

The approach adopted for determining the bending properties of the CLT elements is the so called «γ method». The longitudinal layers are modeled as beam elements connected with "equivalent" fasteners with a stiffness dependent on the rolling shear stiffness of the cross layers.

EN 1995-1-1 Annex B "Mechanically jointed beams": Möhler theory





The transverse layers prevent the longitudinal layers to slip.

Equivalent fasteners

The effective moment of inertia can be taken as:

$$J_{eff} = \sum_{i=1}^{3} J_i + \gamma_i \cdot A_i \cdot a_i^2$$

Design for vertical loads: floors

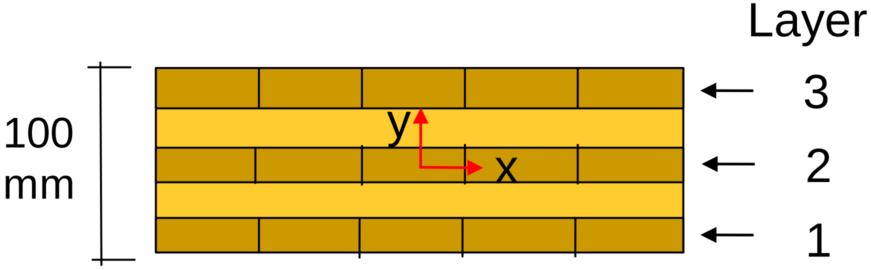
Effective inertia of the section

The bending stiffness of the floor section is

$$J_{eff} = \sum_{i=1}^{3} J_i + \gamma_i \cdot A_i \cdot a_i^2 = 6.02 \cdot 10^7\ mm^4$$ **(MOHLER)**

$$J_{eff} = \sum_{i=1}^{3} J_i + A_i \cdot a_i^2 = 6.6 \cdot 10^7\ mm^4$$ **(EULER-BERNOULLI)**

$$J = \frac{b \cdot h^3}{12} = 8.3 \cdot 10^7\ mm^4 \qquad J_{eff} = 0{,}73 \cdot J$$



The effective section modulus W_{eff} can be calculated (according to ETA) as the ratio of the effective bending stiffness to the mid-thickness of the panel

$$W_{eff} = \frac{J_{eff}}{h/2} = \frac{6.02 \cdot 10^7 mm^4}{100/2} = 1.2 \cdot 10^6\ mm^3$$

Design for vertical loads: floors

Bending stresses: calculation with a simplified method

The maximum normal stress can be calculated dividing the bending moment by the effective section modulus:

$$\sigma_{m,d} = \frac{M}{W_{eff}} = \frac{7.33 \cdot 10^6\ Nmm}{1.2 \cdot 10^6\ mm^3} = 6.11\ MPa$$

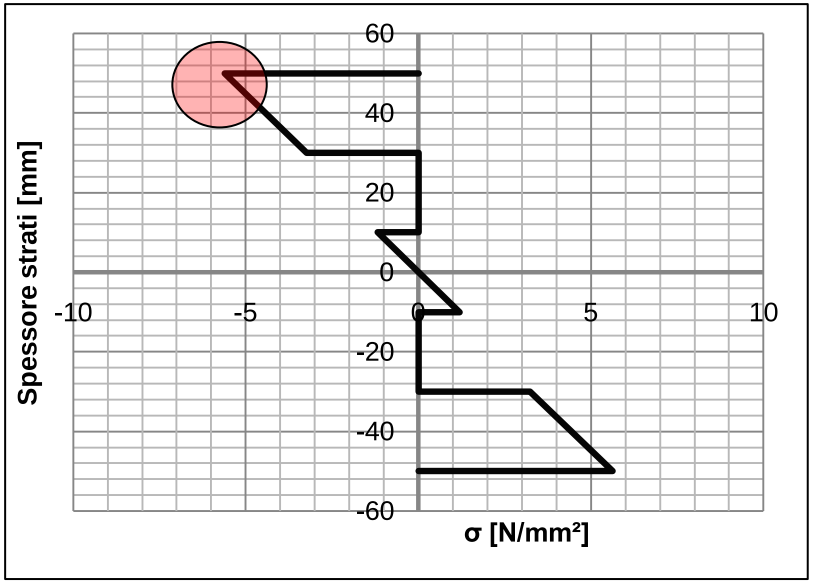
The design bending strength is

$$f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M} = \frac{0.8 \cdot 24}{1.5} = 12.8\ MPa$$

Stress ratio

$$\frac{\sigma_{m,d}}{f_{m,d}} = \frac{6.11\ MPa}{12.8 MPa} = 48\%$$

Distribution of normal stresses in the layers due to the bending action



Load duration class: medium term

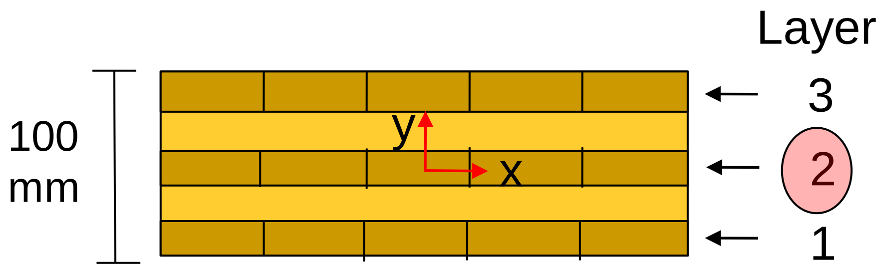
$$k_{sys} = 1.1$$

(for n > 4 parallel interacting boards in the top layers)

Design for vertical loads: floors

Shear stresses in the longitudinal layers

The maximum shear stress occurs at the neutral axis position and shall not exceed the design shear strength. In this case it is localized in the central layer (2):



Shear failure in longitudinal layers

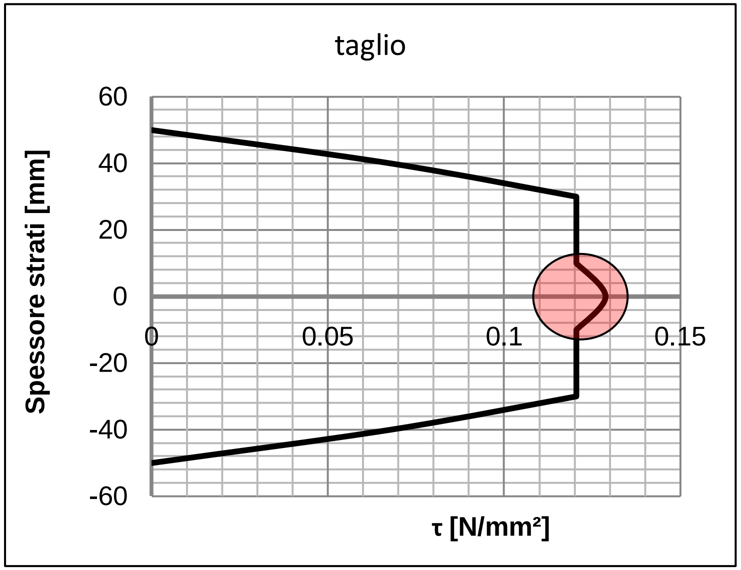
The maximum value of the shear stress can be evaluated using Jourawsky formulation

$$\tau_d = \frac{V \cdot S_{eff}}{J_{eff} \cdot b_2} = \frac{9970 \cdot 777200}{6.02 \cdot 10^7 \cdot 1000} = 0.129\ MPa$$

$$f_{v,d} = \frac{k_{mod} \cdot f_{v,k}}{\gamma_M} = \frac{0.8 \cdot 4}{1.5} = 2.13\ MPa$$

The ratio of the design stress to the shear strength of the material is

$$\frac{\tau_d}{f_{v,d}} = \frac{0.129\ MPa}{2.13\ MPa} = 6.06\%$$

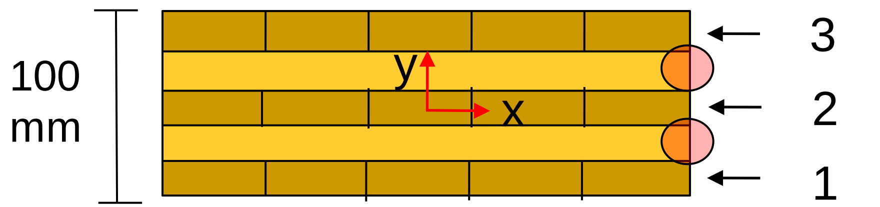


Design for vertical loads: floors

Shear stresses in the transverse layers

$f_{v,R,k} \approx 2\, f_{t,90,k} = 1\ MPa$

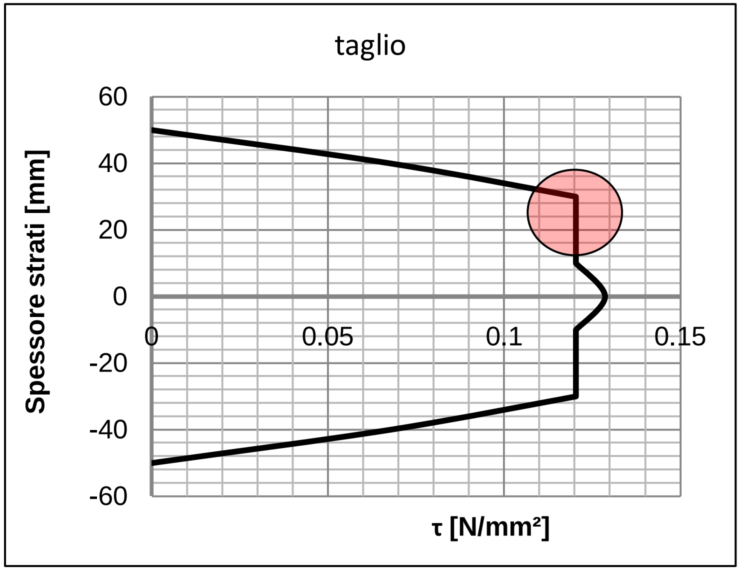
Also the shear stress in the cross layers should be verified



The shear strength for rolling shear is approximately equal to twice the tension strength perpendicular to the grain (EN 1995)

Using Jourawsky formulation we have

$$\tau_d = \frac{V_{Ed} \cdot S_i}{J_{eff} \cdot b} = 0.12\ MPa$$



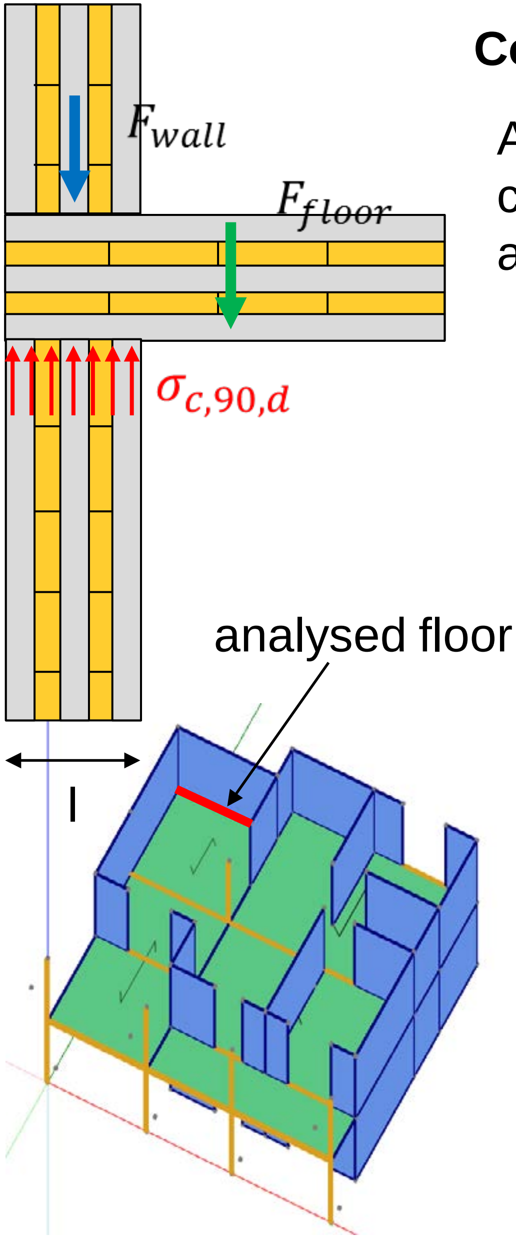
Design rolling shear strength can be calculated as

$$f_{v,d} = \frac{k_{mod} \cdot f_{v,k}}{\gamma_M} = \frac{0.8 \cdot 1}{1.5} = 0{,}53\ MPa$$

$$\frac{\tau_d}{f_{v,d}} = \frac{0.12\ MPa}{0.53\ MPa} = 22.64\%$$

Design for vertical loads: floors

Compression perpendicular to the grain



According to the clause 6.1.5 of Eurocode 5, the design compression stress perpendicular to the grain can be evaluated as

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{F_{wall} + F_{floor}}{l_{ef}}$$

In the load combination $1.3 \cdot G_1 + 1.5 \cdot G_2 + 1.5 \cdot Q_{snow}$

the values of the acting loads are

$$F_{wall} = 13.14\ kN/m$$
$$F_{floor} = 5.79\ kN/m$$

In this case the effective contact length should be assumed equal to the real contact length:

$$l_{ef} = l = 100\ \text{mm}$$

The design compression stress can be calculated as

$$\sigma_{c,90,d} = \frac{F_{wall} + F_{floor}}{l_{ef}} = \frac{(13.14 + 5.79)\ kN/m}{0.1\ m} = 0.19\ MPa$$

Design for vertical loads: floors

Compression perpendicular to the grain

The design compression strength perpendicular to the grain can be evaluated as

$$f_{c,90,d} = \frac{k_{mod} \cdot f_{c,90,k}}{\gamma_M} = \frac{0.9 \cdot 2.50}{1.5} = 1.5\ MPa$$

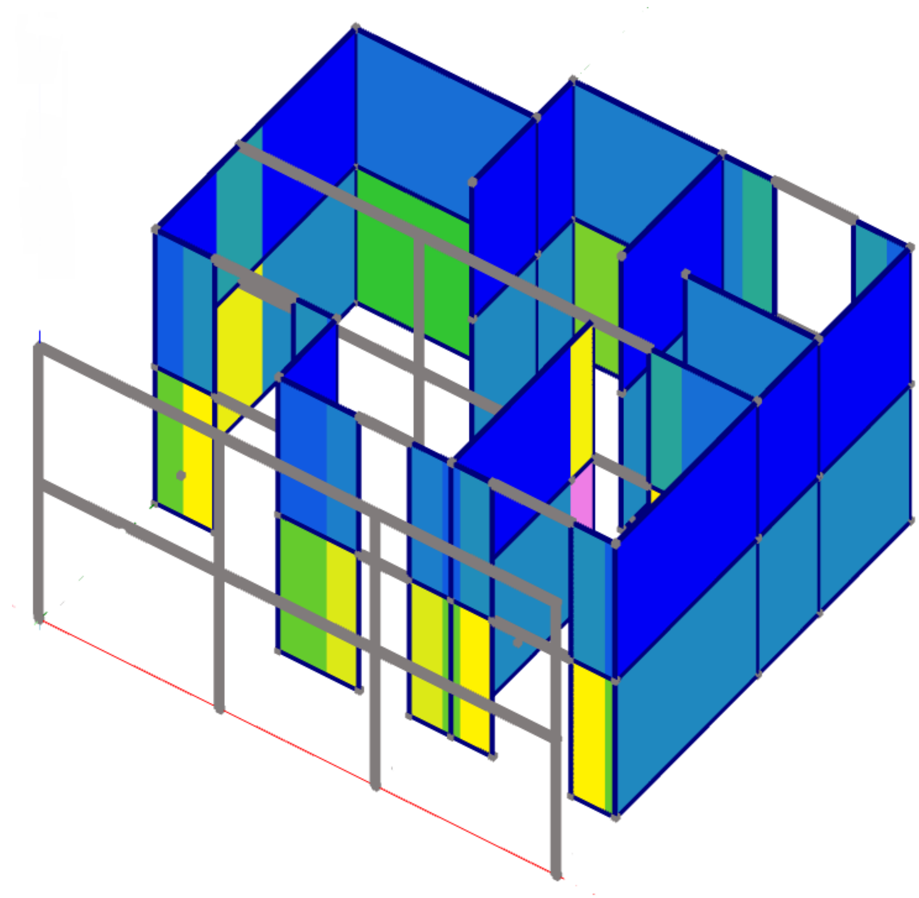
For the considered combination the load-duration class is short-term

Hence we can check the compression stress using the following formula:

$$\frac{\sigma_{c,90,d}}{k_{c,90} \cdot f_{c,90,d}} = \frac{0.19}{1.50 \cdot 1.5} = 8.4\% \ < 100\ \%$$

$k_{c,90} = 1.5$ is a factor taking into account the load configuration and the degree of compressive deformation.

DESIGN FOR VERTICAL LOADS: WALLS

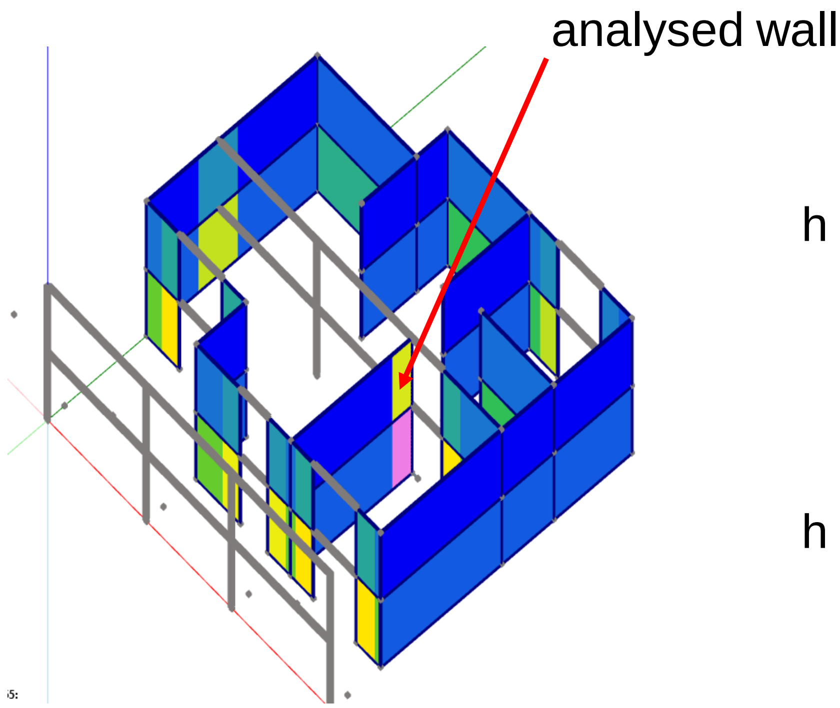


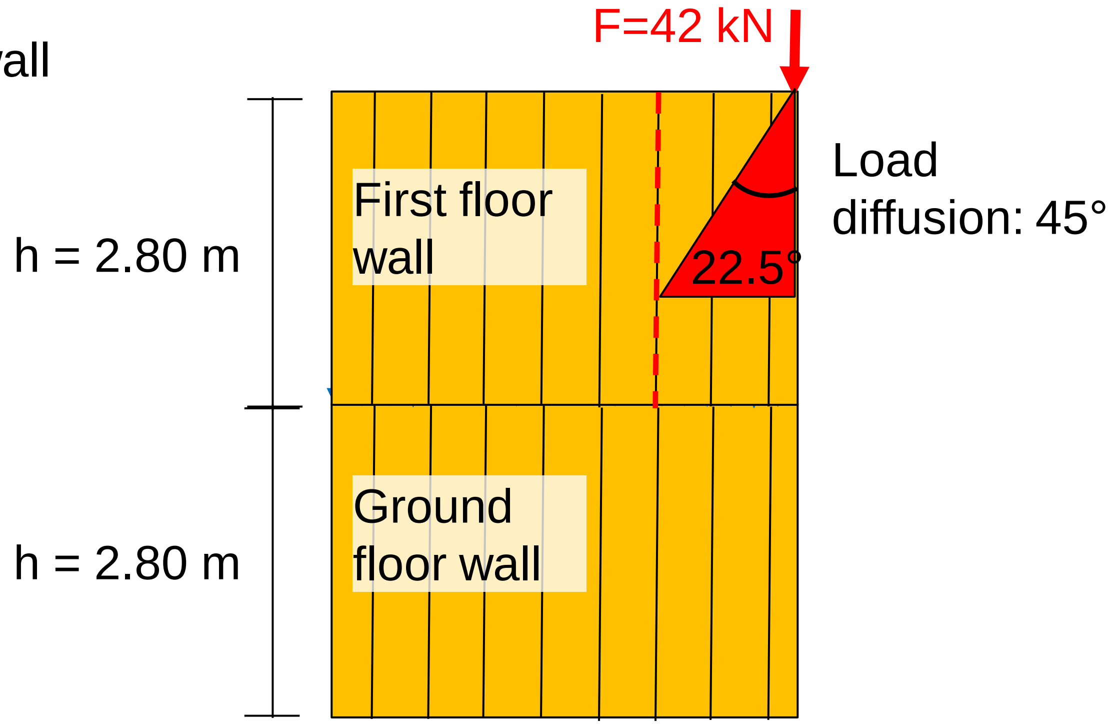
Design of walls for vertical loads: instability

The walls present the following properties:

- Height h = 2.80 m
- Thickness t = 100 mm
- 5-layer CLT panels (20 mm per layer)
- Timber grade C24

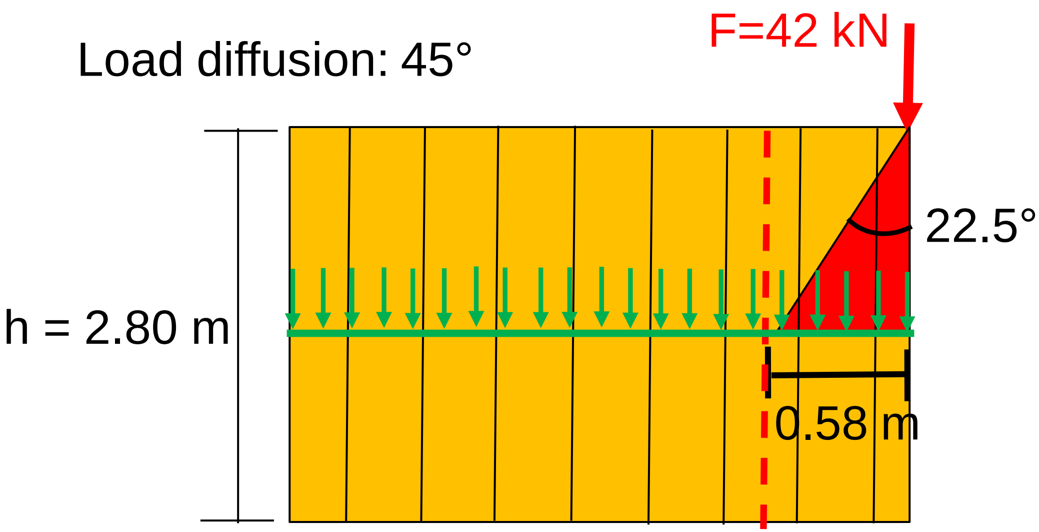
It is checked one of the most stressed wall on which acts a concentrated load, due to the presence of the support of a beam.





Design of walls for vertical loads: instability

The load acting is given by the sum of the self-weight of the wall and the concentrated load, redistributed on a portion of the wall of lenght 0.58 m.



The wall self-weight (h=1.4 m), calculated for the considered ULS combination, is

$$q_{wall} = 2.35\ \mathrm{kN/m}$$

The reaction force of the beam, equal to 42 kN, acting on the verification area is

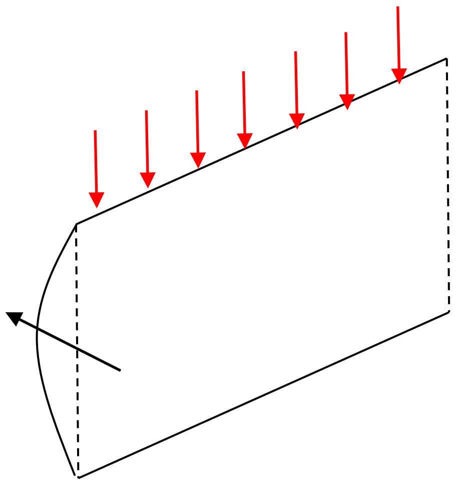
$$q_{point\ load} = \frac{42\ \mathrm{kN}}{0.58\ m} = 72.41\ kN/m$$

Hence the total load acting to be used for instability check is

$$q_{tot} = 2.35 + 72.41 = 74.76\ kN/m$$

Design of walls for vertical loads: instability

For CLT walls instability is possible only out-of-plane. The wall analysed is an internal wall which is not subjected to out-of-plane loads as the wind. To check a wall portion subjected to axial compression, clause 6.3.2 of EN 1995-1-1 can be used.

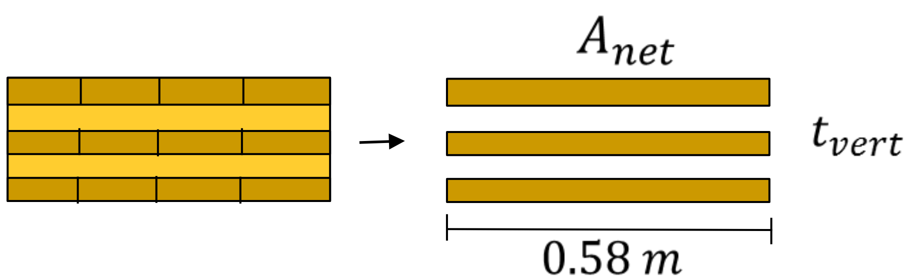


$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \cancel{\frac{\sigma_{m,y,d}}{f_{m,y,d}}} \leq 1$$

There are no loads out-of plane

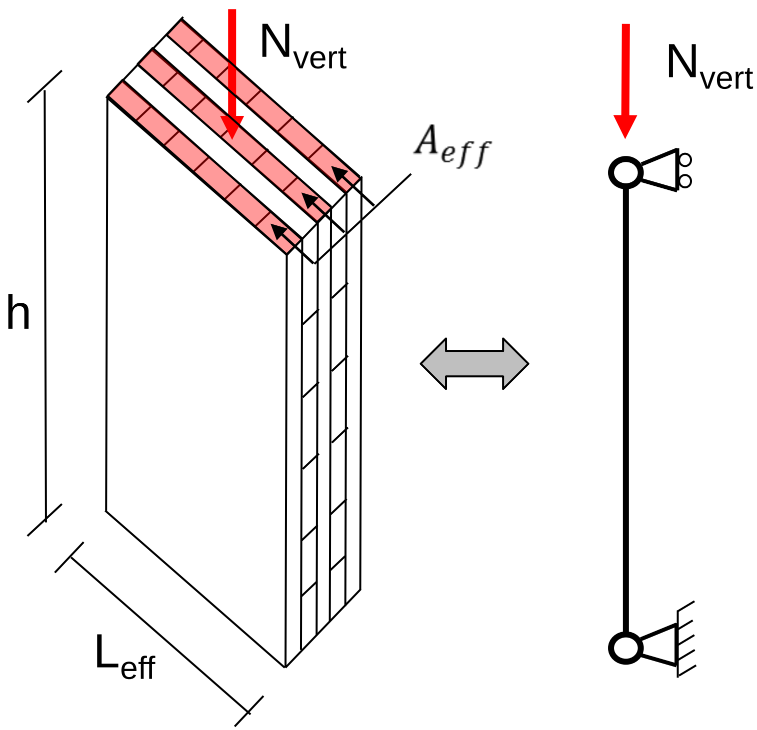
The design compression stress is calculated as the ratio of the total vertical load to the effective cross-section area of the wall portion.

$$\sigma_{c,0,d} = \frac{N_{vert}}{A_{net}} = \frac{q_{tot} \cdot 0.58m}{t_{vert} \cdot 0.58\,m} = \frac{74.76\,N/mm}{60\,mm} = 1.25\,MPa$$



Design of walls for vertical loads: instability

To evaluate the stability of the CLT wall portion we can refer to the clause 6.3.2 of EN 1995-1-1 «*Columns subjected to either compression or combined compression and bending*»



The design compression strength can be evaluated as

$$f_{c,0,d} = \frac{k_{mod} \cdot f_{c,0,k}}{\gamma_M} = \frac{0.9 \cdot 21}{1.5} = 12.6\ MPa$$

Load-duration class: short-term (snow)

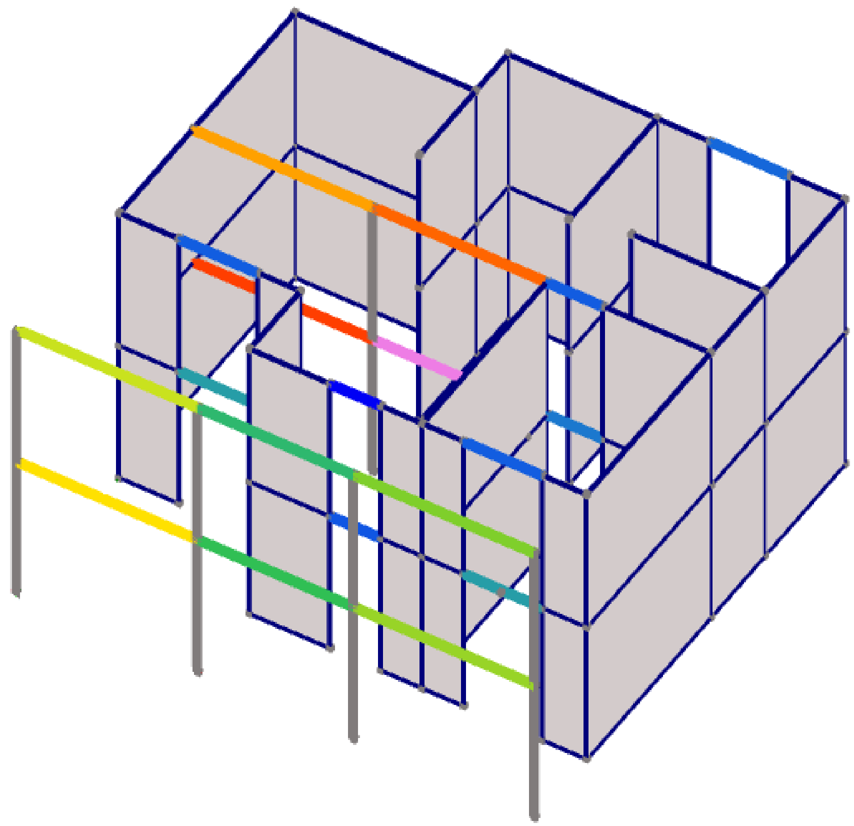
Stability verification

Hence we can check the stability of the wall:

There are no loads perpendicular to the wall plane

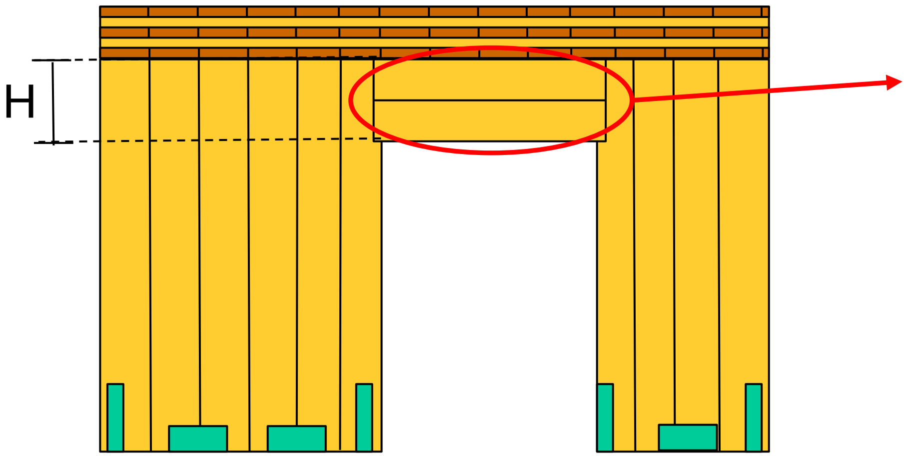
$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} + \cancel{\frac{\sigma_{m,y,d}}{f_{m,y,d}}} = \frac{1{,}25\ MPa}{0.4 \cdot 12.6\ MPa} = 24.80\ \% < 100\ \%$$

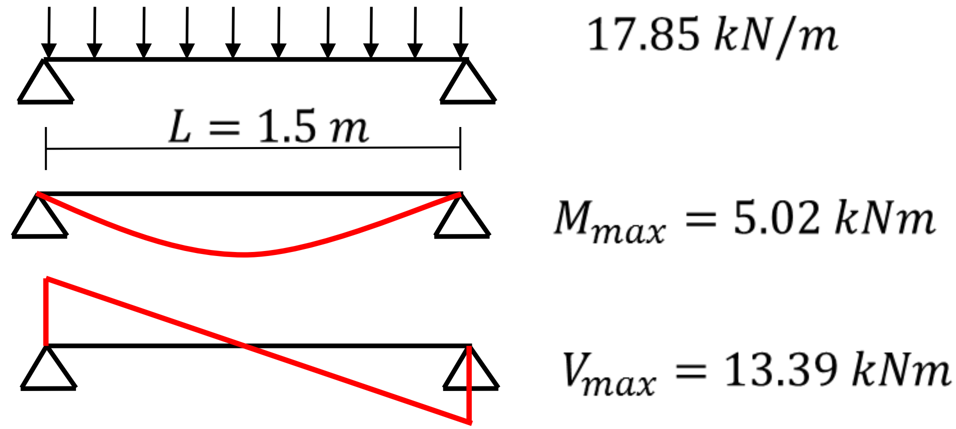
DESIGN FOR VERTICAL LOADS: LINTELS



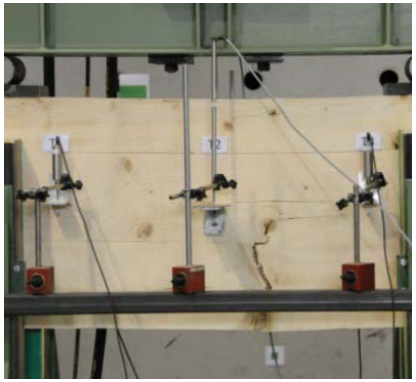
CLT lintels

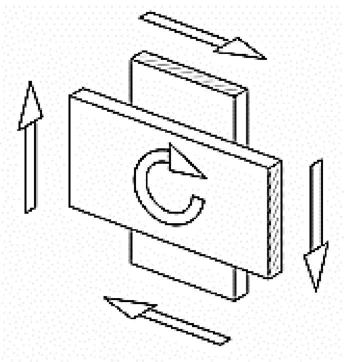
Failure mechanism





In the considered ULS combination the distributed load acting on the analysed lintel is equal to 17.85 kN/m.





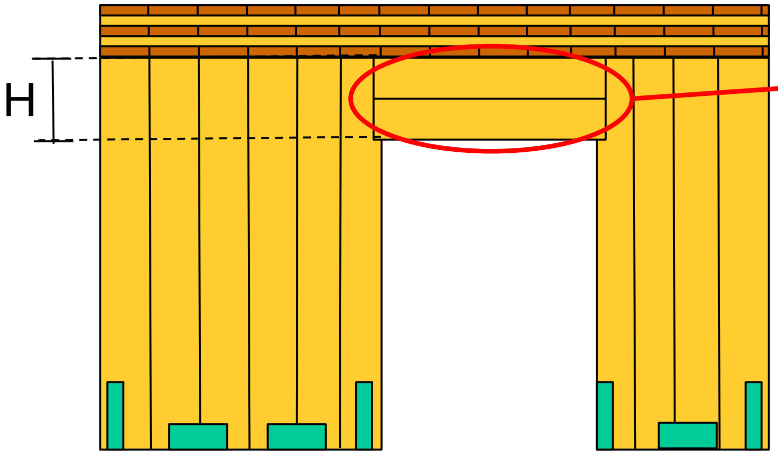
Bending

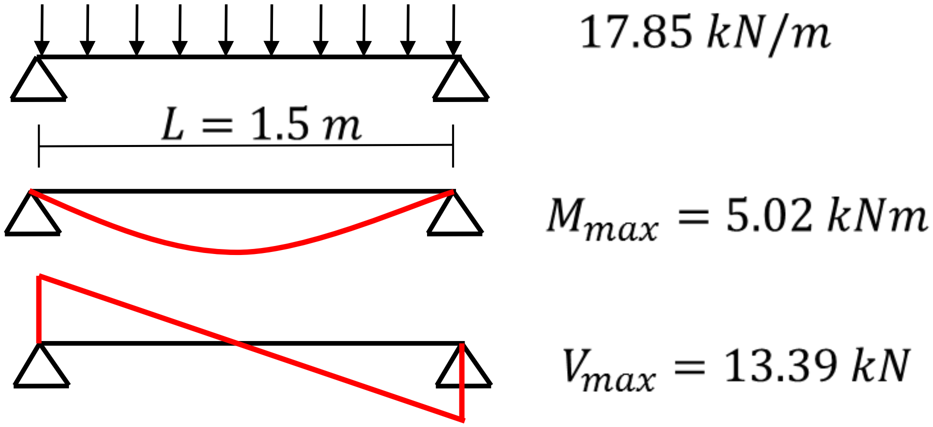
Torsional shear

Shear perp. to grain

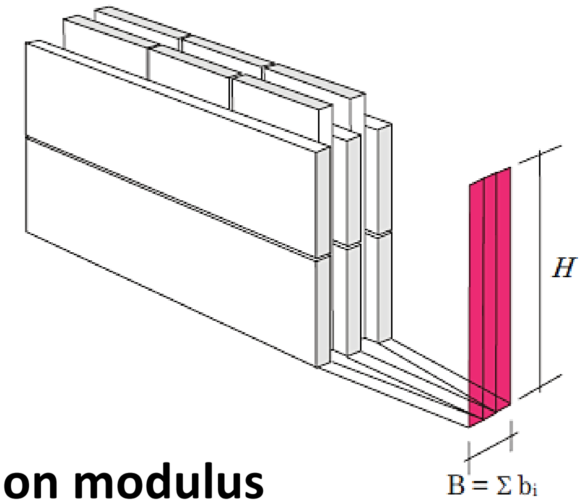
CLT lintels

Bending





The CLT beam has the following characteristics:



- Total section width B_{tot} = 100 mm
- Effective section width B_{eff} = 60 mm
- Section height H = 600 mm
- Span L = 1.5 m

effective modulus of inertia

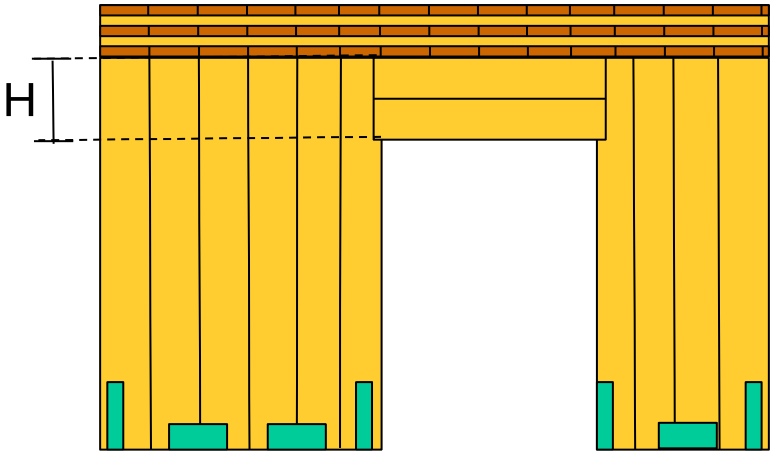
$$I_{net} = \frac{B \cdot H^3}{12} = \frac{60 \cdot 600^3}{12} = 1.08 \cdot 10^9\ mm^3$$

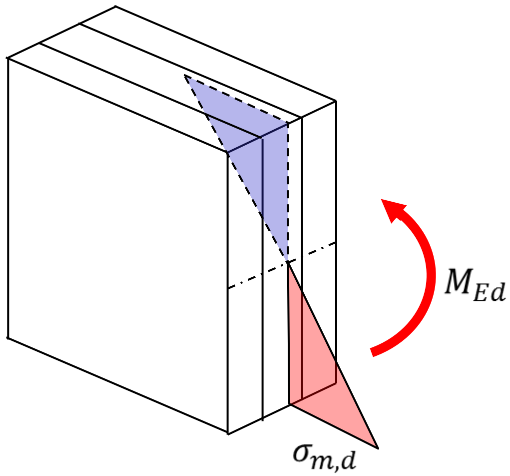
Effective section modulus

$$W_{eff} = \frac{I_{net}}{H/2} = \frac{1.08 \cdot 10^9\ mm^3}{600/2\ mm} = 3.6 \cdot 10^6\ mm^3$$

CLT lintels

Bending





The maximum design normal stress can be calculated using the following expression

$$\sigma_{m,d} = \frac{M_{Ed}}{W_{eff}} = \frac{5.02 \cdot 10^6\ Nmm}{3.6 \cdot 10^6\ mm^3} = 1.39\ MPa$$

The bending strength (load-duration class: medium term) is

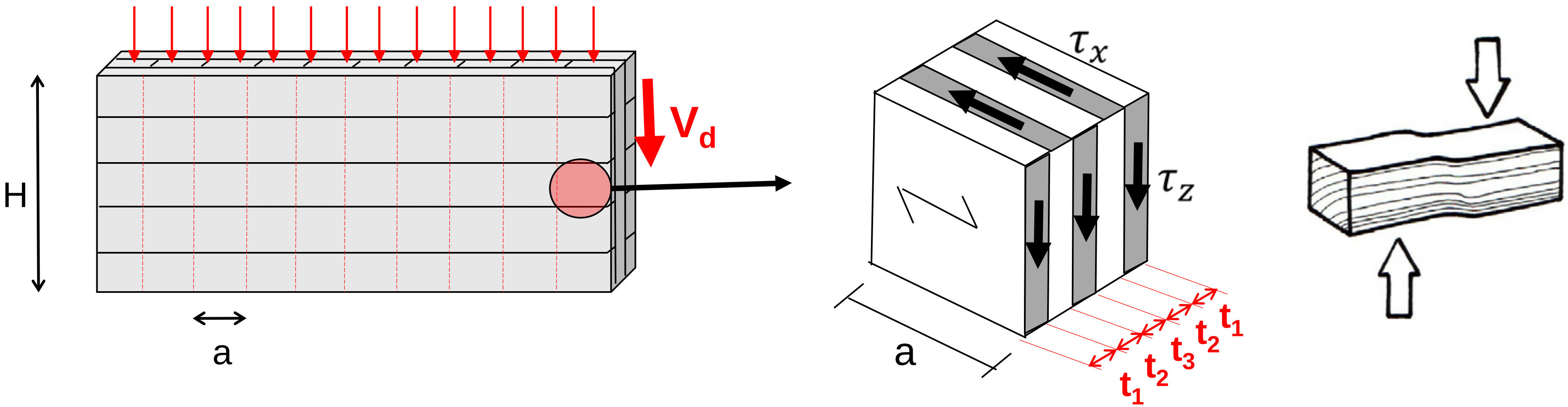
$$f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M} = \frac{0.8 \cdot 24}{1.5} = 12.8\ MPa$$

The stress ratio is

$$\frac{\sigma_{m,d}}{f_{m,d}} = \frac{1.39\ MPa}{12.8\ MPa} = 10.8\ \% < 100\ \%$$

CLT lintels

Shear: failure mechanism for shear perpendicular to grain



The maximum shear per linear meter can be calculated as:

$$v = 1.5 \cdot \frac{V_d}{H} = 1.5 \cdot \frac{13.39\ kN}{0.6\ m} = 33.48\ kN/m$$

$$t_1 = t_2 = t_3 = 20\ mm$$

$$\tau_x = \frac{v}{\sum t_{i,vert}}$$

shear stresses perpendicular to the vertical layers

$$\tau_x = \frac{33.48\ N/mm}{40\ mm} = 0.84\ MPa$$

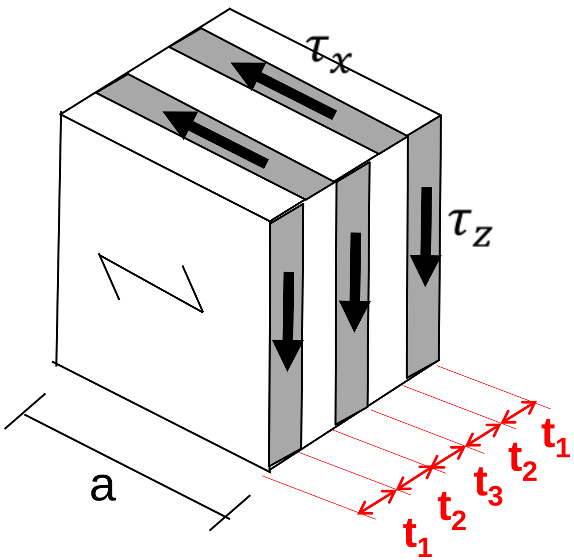
$$\tau_z = \frac{v}{\sum t_{i,oriz}}$$

shear stresses perpendicular to the horizontal layers

$$\tau_z = \frac{33.48\ N/mm}{60\ mm} = 0.56\ MPa$$

CLT lintels

Shear: failure mechanism for shear perpendicular to grain



The maximum shear stress value (τ_x or τ_z) must be considered

$$\tau_d = \max(\tau_x; \tau_z) = 0.84\ MPa$$

The design resistance in shear perpendicular to grain is

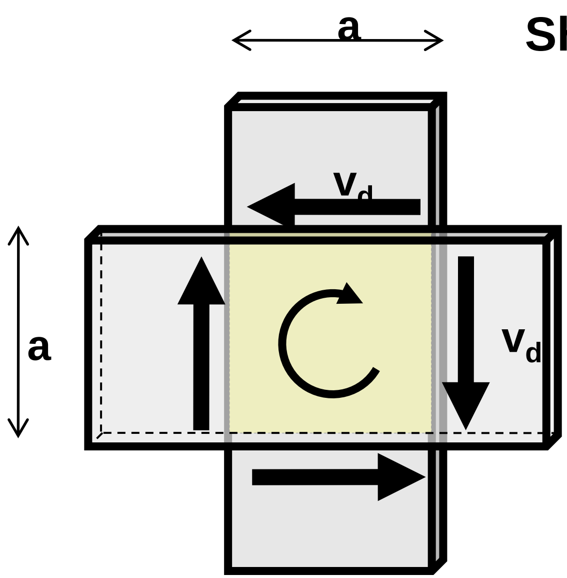
$$f_{v,perp,d} = \frac{k_{mod} \cdot f_{v,perp,k}}{\gamma_M} = \frac{0.8 \cdot 4}{1.5} = 2.13\ MPa$$

The resistance verification to this failure mechanism is as follows

$$\frac{\tau_d}{f_{v,lastra,d}} = \frac{0.84\ MPa}{2.13\ MPa} = 39.44\ \% < 100\ \%$$

CLT lintels

Shear: torsion failure on crossing surfaces



(a is the dimension of the crossing area)

The value of the design torsional moment M_T, acting on the glued surfaces at the intersection between boards, can be calculated using the expression reported in many European Technical Approvals (ETA)

$$M_T = \frac{v \cdot a^2}{n_{glued\,surfaces}} = \frac{v \cdot a^2}{n_{layers-1}}$$

$$M_T = \frac{33.48\,\frac{N}{mm} \cdot 150^2\,mm^2}{4} = 188 \cdot 10^3\,Nmm$$

torsional stress

$$\tau_{T,d} = \frac{M_T}{W} = \frac{188 \cdot 10^3\,Nmm}{1.125 \cdot 10^6\,mm^3} = 0.17\,MPa$$

torsional section modulus

$$W = \frac{a^3}{3} = \frac{150^3}{3} = 1.125 \cdot 10^6\,mm^3$$

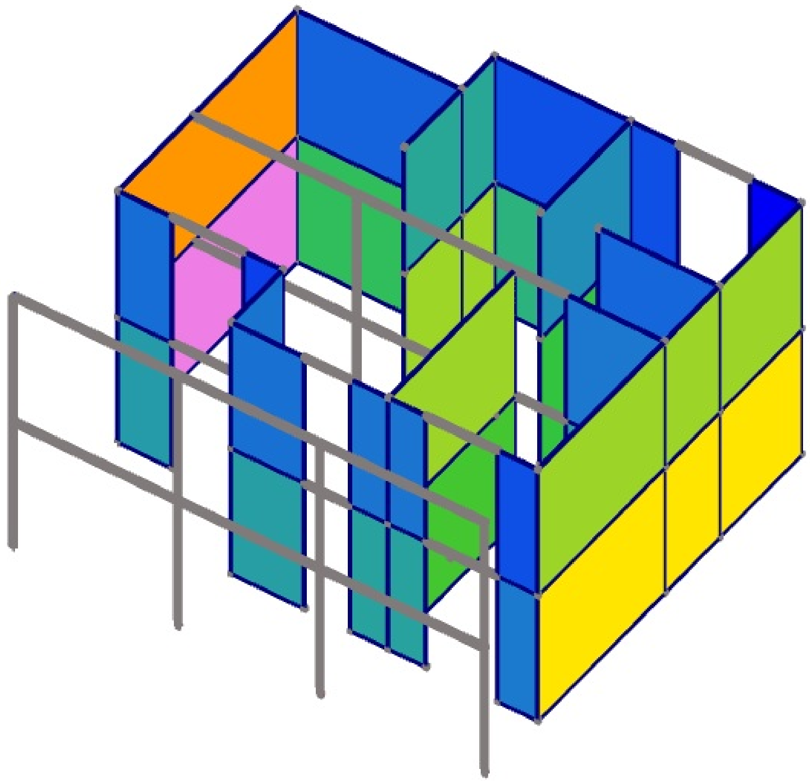
The design torsional strength of the material is

$$f_{T,d} = \frac{k_{mod} \cdot f_{T,k}}{\gamma_M} = \frac{0{,}8 \cdot 2.5}{1.5} = 1.33\,MPa$$

torsional verification

$$\frac{\tau_{T,d}}{f_{T,d}} = \frac{0.17 MPa}{1.33\,MPa} = 13\,\% < 100\,\%$$

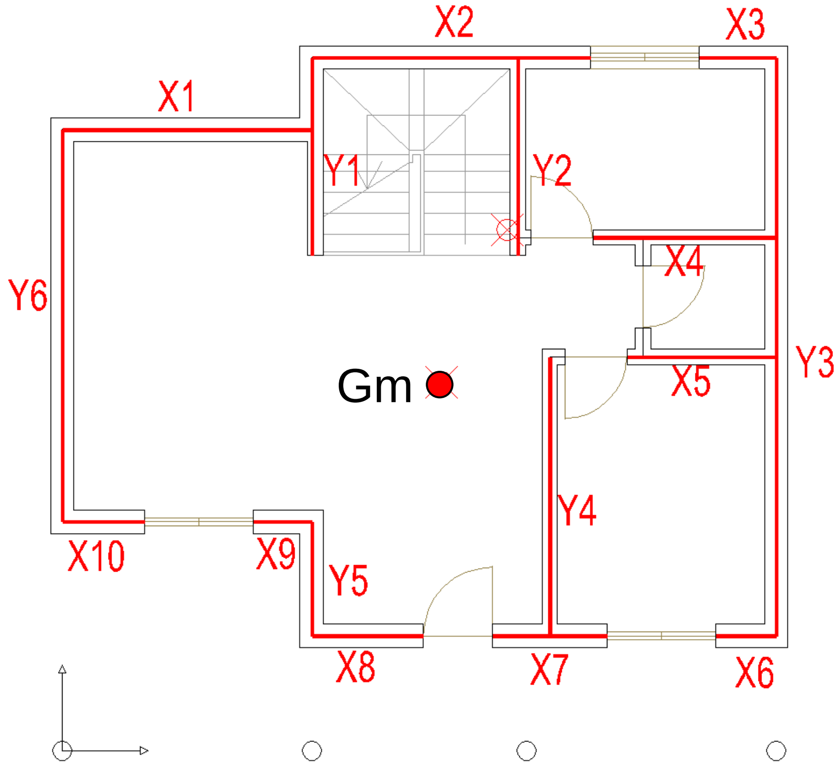
DESIGN FOR HORIZONTAL LOADS: SEISMIC ACTION



Center of mass

The floor diaphragms are considered perfectly rigid in-plane when calculating the distribution of lateral forces. Each diaphragm has three degrees of freedom: translation in x, translation in y and rotation around the z axis.

The figure below shows the position of the center of masses (seismic forces are applied to this point).



Center of mass

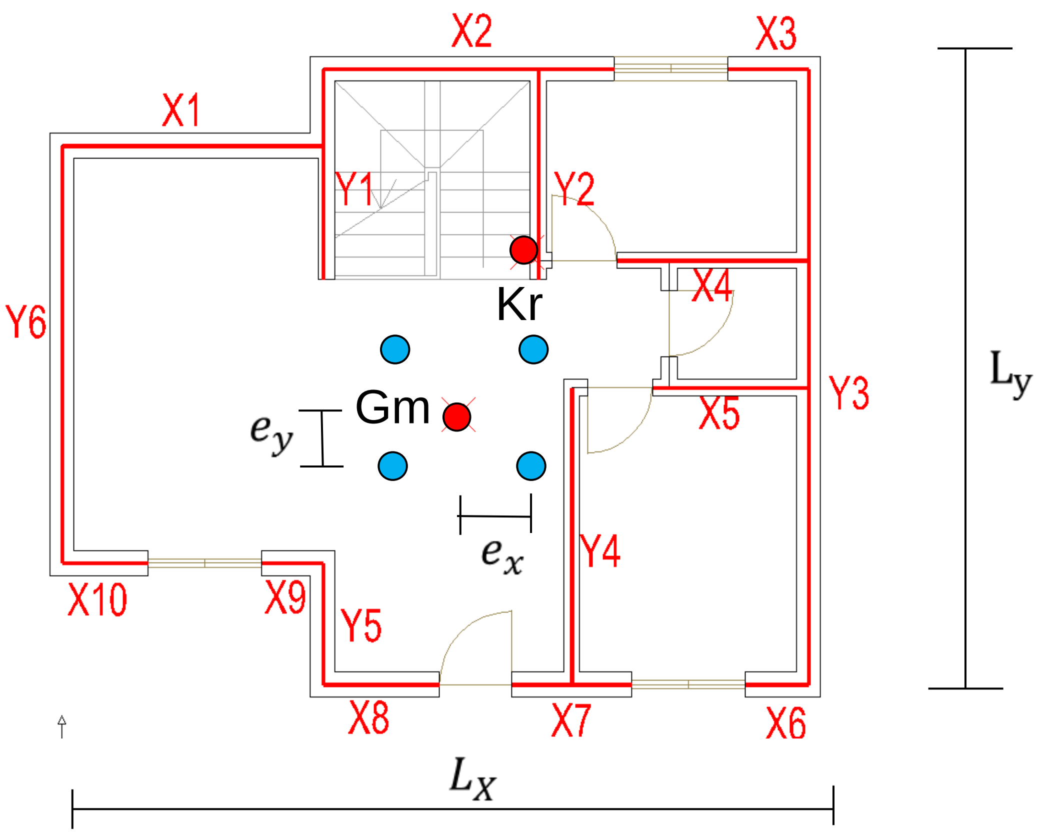
To take into account the uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass at each floor shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity.

Accidental eccentricity

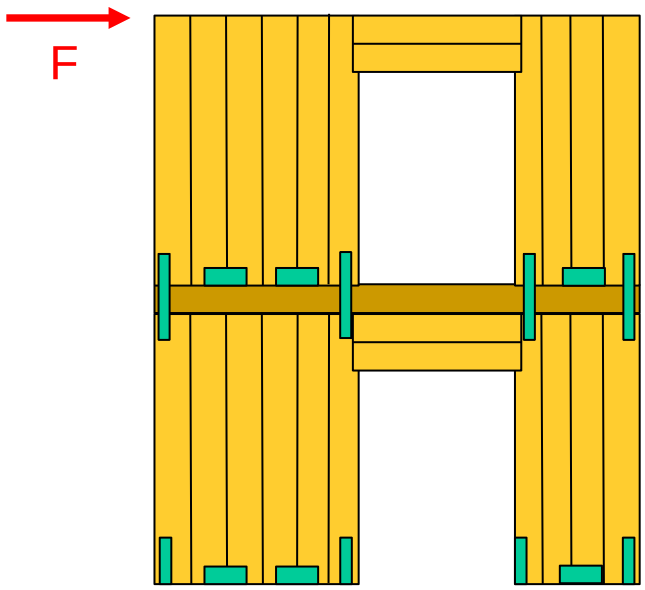
$$e_x = \pm 0{,}05 \cdot L_x$$

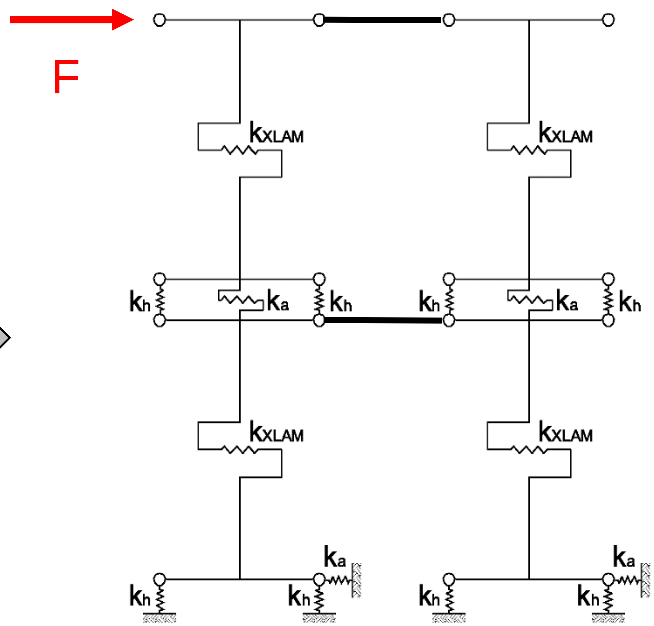
$$e_y = \pm 0{,}05 \cdot L_y$$

where Lx or Ly are the floor dimensions perpendicular to the direction of the seismic action.



Evaluation of the CLT walls stiffness



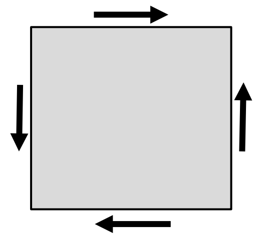


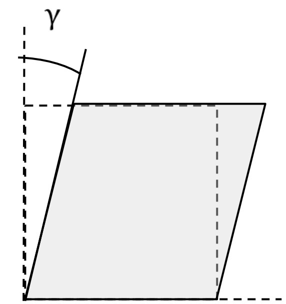
The deformation of the walls under horizontal loads can be evaluated by adding the different contributions of deformation:

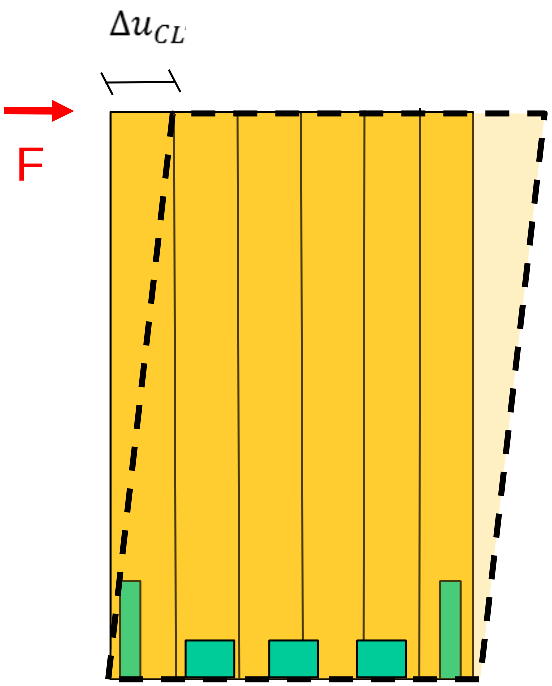
- CLT panels
- tension connections (hold-down/tie-down)
- shear connections (angle brackets)

Evaluation of the CLT walls stiffness

CLT panel deformation







The effective shear modulus of the CLT elements can be calculated using the following equation

$$G_{eff} = G_{0,mean} \cdot \frac{1}{1 + 6 \cdot \alpha_T \cdot \left(\frac{t}{a}\right)^2} = 690 \cdot \frac{1}{1 + 6 \cdot 1.51 \cdot \left(\frac{20}{150}\right)^2} = 594\ MPa$$

In which α_T is a correction factor

$$\alpha_T = 0.32 \cdot \left(\frac{t}{a}\right)^{-0.77} = 0.32 \cdot \left(\frac{20}{150}\right)^{-0.77} = 1.51$$

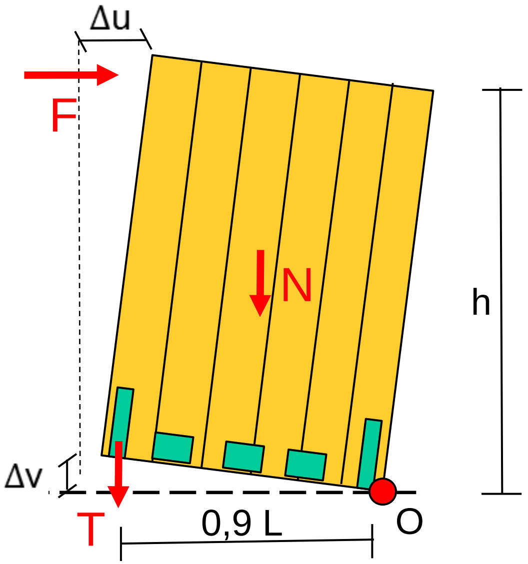
depending on

t: the mean thickness of the boards (20 mm)

a: the width of the boards (if a is not constant for all the boards, a mean value should be used for approximation)

Evaluation of the CLT walls stiffness

Hold-down deformation



With reference to the rigid body rocking and considering the rotational equilibrium around point O, there are:

- an overturning contribution due to the seismic force F
- a stabilizing contribution due to the vertical load N

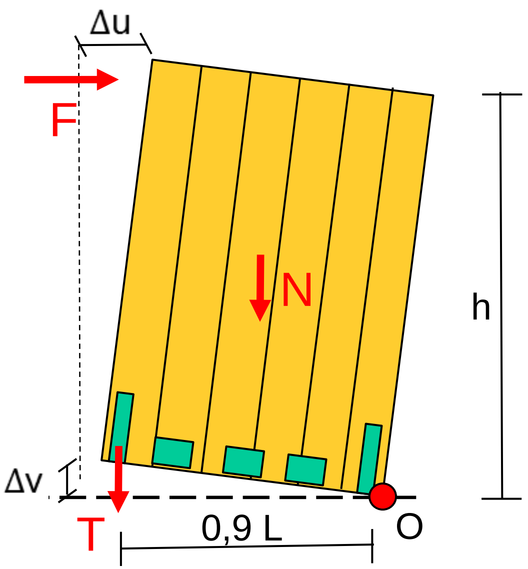
The tension force in the hold-down can be calculated as:

$$\begin{cases} T = \left(\dfrac{F \cdot h}{0{,}9 \cdot L} - \dfrac{N}{2}\right) & \text{active hold-down} \quad F \cdot h > N \cdot \dfrac{0{,}9 \cdot L}{2} \\ T = 0 & \text{not active hold-down} \quad F \cdot h \leq N \cdot \dfrac{0{,}9 \cdot L}{2} \end{cases}$$

- the first equation refers to the case in which the wall rotates because the overturning moment is greater than the stabilizing moment
- the second one refers to the case in which the wall does not rotate because the stabilizing moment is greater than the overturning moment

Evaluation of the CLT walls stiffness

Hold-down deformation



The tension force in the hold-down can be calculated as:

$$T = \left(\frac{F \cdot h}{0{,}9 \cdot L} - \frac{N}{2} \right) \qquad \text{active hold-down} \qquad F \cdot h > N \cdot \frac{0{,}9 \cdot L}{2}$$

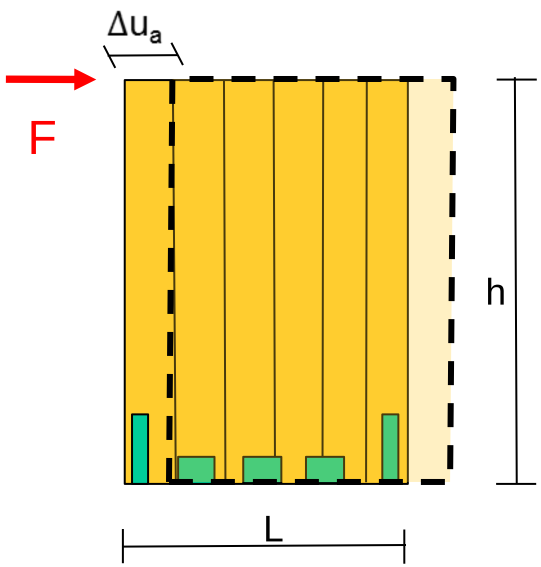
$$T = 0 \qquad \text{not active hold-down} \qquad F \cdot h \leq N \cdot \frac{0{,}9 \cdot L}{2}$$

Introducing the hold-down stiffness it can be calculated the vertical displacement Δv and the horizontal displacement at the top of the wall

$$\Delta v = \frac{T}{k_h} \quad \Leftrightarrow \quad \Delta u_{hold-down} = \frac{h}{0{,}9 \cdot L} \cdot \Delta v$$

Evaluation of the CLT walls stiffness

Shear-connections deformation



The rigid body translation of the wall due to the deformation of the angle brackets can be evaluated using the following expression

$$\Delta u_a = \frac{F}{k_a \cdot n_a}$$

being:

- k_a the stiffness of each connection
- n_a the number of connections

Stiffness of the wall

The total displacement at the top of the wall is given by the sum of all the deformation contributions

$$u_{wall} = \Delta u_{CLT} + \Delta u_{hd} + \Delta u_a$$

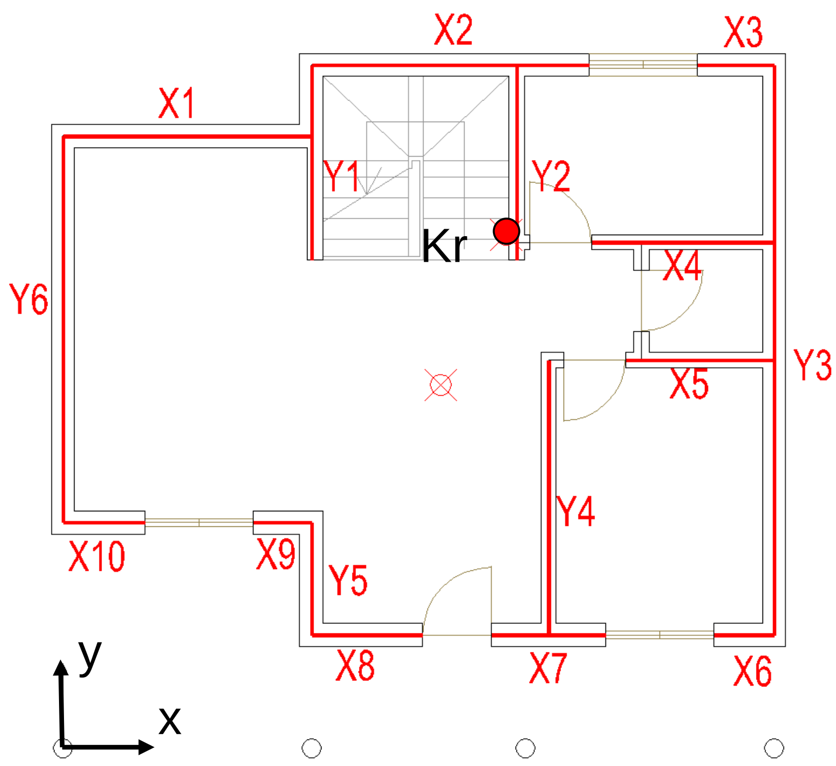
Therefore the stiffness of the wall can be evaluated as

$$k_{wall} = \frac{F}{u_{wall}}$$

Seismic action

Stiffness center

Once calculated the stiffness of the walls, it is possible to locate the centre of stiffness of each floor.



Seismic action

The fundamental period of vibration of the building is

$$T_1 = C_1 \cdot H^{\frac{3}{4}} = 0.05 \cdot 5.6^{\frac{3}{4}} = 0.18\, s$$

Behaviour factor

- Glued wall panels with glued diaphragms $q_0 = 2.00$
- The structure is regular in elevation so the factor K_R can be assumed equal to the unit $K_R = 1$

$$q = q_0 \cdot K_R = 2.00 \cdot 1 = 2.00$$

Floor masses

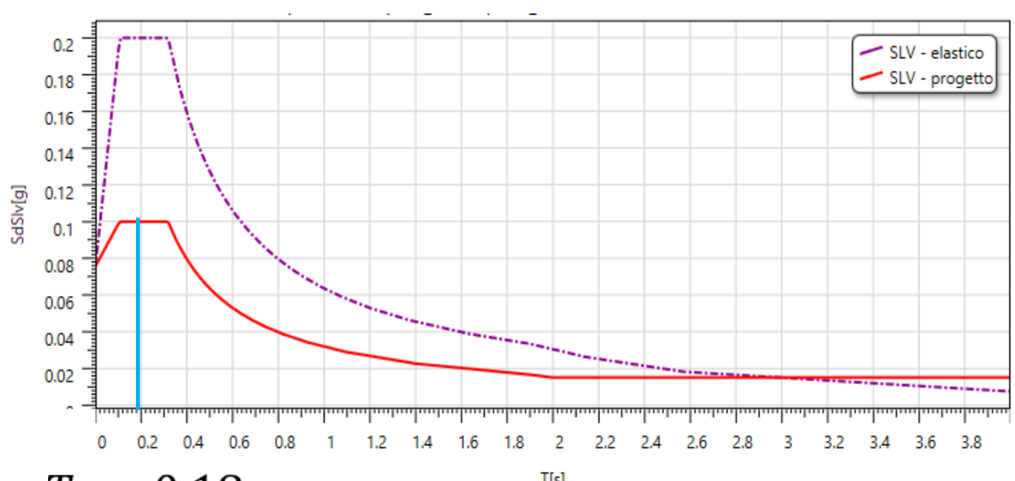
Floor	Height [m]	Translational mass [kg]	Rotational inertia [kg m^2]
1	2.80	39892	591420
2	5.60	28380	448214

Seismic action

Linear-elastic analysis

The seismic analysis is performed according to the so called «lateral force method», which can be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode.

The design spectrum at the ultimate limit state is depicted in the picture below



$$T_1 = 0.18\ s$$

Limit state for the safeguard of human life

$$S_d(T_1) = 0.10\ g$$

The total base shear due to the seismic design action is

$$F_h = S_d(T_1) \cdot \frac{W}{g} = 0.1 \cdot 683\ kN = 68.3\ kN$$

Seismic action

Linear-elastic analysis

The equivalent static lateral forces can be evaluated using the following expression

$$F_i = \frac{F_h \cdot z_i \cdot W_i}{\sum_j z_j \cdot W_j}$$

Total base shear

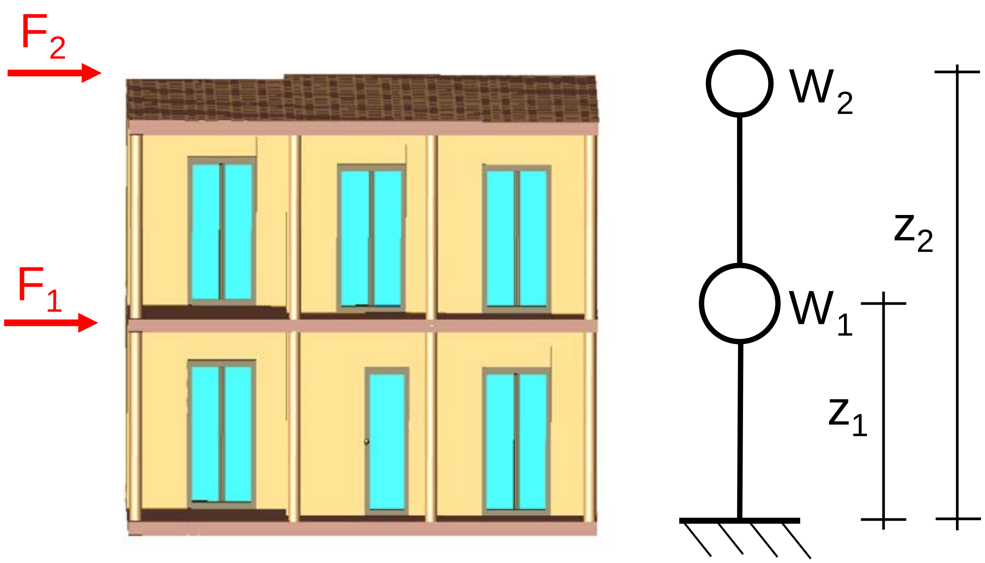
$$F_h = 68.3\ kN$$

First storey

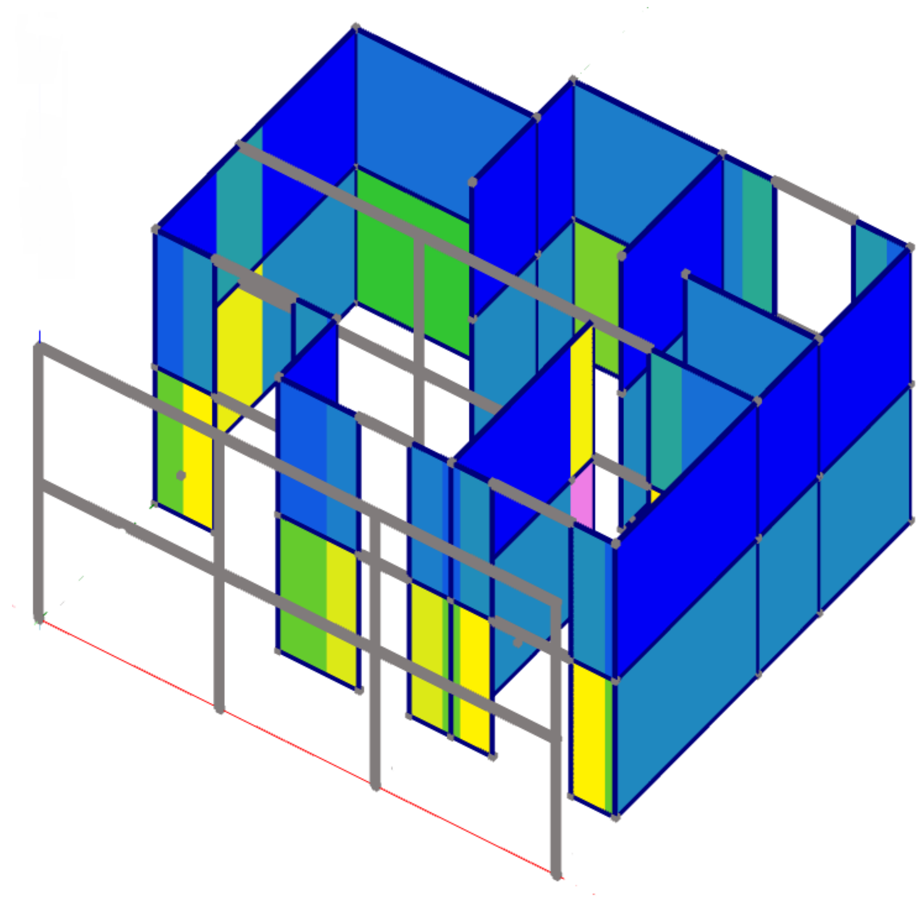
$$F_1 = \frac{68.3 \cdot 2.80 \cdot 399}{2.80 \cdot 399 + 5.60 \cdot 284} = 28.18\ kN$$

Second storey

$$F_2 = \frac{68.3 \cdot 5.60 \cdot 284}{2.80 \cdot 399 + 5.60 \cdot 284} = 40.12\ kN$$



SEISMIC DESIGN: WALLS

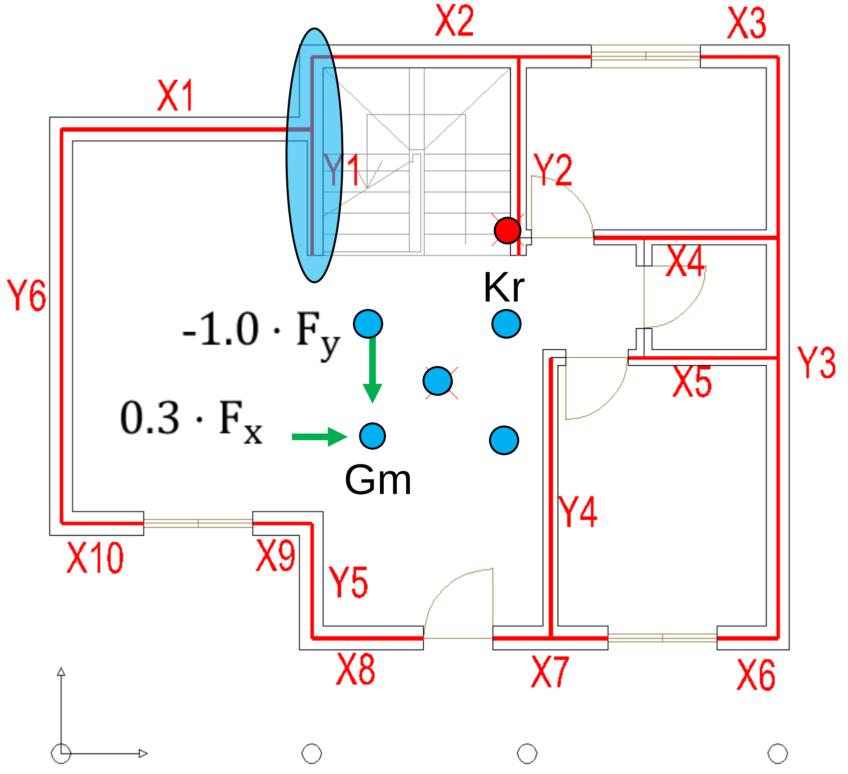


Seismic design: walls

Check of the wall Y1 for the Ultimate Limit State “safeguard of human life”, considering the following spatial combination of the seismic effects:

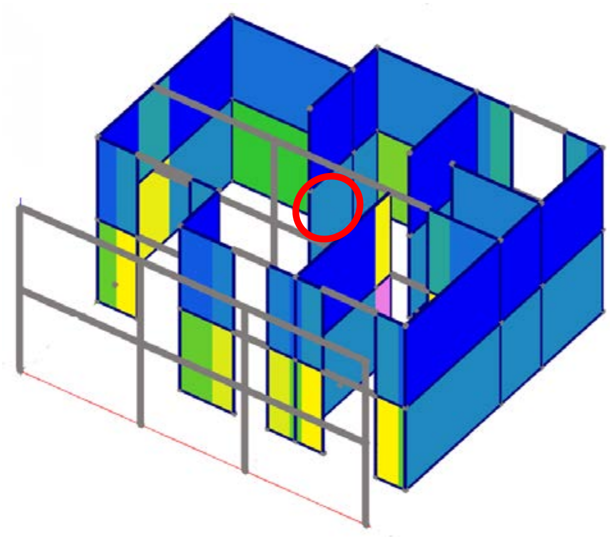
$$0.3 \cdot F_{SLV,x} - 1.0 \cdot F_{SLV,y}$$

with an accidental eccentricity of storey mass equal to –ex –ey.



The shear force acting on the considered wall is

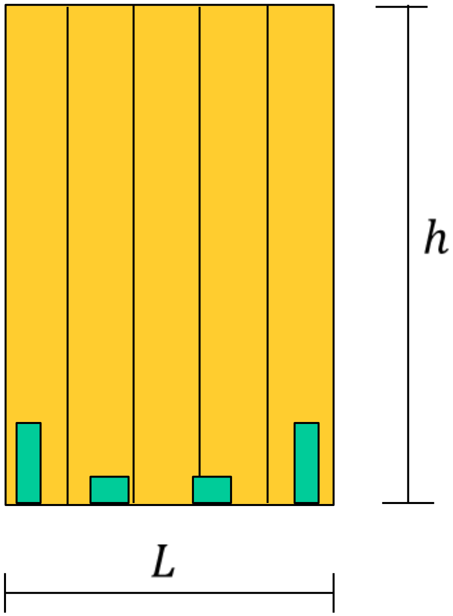
$$V_{wall} = 7.17\ kN$$



Seismic design: walls

The analysed wall has the following characteristics:

- Lenght L = 2.55 m
- Height h = 2.80 m



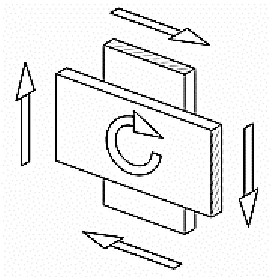
Material CLT

- 5-layer CLT panels (20 mm per layer)
- Timber grade C24

Shear verification of the wall

There are two possible shear failure modes:

- Net shear (shear perpendicular to grain)
- Torsional shear failure of crossing surfaces between the orthogonal boards



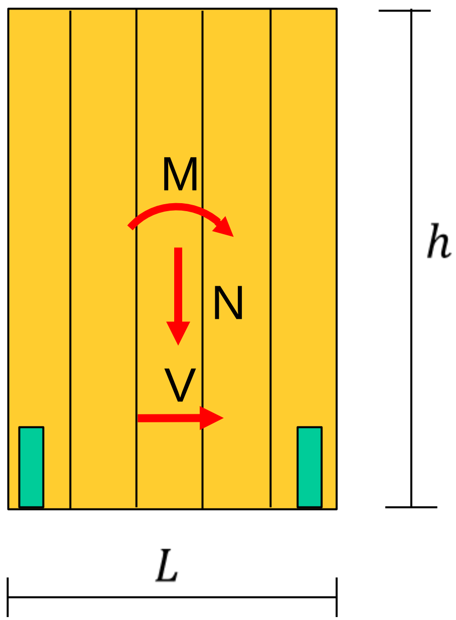
See the shear verification of CLT lintels …

SEISMIC DESIGN: CONNECTIONS

Design of connections: hold-down

The analysed wall has the following characteristics:

- Lenght L = 2.55 m
- Height h = 2.80 m
- 1 hold-down on each wall corner



Acting forces:

- Vertical load N = 17.14 kN
- Shear force V = 7.17 kN
- Overturning moment M = 32.25 kNm

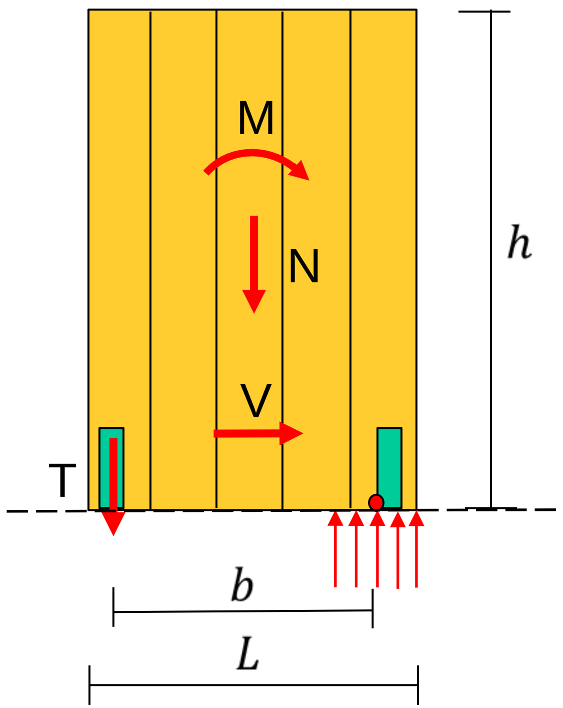
Two different cases are possible:

- the wall rotates because the overturning moment is greater than the stabilizing moment (active hold-down)
- the wall does not rotate because the stabilizing moment is greater than the overturning moment (not active hold-down)

Design of connections: hold-down

The tension force in the hold down is

$$T = \begin{cases} \left(\frac{M}{b} - \frac{N}{2}\right) & \text{Active hold-down} \\ 0 & \text{Not active hold-down} \end{cases}$$



- b is the lever arm taken equal to

$$b = 0.9 \cdot L$$

- N is the vertical load
- M is the overturning moment

In the present case the hold-down is active, therefore the tension force acting on it can be evaluated as

$$T = \left(\frac{M}{b} - \frac{N}{2}\right) = \left(\frac{32.25\ kNm}{0.9 \cdot 2.55\ m} - \frac{17.14\ N}{2}\right) = 5.48\ kN$$

Design of connections: hold-down

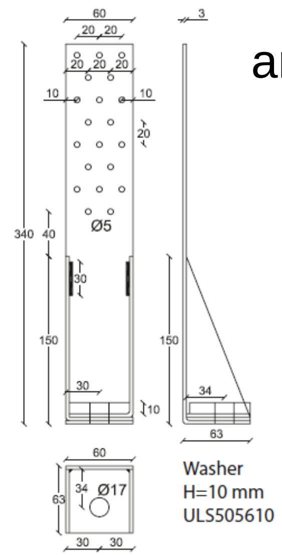
The adopted hold-down

WHT 340

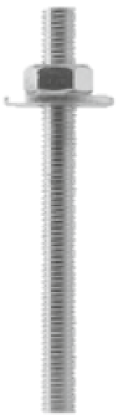
14 «anker» nails 4,0 x 40

Threaded rod M16 5.8

Chemical anchor



annular ring nails



Hold-down resistance

The design resistance of the hold-down can be evaluated as the minimum among the resistance values of the following failure mechanisms:

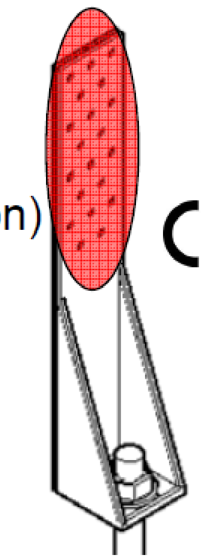
- failure of the nails
- tension failure of the hold down
- tension failure of the threaded steel rod
- pull-out resistance of the anchor

Design of connections: hold-down

Failure of the nailed connection

$$R_{c,d} = \frac{k_{mod} \cdot \mathrm{R}_{c,k}}{\gamma_M} = \frac{1 \cdot 22\ kN}{1.5} = 14.67\ kN$$

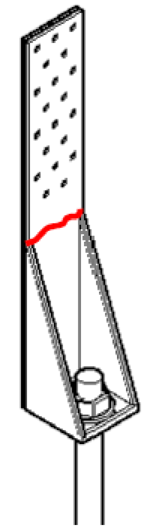
Load-duration class:
Instantaneous (seismic action)



$\mathrm{R}_{c,k}$ is the lateral characteristic resistance of the nailed connection as reported in ETA-11/0086

Tension failure of the hold down

$$R_{c,d} = \frac{\mathrm{R}_{s,k}}{\gamma_{M2}} = \frac{42\ kN}{1.25} = 33.6\ kN$$

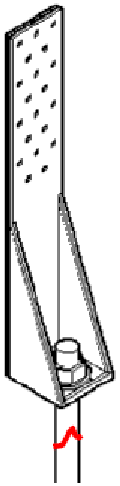


$\mathrm{R}_{s,k}$ is the characteristic value of the tensile load-carrying-capacity of the hold down (steel) as reported ETA-11/0086

γ_M is the partial safety factor for resistance of cross-sections in tension to fracture

Design of connections: hold-down

Tension failure of the threaded steel rod



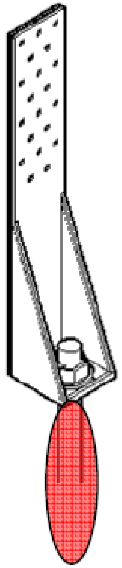
$$R_{t,d} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.9 \cdot 500\,MPa \cdot 157\,mm^2}{1.25 \cdot 1000} = 56.5\,\text{kN}$$

f_{ub} is the ultimate tensile strength of the rod steel

A_s is the tensile stress area of the threaded rod

γ_{M2} is the partial safety factor for resistance of cross-sections in tension to fracture

Pull-out resistance



$$R_{pull,d} = \frac{R_{pull,k}}{\gamma_{Mc}} = \frac{108.57\,kN}{1.8} = 60.32\,kN$$

$R_{pull,k}$ is the characteristic resistance for pull-out failure calculated for cracked concrete as reported in ETA-09/0078

γ_{Mc} is the partial safety factor assumed according to ETA-09/0078

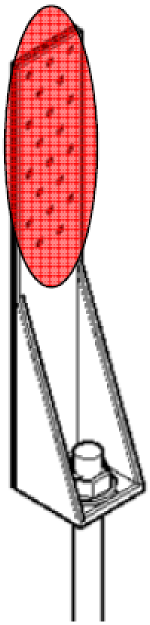
Design of connections: hold-down

Hold down resistance

The design resistance Rd of the hold-down is assumed as the minimum among the resistance values of the different failure modes

$$R_d = min.\left(R_{c,d}; R_{s,d}; R_{t,d}; R_{pull,d}\right) = 14.67\ kN$$

In this case, the failure of the element is due to the nailing.



Check of the hold-down

$$\frac{T_{Ed}}{R_d} = \frac{5.48\ kN}{14.67\ kN} = 37.4\% < 100\%$$

Design of connections: angle brackets

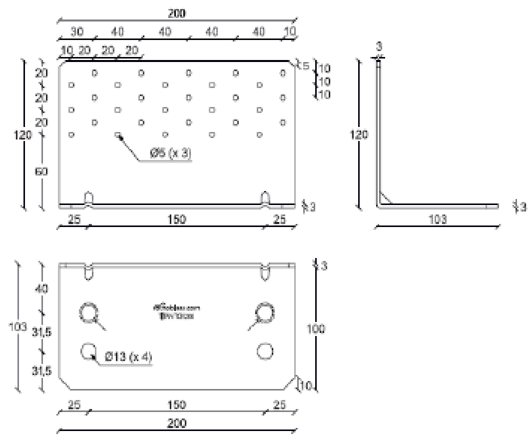
The adopted angle bracket

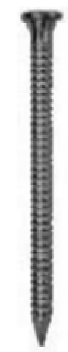
TITAN TCN 200

30 «anker» nails 4,0 x 60

2 threaded rods M16 5.8

Chemical anchor





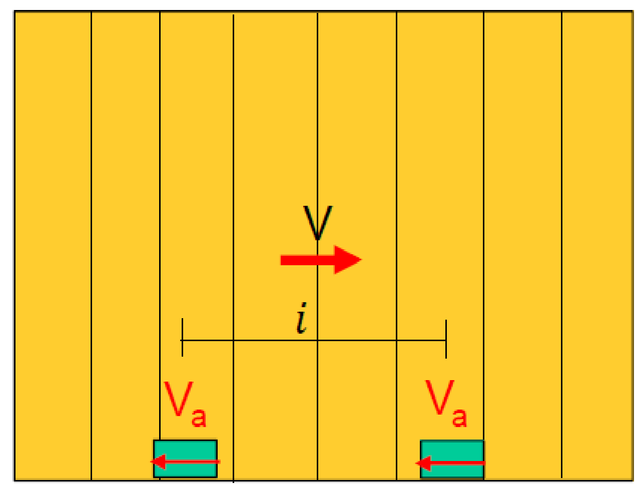
Shear force

The shear force acting on one angle bracket can be evaluated as follows

$$V_a = \frac{V}{n_{anc}} = \frac{7.17}{2} = 3.59\ kN$$

where

- V is the design shear force acting on the wall
- n_{anc} is the number of the shear connections in the wall



Design of connections: angle brackets

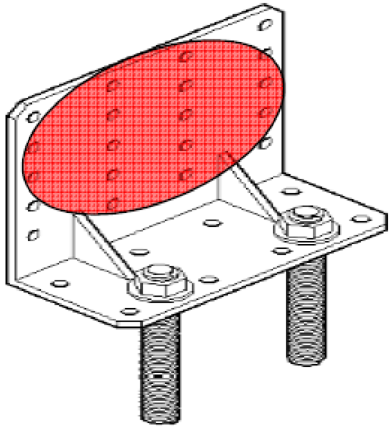
The design resistance R_d of the connection can be calculated as the minimum of the resistance values associated to the following failure modes

- Shear failure of the angle bracket and/or of the nailed connection
- Shear failure of the steel anchors

Angle bracket resistance

The design value of the connection resistance is

$$R_{a,d} = \frac{k_{mod} \cdot R_{a,k}}{\gamma_M} = \frac{1 \cdot 22.1}{1.5} = 14.7\ kN$$



$R_{a,k}$ is the characteristic value of the angle bracket resistance to lateral forces as reported in ETA-11/0496

Check of the angle bracket

$$\frac{V_{Ed}}{R_d} = \frac{3.59}{14.7} = 24.4\% < 100\ \%$$

Thank you for your attention

Roberto Tomasi

Dpt. of Civil, Environmental and Mechanical Engineering University of Trento
Via Mesiano 77 - I 38123 TRENTO
Tel. +39 0461 282529 e-mail roberto.tomasi@unitn.it

In collaboration with

Mauro Andreolli

Timbertech
Via della Villa, 22/A I-38123 – Villazzano – Trento (TN) – Italy
Tel. +39 0461 933376 e-mail info@timbertech.it\