Civil Engineering Department

Master's degree in Civil Engineering
Natural Risk Protection
Structures

Graduation thesis:

"CLT BUILDING DESIGN AND ANALYSIS WITH RFEM"

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Sommario

1. Introduction

The purpose of this report is to illustrate the thesis work carried out during the internship held in the Department of Civil Engineering of the University of Zagreb in kačićeva ulica 26, 10000 Zagreb.

During this period the design and structural analysis of a 10-storey building realized in CLT (or XLAM) was carried out which will be built in Foligno, Perugia’s province.

The modeling was done through Dlubal's RFEM software that leverages the finite elements method to analyze the behavior of the structure.

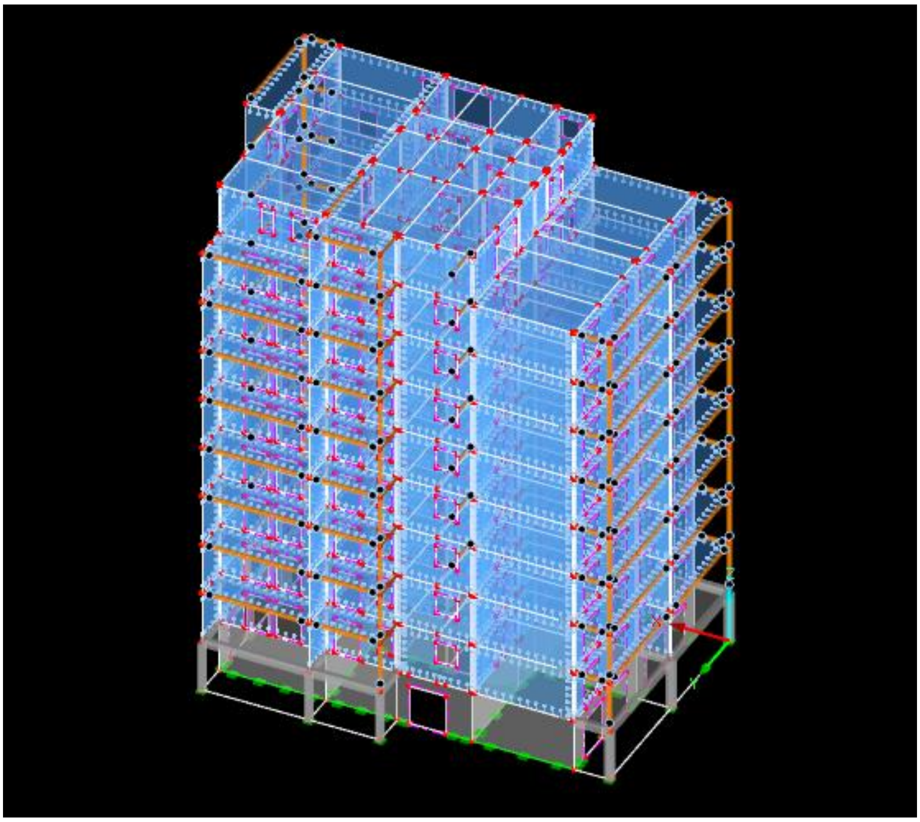


Fig 1.1: Model view on RFEM

The aim of this project was to learn new eco-friendly and environmentally friendly construction techniques in order to apply them in future Italian projects and to spread the culture of wood also in our country.

The public and private construction sector needs energy-efficient sustainable methods. In this context wood is the most important raw material and offers many advantages over other materials that don’t come from renewable sources. The wood regenerates, is recyclable and at the end of its useful cycle can be used to produce bioenergy. In addition, by containing carbon in a linked form, wood products offer an effective means of combating climate change in the long run.

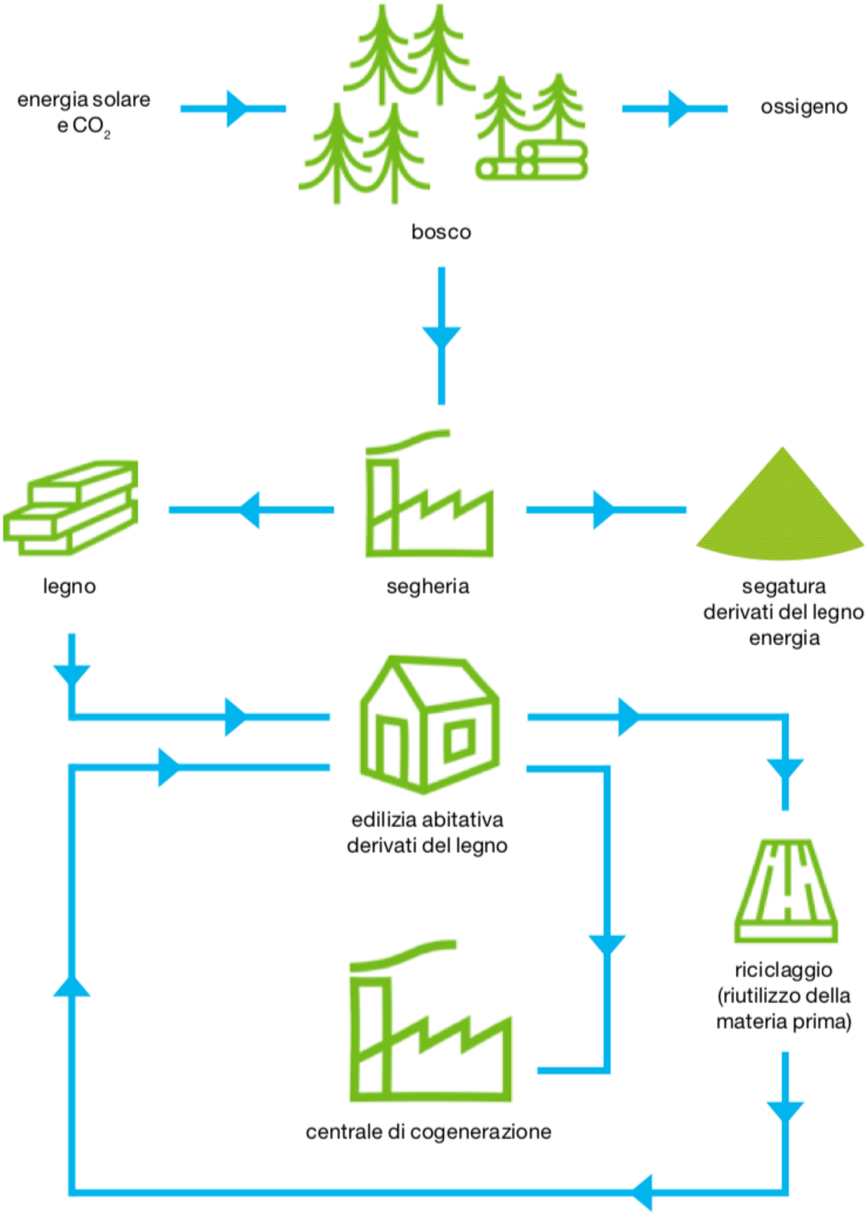


Fig. 1.2 Recycling and reuse of wood

2. Il CLT

The CLT, an acronym for Cross Laminated Timber or X-LAM, is a solid wood product consisting of a minimum of three overlapping single-layer panels and glued on top of each other so that the fiber of each layer is rotated 90 degrees relative to the layers adjacent.
Currently the maximum size produced by Stora Enso company is 2.95x16.00 m.
The CLT panels are then composed of several layers, are available in various thicknesses and are glued with early formaldehyde eco-friendly glues.

The CLT offers virtually unlimited possibilities from the design, stylistic, architectural point of point and lends itself to the construction of external walls such as interior walls, floors and roofs.

Dati tecnici CLT	
Impiego	pannelli per parete, solaio e tetto
Larghezza massima	2,95m (su richiesta fino a 4 m)
Lunghezza massima	16,00 m
Spessore massimo	320 mm
Struttura dei pannelli	3,5,7 o 8 strati
Sagomatura	qualsiasi struttura
Essenze	abete rosso, pino silvestre
Umidità del legno	12% ± 2%
Qualità estetiche	a vista, non a vista e a vista industriale
Superficie	levigata su entrambi i lati
Peso	ca 470kg per m3 di CLT
Coefficiente di resistenza alla diffusione del vapore acqueo	tra 20 e 50 μ
Conduttività termica	0,13W/(mK)
capacità termica specifica c_p	1600 J/(kgK)
classi di utilizzo	utilizzabili per le classi 1 e 2 in conformità alla norma EN 1995-1-1

The use of wood in buildings reduces CO_2 emissions

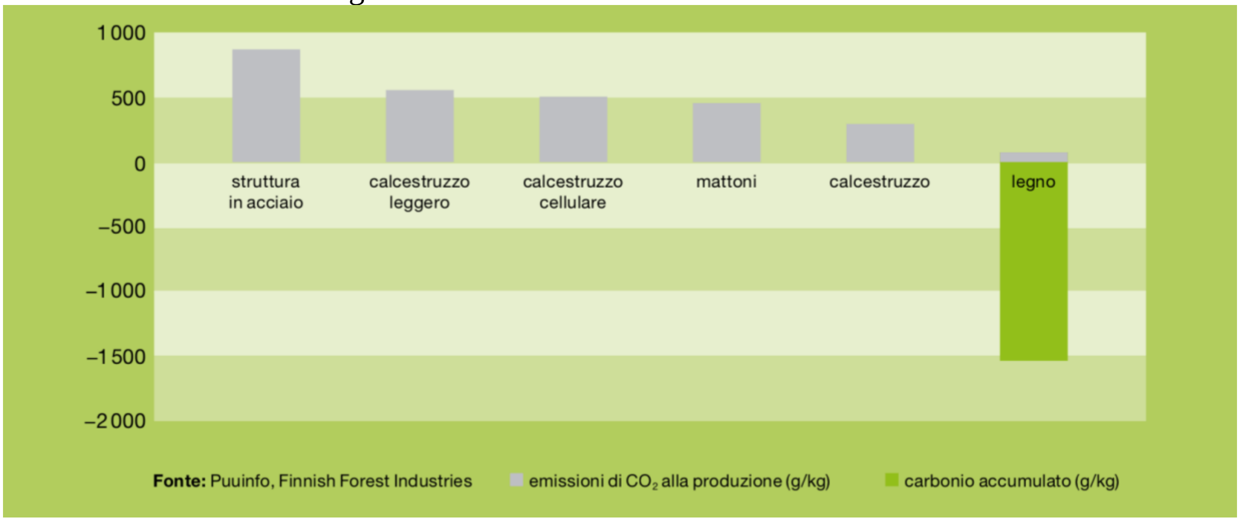


Fig. 2.1 : CO_2 compared

Aluminium, iron, steel and plastics are materials that require enormous energies for their production from the strong harmful emissions into the atmosphere that add up to the environmental damage caused by mineral extraction and paint cycles made with synthetic solvents and more disparate toxic and dangerous components. All this is highlighted daily in the media.

The emission of CO_2 in large quantities worldwide is one of the most pressing environmental problems. Raising the concentration of carbon dioxide in the Earth's atmosphere is one of the key factors that strengthens the greenhouse effect. In the long term, this leads to a general warming of the earth and climate change, unfortunately already taking place.
The solid wood has the characteristic of absorbing moisture from the air and releasing it in the presence of increased drought. For this reason, solid wood dwellings have a particularly healthy environment.

Due to their high static strength and elasticity, CLT massive wood panel buildings are also suitable for areas at seismic risk. As the solid wood is lighter than concrete, the shocks are transmitted to the structure of the building to a much lesser extent.

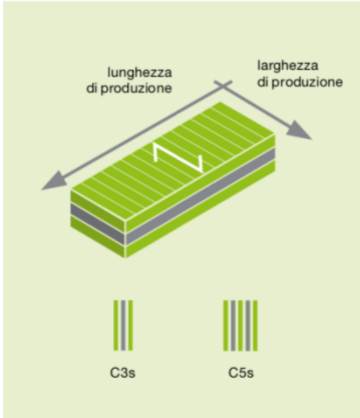
Due to their enormous static load capacity, CLT panels, thanks to the presence of layers of slats glued at crossroads allow to distribute the load along two axes. Even objects and structures to support points are perfectly achievable with the CLT.

Solid wood is more fire resistant than commonly thought. CLT has a moisture content of about 12%. Before the wood burns, it is necessary for the water contained in it to evaporate. In addition, the charred surface protects the innermost layers of the CLT, preventing the solid wood construction from collapsing.

2.1 Strutture standard del CLT

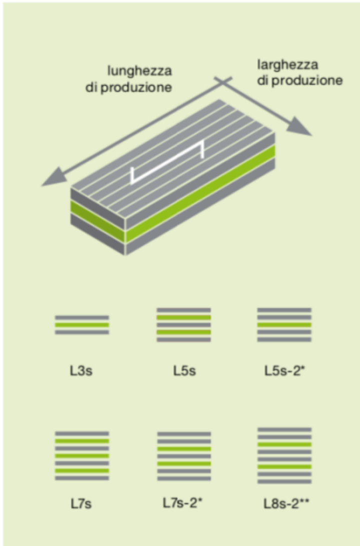
Pannelli C
Le fibre degli strati di copertura decorrono sempre in direzione parallela alla larghezza di produzione.

Spessore [mm]	Tipo di pannello [–]	Strati [–]	Struttura dei pannelli [mm]						
			C***	L	C***	L	C***	L	C***
60	C3s	3	20	20	20				
80	C3s	3	20	40	20				
90	C3s	3	30	30	30				
100	C3s	3	30	40	30				
120	C3s	3	40	40	40				
100	C5s	5	20	20	20	20	20		
120	C5s	5	30	20	20	20	30		
140	C5s	5	40	20	20	20	40		
160	C5s	5	40	20	40	20	40		



Pannelli L
Le fibre degli strati di copertura decorrono sempre ortogonalmente rispetto alla larghezza di produzione.

Spessore [mm]	Tipo di pannello [–]	Strati [–]	Struttura dei pannelli [mm]						
			L	C	L	C	L	C	L
60	L3s	3	20	20	20				
80	L3s	3	20	40	20				
90	L3s	3	30	30	30				
100	L3s	3	30	40	30				
120	L3s	3	40	40	40				
100	L5s	5	20	20	20	20	20		
120	L5s	5	30	20	20	20	30		
140	L5s	5	40	20	20	20	40		
160	L5s	5	40	20	40	20	40		
180	L5s	5	40	30	40	30	40		
200	L5s	5	40	40	40	40	40		
160	L5s-2*	5	60	40	60				
180	L7s	7	30	20	30	20	30	20	30
200	L7s	7	20	40	20	40	20	40	20
240	L7s	7	30	40	30	40	30	40	30
220	L7s-2*	7	60	30	40	30	60		
240	L7s-2*	7	80	20	40	20	80		
260	L7s-2*	7	80	30	40	30	80		
280	L7s-2*	7	80	40	40	40	80		
300	L8s-2**	8	80	30	80	30	80		
320	L8s-2**	8	80	40	80	40	80		



* strati di copertura composti da due strati longitudinali
** strati di copertura e strato centrale composti da due strati longitudinali
*** nei pannelli C la direzione di levigatura è trasversale rispetto alla fibratura

Larghezze di produzione: 245 cm, 275 cm, 295 cm
Lunghezze di produzione: dalla lunghezza minima prodotta di 8,00 m per larghezza di fatturazione fino a max. 16,00 m (con progressione di 10 cm)

Fig. 2.2: Available CLT panels

2.2 Xlam panels are available in the following types of supply:

- <u>Standard non-platoon quality</u>: used when panels are intended to be coated; there may be knots, cracks, resin bags, plate tracks. Non-visible quality is recommended only for non-visible construction elements, coated on both sides.

- <u>Quality not exposed Top</u>: for floors and roofs, with the area piled. The "non-top" quality is at an intermediate level between standard "non-standard" quality and "industrial-view" quality.

- <u>Industrial-eyed:</u>normally industrial quality is performed on one side; on request it can be performed on two sides. The industrial quality on sight is smoothanded and suitable for industrial surfaces, but not for exposed area in residential settings. Suitable for offices, industries and commercial buildings

- Residential view: For residential eye quality, AB-quality slats are used, combed and glued to the width. Residential eye quality is normally performed on the front side of the panel, it is smoothed. The new residential quality plus is also available for use of floor space and roofs. On demand you can have exposed surfaces on both sides.

2.3 Special Xlam panels on demand

- Xlam panel antiqued in three-layer spruce. Steam-treated outer layers to achieve a warm and even color tone that prevents color from changing over time.

- Three-layer spruce Xlam panel with thermal treatment for stability of size, reduced withdrawal and bulge, no additional slit formation when used outside. The application of color is more durable. High resistance to fungi and insects. En 252- Class 2 Durability.

- Xlam composite panel 5 layers: 3-layer spruce plus outer layers in 3mm MDF. Advantages: undeformable, low weight, high flow rate, light color outer layer.

- Xlam panel brushed on one or both faces

- Xlam panel impregnated transparent or colored with ecological protection

- Xlam panel treated with fire-resistant fire-fighting in class 1 of fire reaction as required by UNI 9796 and compliant with D.M.06/09/92. Wooden artifacts are normally classified in class 4 or 5 of fire reaction. The fire-retardant impregnators allow the treated Xlam panels to be retrained in class 1 of fire response, greatly slowing the spread of flames in the event of a fire. Fire protection depends on the type of building. Italian law provides for rules that indicate what services it should provide in order to limit the risk of fire within acceptable limits. In the case of large buildings, the fire requirements are different from those provided for small buildings.

- Xlam panel with longitudinal or transverse grooves relative to the direction of the slats.

2.4 Xlam without formaldehyde

In Xlam panels, the layers of slats are made to adhere using solvent-free and formaldehyde-free PUR glue, which is tested according to DIM 68141 and the strict criteria set by the Stuttgart MPS for the production of building-bearing elements. both indoors and outdoors, according to DIN 1052 and EN 301 standards.

The glue is distributed throughout the surface through an automated process that ensures an optimal amount of glue, while the high pressure exerted ensures a very high quality hold.

2.5 Advantages and disadvantages of using CLT

2.5.1 Advantages

- Short commissioning times, easy assembly and advanced prefabrication level
- 10% gain in terms of housing area by referring to a $100m_2$ dwelling.
- Lighter than concrete and bricks

- Ecological and sustainable construction technique
- Positive CO_2 balance
- Housing comfort and healthy environment climate
- Excellent firefighting behaviour
- High insulation and insulation capabilities
- Great static features
- Dry realization
- Seismic construction
- Ecological and sustainable building material with positive ecological balance
- Healthy and comfortable environment
- The durability of the wooden buildings is absolutely identical to what is expected of masonry buildings, with minimal maintenance over the years. After 40-50 years, some extraordinary maintenance work may be necessary, but also unavoidable in lateral-concrete constructions.
- Certain costs without the possibility of error
- Value stability thanks to controllable elements in terms of construction science
- Maximum freedom of architectural achievement
- Simplicity of planning of individual works and constructions
- Optimal exploitation of the surface of the property through the use of slim build elements
- Statically important elements, large-format and therefore easy to assemble
- Quality of controlled production both internally and externally
- High CNC-cut dimensional precision
- Proven quality products for absolute safety
- Comprehensive solutions thanks to the integration between different construction systems, compatible with steel, glass and other materials.
- Site advice and assistance
- Multi-year experience working with designers and businesses
- Performance Optimization
- Meeting all the specific requirements of wooden buildings (fire protection, design, energy efficiency, resistance, sound insulation, static)
- Recommended for bio-building
- More flexibility without entire-axis constraints
- Excellent static qualities
- Technically approved and CE-certified construction product
- Quality-controlled production
- Direct supply on site
- No improvisation and no errors due to inexperience or imperfection

2.5.2 Disadvantages

The disadvantages and possible defects of wooden houses depend above all:

- From the construction measures of the individual manufacturer, which will have to be aimed at avoiding any infiltration of moisture, through proper disposal of the meteoric waters and adequate waterproofing of the walls, in particular in the area of the ground attack
- From the wood quality used. If the structures are built with substandard materials, too cheap, without static-constructive attention and without thermal insulation and waterproofing they present a rapid degradation.
- The incidence of fixed costs makes the construction of very small buildings unseemly, which prove to be overly expensive compared to masonry.

- There is a whole range of small attentions that may involve unwelcome adaptations, such as the need to "detach" the building from the ground to prevent
- Employing skilled personnel

2.6 CLT or Xlam: the structural system for bio-building

In those years there has been the most important revolution in the construction sector. The need to combine the well-being needs of users, eco-sustainability, with the speed of construction, and above all to optimize costs are leading to the adoption of the Xlam construction system. The possibility of having the building ready in a very few months is in fact attracting more and more users and operators in the sector. Key elements of this revolution are the Xlam panels that offer many advantages and are flanked by the wall constructions framed (Platform frame), very widespread wooden construction technology.

The appearance of the Xlam panels has allowed the introduction of large, flat and massive structural elements into the wooden buildings. The possibility of building with a supporting panel structure with plate and slab functions was added.

The supporting structure of a building in Xlam combines elements with slab behavior and elements connected to each other in order to form three-dimensional structures.

The CLT floors allow more lights than other types of wooden constructions, being particularly suitable for multi-storey residential buildings and for office or commercial use. The Xlam system allows you to make elements with bending carrier capacity in both directions of the plane. It can be interesting to take advantage of this feature for corner element with overhangs in both directions.

The vertical Xlam wall can be seen as a pillar of continuous length. Xlam panels make it easy to build cantilevered structures supported by the walls. This reduces the structural elements size and increases design flexibility. The vertical and horizontal brace are also almost automatically integrated into the structure.

For multi-storey buildings (more than two floors), the best structural system is the one in Xlam.

Xlam panels have total dimensional stability. This allows you to use large panels and makes it possible to work precision in prefabrication without having to consider important tolerances for variations in material size.

Xlam wood panels are used as reinforcement elements but also as non-supporting elements for various types of construction:

- Xlam single-family and multi-family homes
- Multi-storey residential buildings in Xlam
- Public buildings in Xlam
- Hotels and restaurants in Xlam
- Retirement homes in Xlam
- Schools and kindergartens in Xlam
- Administrative buildings and office buildings in Xlam
- Pavilions, ticket offices, info-points for demonstrations in Xlam
- Industrial and commercial construction in Xlam
- Xlam elevations and expansions
- Xlam Bridges

- Exhibition and structural mussal structures

2.7 Xlam supply on building site

The Xlam supply takes place directly on building site.
When you receive the order confirmation, the Xlam panels are produced and cut for the project in question. Once the cuts are made, the panels are loaded on TIR and delivered to the site.

The cutting and molding are specially tailored at the plant, with the latest CNC technology, based on approved production projects and cutting plans.

2.8 Xlam mounting

The well-tailored Xlam elements are delivered to the site and mounted quickly by cranes or lifts by skilled woodconstruction workers.

Xlam panels can be sawn, milled, planed and boarded with all the usual woodworking machines, both stationary and manual.
Assembly time for a 4-story residential building in Xlam:

Total sqm Xla, wall panels	2400 mq
Total smq Xlam slab panels	1520 mq
Time for building assembly	15 giorni

Tempi di montaggio per una casa unifamiliare di 2 piani in Xlam:

Total sqm wall panels Xlam	314 mq
Total smq Xlam slab panels	233 mq
Time for building assembly	2giorni e 1/2

2.9 Xlam seismic contruction system

Xlam is an earthquake-resistant construction system that resists the shock waves of a 100% earthquake: ideal for construction in seismic zone. A house in Xlam holds up better than a concrete one.

In this moment, it is universally recognized buildings in Xlam provide maximum levels of safety and earthquake resistance. The structural system consisting of massive panels of wood glued in cross layers is suitable for the construction of earthquake-resistant buildings and represents the application of state-of-the-art and technical in safe constructions in multi-storey buildings.

Xlam Walls and floors form a box structure of considerable rigidity and structural robustness, allowing to give up the use of pillars and thus avoiding the concentration of agent forces on the structure and foundations. The result is a more robust and less sensitive structure in the case of seismic stresses.

Structural ductility ensures the possibility of energy dissipation of cyclical actions due to the seismic event. The energy of the earthquake is absorbed by the connecting metal elements (plates, joints, special screws, etc.) are released and plasticized without their rupture thanks to their ductile behavior. Buildings with this feature can benefit from a smoothing and plasticization phase without collapse.

Timber is a cost-effective material for seismic construction due to the reduced mass compared to the carrying capacity (the mass-strength ratio is similar to steel structures rather than concrete structures). This means that in the event of an earthquake the stresses on a wooden construction are lower because they are proportional to the mass of the building itself

The timber structures are more flexible than similar structures, made of reinforced concrete or masonry and this is an additional advantage, as a flexible structure better resists the dynamic stresses resulting from an earthquake.

The timber building is never a monolithic body, but is formed by several elements (beams, walls, floors), joined together through metal connections. If these connections are well designed and executed, can make an extremely favourable contribution to the overall behaviour of the building, thanks to the plastic deformations of the metal elements and the friction between the contact surfaces, allowing dissipate significant amounts of energy developed during the earthquake.

Proper design of the supporting structures is essential, taking care of the sizing of all wooden components and the sizing of metal carpentry (hold down, plates, screws, etc.)

A Xlam seismic capacity example is the prototype Sofie made by the CNR as a 100% earthquake-proof house: 7 floors and 21 meters high, entirely in Xlam as an earthquake-proof house; it was developed by Ivalsa-Cnr together with the Province of Trento. The prototype was subjected to the simulation of an earthquake considered among the most dangerous and destructive for civil works: the Kobe earthquake (magnitude 7.2 on the Richter scale), which in 1995 caused the death of more than 6,000 people. The Sofie palace withstood the incredible force of shock without causing damage, coming out unhurt in the mechanical parts and the supporting structure even from a fire resistance test that saw it engulfed in flames for over an hour. The seismic test is the final result of five-year studies and research, which have identified the combination of Xlam and the building's mechanical connections as the best construction technique against earthquakes.

2.10 REI fire resistance certified

In the fire simulation with the ONORM EN 1365-2 test, the Xlam panel slab 5 layers of 140mm resists at least one and a half hours (REI Xlam 90), while the 100mm 5-layer panel lasts an hour (REI60), retaining its mechanical properties and leaving the supporting structure, with performance completely permissible to those of buildings in supporting masonry, and behaves even better than those in reinforced concrete.

Wood is a combustible material, but this doesn’tt mean wooden structures don’t withstand fire or that they suffer more damage than a steel or reinforced concrete structure. It rarely happens that wooden structures act as fuel to fires, but suffer the consequences, manifesting better behavior than other materials. In fact, if you look at the behaviour of a fire-prone structural element, you can see thatWood is a combustible material, but this does not mean that wooden structures do not withstand fire or that they suffer more damage than a steel or reinforced concrete structure. It rarely happens that wooden structures act as fuel to fires, but suffer the consequences, manifesting better behavior than other materials. In fact, if you look at the behaviour of a fire-prone structural element, you can see that:

- The wood burns slowly and the charring process proceeds from the outside to the inside;
- The wood that has not yet burned preserves structural efficiency despite the increase in temperature;

- The rupture is achieved slowly, only when the useful section has been so small that it cannot withstand the load applied.

Thus the loss of efficiency of the fire-prone wooden structure occurs by reduction of the useful section and not by physical-mechanical degradation.

Comparing the behaviour of fire-prone wood with those of other building materials, it is observed that wood offers several advantages:

- Structural steel elements do not burn but the increase in temperature leads to a dangerous increase in ductility and therefore deformations;
- In reinforced concrete constructions, fire resistance depends almost exclusively on the thickness of the cover.

2.11 Xlam economic convenience

The Xlam using cost-effectiveness is the reduction in construction time.

One of the prejudices that precludes the adoption of wood – especially among professional operators in the sector such as designers, companies, real estate developers – is also the supposed higher cost of the Xlam system compared to traditional construction techniques. Anyway, from the experiences developed at European level and confirmed also in our country, it is clear that building in Xlam has a very competitive average cost of construction.

If we consider the time required to build a building with Xlam panels compared to those of a similar structure in c.a. and brick we can see the time of Xlam construction is reduced by 40-50% with significant cost savings of labour, financial burdens, the business issue, with the advantage of having the building –for sale, renting or to live there – much earlier than the traditional system allows.

For many and indisputable advantages offered, the Xlam construction techniques are increasingly adopted in design studios of great prestige, leading to very interesting realizations from the point of view of architectural quality.

2.12 End of life recovery

Thanks to the use of formaldehyde-free glues, Xlam wood panels can be reused after renovation or dismantling.

An energy enhancement at controlled incinerators for energy production is process or possibly even electricity (thermoelectric generators) is considered particularly advantageous thanks to the high calorific power of the wood.

Xlam wood elements can be reused after a dismantling operation. If this is not possible, these elements are to be used for energy enhancement.

The waste disposal code in accordance with the ONORM S2100 waste catalogue rule: 17218 (wood waste, organic treatment).

Landfill is not allowed.

3 Project description

3.1 Territorial framing

Foligno is the third largest city in Umbria, located in the province of Perugia and has about 60,000 inhabitants. The city is the most important commercial and industrial center in the area. The municipality is mainly mountainous, while the town is located at 234m above sea level and extends over a flat area in the middle of the Umbrian valley,surrounded by the Apennine ridge on the Tyrrhenian side and thecolfiorito highlands on the Adriatic side. From a hydrographic point of view the area is included within the Tiber basin and is located at the confluence of the rivers Topino, which crosses the city, and Menotre.

The precise geographic coordinates of the area of interest are:

42.955 N, 12.704 E

Fig. 3.1: Territorial framing

The entire municipality is in an area of high seismic risk. According to the classification parameters provided by P.C.M. No.3274 of 20/03/2003, the area under consideration falls into Zone 1, the so-called "catastrophic risk".

3.2 Surface category and topographical conditions

For the purposes of defining the seismic project action, it is necessary to assess the effect of the local seismic response by means of specific analyses, as stated in Regulation [7.11.3 NTC18]. In the absence of such analyses, a simplified approach can be used for the definition of seismic action, which is based on the identification of reference categories [tab 3.2.II NTC18].
The category of subsoil in this case falls into category B.

Tab. 3.2.II – *Categorie di sottosuolo che permettono l'utilizzo dell'approccio semplificato.*

Categoria	Caratteristiche della superficie topografica
A	*Ammassi rocciosi affioranti o terreni molto rigidi* caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	*Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti*, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 m/s e 800 m/s.
C	*Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti* con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 180 m/s e 360 m/s.
D	*Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti*, con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	*Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D*, con profondità del substrato non superiore a 30 m.

3.3 Structural configuration

The following criteria were taken into account when designing the building:

- Structural simplicity
- Consistency and symmetry
- Distribution of uniform strength and stiffness in the two main directions
- Torsional resistance and stiffness

3.4 Model description

This building has 10 floors and the total high is 30m. The ground floor is designed in concrete while the remaining floors are in CLT. Each floor also has a concrete slab of about 5-7cm depending on the span and the floor referred to; this slab has been inserted in order to increase the rigidity of the structure.

Vengono riportate di seguito le differenti planimetrie dell'edificio:

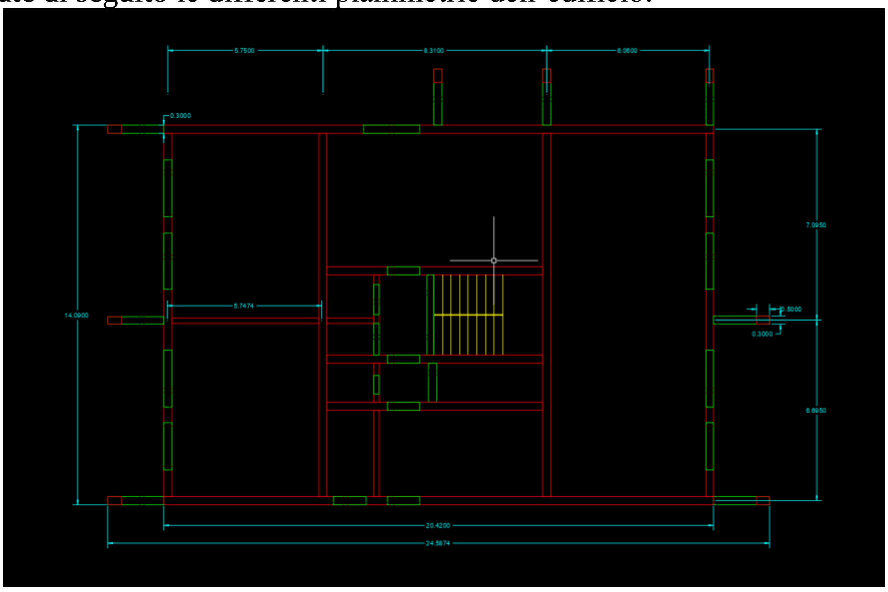


Fig. 3.2 Concrete ground floor plan

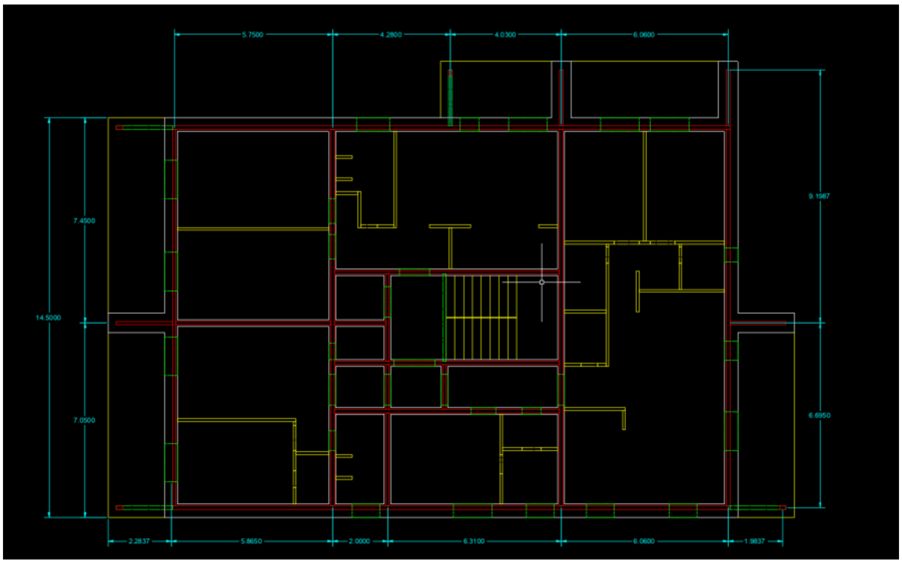


Fig. 3.3 CLT 1-8 Floor Plan

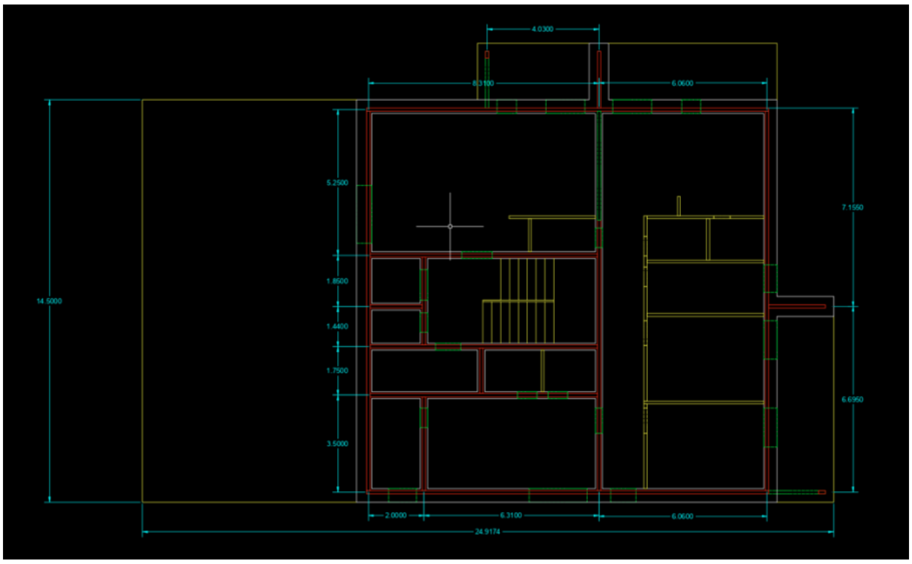


Fig. 3.4 CLT Top floor plan

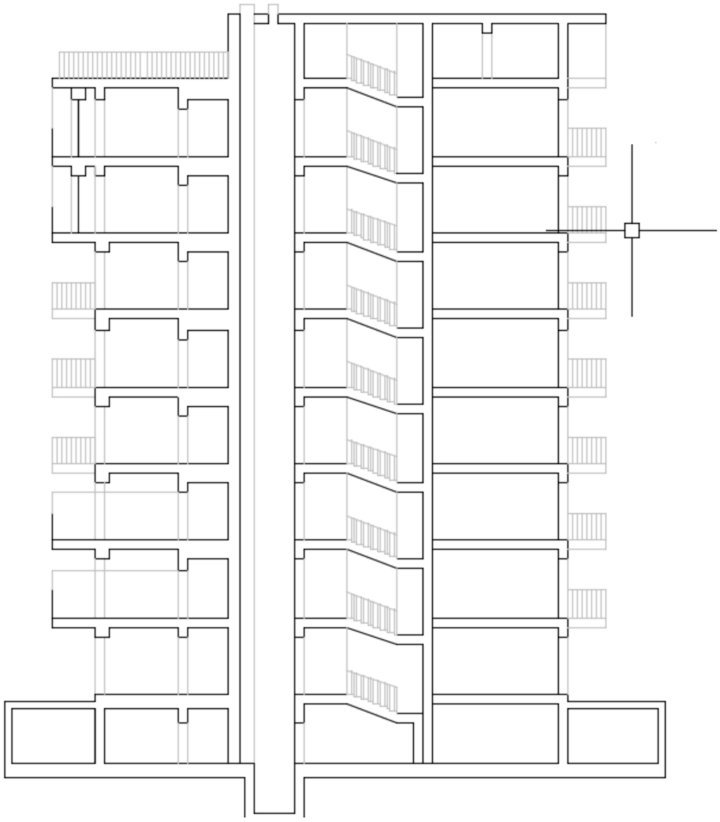


Fig 3.5 Prospect

The plan foundation is located at a depth of 2m and this allows you to overcome any layering of debris and reports of soils of poor characteristics, to overcome the layer of soil subject to the actions of frost and thaw and to get to safety surface water.

This plan has been considered in reinforced concrete and for the design is referred to specific courses. The concrete used for the ground floor and for the slab thrown to each floor is a C30/37 which has the following characteristics:

4 Loads an external actions analysis

4.1 Dead loads

The dead loads calculation was carried out considering a slab band of 1m.
The dead loads inserted into the model on RFEM are:

- CLT interstore floor dead load = 1,25 kN/m2
- CLT balcony dead load = 1,5 kN/m2
- Concrete interstore floor dead load =1 kN/m2

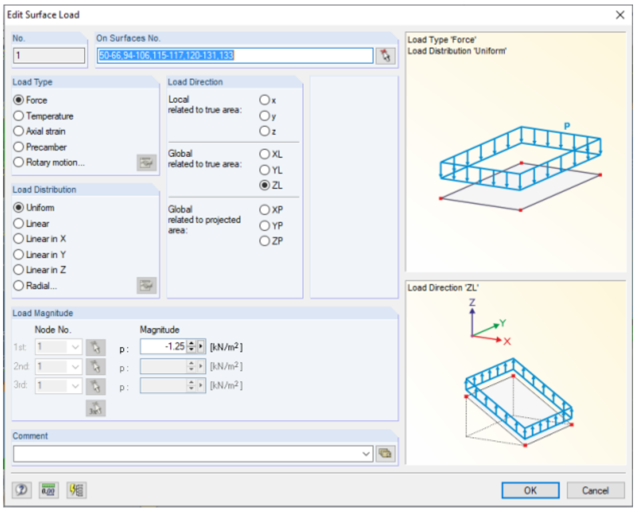


Fig.4.1 CLT interstore floor dead load

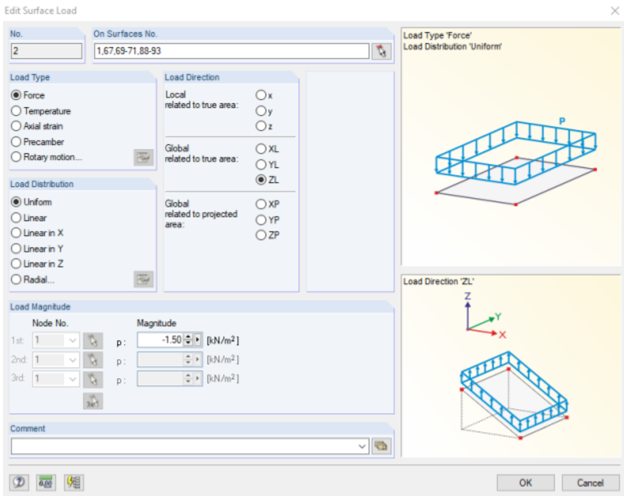


Fig 4.2 CLT balcony dead load

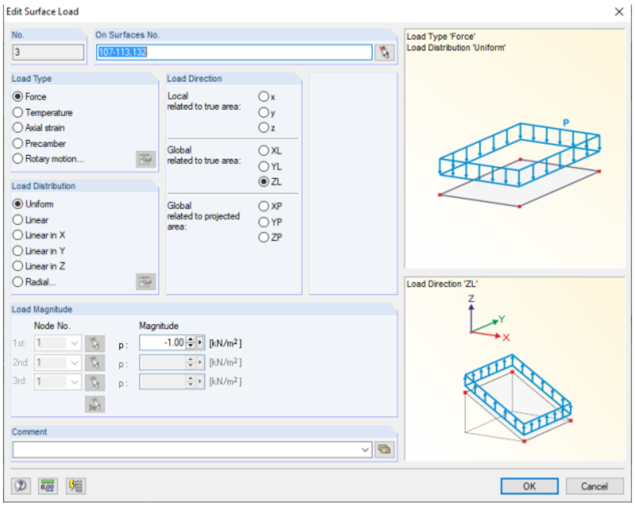


Fig 4.3 Concrete interstore floor dead load

In addition to the loads distributed at m_2, linear loads have been placed on the ladder of linear loads in both the CLT floor and the concrete ground floor.

In CLT plans, the linear load on the scale compartment is 2.61 kN/m obtained as follows:

$$1.3095\ \mathrm{m}\cdot(1.25\ \mathrm{kN/m}_2 + (0.2\ \mathrm{m}\cdot 380\ \mathrm{kg}\cdot 9.81\mathrm{m/s}_2)/1000) = 2.61\mathrm{kN/m}$$

where 1.3095m is half the length of the stairwell
0.2m is the actual thickness of the scale
($380\ \mathrm{kg}\cdot 9.81\mathrm{m/s}_2)/1000 = 3.73\ \mathrm{kN/\ m}_2$ is the specific weightof The CLT

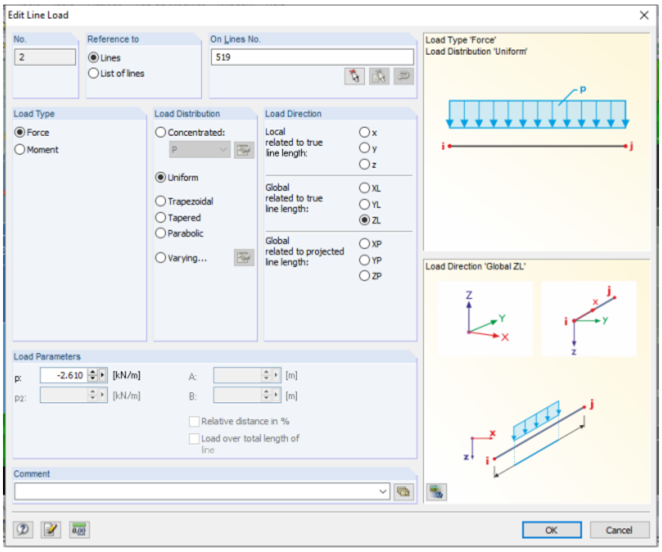


Fig. 4.4 Linear load on the CLT staircase

In the concrete plane, the linear load on the stairwell is 7.86 kN/m obtained as follows:

$$1.3095\ \text{m}\cdot(1\ \text{kN/m}^2 + 0.2\ \text{m}\cdot 25\ \text{kN/m}^3) = 7.68\ \text{kN/m}$$

where 1.3095m is half the length of the staircase
0.2m is the actual thickness of the staircase
25 kN/m3 is the specific weight of the concrete

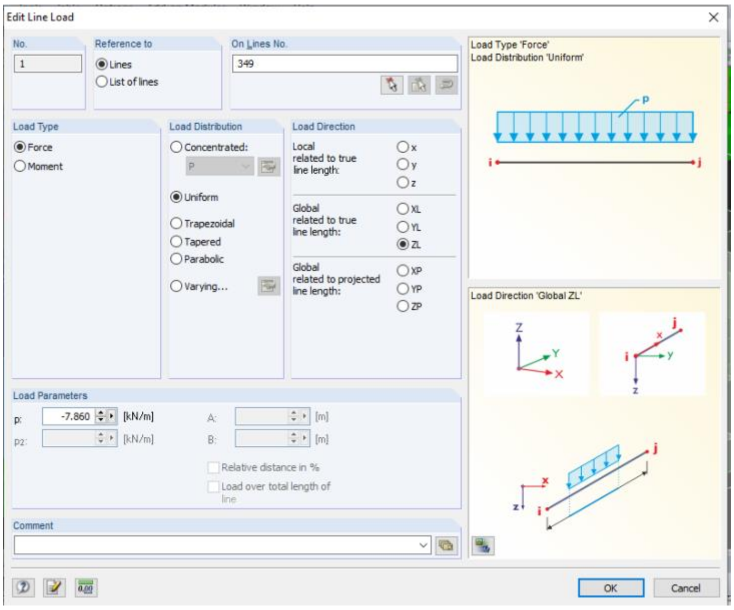


Fig. 4.5 Concrete staircase linear load

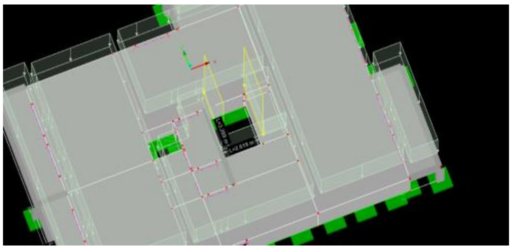


Fig. 4.6 Staircase position

The two newly seen loads were also applied to the wooden beam of the stairwell as "members loads":

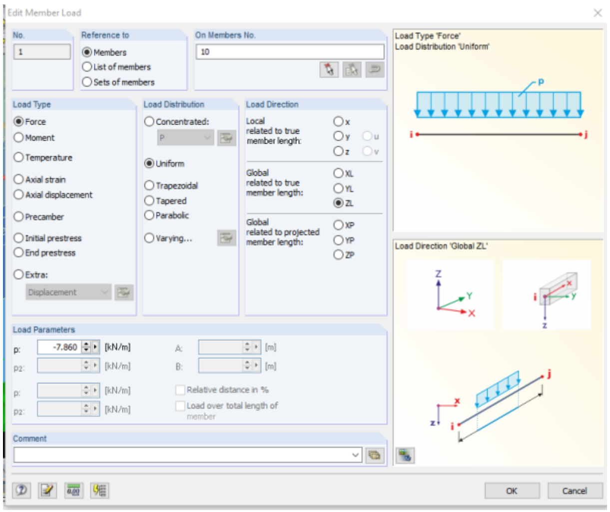


Fig. 4.6 Concreate staircase linear load

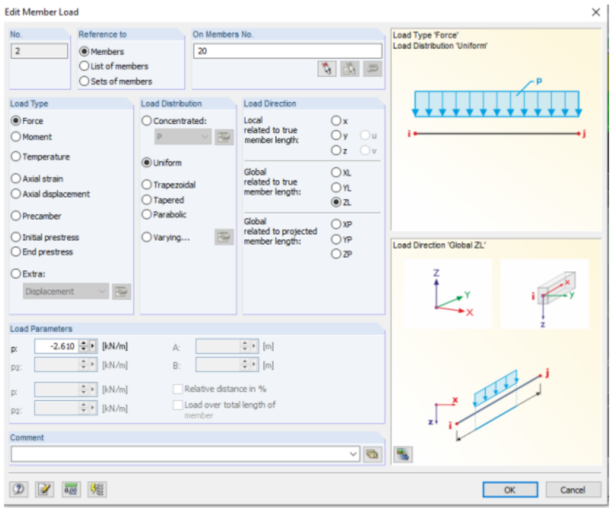


Fig. 4.7 CLT staircase linear load

4.2 Live loads

In step 3.1.4 of the legislation, variable loads related to the use of the premises of the structure are dealt with:

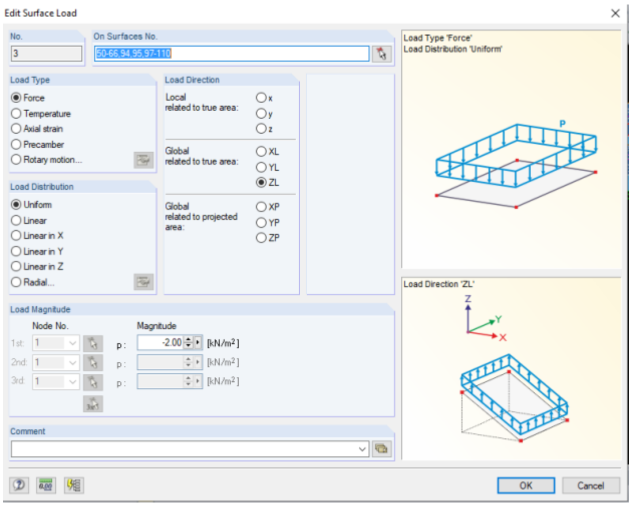
Tabella 3.1.II – *Valori dei carichi d'esercizio per le diverse categorie di edifici*

Cat.	Ambienti	q_k [kN/m²]	Q_k [kN]	H_k [kN/m]
A	**Ambienti ad uso residenziale.** Sono compresi in questa categoria i locali di abitazione e relativi servizi, gli alberghi. (ad esclusione delle aree suscettibili di affollamento)	2,00	2,00	1,00
B	**Uffici.** Cat. B1 Uffici non aperti al pubblico Cat. B2 Uffici aperti al pubblico	 2,00 3,00	 2,00 2,00	 1,00 1,00
C	**Ambienti suscettibili di affollamento** Cat. C1 Ospedali, ristoranti, caffè, banche, scuole Cat. C2 Balconi, ballatoi e scale comuni, sale convegni, cinema, teatri, chiese, tribune con posti fissi Cat. C3 Ambienti privi di ostacoli per il libero movimento delle persone, quali musei, sale per esposizioni, stazioni ferroviarie, sale da ballo, palestre, tribune libere, edifici per eventi pubblici, sale da concerto, palazzetti per lo sport e relative tribune	 3,00 4,00 5,00	 2,00 4,00 5,00	 1,00 2,00 3,00
D	**Ambienti ad uso commerciale.** Cat. D1 Negozi Cat. D2 Centri commerciali, mercati, grandi magazzini, librerie…	 4,00 5,00	 4,00 5,00	 2,00 2,00
E	**Biblioteche, archivi, magazzini e ambienti ad uso industriale.** Cat. E1 Biblioteche, archivi, magazzini, depositi, laboratori manifatturieri Cat. E2 Ambienti ad uso industriale, da valutarsi caso per caso	 ≥ 6,00 —	 6,00 —	 1,00* —
F-G	**Rimesse e parcheggi.** Cat. F Rimesse e parcheggi per il transito di automezzi di peso a pieno carico fino a 30 kN Cat. G Rimesse e parcheggi per transito di automezzi di peso a pieno carico superiore a 30 kN: da valutarsi caso per caso	 2,50 —	 2 x 10,00 —	 1,00** —
H	**Coperture e sottotetti** Cat. H1 Coperture e sottotetti accessibili per sola manutenzione Cat. H2 Coperture praticabili Cat. H3 Coperture speciali (impianti, eliporti, altri) da valutarsi caso per caso	 0,50 secondo categoria di appartenenza —	 1,20 —	 1,00 —
* **	non comprende le azioni orizzontali eventualmente esercitate dai materiali immagazzinati per i soli parapetti o partizioni nelle zone pedonali. Le azioni sulle barriere esercitate dagli automezzi dovranno essere valutate caso per caso			

Live loads values (NTC 2018)

<u>Residential loads</u>

- Interstore floor residential load = 2 kN/m2
- Balcony residential load = 4 kN/m2
- Staircase floor residential load = 3 kN/m2



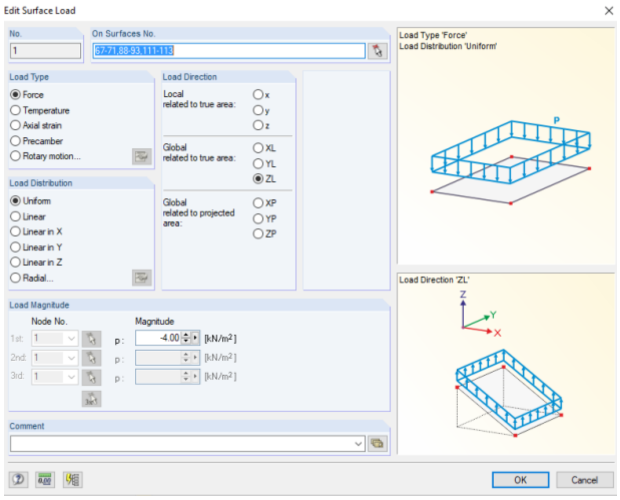


Fig. 4.8 Interstore floor residential load   Fig. 4.9 Balcony residential load

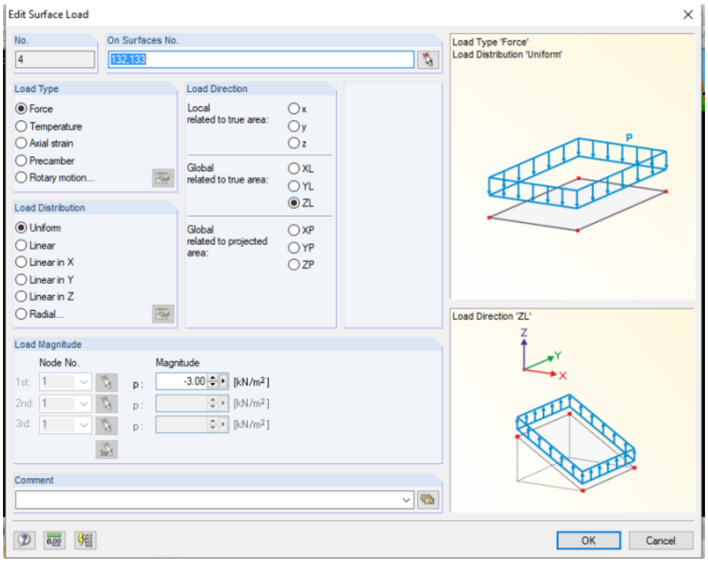


Fig. 4.10 Staircase floor residential load

In addition to the destructiond loads at m_2, linear loads were placed on the scale seal in both the CLT floor and the concrete ground floor in both cases calculated as follows:

$$1.3095m \cdot 3kN/m_2 = 3.93 \ kN/m$$

where 1.3095m is half the length of the staircase
3 kN/m_2 is the live load applied

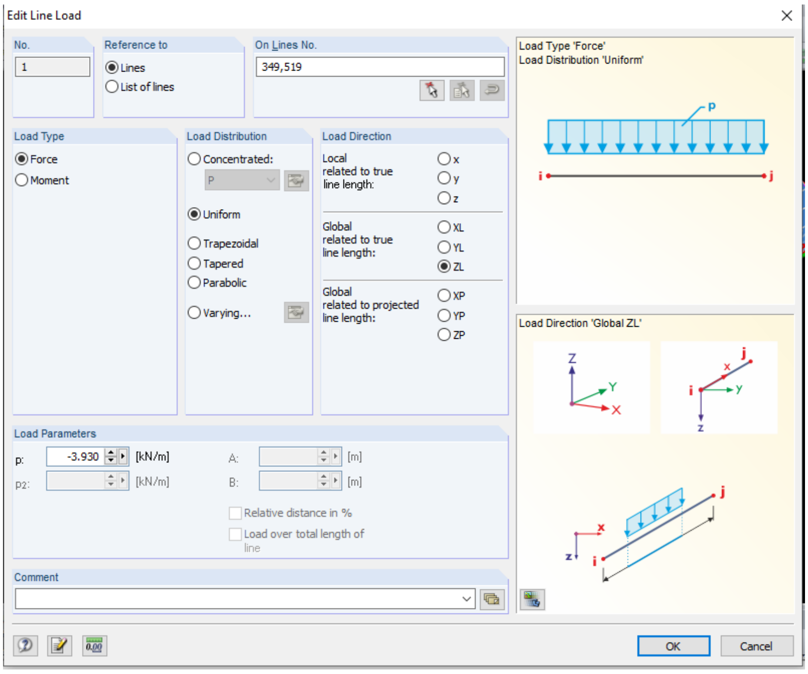


Fig. 4.11 Concreate staircase linear load

The two newly seen loads were also applied to the wooden beam of the stairwell as "members loads":

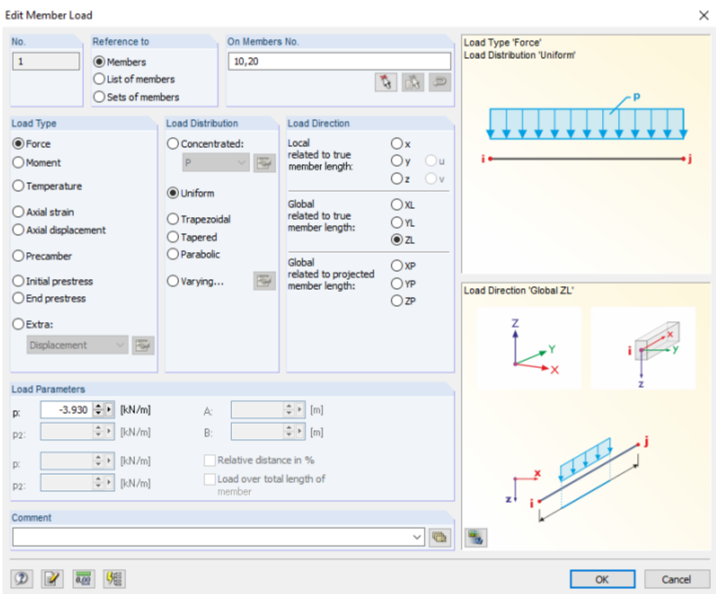


Fig. 4.12 CLT staircase linear load

<u>Snow load</u>

Snow load = 0.854 kN/m2

This live load acts on the balcony and roof ingese and is calculated in thesame way for both following the constraints dictated by the Regulations.

Il carico provocato dalla neve sulle coperture sarà valutato mediante la seguente espressione:

$$q_s = \mu_i \cdot q_{sk} \cdot C_E \cdot C_t \qquad (3.3.7)$$

dove:

q_s è il carico neve sulla copertura;

μ_i è il coefficiente di forma della copertura, fornito al successivo § 3.4.5;

q_{sk} è il valore caratteristico di riferimento del carico neve al suolo [kN/m^2], fornito al successivo § 3.4.2 per un periodo di ritorno di 50 anni;

C_E è il coefficiente di esposizione di cui al § 3.4.3;

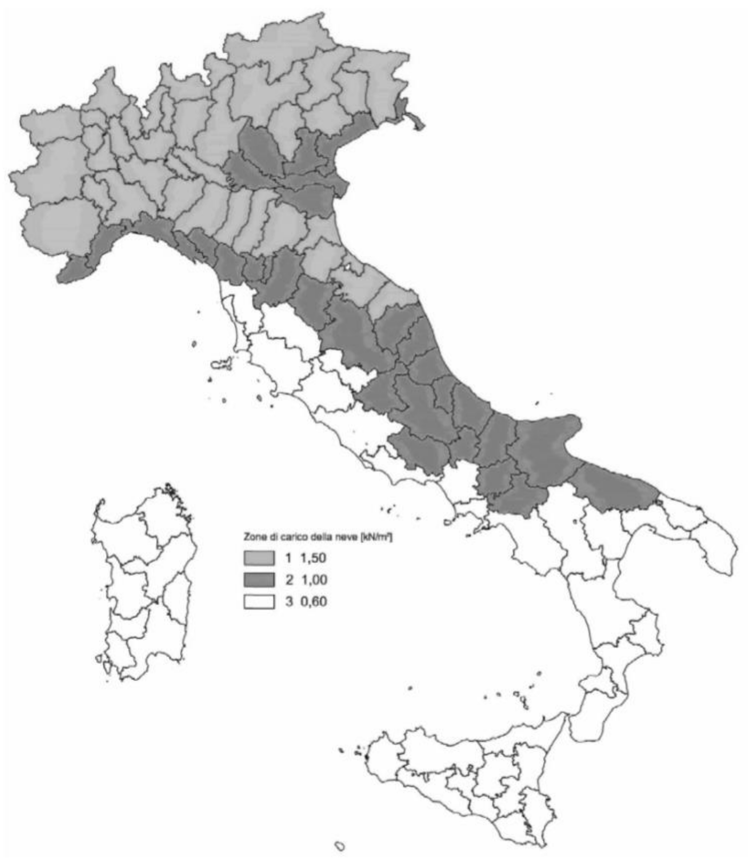
C_t è il coefficiente termico di cui al § 3.4.4.

Si ipotizza che il carico agisca in direzione verticale e lo si riferisce alla proiezione orizzontale della superficie della copertura.

3.4.2. VALORE DI RIFERIMENTO DEL CARICO DELLA NEVE AL SUOLO

Il carico della neve al suolo dipende dalle condizioni locali di clima e di esposizione, considerata la variabilità delle precipitazioni nevose da zona a zona.

In mancanza di adeguate indagini statistiche e specifici studi locali, che tengano conto sia dell'altezza del manto nevoso che della sua densità, il carico di riferimento della neve al suolo, per località poste a quota inferiore a 1500 m sul livello del mare, non dovrà essere assunto minore di quello calcolato in base alle espressioni riportate nel seguito, cui corrispondono valori associati ad un periodo di ritorno pari a 50 anni per le varie zone indicate nella Fig. 3.4.1. Tale zonazione non tiene conto di aspetti specifici e locali che, se necessario, devono essere definiti singolarmente.



Zona II

Arezzo, Ascoli Piceno, Avellino, Bari, Barletta-Andria-Trani, Benevento, Campobasso, Chieti, Fermo, Ferrara, Firenze, Foggia, Frosinone, Genova, Gorizia, Imperia, Isernia, L'Aquila, La Spezia, Lucca, Macerata, Mantova, Massa Carrara, Padova, Perugia, Pescara, Pistoia, Prato, Rieti, Rovigo, Savona, Teramo, Trieste, Venezia, Verona:

$$q_{sk} = 1{,}00 \text{ kN/m}^2 \qquad a_s \leq 200 \text{ m}$$

$$q_{sk} = 0{,}85\,[1 + (a_s/481)^2] \text{ kN/m}^2 \qquad a_s > 200 \text{ m} \qquad [3.4.4]$$

a_s	234
Q_{sk} [kN/m2]	1.067
μ_i	0.8
C_E	1
C_t	1
q_s [kN/m2]	0.854

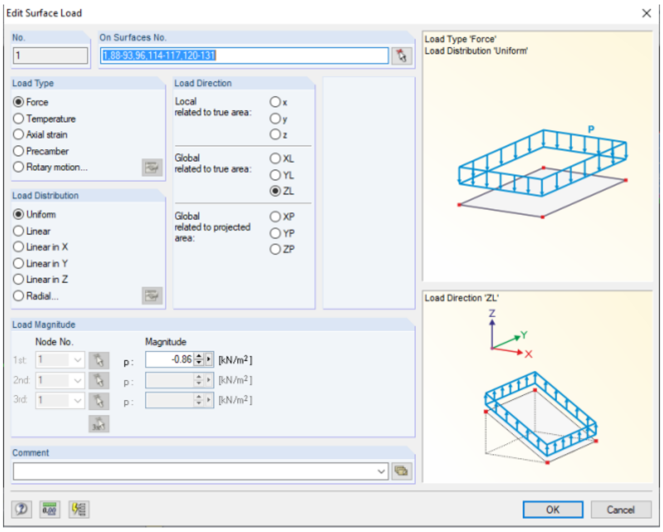


Fig. 4.13 Snow load

Roof load

With regard to the loads applied on the roof slab we have the permanent load, the snow load and the reduced residential load of 0.6 as it is expected that in case of snow there is no total crowd load.

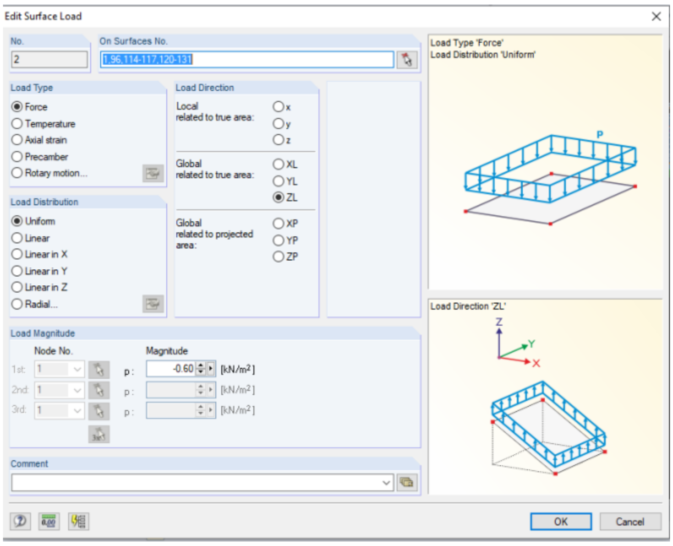


Fig. 4.14 Roof load

Wind load

During the design it was decided to neglect the wind load because, since the seismic action is far greater than that of the wind, it allows to overlook the effect of the latter.

5 Seismic action

5.1 Identifying the site's hazard (FASE 1)

The seismic project actions on the buildings are defined from the basic seismic hazard of the site, which is the primary knowledge element for the determination of the seismic actions themselves.
In this first phase it is therefore necessary to locate the coordinates of the site on which the structure is to be built in the national territory.
The longitude and the reference latitude for this project are those of the municipality of Foligno in the province of Perugia, Umbria:

Longitude 12th 70
Latitude 42-94

The values of the seismic actions obtained are the result of interpolation of the values measured in the nearest national mesh nodes as a site as can be seen at the bottom left of the table below. It has included the coordinates for the region, province and municipality and with this phase 1 has ended successfully.

Fig 5.1: Fase 1: Identifying the site's hazard

5.2 Choosing of strategy design (FASE2)

- Nominal life and reference period

The seismic project actions, on the basis of which to assess the respect of the different limit states considered, are defined from the basic seismic hazard of the construction site. It is the primary knowledge element for the determination of seismic actions. Seismic hazard is defined in terms of the maximum horizontal acceleration expected at_g in free field conditions on a rigid reference site with horizontal topographic surface, as well as ordered elastic response spectrum in acceleration to it corresponding $S_{and}(T)$, referring to fixed probability of surplus P_{VR} (Table 9.2), inthe reference period V_R, whichis obtained, for each type of construction, multiplying the nominal life V_N by the usage coefficient C_U:

$$V_R=V_N \cdot C_U$$

The value of the Use Coefficient$_{C\,U}$ is defined as the class of use changes as shown in Table 9.1

Classe d'uso	I	II	III	IV
Coefficiente C_U	0.7	1	1.5	2.0

Use coefficient values C_U

If $V_R \leq 35$ years still stands V_R =35 years
Because the structure falls into the ordinary structure class:
V_N= 50 years
As a result, the reference period applies:
$V_R=V_N*C_U=50*1=$ 50 anni

- Limit states and their probability of overcoming

Known the reference period, the seismic actions of the project, on the basis of which to assess the respect of the limit status considered, are defined from the basic hazard of the construction site. It is defined in terms of orders of the elastic response spectrum in acceleration If, with reference to certain probability of surplus P_{VR}, in the reference period V_R.
The NTC has four limit states, two operating states and two last states. For class of use II, compliance with all the last limit states is considered to be achieved if the checks relating to the SLV (State Limit of Life Protection) are carried out. The probability of exceeding in the P_{VR} reference period (probability of surplus), to be referred to to detect the agent seismic action in each of the limit states considered are shown:

Stati limite		P_{VR}: Probabilità di superamento nel periodo di riferimento Vr
Stati limite di esercizio	SLO	81%
	SLD	63%
Stati limite ultimi	SLV	10%
	SLC	5%

Therefore, the probability of exceeding in the reporting period, for an SLV check is:

P_{VR}= 10%

The return period of the TR seismic action is therefore:

$$T_R = -\frac{V_R}{\ln(1-P_{VR})} = -\frac{50}{ln(1-0.10)} = 475\ anni$$

STEP 2 summary table

Figura5.2 Fase 2: Choosing of strategy design

a_g, F_0, T_C* values for reference T_R return periods

T_R [anni]	a_g [g]	F_o [-]	T_C^* [s]
30	0,073	2,404	0,272
50	0,095	2,345	0,279
72	0,111	2,345	0,284
101	0,129	2,341	0,288
140	0,147	2,345	0,292
201	0,169	2,354	0,297
475	0,231	2,406	0,313
975	0,293	2,419	0,325
2475	0,390	2,415	0,340

Fig. 5.3 Parameters for reference return periods

5.3 Project action design (FASE 3)

5.3.1 Subsurface category

In the absence of specific analyses of the local seismic response, a simplified approach based on the identification of reference subsurface categories can be referred to for the definition of it.

Categoria	Descrizione
A	*Ammassi rocciosi affioranti o terreni molto rigidi* caratterizzati da valori di $V_{s,30}$ superiori a 800 m/s, eventualmente comprendenti in superficie uno strato di alterazione, con spessore massimo pari a 3 m.
B	*Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti* con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ compresi tra 360 m/s e 800 m/s (ovvero $N_{SPT,30} > 50$ nei terreni a grana grossa e $c_{u,30} > 250$ kPa nei terreni a grana fina).
C	*Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti* con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ compresi tra 180 m/s e 360 m/s (ovvero $15 < N_{SPT,30} < 50$ nei terreni a grana grossa e $70 < c_{u,30} < 250$ kPa nei terreni a grana fina).
D	*Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti*, con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ inferiori a 180 m/s (ovvero $N_{SPT,30} < 15$ nei terreni a grana grossa e $c_{u,30} < 70$ kPa nei terreni a grana fina).
E	*Terreni dei sottosuoli di tipo C o D per spessore non superiore a 20 m*, posti sul substrato di riferimento (con $V_s > 800$ m/s).

Figura 5.4 Subsurface category

We can take a category b for the project under the project. The rules identify four topographical categories depending on the configuration of the surface on which the building stands. This building is located on a flat surface; in fact the category is the T1.
T1 : "flat surface, slopes and isolated reliefs with average inclination i<15"

5.3.2 Behaviour factor

For the project purposes and tests at the SLU, the dissipative capacities of the structures can be taken into account through a reduction in elastic forces, which takes into account in a simplified way the inelastic dissipative capacity of the structure, its over-existence, the increase in the period precisely as a result of plasticization.
The project spectrum is therefore the elastic spectrum corresponding to the probability of overcoming in the period of reference considered, with the orders reduced by the term q, called the structure factor.

The value of the q structure factor depends on the structural type, its degree of hyperstaticity, the design criteria adopted and takes into account the nonlinearities of material. It is defined as:

$$q = q_0 * K_R$$

where:

- q_0 is the maximum value of the behaviour factor that depends on the level of expected ductility, structural type and ratio (relationship between seismic action such as to make the structure labile and seismic action for which the first element plasticizes);
- K_R it's a factor addicted to the regularity in height of the building.

In this case:

$$q_0=2$$
$$K_R=1$$
$$q=2*1=2$$

5.3.3 Plan Regularity

The structure must comply with the four conditions of the 7.2.2 NTC18rule to be adjusted inplan:
Compact and symmetrical configuration with respect to two orthogonal directions, in relation to mass distribution and stiffness;

- The ratio of the sides of the rectangle in which the construction is inscribed is less than 4;
- No size of any indents or overhangs exceeds 25% of the total size of the construction in the corresponding direction;
- Horizontals can be considered infinitely rigid in their plane compared to vertical elements and sufficiently resistant.

In this case:
The first precet is not verified, the structure is not symmetric either on the plane of the x-axis, nor on the plane of the y-axis.
The other three requirements are met.

5.3.4 Height regularity

Alternatively, accelerograms are permitted, as long as they are properly commensurate with the seismic hazard of the site.
For the buildings design in seismic zones, the Italian legislation, expanding what is expected in Eurocode 8, conventionally chooses four tests on resistance parameters called tests to the limit states:

- Ultimate limit states: verification is carried out on parameters that describe the carrying capacity of the structure or other forms of failure that compromise the safety of human life.

- State of life-safety: as a result of the seismic event the construction suffers collapses and ruptures of non-structural and plant components and significant damage of structural components combined with a significant loss of rigidity in the comparisons of horizontal actions; The structure retains residual strength and stiffness for vertical actions and a safety margin for horizontal seismic actions.

- State of the prevention of collapse: after the earthquake the construction suffers serious damage and collapses to non-structural components and serious damage to structural ones;

5.4 Response spectrum

The elastic response spectra of the earthquake horizontal components were derived by the National Institute of Geophysics and Volcanology (INGV)
For definition of spectra, the software provided by the Superior Council of Public Works was used.
The process has 3 steps:

1. Identifying the danger of the site by entering the geographical coordinates of the site of interest;
2. Choosing the Design strategy that defines seismic action by locating the Cu and VN parameters previously defined;
3. Determining the project action for the different boundary states examined on the basis of the subsurface and topographical-categories.

In this we refer to a soil type B tender rocks and deposits of soils with very thickened grain or very fine-grained soils with thicknesses greater than 30, characterized by a gradual improvement of mechanical properties with depth and values of Vs.30 between 360m/s and 800m/s.

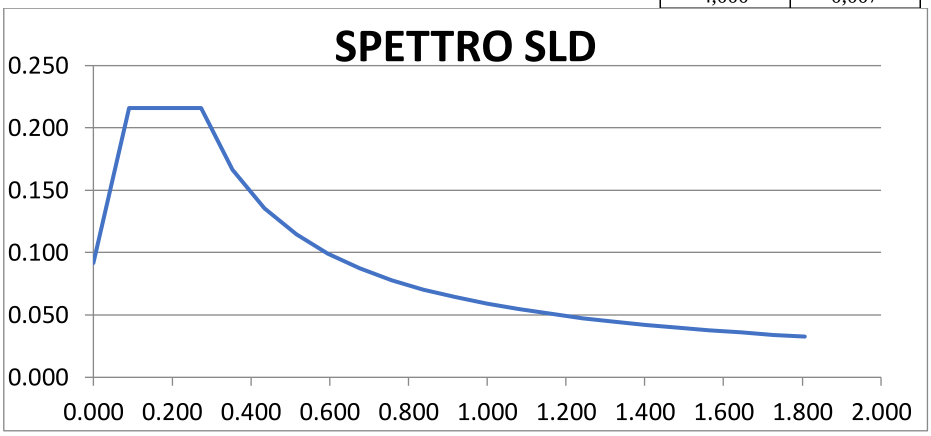
The local seismic response can also be influenced by the topographical configuration of the area where the construction is located: in our study we consider a category T1 (flat surface).

5.4.1 Spectrum parameters for damage limit state:

SLD

	0,000	0,092
TB	0,091	0,216
TC	0,273	0,216
	0,354	0,167
	0,435	0,136
	0,515	0,115
	0,596	0,099
	0,677	0,087
	0,757	0,078
	0,838	0,070
	0,919	0,064
	0,999	0,059
	1,080	0,055
	1,161	0,051
	1,242	0,048
	1,322	0,045
	1,403	0,042
	1,484	0,040
	1,564	0,038
	1,645	0,036
	1,726	0,034
	1,806	0,033

	1,887	0,031
TD	1,968	0,030
	2,065	0,027
	2,161	0,025
	2,258	0,023
	2,355	0,021
	2,452	0,019
	2,548	0,018
	2,645	0,017
	2,742	0,015
	2,839	0,014
	2,935	0,013
	3,032	0,013
	3,129	0,012
	3,226	0,011
	3,323	0,011
	3,419	0,010
	3,516	0,009
	3,613	0,009
	3,710	0,008
	3,806	0,008
	3,903	0,008
	4,000	0,007



Damage limit status (SLD): As a result of the earthquake, the construction as a whole, including structural, non-structural elements and equipment relevant to its function, suffers damage such that it does not put users at risk and does not compromise the ability to resist and stiffen vertical and horizontal actions, while maintaining itself immediately usable even in the interruption of use of part of the apparitions.

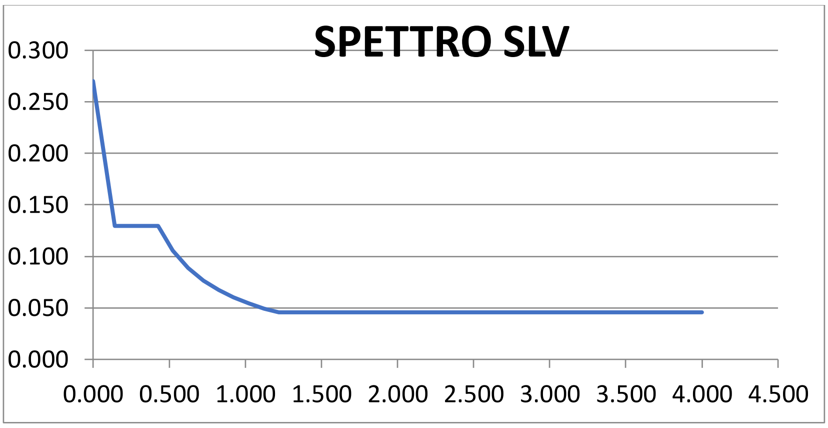
The objective of the SLD is to verify that damage to non-structural components is contained, to avoid the unfit of the structure following a seismic event with a probability of exceeding 63%.

The verification is carried out by checking that the movements of the floor comply with certain limits. For this check, refer to paragraph 12.5 .

5.4.2 Spectrum parameters for the life safety limit state:

SLV

	0,000	0,270
TB	0,142	0,130
TC	0,426	0,130
	0,526	0,105
	0,625	0,089
	0,725	0,076
	0,824	0,067
	0,923	0,060
	1,023	0,054
	1,122	0,049
	1,222	0,046
	1,321	0,046
	1,420	0,046
	1,520	0,046
	1,619	0,046
	1,719	0,046
	1,818	0,046
	1,917	0,046
	2,017	0,046
	2,116	0,046
	2,216	0,046
	2,315	0,046
	2,414	0,046
TD	2,514	0,046
	2,585	0,046
	2,655	0,046
	2,726	0,046
	2,797	0,046
	2,868	0,046
	2,938	0,046
	3,009	0,046
	3,080	0,046
	3,151	0,046
	3,221	0,046
	3,292	0,046
	3,363	0,046
	3,434	0,046
	3,505	0,046
	3,575	0,046
	3,646	0,046
	3,717	0,046
	3,788	0,046
	3,858	0,046
	3,929	0,046
	4,000	0,046



Life safety limit state (SLV): As a result of the earthquake, the structure suffered ruptures and collapses of non-structural and plant components and significant damage to structural components, coupled with a significant loss of rigidity in the comparisons of horizontal actions; the construction, on the other hand, retains some of the resistance and rigidity for vertical actions and a margin of safety against the collapse by horizontal seismic actions.

The SLV is used to design the elastic boundary resistance of the structure

6 Actions combinations

The following actions combinations are defined for the purposes of the limit statechecks:

- Fundamental combination, generally used for ultimate limit states (SLUs):

$$\gamma_{G_1} + \gamma_{G_2} + \gamma_P P + \gamma_{Q1} Q_{k1} + \gamma_{Q2} \Psi_{02} Q_{k2} + \gamma_{Q3} \Psi_{03} Q_{k3} ...$$

Characteristic (rare) combination, generally used for irreversible operating limit states (SLEs), to be used in the checks of eligible voltages referred to in the 2.7 :

$$G_1 + G_2 + P + Q_{k1} + \Psi_{0.2} Q_{k2} + \Psi_{0.3} Q_{k..}$$

- Frequent combination, usually used for reversible operating limit (SLE) states:

$$G_1 + G_2 + P + \Psi_{11} Q_{k1} + \Psi_{22} Q_{k2} + \Psi_{33} Q_{k3} ...$$

- Near-permanent combination (SLE), generally used for long-term effects:

$$G_1 + G_2 + P + \Psi_{21} Q_{k1} + \Psi_{22} Q_{k2} + \Psi_{23} Q_{k3} ...$$

- Combinazione sismica, impiegata per gli stati limite ultimi e di esercizio connessi all'azione sismica E:

$$E + G_1 + G_2 + P + \Psi_{21} Q_{k1} + \Psi_{22} Q_{k2} + \Psi_{23} Q_{k3} ...$$

- Exceptional combination, used for the last limit states related to the exceptional actions of Project Ad:

$$G_1 + G_2 + P + A_d + \Psi_{21} Q_{k1} + \Psi_{22} Q_{k2} + ..$$

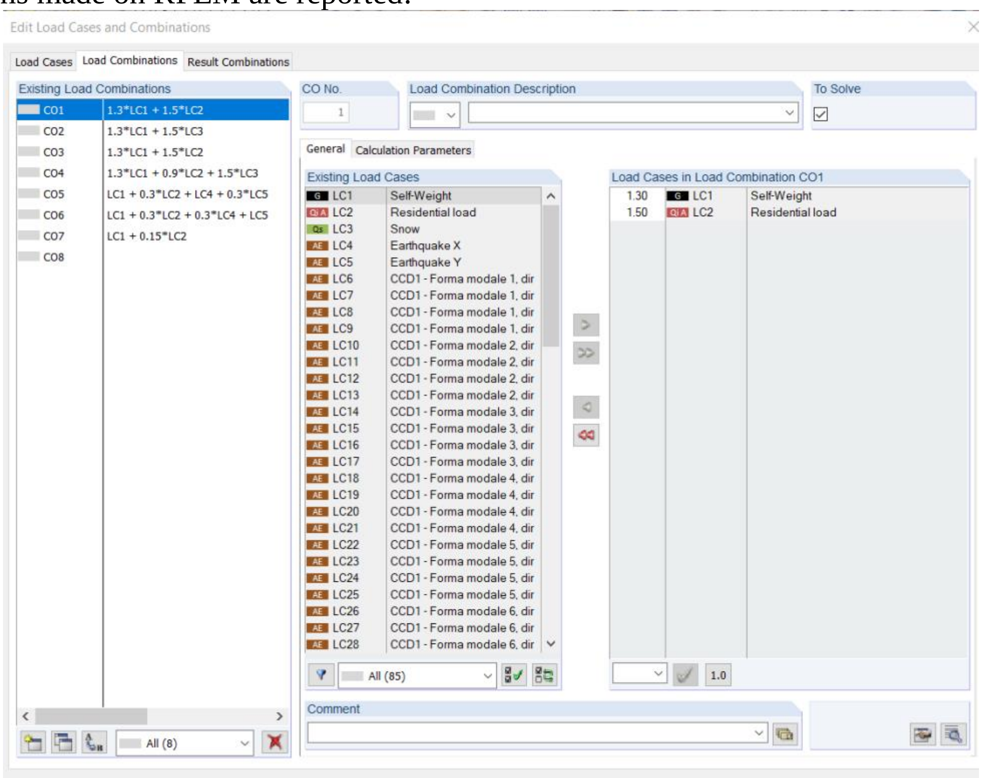
- In SLE combinations, Q_{kj} loads that make a favourable contribution to testing and, where appropriate, G_2 loads areomitted.

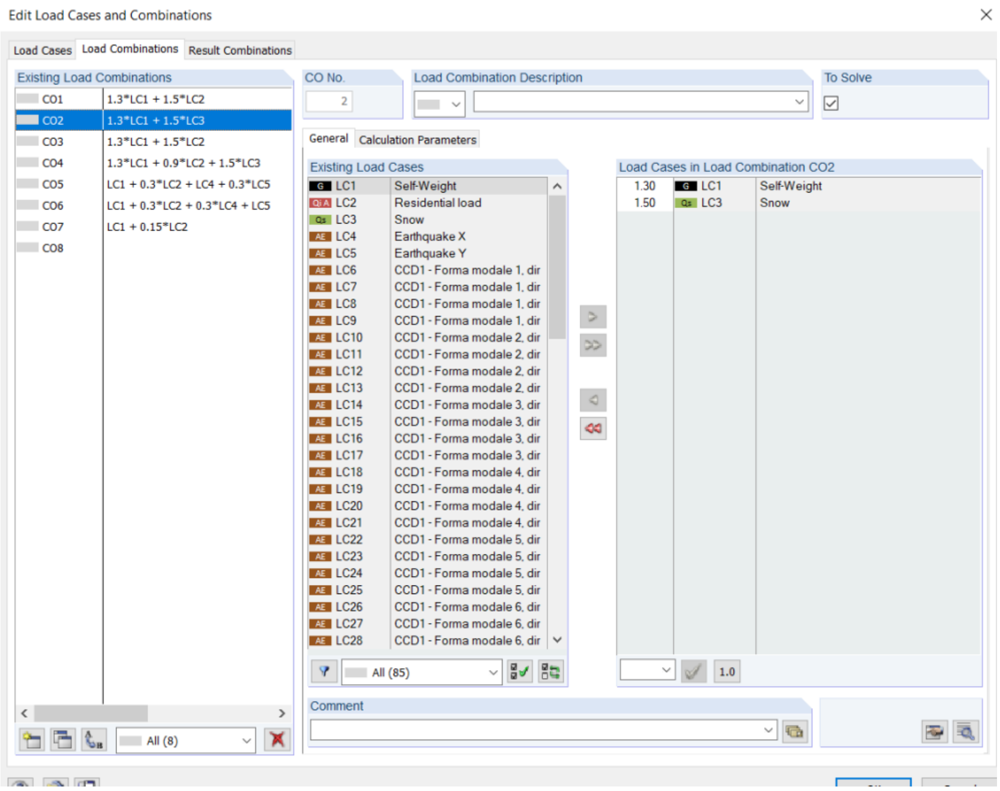
- The partial safety coefficients valus γ_{G_i} and γ_{Q_j} are given by the following table from NTC18 2.5.1 "Combination coefficient values":

Tab. 2.5.I – *Valori dei coefficienti di combinazione*

Categoria/Azione variabile	ψ_{0j}	ψ_{1j}	ψ_{2j}
Categoria A - Ambienti ad uso residenziale	0,7	0,5	0,3
Categoria B - Uffici	0,7	0,5	0,3
Categoria C - Ambienti suscettibili di affollamento	0,7	0,7	0,6
Categoria D - Ambienti ad uso commerciale	0,7	0,7	0,6
Categoria E – Aree per immagazzinamento, uso commerciale e uso industriale Biblioteche, archivi, magazzini e ambienti ad uso industriale	1,0	0,9	0,8
Categoria F - Rimesse , parcheggi ed aree per il traffico di veicoli (per autoveicoli di peso ≤ 30 kN)	0,7	0,7	0,6
Categoria G – Rimesse, parcheggi ed aree per il traffico di veicoli (per autoveicoli di peso > 30 kN)	0,7	0,5	0,3
Categoria H - Coperture accessibili per sola manutenzione	0,0	0,0	0,0
Categoria I – Coperture praticabili	da valutarsi caso per caso		
Categoria K – Coperture per usi speciali (impianti, eliporti, ...)			
Vento	0,6	0,2	0,0
Neve (a quota ≤ 1000 m s.l.m.)	0,5	0,2	0,0
Neve (a quota > 1000 m s.l.m.)	0,7	0,5	0,2
Variazioni termiche	0,6	0,5	0,0

Combinations made on RFEM are reported:

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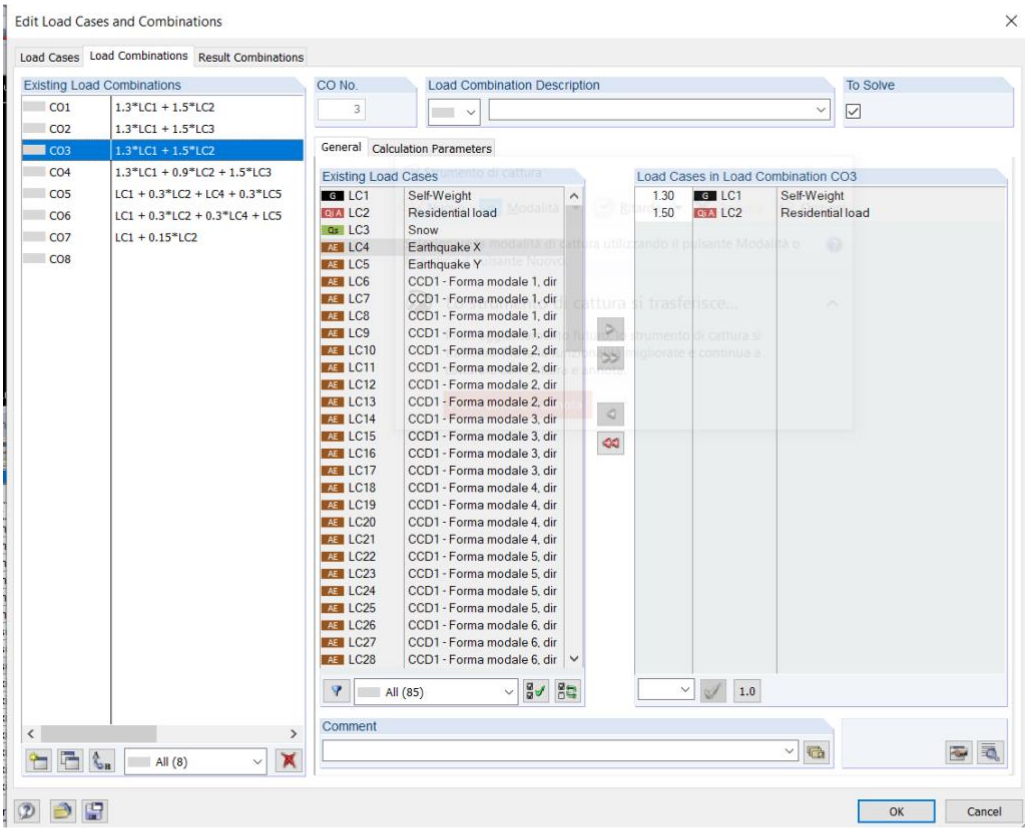
Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
CO1 1.3*LC1 + 1.5*LC2
CO2 1.3*LC1 + 1.5*LC3
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
2
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
Load Cases in Load Combination CO2
1.30 LC1 Self-Weight
1.50 LC3 Snow
All (85)
All (8)
Comment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Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
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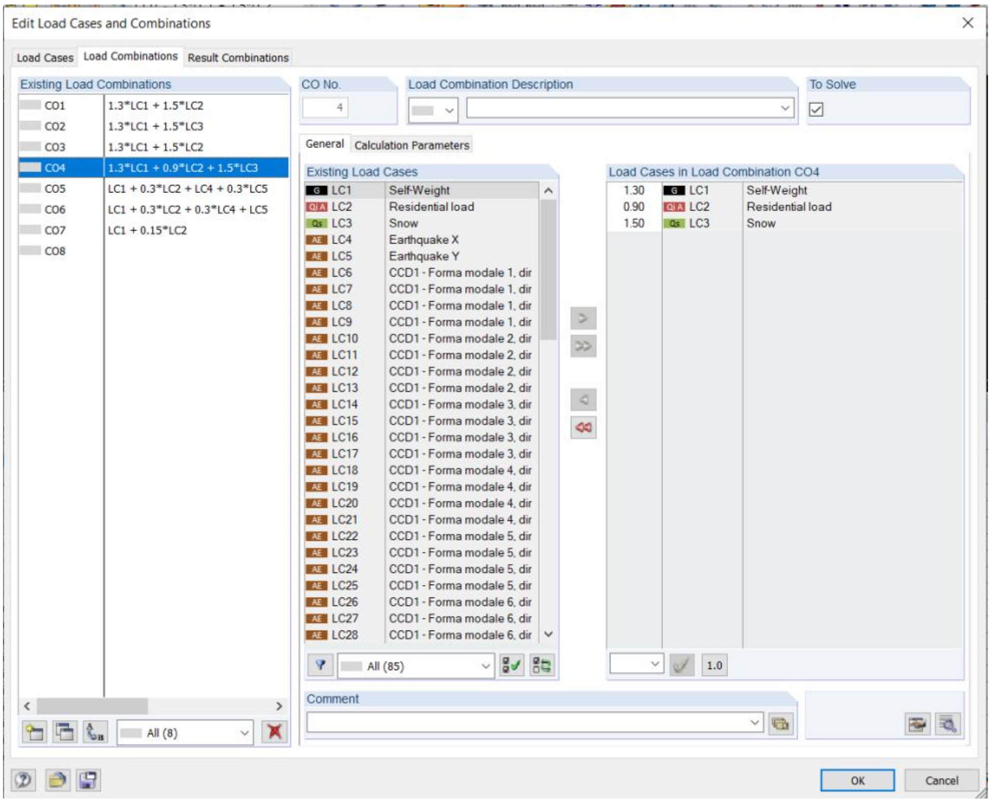
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
3
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
Load Cases in Load Combination CO3
1.30 LC1 Self-Weight
1.50 LC2 Residential load
All (85)
All (8)
Comment
OK
Cancel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Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
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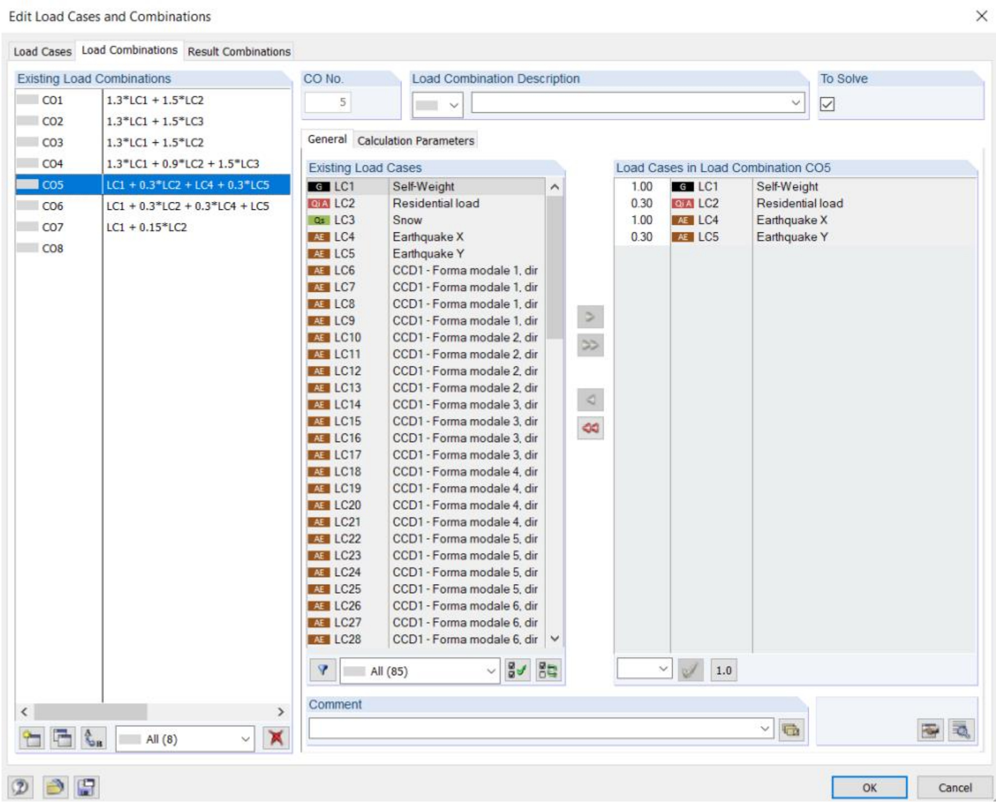
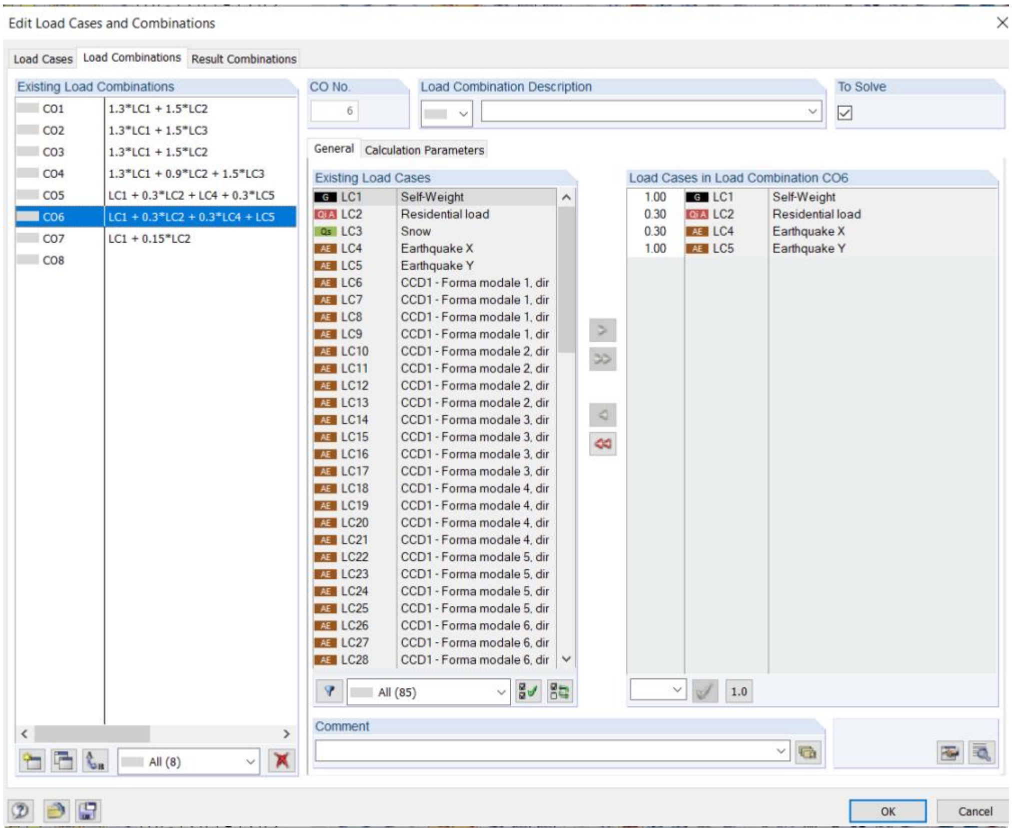
CO2 1.3*LC1 + 1.5*LC3
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
4
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
LC6 CCD1 - Forma modale 1, dir
LC7 CCD1 - Forma modale 1, dir
LC8 CCD1 - Forma modale 1, dir
LC9 CCD1 - Forma modale 1, dir
LC10 CCD1 - Forma modale 2, dir
LC11 CCD1 - Forma modale 2, dir
LC12 CCD1 - Forma modale 2, dir
LC13 CCD1 - Forma modale 2, dir
LC14 CCD1 - Forma modale 3, dir
LC15 CCD1 - Forma modale 3, dir
LC16 CCD1 - Forma modale 3, dir
LC17 CCD1 - Forma modale 3, dir
LC18 CCD1 - Forma modale 4, dir
LC19 CCD1 - Forma modale 4, dir
LC20 CCD1 - Forma modale 4, dir
LC21 CCD1 - Forma modale 4, dir
LC22 CCD1 - Forma modale 5, dir
LC23 CCD1 - Forma modale 5, dir
LC24 CCD1 - Forma modale 5, dir
LC25 CCD1 - Forma modale 5, dir
LC26 CCD1 - Forma modale 6, dir
LC27 CCD1 - Forma modale 6, dir
LC28 CCD1 - Forma modale 6, dir
All (85)
Load Cases in Load Combination CO4
1.30 LC1 Self-Weight
0.90 LC2 Residential load
1.50 LC3 Snow
1.0
Comment
All (8)
OK
Cancel


Edit Load Cases and Combinations
Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
CO1 1.3*LC1 + 1.5*LC2
CO2 1.3*LC1 + 1.5*LC3
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
5
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
LC6 CCD1 - Forma modale 1, dir
LC7 CCD1 - Forma modale 1, dir
LC8 CCD1 - Forma modale 1, dir
LC9 CCD1 - Forma modale 1, dir
LC10 CCD1 - Forma modale 2, dir
LC11 CCD1 - Forma modale 2, dir
LC12 CCD1 - Forma modale 2, dir
LC13 CCD1 - Forma modale 2, dir
LC14 CCD1 - Forma modale 3, dir
LC15 CCD1 - Forma modale 3, dir
LC16 CCD1 - Forma modale 3, dir
LC17 CCD1 - Forma modale 3, dir
LC18 CCD1 - Forma modale 4, dir
LC19 CCD1 - Forma modale 4, dir
LC20 CCD1 - Forma modale 4, dir
LC21 CCD1 - Forma modale 4, dir
LC22 CCD1 - Forma modale 5, dir
LC23 CCD1 - Forma modale 5, dir
LC24 CCD1 - Forma modale 5, dir
LC25 CCD1 - Forma modale 5, dir
LC26 CCD1 - Forma modale 6, dir
LC27 CCD1 - Forma modale 6, dir
LC28 CCD1 - Forma modale 6, dir
All (85)
Load Cases in Load Combination CO5
1.00 LC1 Self-Weight
0.30 LC2 Residential load
1.00 LC4 Earthquake X
0.30 LC5 Earthquake Y
1.0
Comment
All (8)
OK
Cancel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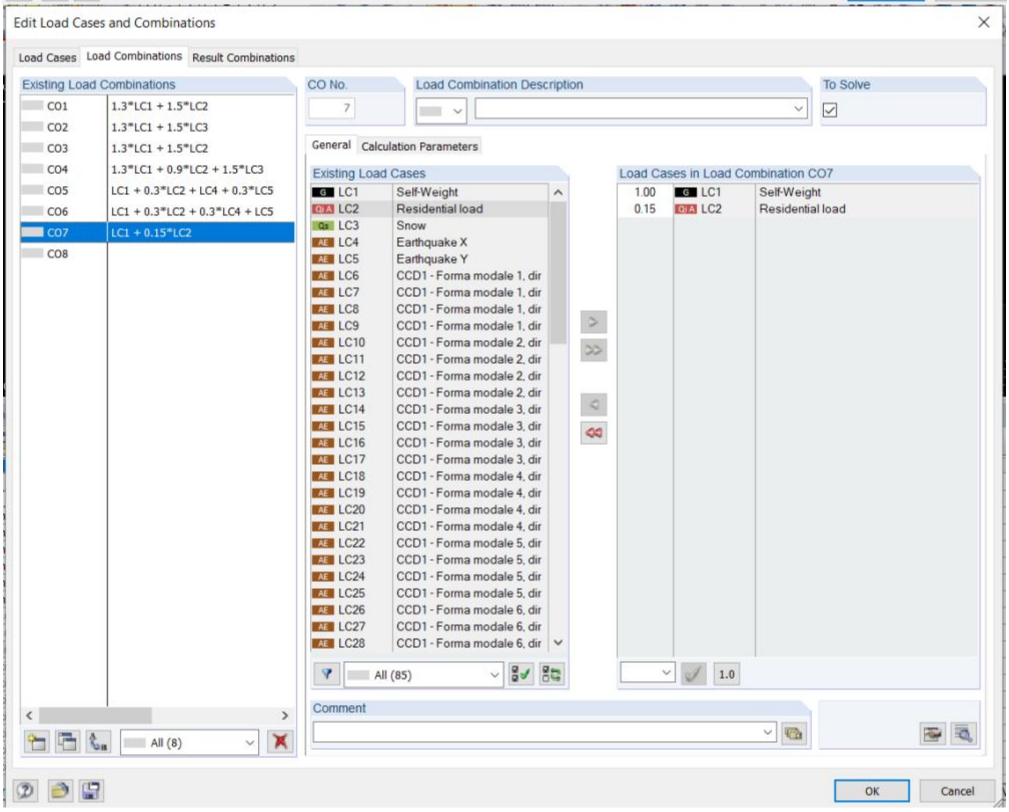
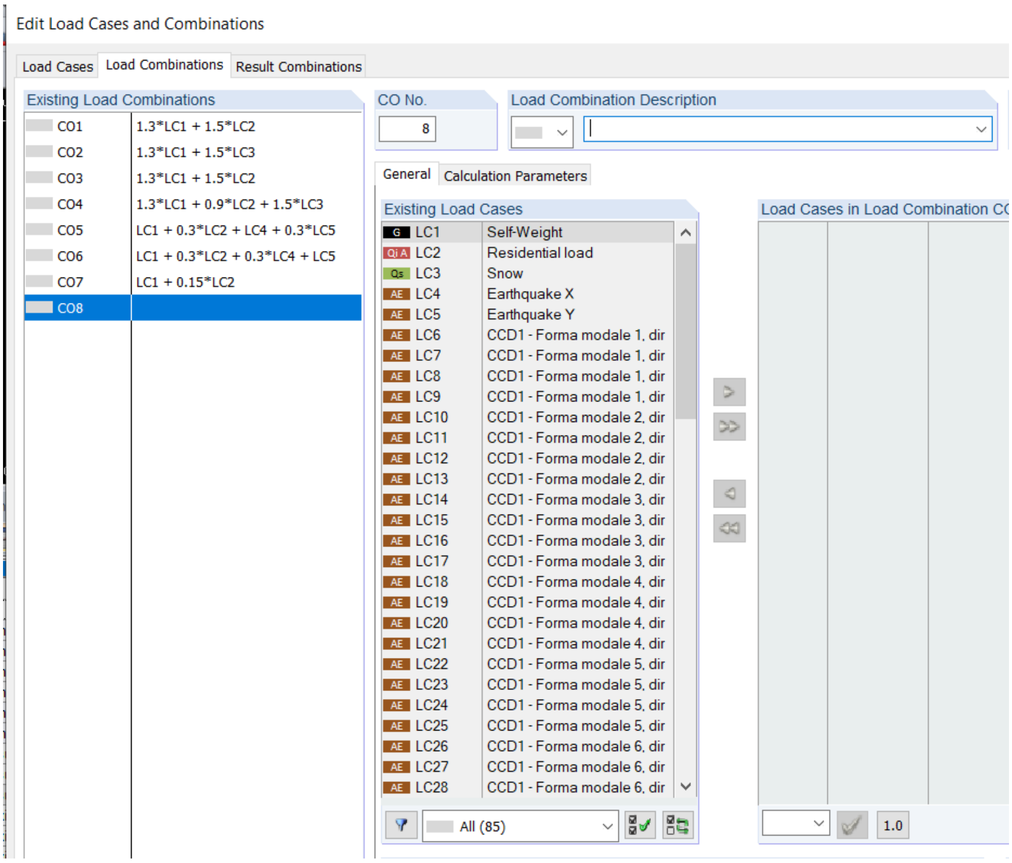
Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
CO1 1.3*LC1 + 1.5*LC2
CO2 1.3*LC1 + 1.5*LC3
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
6
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
LC6 CCD1 - Forma modale 1, dir
LC7 CCD1 - Forma modale 1, dir
LC8 CCD1 - Forma modale 1, dir
LC9 CCD1 - Forma modale 1, dir
LC10 CCD1 - Forma modale 2, dir
LC11 CCD1 - Forma modale 2, dir
LC12 CCD1 - Forma modale 2, dir
LC13 CCD1 - Forma modale 2, dir
LC14 CCD1 - Forma modale 3, dir
LC15 CCD1 - Forma modale 3, dir
LC16 CCD1 - Forma modale 3, dir
LC17 CCD1 - Forma modale 3, dir
LC18 CCD1 - Forma modale 4, dir
LC19 CCD1 - Forma modale 4, dir
LC20 CCD1 - Forma modale 4, dir
LC21 CCD1 - Forma modale 4, dir
LC22 CCD1 - Forma modale 5, dir
LC23 CCD1 - Forma modale 5, dir
LC24 CCD1 - Forma modale 5, dir
LC25 CCD1 - Forma modale 5, dir
LC26 CCD1 - Forma modale 6, dir
LC27 CCD1 - Forma modale 6, dir
LC28 CCD1 - Forma modale 6, dir
Load Cases in Load Combination CO6
1.00 LC1 Self-Weight
0.30 LC2 Residential load
0.30 LC4 Earthquake X
1.00 LC5 Earthquake Y
All (85)
1.0
Comment
All (8)
OK
Cancel

Edit Load Cases and Combinations
Load Cases
Load Combinations
Result Combinations
Existing Load Combinations
CO1 1.3*LC1 + 1.5*LC2
CO2 1.3*LC1 + 1.5*LC3
CO3 1.3*LC1 + 1.5*LC2
CO4 1.3*LC1 + 0.9*LC2 + 1.5*LC3
CO5 LC1 + 0.3*LC2 + LC4 + 0.3*LC5
CO6 LC1 + 0.3*LC2 + 0.3*LC4 + LC5
CO7 LC1 + 0.15*LC2
CO8
CO No.
7
Load Combination Description
To Solve
General
Calculation Parameters
Existing Load Cases
LC1 Self-Weight
LC2 Residential load
LC3 Snow
LC4 Earthquake X
LC5 Earthquake Y
LC6 CCD1 - Forma modale 1, dir
LC7 CCD1 - Forma modale 1, dir
LC8 CCD1 - Forma modale 1, dir
LC9 CCD1 - Forma modale 1, dir
LC10 CCD1 - Forma modale 2, dir
LC11 CCD1 - Forma modale 2, dir
LC12 CCD1 - Forma modale 2, dir
LC13 CCD1 - Forma modale 2, dir
LC14 CCD1 - Forma modale 3, dir
LC15 CCD1 - Forma modale 3, dir
LC16 CCD1 - Forma modale 3, dir
LC17 CCD1 - Forma modale 3, dir
LC18 CCD1 - Forma modale 4, dir
LC19 CCD1 - Forma modale 4, dir
LC20 CCD1 - Forma modale 4, dir
LC21 CCD1 - Forma modale 4, dir
LC22 CCD1 - Forma modale 5, dir
LC23 CCD1 - Forma modale 5, dir
LC24 CCD1 - Forma modale 5, dir
LC25 CCD1 - Forma modale 5, dir
LC26 CCD1 - Forma modale 6, dir
LC27 CCD1 - Forma modale 6, dir
LC28 CCD1 - Forma modale 6, dir
Load Cases in Load Combination CO7
1.00 LC1 Self-Weight
0.15 LC2 Residential load
All (85)
1.0
Comment
All (8)
OK
Cancel



7 Analyses types to do

In the legislation, 4 methods of analysis characterized by increasing complexity and precision are allowed, the choice depends on the characteristics of the structure, such as regularity and its own period, and its importance related to the intended use. First, the analysis of structures subject to seismic action can be linear or non-linear. Linear analysis is used to calculate the effects of the earthquake both in the case of dissipative systems and in the case of non-dissipative systems, while non-linear systems for dissipative systems take into account geometric and material nonlinearities. In addition to the fact that the analysis is linear or non-linear, the methods of analysis consider the fact that the balance is treated statically, that is, through the application of certain forces, or dynamically through the use of spectra of Calculation. So the 4 methods mentioned above are as follows:

- Linear static analysis;
- Not linear static analysis;
- Linear dynamic analysis;
- Not linear dynamic analysis.

7.1 Equivalent linear static analysis

Equivalent linear static analysis consists of the application of static forces equivalent to the induced inertia forces of seismic action and can be carried out for constructions that meet specific requirements such as:

- Structural height regularity;
- The foundamental period in T1 direction doesn't exceed 2,5TC orTD.

For civil or industrial constructions that do not exceed 40m in height and whose mass is evenly distributed along the height, T1 can be estimated in the absence of more detailed calculations using the following formula:

$$T_1 = C_1 \cdot H^{\frac{3}{4}}$$

With:

H: height of the building in meters from the foundation floor, or from thetop of a rigid plinth, equal to 30-3 to 27 m;
$C_1 = 0{,}05$ for constructions with a wall structure glued

$$T_1 = 0{,}59s$$

Alternatively, the estimate of T1 (in seconds) can be made using the following expression:

$$T_1 = 2 \cdot \sqrt{d}$$

Where:
d is the elastic lateral displacement of the highest point of the building expressed in meters, due to the weights of its own applied in the horizontal direction

In case you consider a horizontal action equal to 100% in the direction of x and 30% in the y-direction we have that d-93.7mm
Then

$$T_1 = 2 \cdot \sqrt{0.0937} = 0{,}6s$$

In case you consider a horizontal action equal to 100% in the y-direction and 30% in the x-direction we have that d-123.4mm

$$T_1 = 2 \cdot \sqrt{0.1234} = 0{,}7s$$

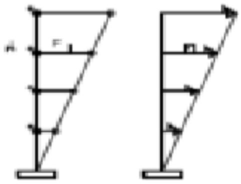
As both conditions were met, the analysis was carried out. The equivalent linear static analysis consists of an envelope between the vertical load combinations and the load combinations resulting from the application of the horizontal static forces on the various planes representing the earthquake in static form.

EQUIVALENT STATIC ENVELOPE - ENVELOPE (ENVELOPE STATIC, EARTHQUAKE ENVELOPE X, EARTHQUAKE ENVELOPE Y.

7.1.1 Horizontal load combination (Linear static analysis)

The forces system to be applied on the structure and precisely on the special joint was determined by the order of the project spectrum corresponding to the T1 period and their distribution on the structure follows the form of the main vibrating mode in the direction under review. The distribution of them is given by the
following formula:

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



- Figura 7.1: Forces breakdown graphical representation on eachplane

Where:

$F_h = S_d(T_1) \cdot W \cdot \frac{\lambda}{g}$ is the force referring to the equivalent linear oscillator corresponding to the structure;

- $S_d(T_1)$ is the ordered project response spectrum corresponding to the T1 period;
- W is the overall weight of the construction;
- λ is a coefficient of 0.85 if the construction has at least three horizontal.

The division of forces on each plane is schematized in the following table:
Where W_i represents the weight of the i-th floor.

Piano	zi [m]	Wi [kN]	zi*Wi [kNm]
1	3	510,6	1531,8
2	6	506,8	3040,8
3	9	499,6	4496,4
4	12	482,7	5792,4
5	15	460,1	6901,5
6	18	400	7200
7	21	357,8	7513,8
8	24	275	6600
9	27	90,5	2443,5
10	30	35,2	1056
			46576,2

Forza sismica	Direzione x	Direzione y
F1	122	89,5
F2	243	178
F3	300	205
F4	359	263
F5	410	290
F6	450	320
F7	500	362
F8	548	401
F9	138	101
F10	52,3	38,3

Tab: Breakdown of forces on each plane (Linear Static Analysis).

The best choice is the seismic combination:

$$E + G_1 + G_2 + \varphi_{2j} Q_{kj}$$

Where."E" corresponds to the equivalent static forces.

32 Es have been identified for each plan. This value is due to the combinations that are created from the moment when the legislation requires that " to take into account the *spatial variability of seismic motion, as well as any uncertainties in the localization of the masses, an accidental eccentricity must be attributed to the center of mass, compared to the position derived from the calculation. For buildings the accidental eccentricity to be considered should not be less than 0.05 times the size of the building measured perpendicular to the direction of application of seismic action"* [7.2.6 – NTC]. This eccentricity is assumed constant by entity and direction, on all horizontals.

The center of mass has therefore been moved to four different points:

y length (m)	13,786
x length (m)	2
x eccentricity (m)	+-1,104
y eccentricity (m)	+-0,689

Azione sismica principale	segno	eccentricità	Azione sismica secondaria	eccentricità	N° comb.
E_x	+	$+e_y$	$+0.3\ E_y$	$+e_x$	1
				$-e_x$	2
			$-0.3\ E_y$	$+e_x$	3
				$-e_x$	4
		$-e_y$	$+0.3\ E_y$	$+e_x$	5
				$-e_x$	6
			$-0.3\ E_y$	$+e_x$	7
				$-e_x$	8
	−	$+e_y$	$+0.3\ E_y$	$+e_x$	9
				$-e_x$	10
			$-0.3\ E_y$	$+e_x$	11
				$-e_x$	12
		$-e_y$	$+0.3\ E_y$	$+e_x$	13
				$-e_x$	14
			$-0.3\ E_y$	$+e_x$	15
				$-e_x$	16
E_y	+	$+e_x$	$+0.3\ E_x$	$+e_y$	17
				$-e_y$	18
			$-0.3\ E_x$	$+e_y$	19
				$-e_y$	20
		$-e_x$	$+0.3\ E_x$	$+e_y$	21
				$-e_y$	22
			$-0.3\ E_x$	$+e_y$	23
				$-e_y$	24
	−	$+e_x$	$+0.3\ E_x$	$+e_y$	25
				$-e_y$	26
			$-0.3\ E_x$	$+e_y$	27
				$-e_y$	28
		$-e_x$	$+0.3\ E_x$	$+e_y$	29
				$-e_y$	30
			$-0.3\ E_x$	$+e_y$	31
				$-e_y$	32

In addition, to take into account that the earthquake could come from both directions at the same time combine the maximum values obtained for the action applied in one direction with 30% of the maximums obtained for the action applied in the other direction. This causes the above 32 combinationsto becreated. These combinations are automatically obtained from the software by entering 0.3 as the multiplicative coefficient.

7.2 Linear dynamic analysis

Linear dynamic analysis consists of:

- To determine the ways of vibrating the construction (modal analysis);
- To calculate the effects of seismic action represented by the project response spectrum for each of the vibrating modes identified;
- In the combination of these effects.

The legislation requires that all modes be considered with a participating mass of more than 5% and that the sum does not exceed 85%.

In the case of forces considered 100% in the x-direction 30% in the y direction we have a shift in the maximum sum of 93.7mm

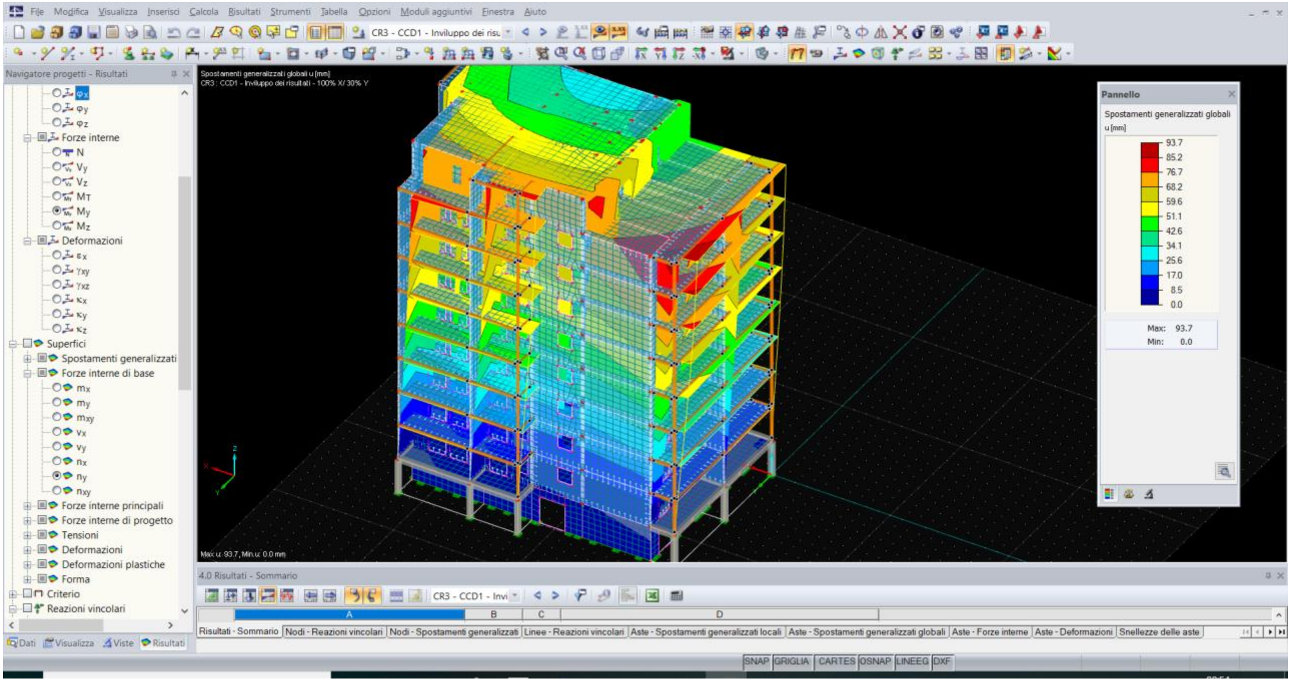


Fig. 7.1 Dynamic analysis 100%X-30%Y results

In the case of forces considered 100% in the y-direction and 30% in the x-direction we have a maximum displacement of 123.4mm

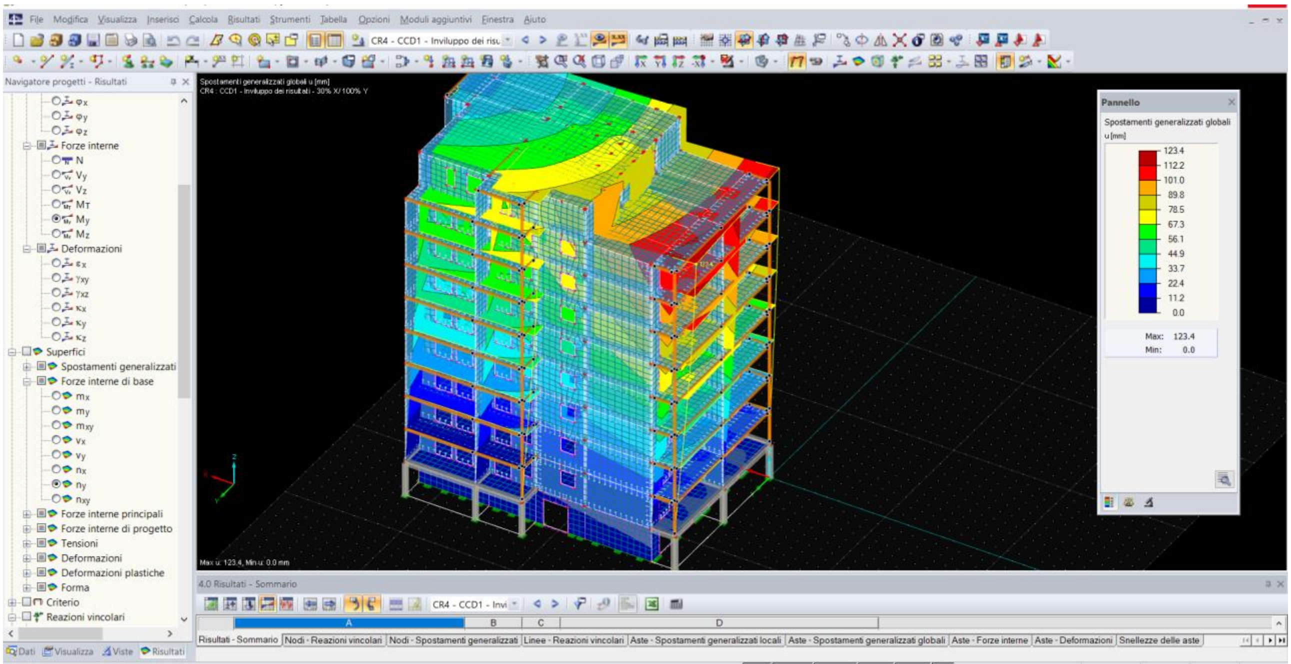


Fig. 7.2 Dynamic analysis 100%Y-30%X results

The graph resulting from that analysis is reported:

- Vibration way

Modo nr.	Autovalore λ [1/s^2]	Frequenza angolare ϖ [rad/s]	Frequenza naturale f [Hz]	Periodo proprio T [s]
1	196,246	14,009	2,230	0,449
2	268,068	16,373	2,606	0,384
3	350,179	18,713	2,978	0,336
4	1867,086	43,210	6,877	0,145
5	2504,608	50,046	7,965	0,126
6	3158,626	56,202	8,945	0,112
7	4187,644	64,712	10,299	0,097
8	4373,069	66,129	10,525	0,095
9	4634,710	68,079	10,835	0,092
10	5120,762	71,560	11,389	0,088
11	5595,469	74,803	11,905	0,084
12	5791,505	76,102	12,112	0,083
13	6671,752	81,681	13,000	0,077
14	7211,990	84,923	13,516	0,074
15	7778,295	88,195	14,037	0,071
16	8621,017	92,849	14,777	0,068
17	8956,335	94,638	15,062	0,066
18	9905,285	99,525	15,840	0,063
19	10447,289	102,212	16,268	0,061
20	11450,221	107,006	17,030	0,059

- Partecipant mass

Mode	Modal Mass	Effective Modal Mass						Effective Modal Mass Factor		
No.	M_i [kg]	m_{eX} [kg]	m_{eY} [kg]	m_{eZ} [kg]	$m_{\varphi X}$ [kg.m^2]	$m_{\varphi Y}$ [kg.m^2]	$m_{\varphi Z}$ [kg.m^2]	f_{meX} [-]	f_{meY} [-]	f_{meZ} [-]
1	223777,79	4919,34	717974,45	0,00	48785143,92	311158,53	5552546,36	0,003	0,486	0,000
2	222986,71	654908,62	35330,45	0,00	2983321,93	45299970,27	8394986,37	0,443	0,024	0,000
3	124297,52	155413,59	57027,44	0,00	5292896,04	10897949,28	39659606,10	0,105	0,039	0,000
4	188680,53	970,10	108635,76	0,00	14430935,13	139217,27	678176,46	0,001	0,073	0,000
5	229321,44	96058,08	7170,43	0,00	984992,00	13640769,20	1209633,78	0,065	0,005	0,000
6	90744,89	24094,15	8477,45	0,00	1437740,88	3741285,67	5171759,69	0,016	0,006	0,000
7	1438,66	16,48	228,30	0,00	13239,41	1763,28	18435,97	0,000	0,000	0,000
8	1242,86	18,15	43,23	0,00	5264,90	3486,87	19815,90	0,000	0,000	0,000
9	1034,13	19,28	3694,27	0,00	227789,93	2571,80	3080,16	0,000	0,002	0,000
10	969,41	5,16	175,50	0,00	3028,52	524,14	782,34	0,000	0,000	0,000
11	8403,57	605,29	32476,11	0,00	1463067,28	35709,04	305164,31	0,000	0,022	0,000
12	870,24	29,13	445,40	0,00	17170,56	1443,70	22536,54	0,000	0,000	0,000
13	850,68	20,68	62,14	0,00	6342,89	881,47	2116,50	0,000	0,000	0,000
14	146556,42	28718,40	2958,03	0,00	110075,54	1188387,49	473985,65	0,019	0,002	0,000
15	694,39	45,18	3,08	0,00	103,40	1859,86	1229,41	0,000	0,000	0,000
16	63552,92	8642,81	1284,07	0,00	14336,16	344439,27	1737563,97	0,006	0,001	0,000
17	550,32	25,87	61,71	0,00	4155,08	1030,84	8847,52	0,000	0,000	0,000
18	360,65	5,81	105,09	0,00	10680,39	254,69	1243,31	0,000	0,000	0,000
19	25092,01	544,76	18743,27	0,00	1867123,84	35201,50	25965,61	0,000	0,013	0,000
20	163599,29	36,07	555,90	0,00	41349,89	6313,64	5086,27	0,000	0,000	0,000
Sum	1495024,43	975096,95	995452,09	0,00	77698757,70	75654217,78	63292562,23	0,660	0,673	0,000

8 CLT walls check

8.1 Instability checks

The CLT wall instability checks were carried out with reference to paragraph 6.3.2 of UNI EN 1995-1-1.

It is recommended that in all cases the tensions that will be increased as a result of the inflection arrow satisfy the following expression:

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

8.2 CLT calculation model

The calculation model adopted is that of a composite structure with a deformable connection. The layers oriented in the calculation direction of the CLT panel are yieldingly connected by the orthogonal layers.
The panel is calculated as a composite structure with deformable connection in accordance with Appendix B of the EN 1995-1-1 Standard by factors γ dependent on the thickness of the orthogonal layers, from the cutting module to the "rolling shear" and by the spans length calculated using Möhler's theory –panels with up to three layers oriented in the direction of calculation – and Shelling – panels with more than three layers oriented in the calculation direction – .

Effective flexion stiffness is :

$$EJ_{eff} = \sum_{i=1}^{n}(E_i J_i + \gamma_i E_i A_i a_i^2)$$

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i}{G_R \frac{b}{d} l_{ref}^2}\right]^{-1}$$

where

J_i	represents the moment of inertia of the generic layer
A_i	represents the moment of inertia of the generic layer
a_i	is the distance between the center of gravity of the i-th layer and the center of the center of gravity of the section
l_{ref}	is the reference span length
G_R	is the rolling shear cutting module

8.3 Fiber- perpendicular compression checks

Near the support of the walls presents the situation of risk of orthogonal crushing to the fiber. In order for the verification to be met, it must be ensured that the stress voltage is lower than the resistance of the material according to the following expression:

$$\sigma_{c,90,d} \leq k_{c,90,d} \cdot f_{c,90,d}$$

With:

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{full}}$$

where

$\sigma_{c,90,d}$	is the compression project tension in the effective contact area, perpendicular to the fiber
$F_{c,90,d}$	is the compression project load perpendicular to the fiber
$f_{c,90,d}$	is the compression project resistance, perpendicular to the fiber
A_{full}	is the contact area on which the perpendicular compression to the fiber acts
$k_{c,90,d}$	it is a coefficient that takes into account the load configuration, the possibility of splitting breakage, as well as the degradation of compression strain

8.4 Shear checks

The shearing stress on the CLT leads to a shearing stress on the slats and a twisting stress on the glued cross-sides that can lead to the CLT breaking in two different modes.

8.4.1 Checking the break mechanism for shearing

Shearing stresses on slats can be determined by the following expressions:

$$\tau_y = \frac{v_2}{\sum t_{i,int}}$$

$$\tau_z = \frac{v_2}{\sum t_{i,ext}}$$

Where:

τ_y is the agent shearing tension on layers that are oriented parallel to the inner layers
τ_z is the agent shearing tension on layers that are oriented parallel to the outer layers
v_2 is the linear meter agent shear on the element in CLT
$t_{i,int}$ is the thickness of the i-th layer with parallel orientation to the inner layers
$t_{i,ext}$ is the thickness of the i-th layer with parallel orientation to the outer layers

The tension to be used in the verification is the maximum between τ_y and τ_z

$$\tau_d = \max(\tau_z; \tau_y)$$

The verification results in the following inequality

$$\tau_d \leq f_{v,lastra,k}$$

with

$f_{v,d}$ the project slab shear resistance calculated by using the

$$f_{v,lastra,d} = \frac{k_{mod} \cdot f_{v,lastra,k}}{\gamma_M}$$

8.4.2 Checking the torsion break mechanism

The tension due to the torsion can be derived from the relationship between the torsion moment agent and the polar resistant moment

$$\tau_d = \frac{M_T}{W}$$

The W value is determined using the following expression

$$W = \frac{a_{ref}^2}{n_{strati-1}}$$

The verification is:

$$\tau_{T,d} \leq f_{T,d}$$

with

$f_{T,d}$ the torsion resistance project value at intersections

$$f_{T,d} = \frac{k_{mod} \cdot f_{T,k}}{\gamma_M}$$

The CLT wall check was carried out through the program “CALCULATIS by Stora Enso”
Following some examples for demonstration:
To obtain the verification you draw the wall on calculatis, enter the values of the agent loads and perform the above checks.

Walls

- 28

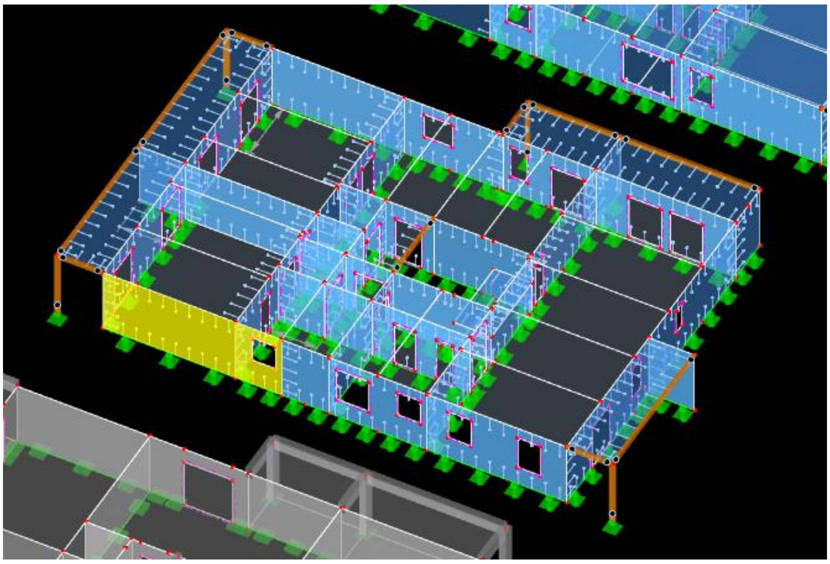
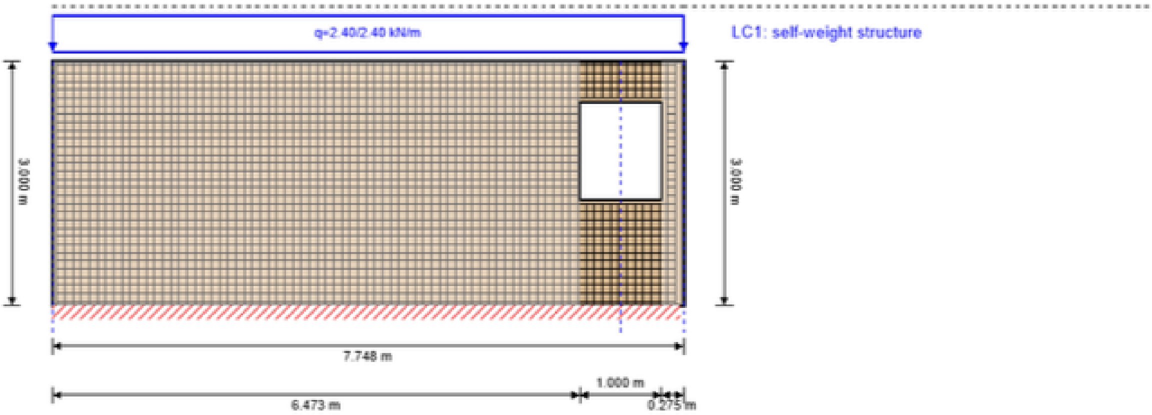


Fig. 8.1 Wall 28 position

system



global utilization ratio							6 %
ULS	3 %	ULS fire	6 %	SLS	1 %		

section: CLT 160 L5s - 2

layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	0°	C24 spruce ETA (2014)
3	40.0 mm	90°	C24 spruce ETA (2014)
4	30.0 mm	0°	C24 spruce ETA (2014)
5	30.0 mm	0°	C24 spruce ETA (2014)
t_{CLT}	**160.0 mm**		

section fire: CLT 160 L5s - 2

layer	thickness	orientation	material
1	30.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	0°	C24 spruce ETA (2014)
3	34.0 mm	90°	C24 spruce ETA (2014)
t_{CLT}	**94.0 mm**		
time	**90 min**		

fire resistance class:R 90

fire protection layering : 12.5 mm gypsum plasterboard Type F + 40 mm rock wool

gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)

Die Steinwolle-Dämmung der Installationsebene muss eine Mindestrohdichte von 26 kg/m3 und einen Schmelzpunkt >1000 °C aufweisen.

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
29	32	50	26	1	7	59.0	66.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2014)	24.00	14.00	0.35	21.00	2.40	4.00	1.25	12,500.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanet	0.6	1	1.3	1	1	1

LC1:self-weight structure

trapezoidal load

distance from start	$q_{k,a}$	load at end	laod length
[m]	[kN/m]		[m]
0.000	2.4	2.40	7.748

ULS combinations

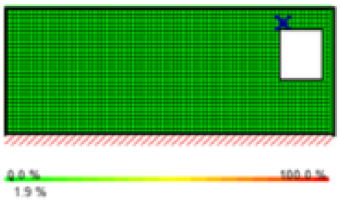
	combination rule
LCO1	1.30/1.00 * LC1

ULS combinations fire

	combination rule
LCO1	1.00/1.00 * LC1

Ultimate limit state (ULS) - design results

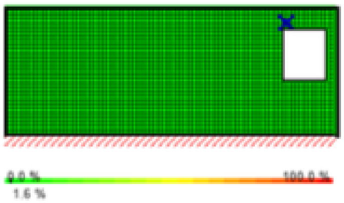
utilization rate of shear stress in plane on net section



LCO1

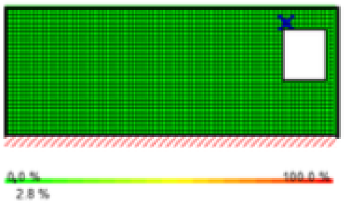
Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	0.6	8.0	0.38	0.06	2 %

utilization rate of shear stress in plaen of gross section



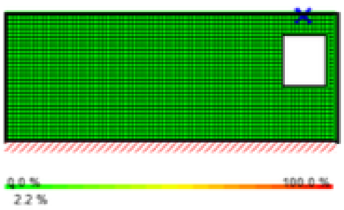
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	0.6	3.5	0.38	0.02	2 %

utilization rate of torsional shear stress in face glued surfaces



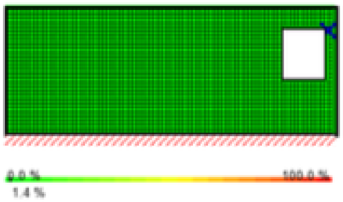
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	0.6	2.5	0.38	0.03	3 %

utilization rate of axail force horizontal



LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
2380	6.95	2.95	0.6	24.0	0.8638	0.0000	0.22	2 %

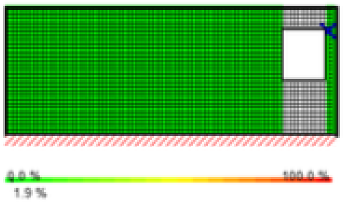
utilization rate of axail force vertical



LCO1

Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
2001	7.55	2.45	0.6	24.0	1.6200	0.0000	0.13	1 %

utilization rate for buckling

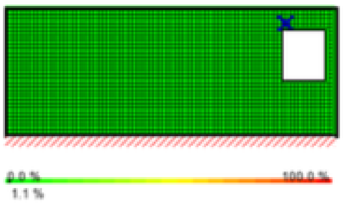


LCO1

Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²	N/mm²	N/mm²	[%]
2001	7.55	2.45	3.0	57	0.1	0.839	8.69	0.13	0.00	2 %

Ultimate limit state (ULS) fire design - results

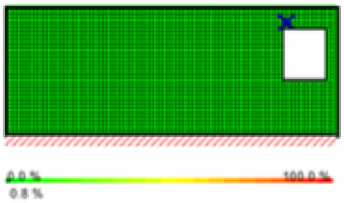
utilization rate of shear stress in plane on net section



LCO1

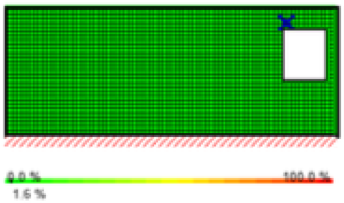
Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	1	8.0	0.30	0.10	1 %

utilization rate of shear stress in plaen of gross section



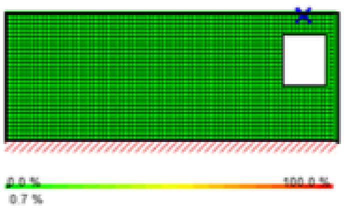
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	1	3.5	0.30	0.03	1 %

utilization rate of torsional shear stress in face glued surfaces



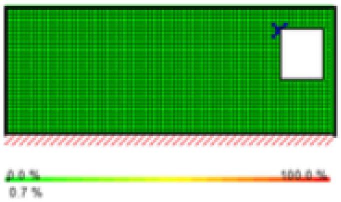
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2145	6.55	2.65	1	2.5	0.30	0.05	2 %

utilization rate of axail force horizontal



LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
2380	6.95	2.95	1	24.0	0.6676	0.0000	0.20	1 %

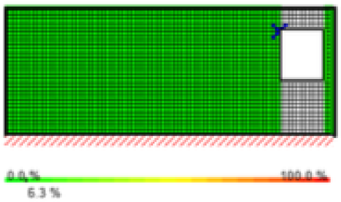
utilization rate of axail force vertical



LCO1

Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
1990	6.45	2.45	1	24.0	1.1611	0.0000	0.19	1 %

utilization rate for buckling

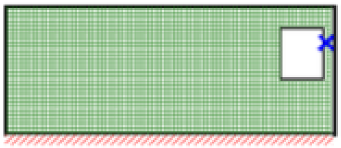


LCO1

Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[N/mm²]	[%]
1990	6.45	2.45	3.0	173	0.1	0.127	24.15	0.19	0.00	6 %

Service limit state design (SLS) - design results

horizontal deformation



LCO1

Id	X	Z	w_{limit}	limit	$v_{h,max}$	ratio
[-]	[m]	[m]	[mm]	[mm]	[mm]	[%]
1770	7.55	2.15	10.0	L/300 = 10.0	0.0131	0.1 %

w_{inst} = w[char]

LCO1

Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
2073	7.05	2.55	0.8	1.0	1/300	3.3	0.0	1 %

w_{fin} = w[char] + w[q.p.]*kdef

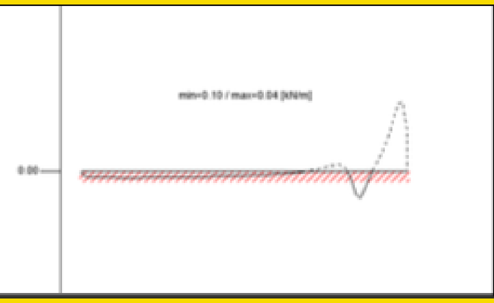
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
2073	7.05	2.55	0.8	1.0	1/200	5.0	0.0	1 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

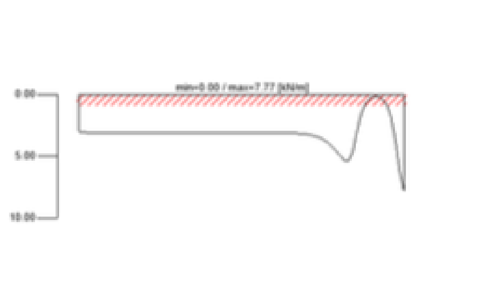
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[m]	[mm]	
2073	7.05	2.55	0.8	1.0	L/250	4.0	0.0	1 %

support reaction

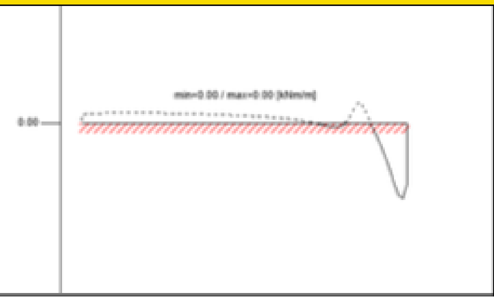
support reaction horizontal min/max



support reaction vertical min/max



support reaction moment min/max



reference documents for this analysis	
English title	description
EN 338	EN 338 - Structural timber — Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520
EN 1990	EN 1990 - Eurocode — Basis of structural design
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guildeline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
Analysis of CLT wall elements, using a beam grid model - TU-Graz - focus_sts 113_1_SF_12	Analysis of CLT shear walls with beam grid models - TU-Graz - focus_sts 113_1_SF_12
UNI EN 1995-1-2_NA	UNI EN 1995-1-2 - Italy - National Annex - Eurocode 5: Design of timber structures — Part 1-2: General — Structural fire design — National specifications concerning UNI EN 1995-1-2, national comments and national supplements
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings

reference documents for this analysis	
English title	description
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
Expertise shear in plane of CLT, H.J. Blass	Expertise - revision of DIBt technical approval Z-9.1/599 - shear in the plane of CLT

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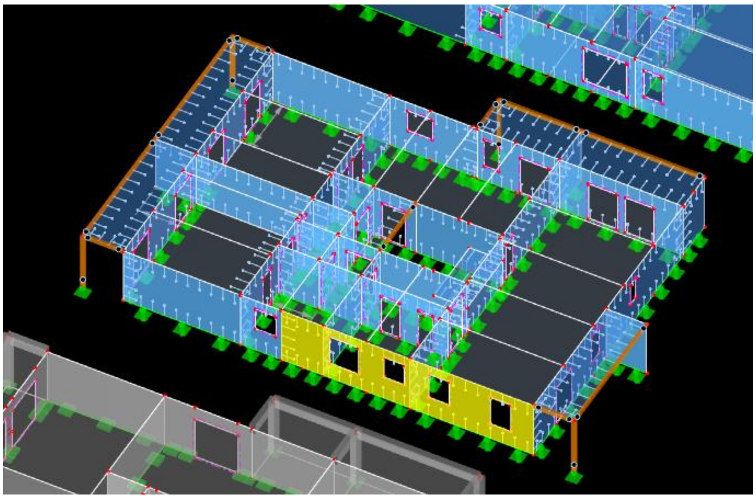
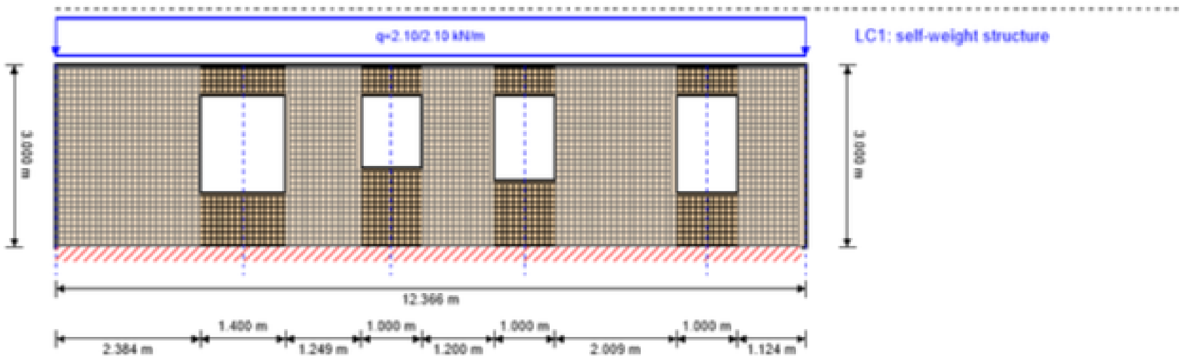


Fig. 8.2 Wall 72 position

system



global utilization ratio							4 %
ULS	3 %	ULS fire	4 %	SLS	1 %		

section: CLT 140 C5s

layer	thickness	orientation	material
1	40.0 mm	90°	C24 spruce ETA (2014)
2	20.0 mm	0°	C24 spruce ETA (2014)
3	20.0 mm	90°	C24 spruce ETA (2014)
4	20.0 mm	0°	C24 spruce ETA (2014)
5	40.0 mm	90°	C24 spruce ETA (2014)
t_{CLT}	**140.0 mm**		

section fire: CLT 140 C5s

layer	thickness	orientation	material
1	40.0 mm	90°	C24 spruce ETA (2014)
2	20.0 mm	0°	C24 spruce ETA (2014)
3	18.0 mm	90°	C24 spruce ETA (2014)
t_{CLT}	**78.0 mm**		
time	**90 min**		

fire resistance class:R 90

fire protection layering : 12.5 mm gypsum plasterboard Type F + 40 mm rock wool
gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)
Die Steinwolle-Dämmung der Installationsebene muss eine Mindestrohdichte von 26 kg/m3 und einen Schmelzpunkt >1000 °C aufweisen.

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
29	32	50	26	1	7	55.0	62.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2014)	24.00	14.00	0.35	21.00	2.40	4.00	1.25	12,500.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanet	0.6	1	1.3	1	1	1

LC1:self-weight structure

trapezoidal load

distance from start	$q_{k,a}$	load at end	laod length
[m]	[kN/m]		[m]
0.000	2.1	2.10	12.366

ULS combinations

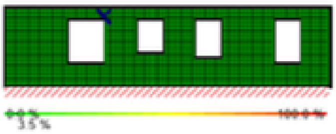
	combination rule
LCO1	1.30/1.00 * LC1

ULS combinations fire

	combination rule
LCO1	1.00/1.00 * LC1

Ultimate limit state (ULS) - design results

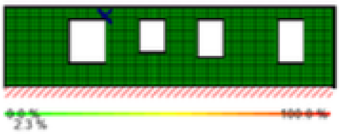
utilization rate of shear stress in plane on net section



LCO1

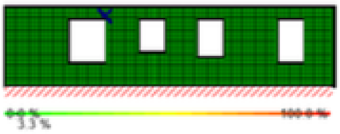
Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	0.6	8.0	-0.46	0.11	3 %

utilization rate of shear stress in plaen of gross section



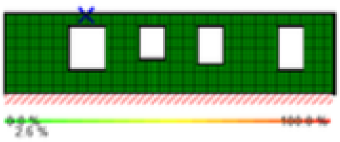
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	0.6	3.5	-0.46	0.03	2 %

utilization rate of torsional shear stress in face glued surfaces



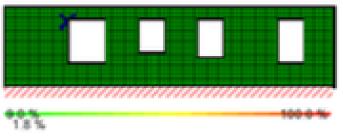
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	0.6	2.5	-0.46	0.03	3 %

utilization rate of axail force horizontal



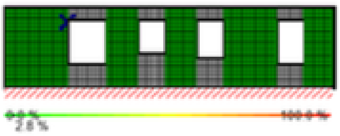
LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
3721	3.05	2.95	0.6	24.0	1.0309	0.0000	0.26	3 %

utilization rate of axail force vertical



LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
3099	2.35	2.45	0.6	24.0	1.7714	0.0000	0.18	2 %

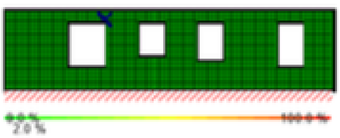
utilization rate for buckling



LCO1										
Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²	[N/mm²	[N/mm²	[%]
3099	2.35	2.45	3.0	65	0.1	0.735	8.69	0.18	0.00	3 %

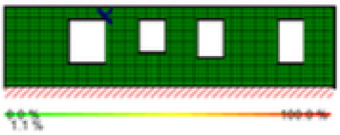
Ultimate limit state (ULS) fire design - results

utilization rate of shear stress in plane on net section



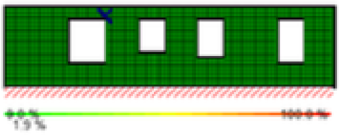
LCO1							
Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	1	8.0	-0.36	0.18	2 %

utilization rate of shear stress in plaen of gross section



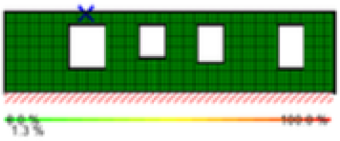
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	1	3.5	-0.36	0.05	1 %

utilization rate of torsional shear stress in face glued surfaces



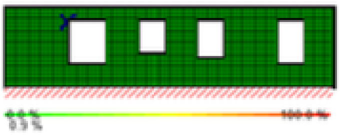
LCO1							
Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
3359	3.75	2.65	1	2.5	-0.36	0.05	2 %

utilization rate of axail force horizontal



LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
3721	3.05	2.95	1	24.0	0.7369	0.0000	0.37	1 %

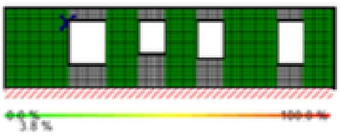
utilization rate of axail force vertical



LCO1

Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
3099	2.35	2.45	1	24.0	1.3623	0.0000	0.23	1 %

utilization rate for buckling

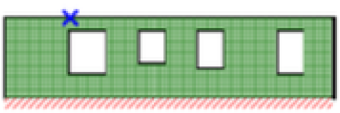


LCO1

Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²	N/mm²	N/mm²	[%]
3099	2.35	2.45	3.0	121	0.1	0.254	24.15	0.23	0.00	4 %

Service limit state design (SLS) - design results

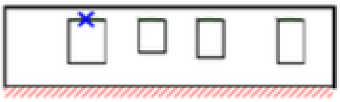
horizontal deformation



LCO1

Id	X	Z	w_{limit}	limit	$v_{h,max}$	ratio
[-]	[m]	[m]	[mm]	[mm]	[mm]	[%]
3838	2.45	3	10.0	L/300 = 10.0	0.0096	0.1 %

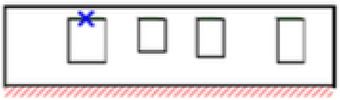
w_{inst} = w[char]



LCO1

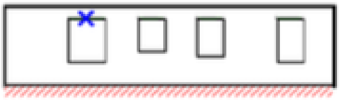
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
3229	3.05	2.55	0.8	1.4	1/300	4.7	0.0	1 %

w_{fin} = w[char] + w[q.p.]*kdef



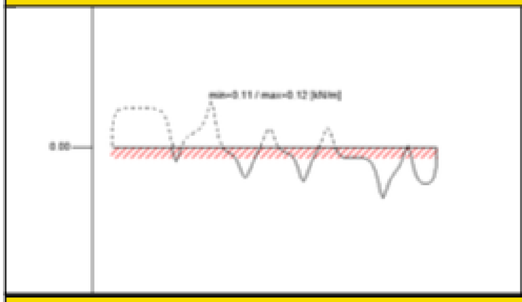
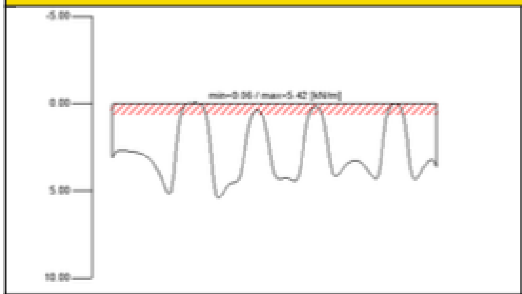
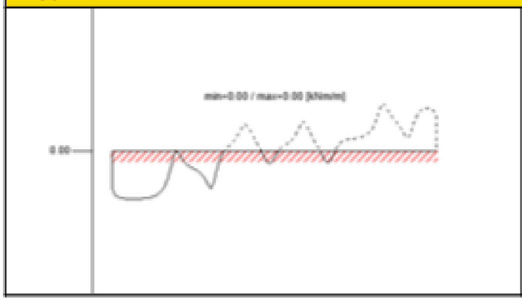
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
3229	3.05	2.55	0.8	1.4	1/200	7.0	0.1	1 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef



Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[m]	[mm]	
3229	3.05	2.55	0.8	1.4	L/250	5.6	0.1	1 %

support reaction

support reaction horizontal min/max	
	
support reaction vertical min/max	
	
support reaction moment min/max	
	

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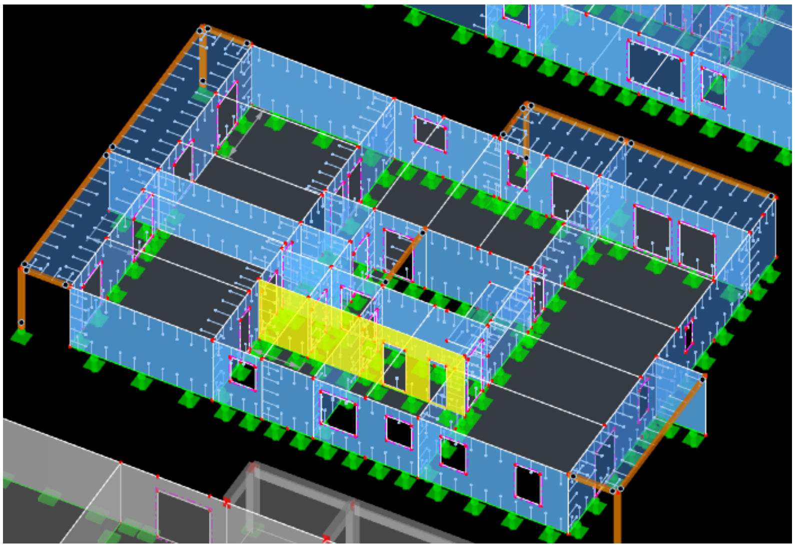
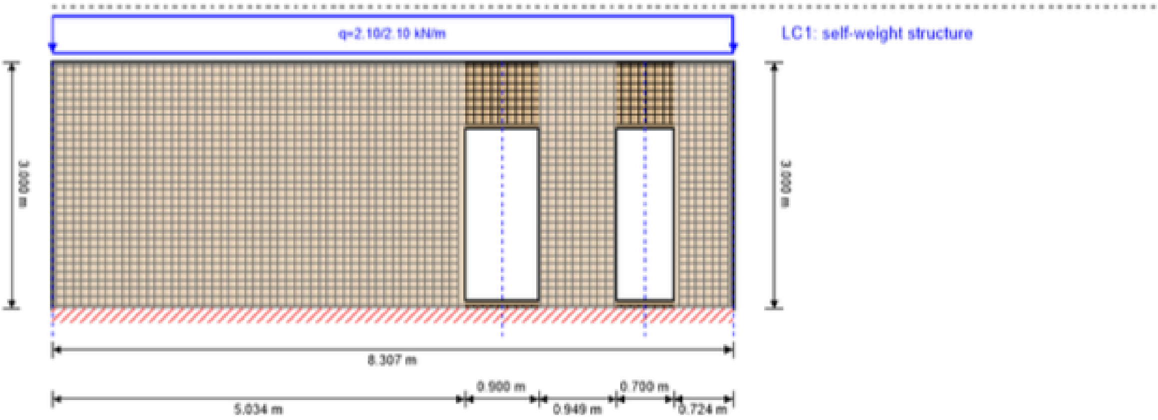


Fig. 8.3 Wall 43 position

system



global utilization ratio							2 %
ULS	1 %	ULS fire	2 %	SLS	0 %		

section: CLT 140 C5s

layer	thickness	orientation	material
1	40.0 mm	90°	C24 spruce ETA (2014)
2	20.0 mm	0°	C24 spruce ETA (2014)
3	20.0 mm	90°	C24 spruce ETA (2014)
4	20.0 mm	0°	C24 spruce ETA (2014)
5	40.0 mm	90°	C24 spruce ETA (2014)
t_{CLT}	**140.0 mm**		

section fire: CLT 140 C5s

layer	thickness	orientation	material
1	40.0 mm	90°	C24 spruce ETA (2014)
2	20.0 mm	0°	C24 spruce ETA (2014)
3	18.0 mm	90°	C24 spruce ETA (2014)
t_{CLT}	**78.0 mm**		
time	**90 min**		

fire resistance class:R 90

fire protection layering : 12.5 mm gypsum plasterboard Type F + 40 mm rock wool
gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)
Die Steinwolle-Dämmung der Installationsebene muss eine Mindestrohdichte von 26 kg/m3 und einen Schmelzpunkt >1000 °C aufweisen.

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
29	32	50	26	1	7	55.0	62.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2014)	24.00	14.00	0.35	21.00	2.40	4.00	1.25	12,500.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanet	0.6	1	1.3	1	1	1

LC1:self-weight structure

trapezoidal load

distance from start	$q_{k,a}$	load at end	laod length
[m]	[kN/m]		[m]
0.000	2.1	2.10	8.307

ULS combinations

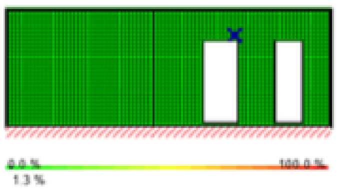
	combination rule
LCO1	1.30/1.00 * LC1

ULS combinations fire

	combination rule
LCO1	1.00/1.00 * LC1

Ultimate limit state (ULS) - design results

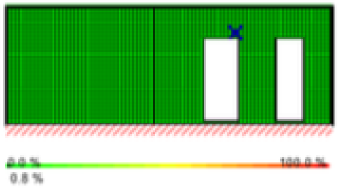
utilization rate of shear stress in plane on net section



LCO1

Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	0.6	8.0	-0.17	0.04	1 %

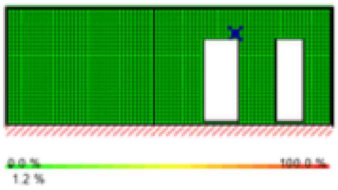
utilization rate of shear stress in plaen of gross section



LCO1

Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	0.6	3.5	-0.17	0.01	1 %

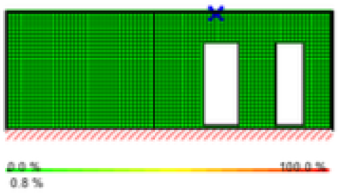
utilization rate of torsional shear stress in face glued surfaces



LCO1

Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	0.6	2.5	-0.17	0.01	1 %

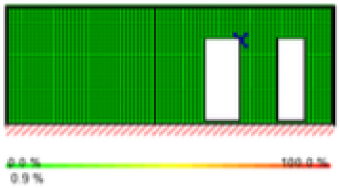
utilization rate of axail force horizontal



LCO1

Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
2544	5.35	2.95	0.6	24.0	0.3179	0.0000	0.08	1 %

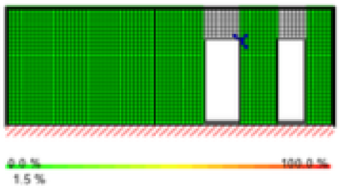
utilization rate of axail force vertical



LCO1

Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
1886	5.95	2.15	0.6	24.0	0.9350	0.0000	0.09	1 %

utilization rate for buckling

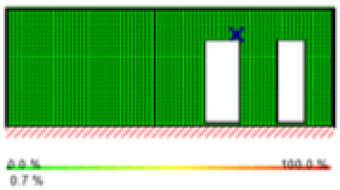


LCO1

Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²	[N/mm²	[N/mm²	[%]
1886	5.95	2.15	3.0	65	0.1	0.735	8.69	0.09	0.00	1 %

Ultimate limit state (ULS) fire design - results

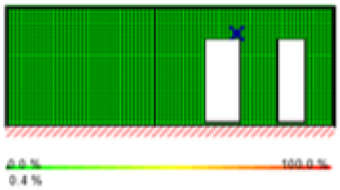
utilization rate of shear stress in plane on net section



LCO1

Id	X	Z	k_{mod}	$f_{IP,Netto,k}$	Q	$\tau_{IP,Net,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	1	8.0	-0.14	0.07	1 %

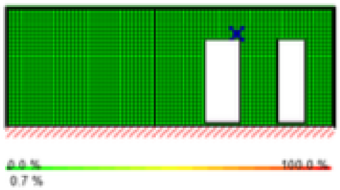
utilization rate of shear stress in plaen of gross section



LCO1

Id	X	Z	k_{mod}	$f_{v,IP,Brutto,k}$	Q	$\tau_{IP,Gross,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	1	3.5	-0.14	0.02	0 %

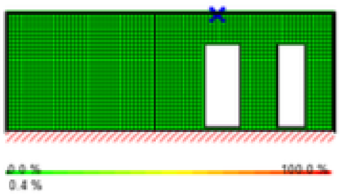
utilization rate of torsional shear stress in face glued surfaces



LCO1

Id	X	Z	k_{mod}	$f_{v,IP,T,k}$	Q	$\tau_{T,Node,d}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[N/mm²]	[%]
2051	5.85	2.35	1	2.5	-0.14	0.02	1 %

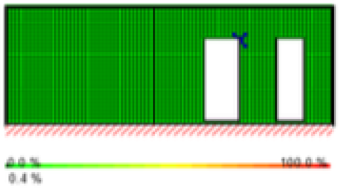
utilization rate of axail force horizontal



LCO1

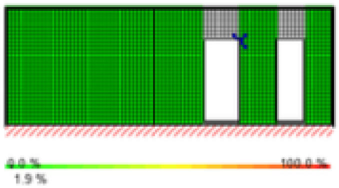
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{h,max}$	M_y	$\sigma_{h,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
2544	5.35	2.95	1	24.0	0.2150	0.0000	0.11	0 %

utilization rate of axail force vertical



LCO1								
Id	X	Z	k_{mod}	$f_{m,k}$	$N_{v,max}$	M_y	$\sigma_{v,max}$	ratio
[-]	[m]	[m]	[-]	[N/mm²]	[kN]	[kNm]	[N/mm²]	[%]
1886	5.95	2.15	1	24.0	0.6901	0.0000	0.12	0 %

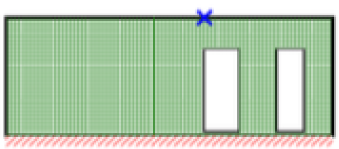
utilization rate for buckling



LCO1										
Id	X	Z	l_k	λ_y	β_c	$k_{c,y}$	$f_{c,d}$	$\sigma_{c,0,d}$	$\sigma_{m,y,d}$	ratio
[-]	[m]	[m]	[m]	[-]	[-]	[-]	[N/mm²	[N/mm²	[N/mm²	[%]
1886	5.95	2.15	3.0	121	0.1	0.254	24.15	0.12	0.00	2 %

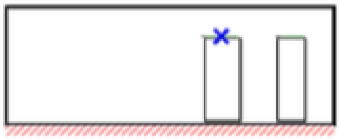
Service limit state design (SLS) - design results

horizontal deformation



LCO1						
Id	X	Z	w_{limit}	limit	$v_{h,max}$	ratio
[-]	[m]	[m]	[mm]	[mm]	[mm]	[%]
2624	5.05	3	10.0	L/300 = 10.0	0.0031	0.0 %

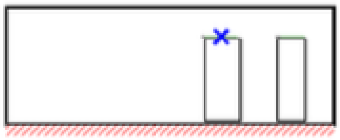
w_{inst} = w[char]



LCO1

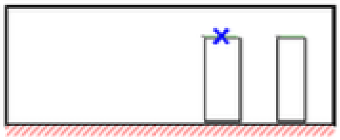
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
1964	5.45	2.25	0.8	0.9	1/300	3.0	0.0	0 %

w_{fin} = w[char] + w[q.p.]*kdef



Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[mm]	[mm]	
1964	5.45	2.25	0.8	0.9	1/200	4.5	0.0	0 %

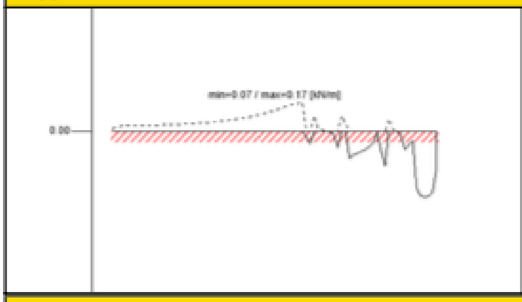
$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef



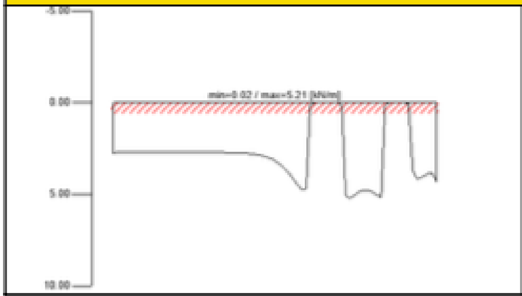
Id	X	Z	K_{def}	L_{ref}	limit	w_{limit}	$w_{calc.}$	ratio
[-]	[m]	[m]		[m]	[-]	[m]	[mm]	
1964	5.45	2.25	0.8	0.9	L/250	3.6	0.0	0 %

support reaction

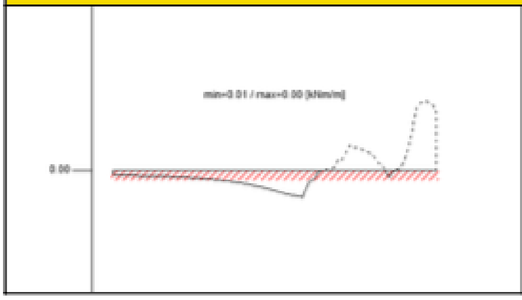
support reaction horizontal min/max



support reaction vertical min/max



support reaction moment min/max



Initially, walls were used to be 140 mm thick; as a result of the dynamic analysis, changes were made and 160mm walls were permanently used for the first 3 floors, 140mm for the 4-6 floors and 120mm for the last 3 floors 7-9.

9 CLT floor check

9.1 CLT model

The calculation model adopted is a composite structure with a deformable connection. The layers oriented in the calculation direction of the CLT panel are yieldingly connected by the orthogonal layers. The panel is calculated as a composite structure with deformable connected according to Appendix B of the EN 1995-1-1 by factors γ depending on the thickness of the orthogonal layers, the rolling shear cutting module and the length of the spans, calculated using Möhler's theory (panels with up to three layers oriented in the direction of calculation) and Shelling (panels with more than three layers oriented in the calculation direction).

Effective stiffness is:

$$EJ_{eff} = \sum_{i=1}^{n}(E_i J_i + \gamma_i E_i A_i a_i^2)$$

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i}{G_R \frac{b}{d} l_{ref}^2}\right]^{-1}$$

Where

J_i represents the moment of inertia of the generic layer
A_i represents the moment of inertia of the generic layer
a_i is the distance between the center of gravity of the i-th layer and the center of the center of gravity of the section
l_{ref} is the reference span length
G_R is the rolling shear cutting module

Generally the spans reference length l_{ref} is assumed, according to the static pattern, as shown in the following table, in which the actual length of the span is indicated by l.

Static system	Reference length
Simpre support beam	l_{ref} =l
Continuos beam span	l_{ref} = 0.8 l
Continuous beam interla support	l_{ref} = 0.8 l_{min}
Cantiliever beam	l_{ref} = 2l

9.2 Bending strength

The bending tests are carried out on a floor band parallel to the calculation direction with reference to paragraph 6.1.6 of the UNI EN 1995-1-1 standard. The following expression must be met:

$$\frac{\sigma_{m,d}}{f_{m,d}} \le 1$$

where:

$\sigma_{m,d}$ is the project tension in bending
$f_{m,d}$ is the material bending project resistance

9.3 Shear resistance checks

9.3.1 Pannel layer shear resistance checks to the calculation direction

Shearing checks are carried out with reference to paragraph 6.1.7 of the UNI En 1995-1-

Deve essere soddisfatta la seguente espressione:

$$\frac{\tau_{v,d}}{f_{v,d}} \leq 1$$

dove:

$\tau_{v,d}$ is the shear project tension
$f_{v,d}$ is the shear project resistance

The value of the maximum project shear effort in the longitudinal layers is evaluated with the following formula:

$$\tau_{v,d} = \frac{V_d S_{max}}{J_{eff} \cdot b}$$

where:

V_d is the agent shear in the section check
S_{max} is the static moment associated with maximum shearing tension
J_{eff} is the CLT cross section panel effective inertial moment
B is the CLT cross section panel base (K_{cr}=1)

- ### 9.3.2 Rolling shear checks

Cross-shearing checks are carried out with reference to paragraph 6.1.7 of the UNI EN 1995-1-1 standard. The following expression must be met:

$$\frac{\tau_{R,d}}{f_{v,R,d}} \leq 1$$

where:

$\tau_{R,d}$ is the cross-shearing project tension
$f_{v,R,d}$ is cross-shearing project resistance

The maximum project shear effort value in the longitudinal layers is evaluated with the following fomula:

$$\tau_{R,d} = \frac{V_d S_{R,max}}{J_{eff} \cdot b}$$

where:

V_d is the agent shear in the specific section

$S_{R,max}$ is the static moment associated with maximum shearing tension
J_{eff} is the CLT cross section panel effective inertial moment
B is the CLT cross section panel base (K_{cr}=1)

9.4 Deformation floor checks (SLE)

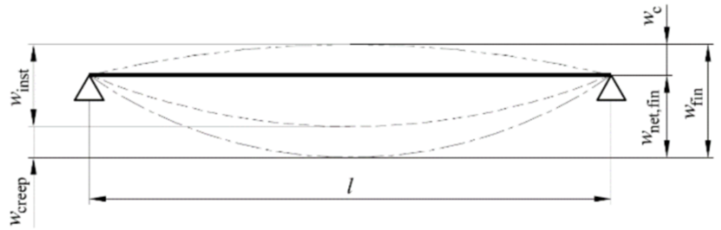
It occurs that the deformation of the structure resulting from the effects of actions and humidity remains within the appropriate limits. The deformation checks are carried out with reference to paragraph 2.2.3 of UNI EN 1995-1-1.

The net arrow $w_{net,fin}$ is:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

where:

$w_{net,fin}$	is the net end arrow
w_{inst}	is the instant arrow
w_{creep}	it's the viscoelastic arrow
w_c	is the beam length
w_{fin}	is the end arrow



The arrow limit values are assumed as shown in the following table.

Condition	w_{ist}	$w_{net,fin}$
Two supports beam	l/300	l/250
Cantiliever beam	l/500	l/125

- Instant deformation

The instant deformation w_{inst} is calculated for the rare combination of actions

- End deformation

The end deformation $w_{net,fin}$ is calculated whereas the almost permanent components of the actions cause over time a viscoelastic deformation w_{creep} that can be calculated using the average values of the elastic modules appropriately reduced by the factor ($1k_{def}$)

For consistent structures of elements, components and connections having the same viscoelastic behavior and under the assumption of a linear correlation between the actions and the deformations corresponding to the final deformation w_{fin}, can be considered as:

$$w_{fin} = w_{fin,G} + w_{fin,Q1} + \sum w_{fin,Qi}$$

where:

$w_{fin,G} = w_{inst,,G} \cdot (1 + k_{def})$	for a dead load G
$w_{fin,Q1} = w_{inst,Q1} \cdot (1 + \psi_{2,1} k_{def})$	for a primary live load Q_1
$w_{fin,Qi} = w_{inst,Qi} \cdot (1 + \psi_{2,1} k_{def})$	for secondary live load, Q_i (i >1)

The CLT floor check was carried out through the "CALCULATIS by Stora Enso" program, of which all the results are reported.
For the CLT floor in addition to the panel it was chosen to insert a concrete slab on each floor in order to ensure a greater rigidity of the structure

Floors1-8

Here are all the checks on the floors described above:

- 50-59

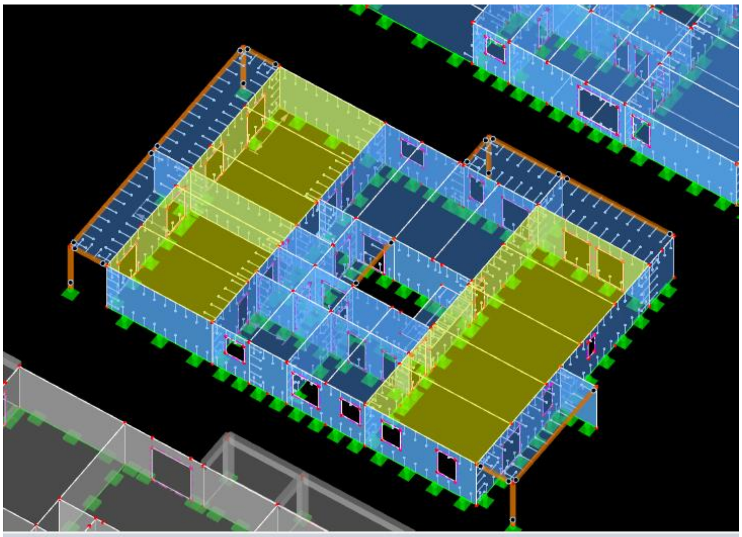
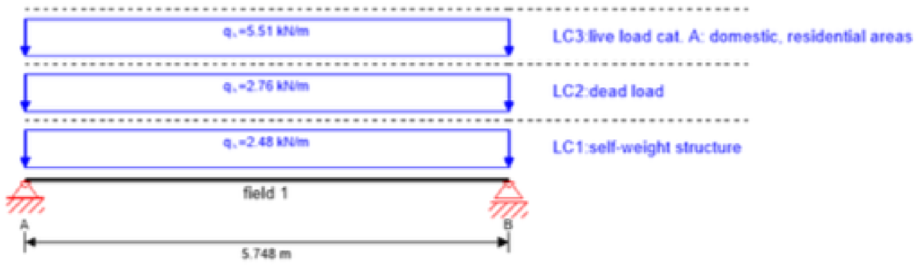


Fig. 9.1 50-59 Floor panels position

system

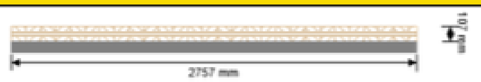


global utilization ratio									89 %
ULS	34 %	ULS fire	18 %	SLS	69 %	SLS vibration	89 %	support	3 %

section: CLT 180 L5s

layer	thickness	orientation	material
1	40.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	90°	C24 spruce ETA (2014)
3	40.0 mm	0°	C24 spruce ETA (2014)
4	30.0 mm	90°	C24 spruce ETA (2014)
5	40.0 mm	0°	C24 spruce ETA (2014)
t_{CLT}	**180.0 mm**		

section fire: CLT 180 L5s



layer	thickness	orientation	material
1	40.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	90°	C24 spruce ETA (2014)
3	37.0 mm	0°	C24 spruce ETA (2014)
t_{CLT}	**107.0 mm**		
time	**90 min**		

fire resistance class:R 90

fire protection layering : 15.0 mm gypsum plasterboard Type F + 40 mm rock wool

gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)

Die Steinwolle-Dämmung der Installationsebene muss eine Mindestrohdichte von 26 kg/m3 und einen Schmelzpunkt >1000 °C aufweisen.

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
27	27	46	25	1	7	66.0	73.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2014)	24.00	14.00	0.35	21.00	2.40	4.00	1.25	12,500.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanet	0.6	1	1.3	1	1	1
LC2	dead load	G	permanet	0.6	1	1.3	1	1	1
LC3	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

continous load	
field	load at start
	[kN/m]
1	2.48

LC2:dead load

continous load	
field	load at start
	[kN/m]
1	2.76

LC3:live load cat. A: domestic, residential areas

continous load	
field	load at start
	[kN/m]
1	5.51

ULS combinations

	combination rule
LCO1	1.30/1.00 * LC1 + 1.30/1.00 * LC2
LCO2	1.30/1.00 * LC1 + 1.30/1.00 * LC2 + 1.50/0.00 * LC3

ULS combinations fire

	combination rule
LCO3	1.00/1.00 * LC1 + 1.00/1.00 * LC2
LCO4	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.30 * LC3

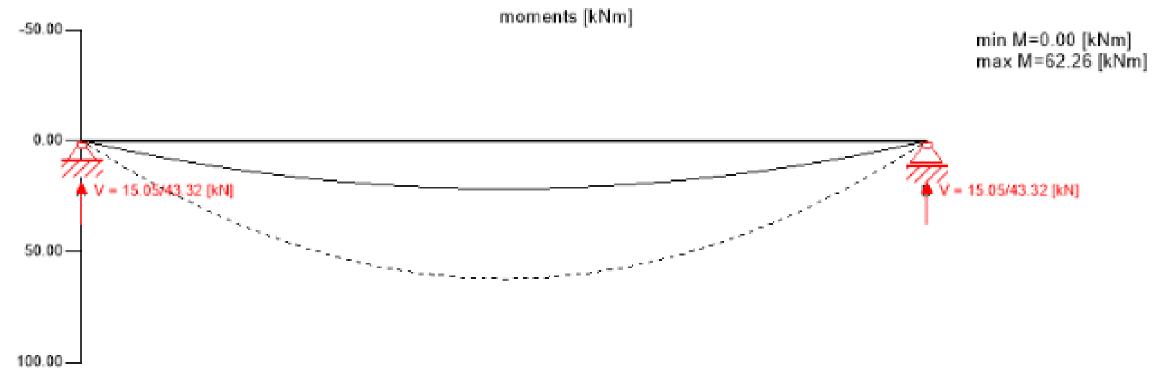
SLS characteristic combination

	combination rule
LCO5	1.00/1.00 * LC1 + 1.00/1.00 * LC2
LCO6	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * LC3

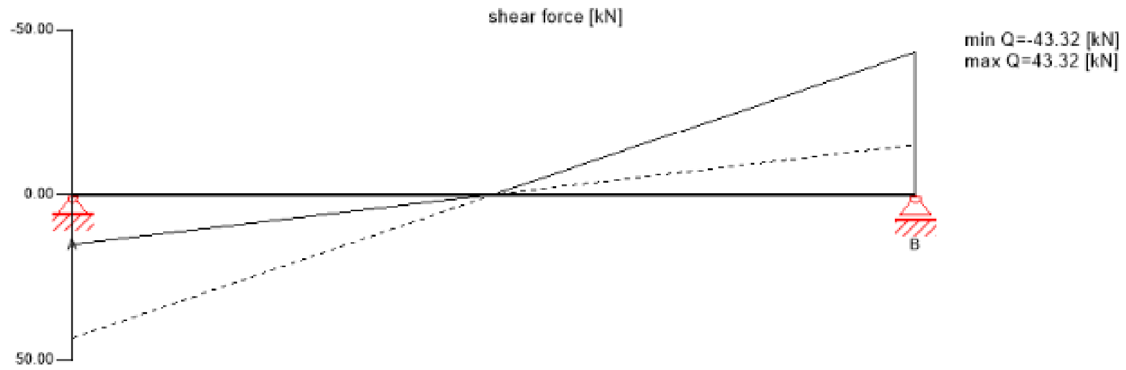
SLS quasi-permanent combination

	combination rule
LCO7	1.00/1.00 * LC1 + 1.00/1.00 * LC2
LCO8	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/0.00 * 0.30 * LC3

Ultimate limit state (ULS) - design results



Ultimate limit state (ULS) - design results



ULS flexural design

field	dist.	$f_{m,k}$	γ_m	k_{mod}	$k_{sys,y}$	$f_{m,y,d}$	$M_{y,d}$	$\sigma_{m,y,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kNm]	[N/mm²]		
1	2.87	24.00	1.45	0.80	1.10	14.57	62.26	-4.98	34 %	LCO2

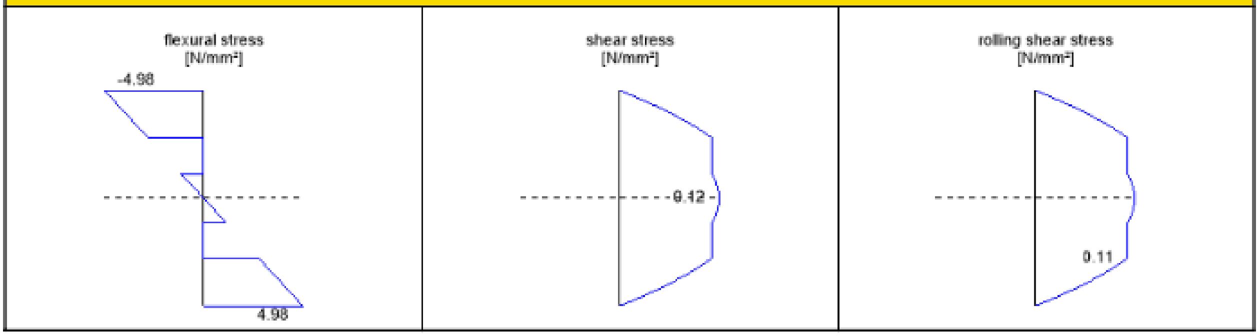
ULS shear analysis

field	dist.	$f_{v,k}$	γ_m	k_{mod}	$f_{v,d}$	V_d	$\tau_{v,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	5.75	4.00	1.45	0.80	2.21	-43.32	0.12	5 %	LCO2

ULS rolling shear

field	dist.	$f_{r,k}$	γ_m	k_{mod}	$f_{r,d}$	V_d	$\tau_{r,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	5.75	1.15	1.45	0.80	0.63	-43.32	0.11	17 %	LCO2

stress diagram



flexural stress analysis

$M_{y,d}$ =	62.26	kNm		$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.45	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				k_{hm} =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,d}$ =	7.72	N/mm²	
$\sigma_{m,y,d}$ =	-4.98	N/mm²	<	$f_{m,y,d}$ =	14.57	N/mm²	✓

utilization ratio 34 %

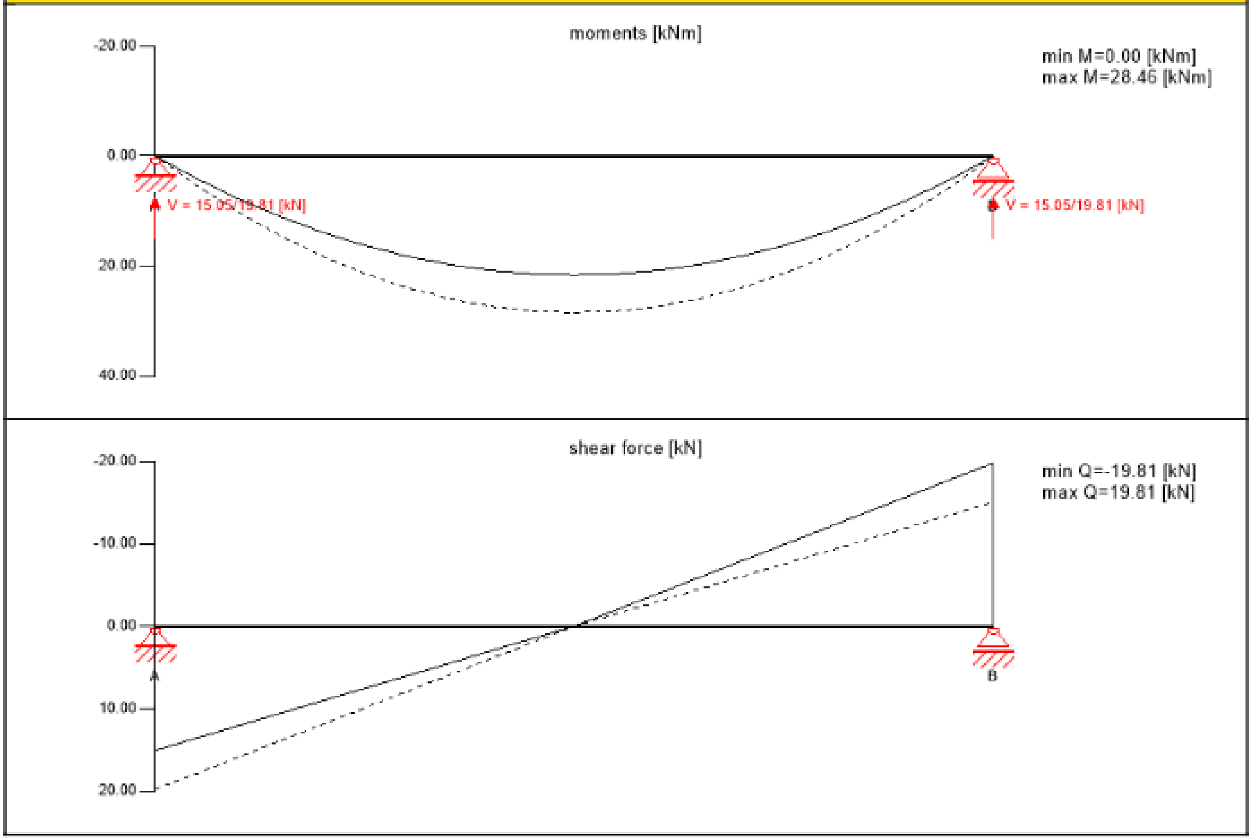
shear stress analysis

V_d =	-43.32	kN		$f_{v,k}$ =	4.00	N/mm²
				γ_m =	1.45	
				k_{mod} =	0.80	
$\tau_{v,d}$ =	0.12	N/mm²	<	$f_{v,d}$ =	2.21	N/mm² ✓
utilization ratio						5 %

rolling shear analysis

V_d =	-43.32	kN		$f_{r,k}$ =	1.15	N/mm²
				γ_m =	1.45	-
				k_{mod} =	0.80	-
$\tau_{r,d}$ =	0.11	N/mm²	<	$f_{r,d}$ =	0.63	N/mm² ✓
utilization ratio						17 %

Ultimate limit state (ULS) fire design - results



ULS fire flexural design

field	dist.	$f_{m,k}$	γ_m	k_{mod}	$k_{sys,y}$	k_{fi}	$f_{m,y,d}$	$M_{y,d}$	$\sigma_{m,y,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[-]	[N/mm²]	[kNm]	[N/mm²]		
1	2.87	24.00	1.00	1.00	1.10	1.15	30.36	28.46	5.60	18 %	LCO4

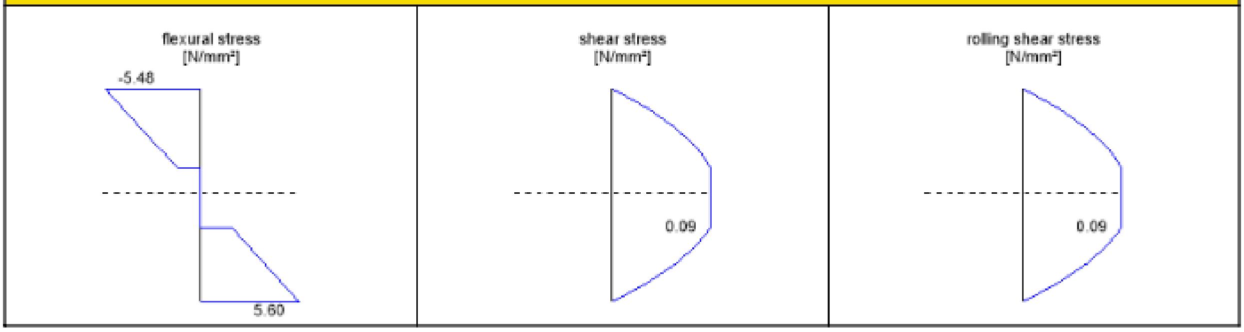
ULS fire shear analysis

field	dist.	$f_{v,k}$	γ_m	k_{mod}	k_{fi}	$f_{v,d}$	V_d	$\tau_{v,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	5.75	4.00	1.00	1.00	1.15	4.60	-19.81	0.09	2 %	LCO4

ULS fire rolling shear

field	dist.	$f_{r,k}$	γ_m	k_{mod}	k_{fi}	$f_{r,d}$	V_d	$\tau_{r,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	5.75	1.15	1.00	1.00	1.15	1.32	-19.81	0.09	7 %	LCO4

stress diagram



flexural stress analysis fire

$M_{y,d}$ =	28.46	kNm		$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				k_{hm} =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	5.60	N/mm²	<	$f_{m,y,d}$ =	30.36	N/mm²	✓
utilization ratio							18 %

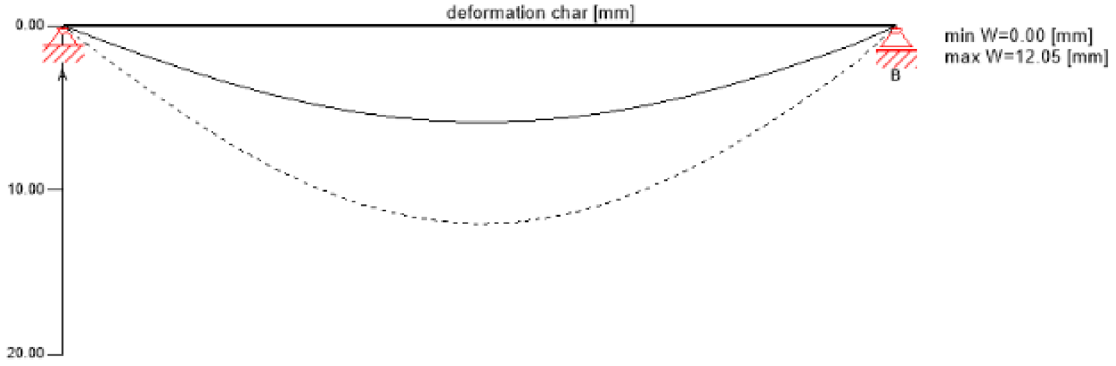
shear stress analysis fire

V_d =	- 19.81	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.00		
				k_{mod} =	1.00		
				k_{fi} =	1.15		
$\tau_{v,d}$ =	0.09	N/mm²	<	$f_{v,d}$ =	4.60	N/mm²	✓
utilization ratio							2 %

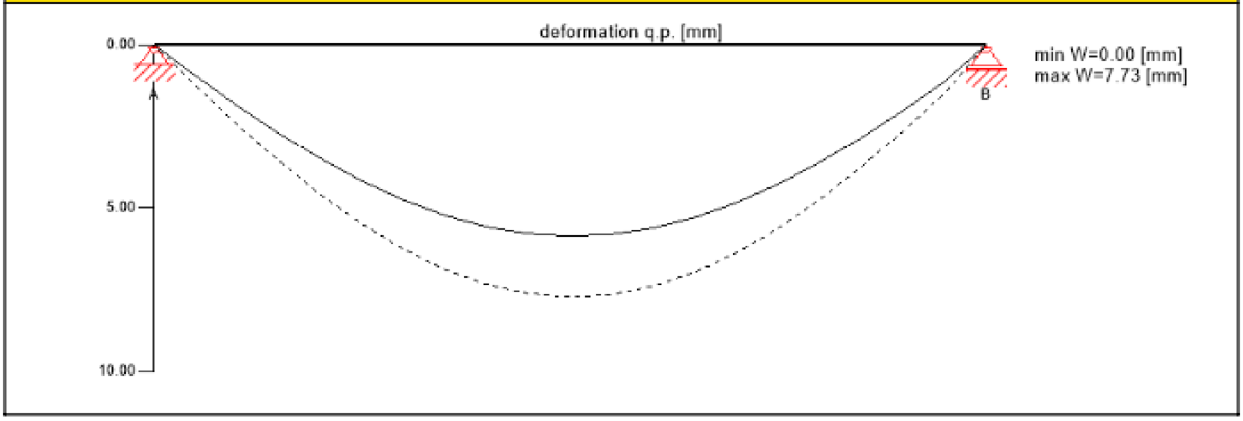
rolling shear analysis fire

V_d =	-19.81	kN		$f_{r,k}$ =	1.15	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.09	N/mm²	<	$f_{r,d}$ =	1.32	N/mm²	✓
utilization ratio							7 %

Service limit state design (SLS) - design results



Service limit state design (SLS) - design results



w_{inst} = w[char]					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/300	19.2	12.1	63 %

w_{fin} = w[char] + w[q.p.]*kdef					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/200	28.7	19.8	69 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/250	23.0	15.5	67 %

vibration analysis

general		
total mass	8.42	[t]
tributary width	3.5	[m]
stiffness longitudinal direction	14060.7	[kNm²]
stiffness cross direction	2688.8	[kNm²]
modal damping	4.0	[%]
α	0.1	[-]
man weight	700.0	[N]
modal mass	2527.8	[kg]

analysis				
criterion	calc.	class II	class II	cl. II
frequency criterion min	6.728 [Hz]	4.5 [Hz]	67 %	✓
frequency criterion	6.728 [Hz]	6.0 [Hz]	89 %	✓
acceleration criterion	0.094 [m/s²]	0.1 [m/s²]	94 %	✓
stiffness criterion	0.081 [mm]	0.5 [mm]	16 %	✓

support design

nr.	type	width	area	k_{mod}	γ_m	$k_{c,90,k}$	$f_{c,k}$	$f_{c,d}$	V_{max}	V_{min}	$\sigma_{c,d}$		ratio
		[mm]	[cm²]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[kN]	[kN]	[N/mm²]		
A	rigid plate	200	6341.10	0.80	1.45	1.50	2.40	1.99	43.32	0.00	0.07	LCO2	3 %
B	rigid plate	200	6341.10	0.80	1.45	1.50	2.40	1.99	43.32	0.00	0.07	LCO2	3 %

support reaction

load case category	k_{mod}	A_V	B_V
		[kN]	
self-weight structure	0.6	7.13	7.13
		7.13	7.13
dead load	0.6	7.92	7.92

support reaction			
load case category	k_{mod}	A_V	B_V
		[kN]	
		7.92	7.92
live load cat. A: domestic, residential areas	0.8	15.84	15.84
		0.00	0.00

reference documents for this analysis	
English title	description
EN 338	EN 338 - Structural timber — Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 1990	EN 1990 - Eurocode — Basis of structural design
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guildeline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
UNI EN 1995-1-2_NA	UNI EN 1995-1-2 - Italy - National Annex - Eurocode 5: Design of timber structures — Part 1-2: General — Structural fire design — National specifications concerning UNI EN 1995-1-2, national comments and national supplements
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings; chapter 7.3

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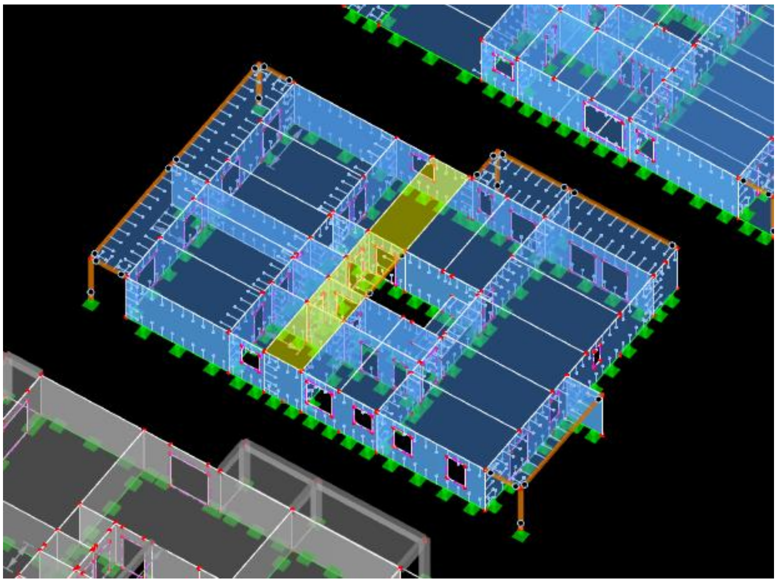
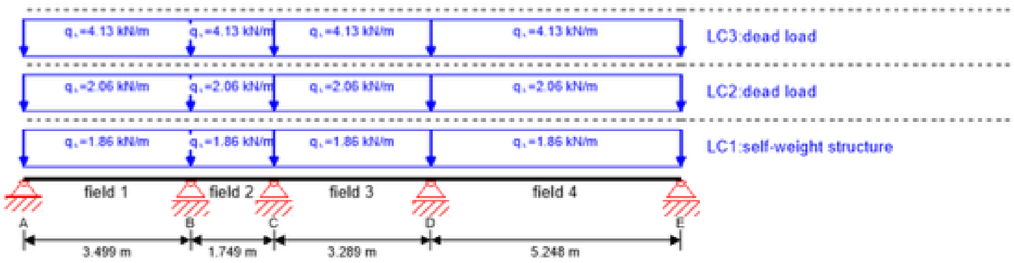


Fig. 9.2 62 floor panel position

system



global utilization ratio 97 %

ULS	25 %	ULS fire	18 %	SLS	51 %	SLS vibration	97 %	support	4 %

section: CLT 180 L5s

layer	thickness	orientation	material
1	40.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	90°	C24 spruce ETA (2014)
3	40.0 mm	0°	C24 spruce ETA (2014)
4	30.0 mm	90°	C24 spruce ETA (2014)
5	40.0 mm	0°	C24 spruce ETA (2014)
t_{CLT}	**180.0 mm**		

section fire: CLT 180 L5s

layer	thickness	orientation	material
1	40.0 mm	0°	C24 spruce ETA (2014)
2	30.0 mm	90°	C24 spruce ETA (2014)
3	37.0 mm	0°	C24 spruce ETA (2014)
t_{CLT}	**107.0 mm**		
time	**90 min**		

fire resistance class:R 90

fire protection layering : 15.0 mm gypsum plasterboard Type F + 40 mm rock wool

gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)
Die Steinwolle-Dämmung der Installationsebene muss eine Mindestrohdichte von 26 kg/m3 und einen Schmelzpunkt >1000 °C aufweisen.

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
27	27	46	25	1	7	66.0	73.0

material values

material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2014)	24.00	14.00	0.35	21.00	2.40	4.00	1.25	12,500.00	690.00	50.00

load

load case groups

	load case category	Typ	duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanet	0.6	1	1.3	1	1	1
LC2	dead load	G	permanet	0.6	1	1.3	1	1	1
LC3	dead load	G	permanet	0.6	1	1.3	1	1	1

LC1:self-weight structure

continous load	
field	load at start
	[kN/m]
1	1.86
2	1.86
3	1.86
4	1.86

LC2:dead load

continous load	
field	load at start
	[kN/m]
1	2.06
2	2.06
3	2.06
4	2.06

LC3:dead load

continous load	
field	load at start
	[kN/m]
1	4.13
2	4.13
3	4.13
4	4.13

ULS combinations

	combination rule
LCO1	1.30/1.00 * LC1 + 1.30/1.00 * LC2 + 1.30/1.00 * LC3

ULS combinations fire

	combination rule
LCO2	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/1.00 * LC3

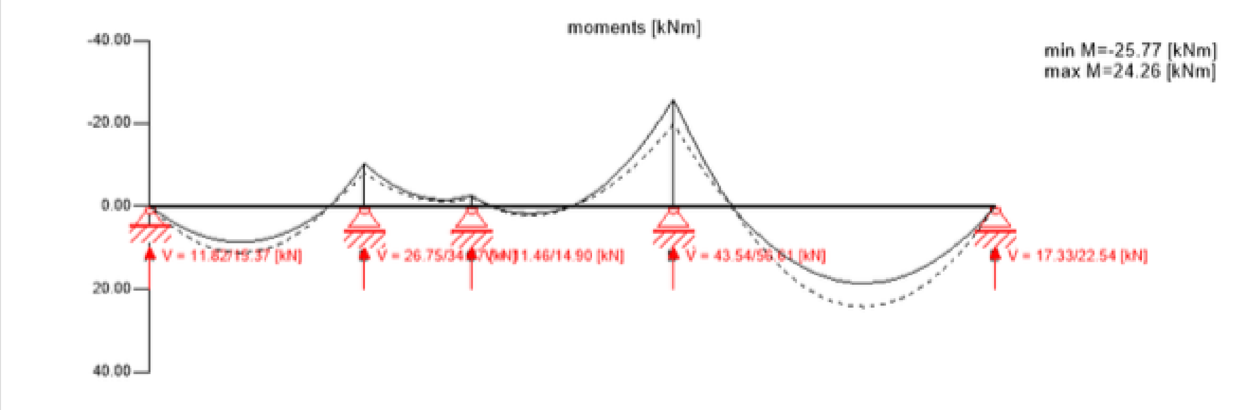
SLS characteristic combination

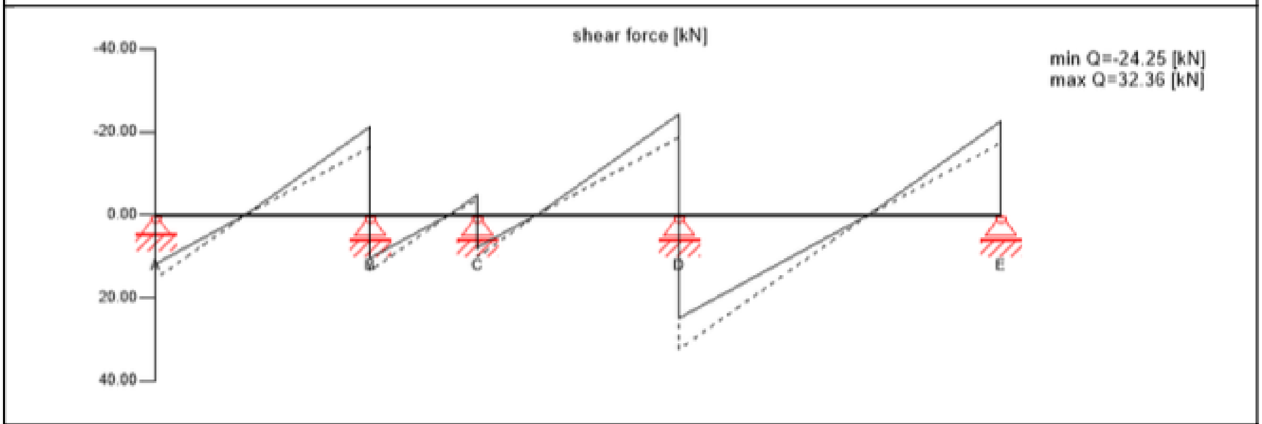
	combination rule
LCO3	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/1.00 * LC3

SLS quasi-permanent combination

	combination rule
LCO4	1.00/1.00 * LC1 + 1.00/1.00 * LC2 + 1.00/1.00 * LC3

Ultimate limit state (ULS) - design results





ULS flexural design

field	dist.	$f_{m,k}$	γ_m	k_{mod}	$k_{sys,y}$	$f_{m,y,d}$	$M_{y,d}$	$\sigma_{m,y,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kNm]	[N/mm²]		
1	1.4	24.00	1.45	0.60	1.10	10.92	11.26	-1.20	11 %	LCO1
2	0.0	24.00	1.45	0.60	1.10	10.92	-10.26	1.10	10 %	LCO1
3	3.29	24.00	1.45	0.60	1.10	10.92	-25.77	2.75	25 %	LCO1
4	0.0	24.00	1.45	0.60	1.10	10.92	-25.77	2.75	25 %	LCO1

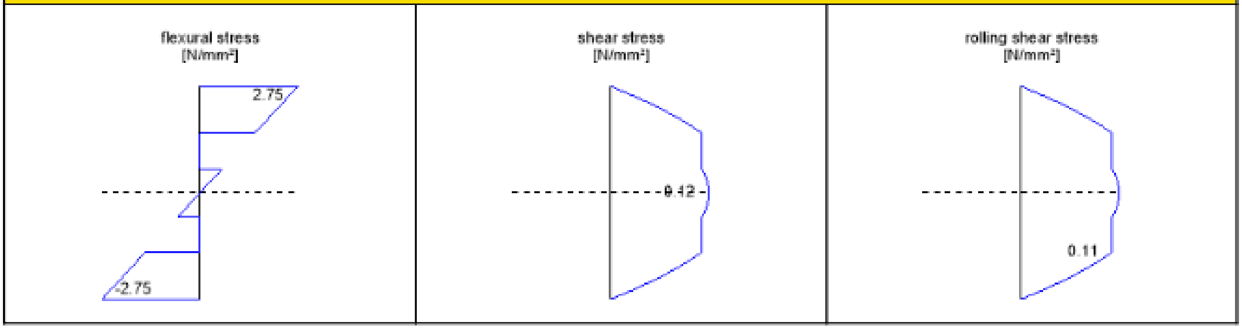
ULS shear analysis

field	dist.	$f_{v,k}$	γ_m	k_{mod}	$f_{v,d}$	V_d	$\tau_{v,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	3.5	4.00	1.45	0.60	1.66	-21.23	0.08	5 %	LCO1
2	0.0	4.00	1.45	0.60	1.66	13.54	0.05	3 %	LCO1
3	3.29	4.00	1.45	0.60	1.66	-24.25	0.09	5 %	LCO1
4	0.0	4.00	1.45	0.60	1.66	32.36	0.12	7 %	LCO1

ULS rolling shear

field	dist.	$f_{r,k}$	γ_m	k_{mod}	$f_{r,d}$	V_d	$\tau_{r,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	3.5	1.15	1.45	0.60	0.48	-21.23	0.07	15 %	LCO1
2	0.0	1.15	1.45	0.60	0.48	13.54	0.05	9 %	LCO1
3	3.29	1.15	1.45	0.60	0.48	-24.25	0.08	17 %	LCO1
4	0.0	1.15	1.45	0.60	0.48	32.36	0.11	23 %	LCO1

stress diagram



flexural stress analysis

$M_{y,d}$ =	-25.77	kNm		$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.45	-	
				k_{mod} =	0.60	-	
				$k_{sys,y}$ =	1.10	-	
				k_{hm} =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,d}$ =	5.79	N/mm²	
$\sigma_{m,y,d}$ =	2.75	N/mm²	<	$f_{m,y,d}$ =	10.92	N/mm²	✓
utilization ratio							25 %

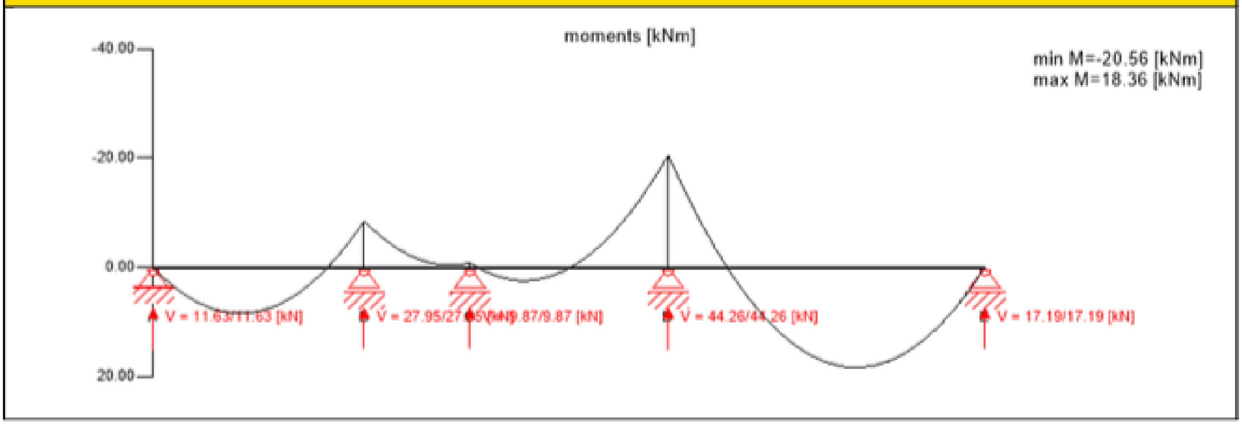
shear stress analysis

V_d =	32.36	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.45		
				k_{mod} =	0.60		
$\tau_{v,d}$ =	0.12	N/mm²	<	$f_{v,d}$ =	1.66	N/mm²	✓
utilization ratio							7 %

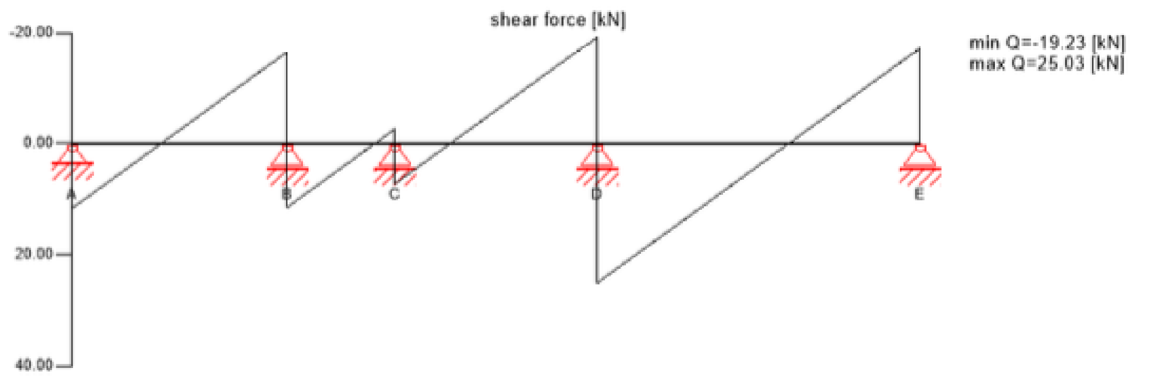
rolling shear analysis

V_d =	32.36	kN		$f_{r,k}$ =	1.15	N/mm²	
				γ_m =	1.45	-	
				k_{mod} =	0.60	-	
$\tau_{r,d}$ =	0.11	N/mm²	<	$f_{r,d}$ =	0.48	N/mm²	✓
utilization ratio							23 %

Ultimate limit state (ULS) fire design - results



Ultimate limit state (ULS) fire design - results



ULS fire flexural design

field	dist.	$f_{m,k}$	γ_m	k_{mod}	$k_{sys,y}$	k_{fi}	$f_{m,y,d}$	$M_{y,d}$	$\sigma_{m,y,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[-]	[N/mm²]	[kNm]	[N/mm²]		
1	3.5	24.00	1.00	1.00	1.10	1.15	30.36	-8.54	-2.24	7 %	LCO2
2	0.0	24.00	1.00	1.00	1.10	1.15	30.36	-8.54	-2.24	7 %	LCO2
3	3.29	24.00	1.00	1.00	1.10	1.15	30.36	-20.56	-5.40	18 %	LCO2
4	0.0	24.00	1.00	1.00	1.10	1.15	30.36	-20.56	-5.40	18 %	LCO2

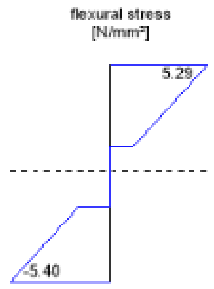
ULS fire shear analysis

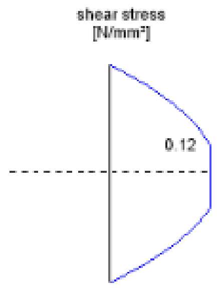
field	dist.	$f_{v,k}$	γ_m	k_{mod}	k_{fi}	$f_{v,d}$	V_d	$\tau_{v,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	3.5	4.00	1.00	1.00	1.15	4.60	-16.52	0.11	2 %	LCO2
2	0.0	4.00	1.00	1.00	1.15	4.60	11.44	0.07	2 %	LCO2
3	3.29	4.00	1.00	1.00	1.15	4.60	-19.23	0.12	3 %	LCO2
4	0.0	4.00	1.00	1.00	1.15	4.60	25.03	0.16	3 %	LCO2

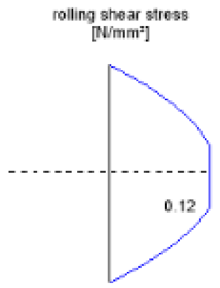
ULS fire rolling shear

field	dist.	$f_{r,k}$	γ_m	k_{mod}	k_{fi}	$f_{r,d}$	V_d	$\tau_{r,d}$	ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	3.5	1.15	1.00	1.00	1.15	1.32	-16.52	0.11	8 %	LCO2
2	0.0	1.15	1.00	1.00	1.15	1.32	11.44	0.07	6 %	LCO2
3	3.29	1.15	1.00	1.00	1.15	1.32	-19.23	0.12	9 %	LCO2
4	0.0	1.15	1.00	1.00	1.15	1.32	25.03	0.16	12 %	LCO2

stress diagram





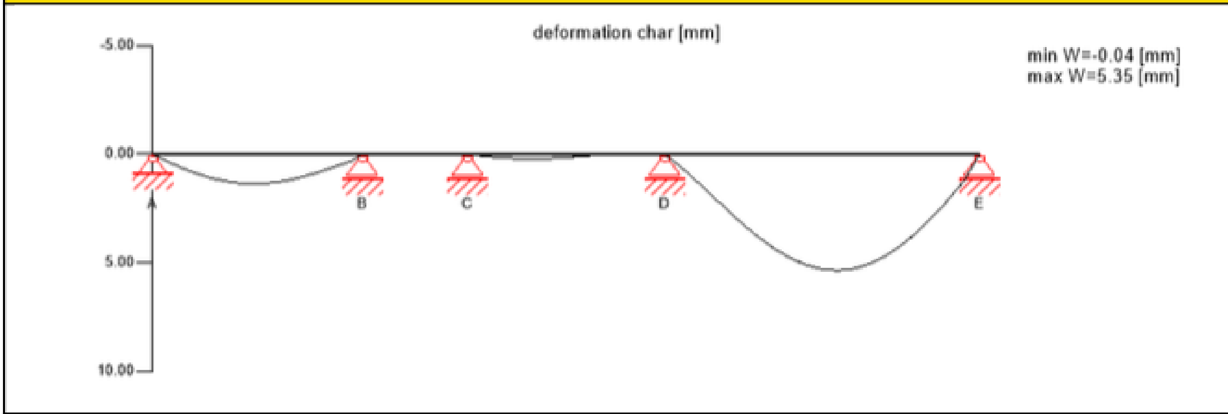


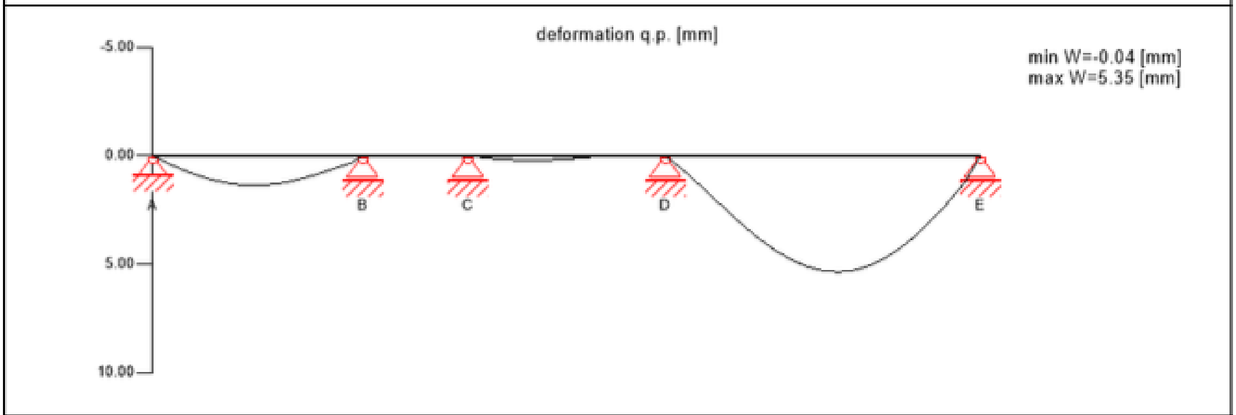
flexural stress analysis fire							
$M_{y,d}$ =	-20.56	kNm		$f_{m,k}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				k_{hm} =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	-5.40	N/mm²	<	$f_{m,y,d}$ =	30.36	N/mm²	✓
utilization ratio							18 %

shear stress analysis fire							
V_d =	25.03	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.00		
				k_{mod} =	1.00		
				k_{fi} =	1.15		
$\tau_{v,d}$ =	0.16	N/mm²	<	$f_{v,d}$ =	4.60	N/mm²	✓
utilization ratio							3 %

rolling shear analysis fire							
V_d =	25.03	kN		$f_{r,k}$ =	1.15	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.16	N/mm²	<	$f_{r,d}$ =	1.32	N/mm²	✓
utilization ratio							12 %

Service limit state design (SLS) - design results





w_{inst} = w[char]					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/300	11.7	1.4	12 %

w_{inst} = w[char]					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
2	1	L/300	5.8	0.0	1 %
3	1	L/300	11.0	0.2	2 %
4	1	L/300	17.5	5.4	31 %

w_{fin} = w[char] + w[q.p.]*kdef					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/200	17.5	2.7	16 %
2	1	L/200	8.7	0.1	1 %
3	1	L/200	16.4	0.4	3 %
4	1	L/200	26.2	10.7	41 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef					
field	K_{def}	limit	w_{limit}	$w_{calc.}$	ratio
		[-]	[mm]	[mm]	
1	1	L/250	14.0	2.7	19 %
2	1	L/250	7.0	0.1	1 %
3	1	L/250	13.2	0.4	3 %
4	1	L/250	21.0	10.7	51 %

vibration analysis

general		
total mass	75.51	[t]
tributary width	3.2	[m]
stiffness longitudinal direction	10526.4	[kNm²]
stiffness cross direction	2013.1	[kNm²]
modal damping	4.0	[%]
α	0.0	[-]
man weight	700.0	[N]
modal mass	3261.8	[kg]

analysis							
criterion	calc.	class I	class II	class I	class II	cl. I	cl. II
frequency criterion min	8.219 [Hz]	4.5 [Hz]	4.5 [Hz]	55 %	55 %	✓	✓
frequency criterion	8.219 [Hz]	8.0 [Hz]	6.0 [Hz]	97 %	73 %	✓	✓
acceleration criterion	0.04 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	80 %	40 %	✓	✓
stiffness criterion	0.091 [mm]	0.25 [mm]	0.5 [mm]	36 %	18 %	✓	✓

support design

nr.	type	width	area	k_{mod}	γ_m	$k_{c,90,k}$	$f_{c,k}$	$f_{c,d}$	V_{max}	V_{min}	$\sigma_{c,d}$		ratio
		[mm]	[cm²]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[kN]	[kN]	[N/mm²]		
A	rigid plate	300	6811.20	0.60	1.45	1.50	2.40	1.49	15.37	0.00	0.02	LCO1	2 %
B	rigid plate	300	7430.40	0.60	1.45	1.80	2.40	1.79	34.77	0.00	0.05	LCO1	3 %
C	rigid plate	300	7430.40	0.60	1.45	1.80	2.40	1.79	14.90	0.00	0.02	LCO1	1 %
D	rigid plate	300	7430.40	0.60	1.45	1.80	2.40	1.79	56.61	0.00	0.08	LCO1	4 %
E	rigid plate	300	6811.20	0.60	1.45	1.50	2.40	1.49	22.54	0.00	0.03	LCO1	2 %

support reaction

load case category	k_{mod}	A_V	B_V	C_V	D_V	E_V
		[kN]				
self-weight structure	0.6	2.73	6.18	2.65	10.05	4.00
		2.73	6.18	2.65	10.05	4.00
dead load	0.6	3.03	6.85	2.93	11.15	4.44
		3.03	6.85	2.93	11.15	4.44
dead load	0.6	6.06	13.72	5.88	22.34	8.89
		6.06	13.72	5.88	22.34	8.89

reference documents for this analysis

English title	description
EN 338	EN 338 - Structural timber — Strength classes

10 Timber beam check

10.1 Bending resistance checks

Bending resistance checks are conducted with reference to paragraph 6.3.3 of the UNI EN 1995-1-1 standard. The following expression must be met:

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1$$

Where:

$\sigma_{m,d}$	is the project tension in bending
$f_{m,d}$	is the project resistance in bending
k_{crit}	is a coefficient that takes into account the low-flexing resistance due to lateral dislike

Il k_{crit} coefficient is assumed to be 1 for beams in which the lateral displacement of the compressed edge is prevented on the entire length and the torsional rotation is prevented from the supports. Otherwise, the coefficient is determined according to the following expression:

$$k_{crit} = \begin{cases} 1 & per\ \lambda_{rel,m} \leq 0{,}75 \\ 1{,}56 - 0{,}75\lambda_{rel,m} & per\ 0{,}75 \leq \lambda_{rel,m} \leq 1{,}4 \\ \dfrac{1}{\lambda_{rel,m}^2} & per\ 1{,}4 < \lambda_{rel,m} \end{cases}$$

Where relative slenderness, $\lambda_{rel,m}$ is:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$$

$\sigma_{m,crit}$ is the critical tension bending calculated according to the classical theory of assumed stability of:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi\sqrt{E_{0,05} I_z G_{0,05} I_{tor}}}{l_{eff} W_y}$$

where

$E_{0,05}$	is the elasticity module fifth percentile value parallel to the fiber;
$G_{0,05}$	is the shearing module fifth percentile value parallel to the fiber;
I_z	it's time for inertia around the weak axis z
I_{tor}	it's the torsional inertia moment
l_{eff}	is the beam effective length depending on the support conditions and the load configuration
W_y	is the module of the section around the strong y axis

10.2 Shear resistance checks

Shearing ckecks are carried out with reference to paragraph 6.1.7 of the UNI EN 1995-1-1 standard. The following expression must be met:

$$\frac{\tau_d}{f_{v,d}} \leq 1$$

where:

τ_d is the shear project tension

$f_{v,d}$ is the resistance project tension

For the resistance shear checking of the flexed elements, the influence of the cracks is taken into account using an effective width of the element given by:

$$b_{eff} = k_{cr} \cdot b$$

where:

b_{eff} is the beam section width

The following values of the coefficient k_{cr} are used:

$k_{cr} = 0{,}67$ for solid wood and laminated wood glued

The maximum stress effort value in a rectangular section is evaluated with the following formula:

$$\tau_d = \frac{3}{2} \cdot \frac{V_d}{k_{cr} \cdot A}$$

where:

A is the cross section beam area

10.3 Beam deformation checks (SLE)

It occurs that the deformation of the structure resulting from the effects of actions and humidity remains within the appropriate limits. The deformation checks are carried out with reference to paragraph 2.2.3 of UNI EN 1995-1-1.

The net arrow $w_{net,fin}$ is:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

where:

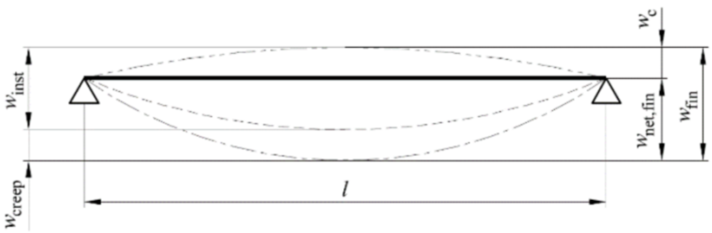
$w_{net,fin}$ is the net end arrow

w_{inst} is the instant arrow

w_{creep} it's the viscoelastic arrow

w_c is the beam length

w_{fin} is the end arrow



The arrow limit values are assumed as shown in the following table.

Condition	w_{ist}	$w_{net,fin}$
Two supports beam	l/300	l/250
Cantiliever beam	l/500	l/125

- Instant deformation

The instant deformationnea w_{inst} is calculated for the rare combination of actions

- End deformation

The end deformation $w_{net,fin}$ is calculated whereas the almost permanent components of the actions cause over time a viscoelastic deformation w_{creep} that can be calculated using the average values of the elastic modules appropriately reduced by the factor (1k_{def})

For consistent structures of elements, components and connections having the same viscoelastic behavior and under the assumption of a linear correlation between the actions and the deformations corresponding to the final deformation w_{fin}, can be considered as:

$$w_{fin} = w_{fin,G} + w_{fin,Q1} + \sum w_{fin,Qi}$$

where:

$w_{fin,G} = w_{inst,,G} \cdot (1 + k_{def})$ for a dead load G
$w_{fin,Q1} = w_{inst,Q1} \cdot (1 + \psi_{2,1} k_{def})$ for a primary live load Q_1
$w_{fin,Qi} = w_{inst,Qi} \cdot (1 + \psi_{2,1} k_{def})$ for secondary live load, Q_i (i >1)

11 Timber pillars checks

11.1 Instability checks

The pillars' instability checks were carried out with reference to paragraph 6.3.2 of the UNI EN 1995-1-1 rule.
The rules recommend that relative leanness ratios be assumed as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

and

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

where :

λ_y and $\lambda_{rel,y}$ are the slenderness ratios corresponding to the flexing around the y-axis (arrow in the z-direction)
λ_z and $\lambda_{re,z}$ sono i rapporti di snellezza corrispondenti alla flessione intorno all'asse z (freccia in direzione y)

It is recommended that the tensions that will be increased as a result of the inflection arrow meet the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Where:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

$$k_y = 0{,}5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel,y} - 0{,}3\right) + \lambda_{rel,y}^2\right)$$

$$k_z = 0{,}5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel,z} - 0{,}3\right) + \lambda_{rel,z}^2\right)$$

where:
β_c is a coefficient for elements within the straightness limits defined in section 10 of the UNI EN 1995-1-1 standard and assumes the following values:

$$\beta_c = \begin{cases} 0{,}2 \; per\ legno\ massiccio \\ 0{,}1 \; per\ legno\ lamellare\ incollato\ e\ LVL \end{cases}$$

This check was carried out with Calculatis of Stora Enso

12 The connections

12.1 The connections role in seismic behavior and modeling criteria

The X-lam wood multi-storey buildings represent a type of construction that is spreading rapidly in the territory thanks to its excellent earthquake resistance skills also of considerable intensity (Dujic et al., 2010; Ceccotti et al., 2008). In fact, the combination of structural lightness (about 1/5-1/4 of the weight of a c.a. structure) and consequently the reduced seismic actions related to this mass, sustainability and rapid execution make these structures very competitive compared to traditional ones. On the other hand, however, the low ductility possessed by structural wood, and the presence of many areas of connection between the prefabricated elements could lead to losses of stability if not perfectly connected to each other.

The overall ductility of the structure is now a fundamental resource for overcoming seismic events with a high return period. In the specific case of wooden constructions it can be obtained thanks to

the plastic behaviour that is concentrated in the areas of mechanical connection between the limbs, which represent the only source of plastic dissipation.
In traditional X-lam massive panel buildings, global ductile behaviour is achieved through the plasticization of connection zones. Hold-downs, designed for the absorption of traction induced by tipping actions, dissipate energy with cyclical traction plasticization, while the anglers, which absorb the floor cut, can dissipate by cutting. The compression, induced by both vertical loads and horizontal actions is absorbed by the contact wood-wood or foundation wood (when on the ground floor are not interposed wooden sleepers between panel and foundation), which because of the tipping actions causes wood to crush (irreversible) in an orthogonal direction to the fibers.

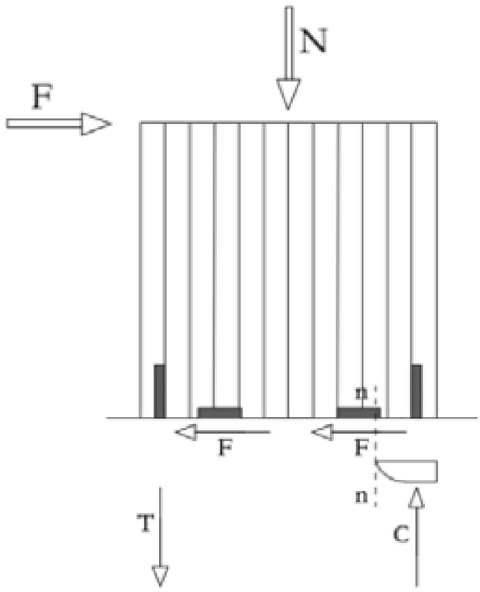


Fig.12.1 Connection zone plastic behavior

The high stiffness and strength of the panel in its plane with respect to the connections leads to the hypothesis of infinitely rigid panel and that therefore all the elastic and inelastic field behavior in terms of both resistance and ductility is governed by connection systems.

12.2 Resistance and flexibility of connections: general notes

The wooden constructions being made with a basic material –structural wood – with distinctly fragile behavior for stresses parallel to the fibercould not in itselfmanifest a dissipative behavior under seismic actions, but the presence of mechanical connections makes these structures capable of exhibiting a plastic response not negligible. As is well known, the links of the wooden structures are divided into two major categories: traditional carpentry unions and mechanical unions.

- Traditional unions, which are found mostly in ancient wooden structures, do not involve the use of metal devices and are obtained by carvings practiced in wooden limbs that allow to create "interlocking" unions between the Elements. These processes, which used to be made by axe, are now carried out with numerically controlled machines.

The traditional union allows to transfer, by contact, mainly compression stresses (e.g. simple tooth joins, double etc. – Fig. 12.2 a), but also cutting (half-wood or swallowtail unions – Fig. 12.2 d and Fig. 12.3 e) or traction (Jupiter dart, right tooth joins - Fig. 12.2b and Fig. 12.2 c). These types of connections, which at most make a monolateral constraint, do not offer any kind of energy dissipation (unless scrolling between the parts) and therefore cannot be used for buildings in seismic zone.

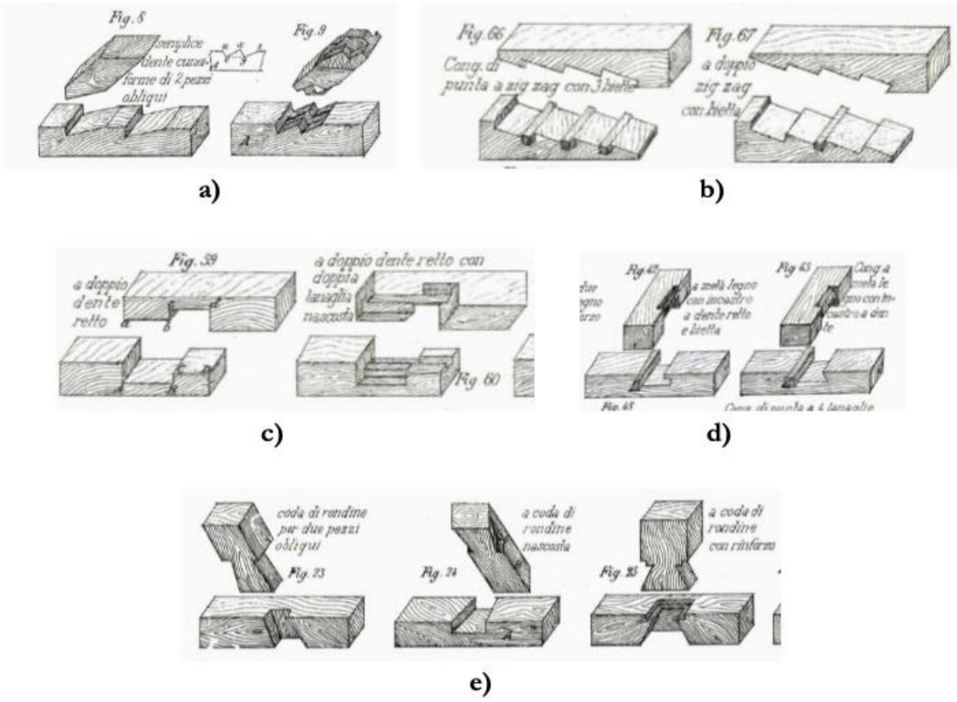


Fig.12.2 Traditional carpentry unions: (a) double-toothed, b) Jupiter's dart, c)a right tooth, d) half wood, e) swallow-tailed

Mechanical connections are made with metal connectors –cylindrical stem, clots, staples, toothed plates etc. –generally positioned in order to absorb cutting stresses, able to connect two or more pieces of wood (wood-wood unions) or wooden elements with metal plates (union-steel wood).

These connections can exhibit significant ductile behavior that develops thanks to the interaction between metal element unnerving and wood displacement. However the ductile mechanisms can be activated only in the extent the connection is designed in order to ward off dangerous fragile breaks (splitting, group effects, etc.), obtained respecting the distance of the connectors with respect to the edges of the connection and the spacing between them.

The ductile behavior and resistance of the connection, in the case of joining with cylindrical stem elements depends on multiple factors including:
- connectors slenderness;
- connector type;
- connector angle within the link.

Figure 12.2 shows a comparison of the force-moving cyclical bond of a lean connector union (case "a") and a stocky connector (case "b"), where slenderness refers to the ratio of the diameter of the connector to the thickness element.

- In the first case the crisis occurs with the unnerving of the connector accompanied by the displacement of the wood: the bond F-δ is characterized by a hysterical cycle quite "wide", but with slight pinching due to the gap that must be recovered in the hole in the unloading phase.

- Instead in the second case the cycle has less hysterical dissipation and a much more pronounced pinching zone, the result of a crisis for wood-only, with a much wider gap due to the connector sliding. Ultimately, connections with leamy connectors have better dissipative capabilities than those with large diameter connectors.

12.2.1 Types available

Metal connections within the XLAM structural system play a key role as they are tasked with transmitting efforts between panel and panel and between panel and elements such as foundations. These connections must transmit the stresses, behave sufficiently plastic, but they must also be easy to find and assemble, not affecting the costs of the overall structure.
These connections are made through the use of cylindrical stem connectors such as screws, nails, threaded bars that are designed to transmit cutting or tipping efforts. They are highly flexible and optimal for solving the problems of a multi-storey wooden building.

- Screws

The screws are cylindrical metal elements threaded with flat or hexagonal head. The thread can be a simple, full-threaded thread –the entire length is threaded – or double-threaded (threaded the beginning and the end). The diameter varies from 4 to 12mm, while the length from 25 to 600mm. They have self-drilling tips and can be mounted using screwdrivers.

(a) Tirafondo. (b) Auto-forante filetto parziale. (c) Auto-forante con filetto totale. (d) Auto-forante doppio filetto.

Fig.12.3 Screws types

- Nails

The nails have a diameter of 2.75 to 8mm and a length between 40 and 200mm. They can be smooth stem or marked stem and be laid with the help of pneumatic or electric nails. They are used for elements that are subject only to cutting.

Fig.12.4 Nail

- Plugs

The plugs are cylindrical elements that can be threaded or smooth-surfaced and must be inserted into previously made punctures. They are mainly used for laminated wood elements.

STA SPINOTTO LISCIO | **SBD** SPINOTTO AUTOFORANTE | **WS** SPINOTTO AUTOFORANTE

Fig.12.5 Plugs types

- Self-drilling pins

Self-drilling pins are cylindrical elements with a special tip that allows you to pierce both wood and metal plates and are typically one centimeter thick. These are used as they ensure an effective connection between plates and panels and reduce assembly time by not having to pre-drill the elements.

- Plates

Steel plates are L-like or rectangular elements, galvanized with heat to avoid oxidation. Through cylindrical connectors, which are arranged in special holes in the plate, they transmit stresses between adjacent panels or between panels and other elements. Having to connect two elements, the plates have a double puncture, one for each element. The angles are used both to counter cutting forces and to lock panels on the concrete plinth.
Elongated angular plates are called "hold-downs".

- Wall-foundation joints

The panels are solidified with the concrete foundation elements in order to counteract the effect of horizontal actions on the building, typically the wind and the earthquake and to transmit the vertical loads.
These connections are divided into two large groups: the cut ones, which are mainly tasked with preventing a horizontal translating of the panel, when it is subjected to a horizontal force, and those with traction, which have the function of preventing a horizontal translating of the panel. rotation of the panel around one of its edges.
The panels can be placed on a screed or on a wooden threshold, usually in larch, with interposition of insulating material to protect the wood from moisture coming from the foundation.

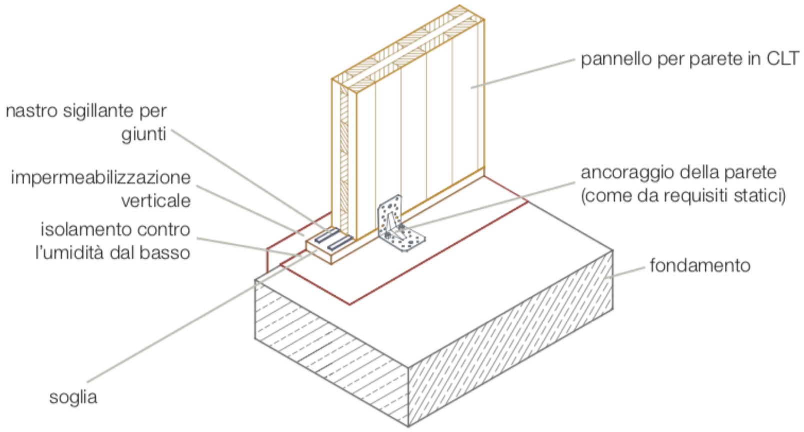


Fig. 12.5 Wall-foundation joints

- Panel-to-panel joints

Panel-panel joints cover both vertical partitions and floors and can be of different types: at an angle, a T, joint, swinged. These are very simple joints that take place through threaded screws and insulation strips to ensure the air of the building. In some types of joint the panels must be grooved in the head to be accommodated in interlocking with other panels or for the interlocking of joint boards.
Some images from the CLT technical folder of Stora Enso are found:

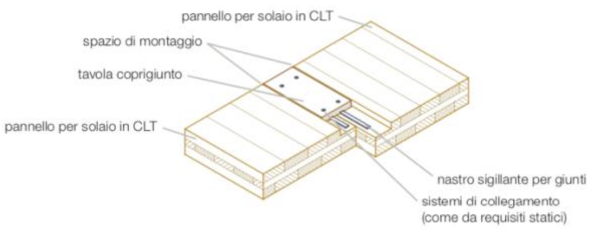


Fig.12.6 Joint for attic (joint table)

Fig.12.7 Application

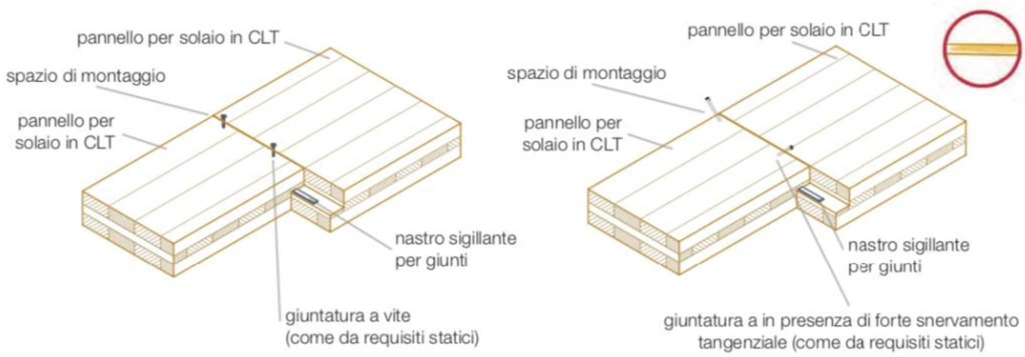


Fig.12.8 Coming for floor swing

Fig.12.9 Applications

The panels side sizes can be limited by production reasons, so it is necessary to connect vertically or horizontally multiple panels of cross boards to make a wall of a certain length or simply a slab.
Some details of the possibilities of connecting wall-wall or slab elements are illustrated in the following figures. These connections must be sized to transmit the cutting forces that you have between one panel and another.

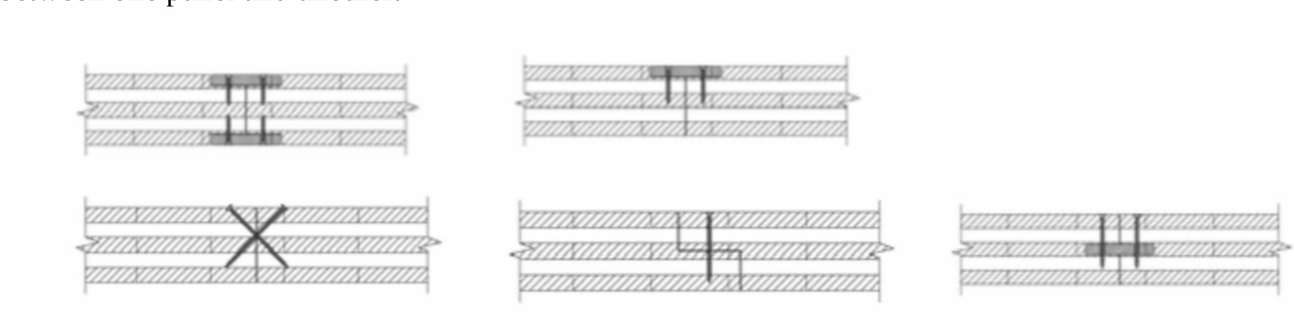


Fig.12.10 floor-floor and wall-wall possible details

- Wall-to-floor-wall joint

CLT buildings are prefabricated systems, where the elements are mounted in operation and then connected to each other by junctions. The construction process is repeated floor by floor: the vertical panels that form the walls are mounted, the floor is closed with horizontal panels and these panels act as a platform for the placement of the vertical panels of the next floor. CLT systems are therefore platform systems, where intermediate horizontaling intersects vertical elements. In the wall-slab-wall node, structural continuity must then be restored through junction systems similar to those seen for connection with the foundation works. Therefore, two different types of connections must be used in these nodes, one for the transmission of cutting efforts and one for lifting the panel. The slab panel and the wall panel below can be reached by self-threading screws inserted into the head surface of the attic. While the panels of the upper floor are fixed with metal corners. These junctions allow both horizontal loads to be transferred to the floors and traction forces to the floors. Insulation sheaths must be placed at each intersection to ensure hermeticity. For the same reason along the inner or outer junctions can be applied sealing stickers.

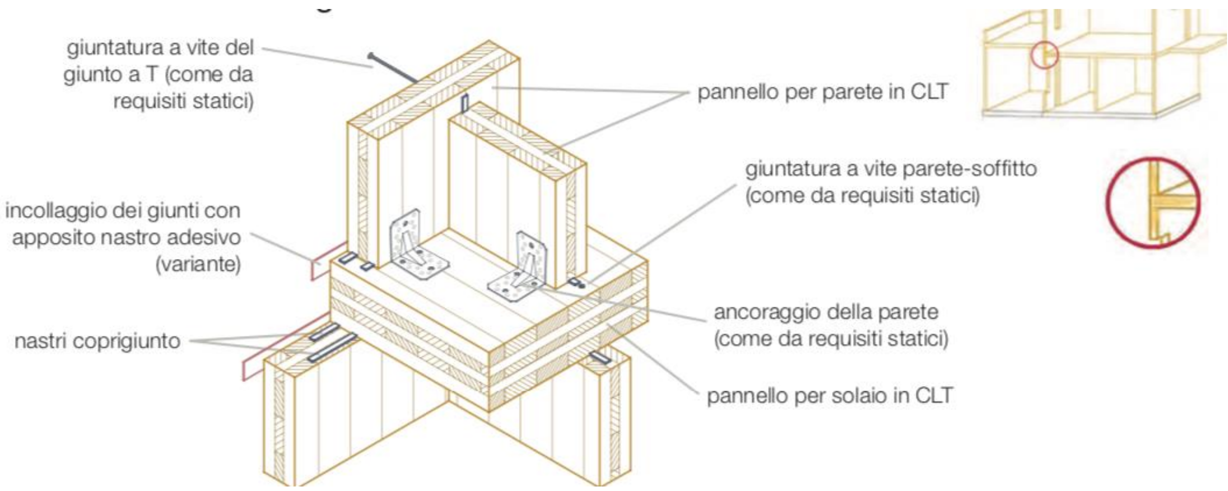
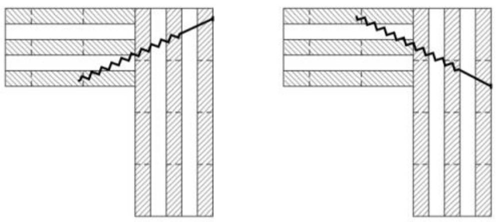


Fig.12.11 Wall-to-slab-wall connection

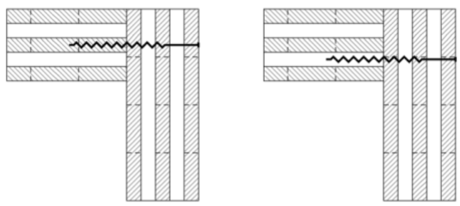
- Corner joints

The corner connection between orthogonal walls is essential to ensure greater robustness to the entire construction. In addition, this constraint can be a garrison for the out-of-floor forces of the walls, due, for example, to the forces of lateral stabilization or the effect of the wind on the walls.

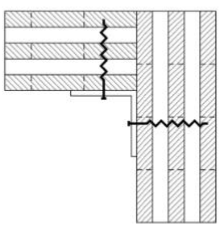
If you use self-threading screws you have to be careful how they are inserted into a layer of the panel where the fibers are parallel to the axis of the screw, the junction can be considered ineffective or otherwise with reduced resistance. Because it is not always easy to pinpoint the exact spot where to insert the screw, it is recommended to insert it slightly inclined to the axis of the panel in order to intersect multiple layers.



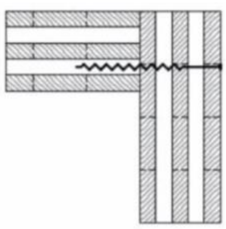
Corner joints: correct screws insertion



Corner joints: inserting screws NOT correct



Corner joints: correct connection via brackets



Corner joints:
link NOT correct

Fig.12.12 Corner joints

12.3 Connection behaviour patterns in CLT or Xlam structures

In Fig. 12.13, a comparison of the force-moving cyclical bond of a lean connector union (case "a") and a stocky connector (case "b") is reported, where leanness refers to the ratio of the diameter of the connector to the thickness of the wooden elemento.

- In the first case the crisis occurs with the unnerving of the connector accompanied by the displacement of the wood: the bond F-δ it is characterized by a hysterical cycle quite "wide", but with slight pinching due to the gap that must be recovered in the hole in the unloading phase.

- Instead in the second case the cycle has less hysterical dissipation and a much more pronounced pinching zone, the result of a crisis for wood-only, with a much wider gap due to the connector sliding. Ultimately, connections with leamy connectors have better dissipative capabilities than those with large diameter connectors.

The connectors interaction with wood penalizes the cyclical behavior of the connection, in fact if the crisis were governed only by the cyclical unnerving of the connector the cycle would correspond to that of steel.
The flexibility of the connection is also influenced by the number of connectors used, in fact the use of a large number of means of union is a factor in the infragilit of the connection.
Finally, the type of connector used also changes the behavior of the connection, in fact for the same conditions connectors such as nails and screws allow for a more accentuated ductile behavior than pins, bolts or plugs.

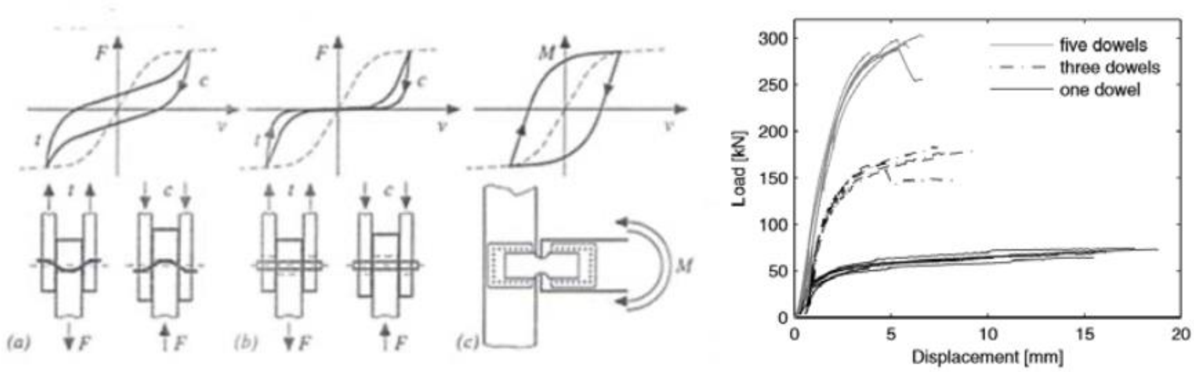


Fig.12.13 Connection cyclical behavior

More recently, shear connections have also been introduced that provide for the arrangement of cylindrical stem connectors arranged in a sloping manner with respect to the fiber of the wood. This arrangement, compared to the classic one that sees the connectors arranged perpendicular to the fibers, allows to achieve higher levels of resistance, but with a consequent reduction in ductility.

It is interesting to note that at present, analytical formulations are available for the calculation of mechanical connections, which can also be found in national and international regulatory codes, which allow us to determine the strength of the connection (Johansen, 1949); on the contrary, there are no formulations that lead to the definition of the level of ductility achieved by a given connection and it is necessary to refer to experimental tests.

The connection flexibility is a fundamental requirement for the wooden structures seismic response as they represent the only place of formation of plastic "hinges". In fact wooden elements are characterized by an elasto-fragile behavior and do not allow any kind of plasticization (Fig. 12.14).

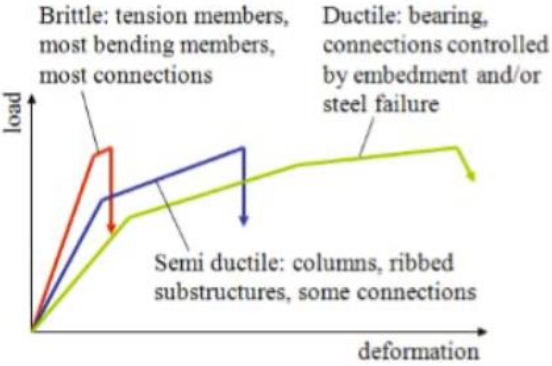


Fig.12.14 Connection Behavior

The seismic behaviour of X-lam-panel buildings is significantly influenced by the mechanical connections between the panels, so a proper understanding, first, and modelling, then, of mechanical connections is a step inescapable in the overall schematization of the walls.
In the literature, results of experimental tests have been found which investigated the cyclical behaviour of hold-down and angular connections, on the basis of which it was possible to calibrate theoretical cyclical models useful for the purpose of carrying out analysis on individual panels or on whole walls.
For example, an experimental campaign carried out by Gavric et al. (2011a) analysed the behaviour of hold-downs and angles stressed by traction and cyclical cutting respectively, highlighting their excellent behavioural performance.
In more detail, in Fig. 12.15 and 12.16, force-shift diagrams for hold-downs and angles are represented. The hysterical cycle of these connections is characterized by significant dissipation of energy, pinching, degradation of resistance and stiffness and a softening branch after peak resistance. Hold-downs, unlike anglers, have non-symmetric behavior because local instability renders their contribution for compression stresses null.

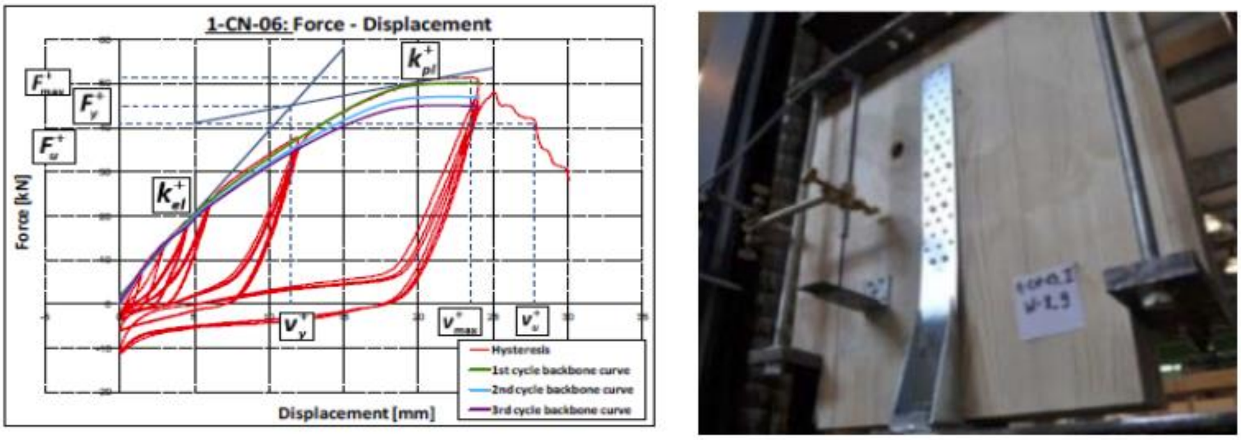


Fig.12.15 A hold down connection cycling behavior

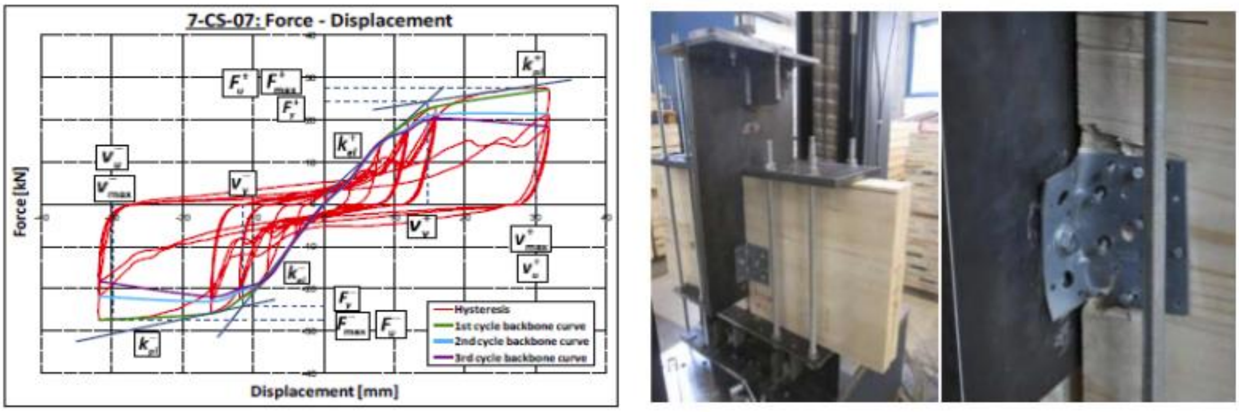


Fig.12.16 A bracket connection cycling behavior

The results of the experimental tests then allowed to develop theoretical cyclical models for both hold-downs and angles that can be adopted as plastic "hinges" for conducting non-linear analyses of multi-story X-lam walls. In Fig. 12.17 and 12.18, the theoretical bonds obtained by Rinaldin et al are represented. (2013) derived as a back-bone curve of experimental bonding, which take into account pinching, degradation ofreturnand stiffness and post-peak softening. In particular, this envelope curve was obtained with reference to the third load cycle in order to take into account, in a conservative way, the effects of the cumulative damage during cyclical behaviour and therefore the degradation of resistance between the first and third cycles. These links can be usedtodefine the cyclical behavior of non-linear springs that simulate the behavior of connections in the panel-panel and panel-foundation contact section.

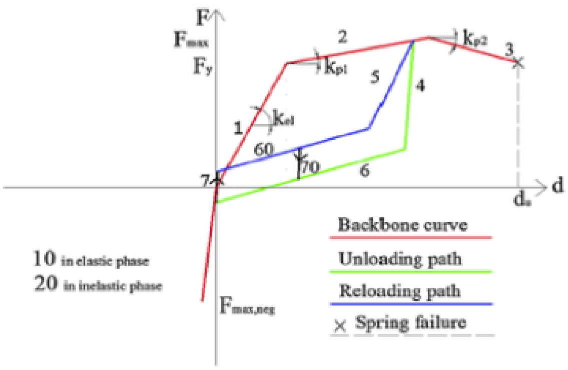


Fig.12.17 Hysterical hold down law

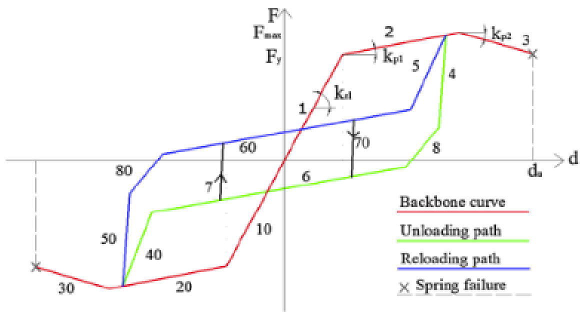


Fig.12.18 Bracket hysterical law

This mechanical pattern of connection behaviour is, among those proposed in the literature, that describes more accurately the real cyclical response of hold-downs and angulars and which is well suited to describe the behavior of wooden panels.
Other authors also use force-shifting bonds characterized by an envelope curve similar to that propagated by Gavric et al. For example, Dujic et al. (2010) propose a legmane of the type represented in Figure 12.19 for angular hold-downs respectively. The authors themselves point out, however, that the complexity of the bond leads, very often, to problems of numerical instability of the solver used for analysis and therefore adopt a simplification of this link, as represented in Figure 12.20.

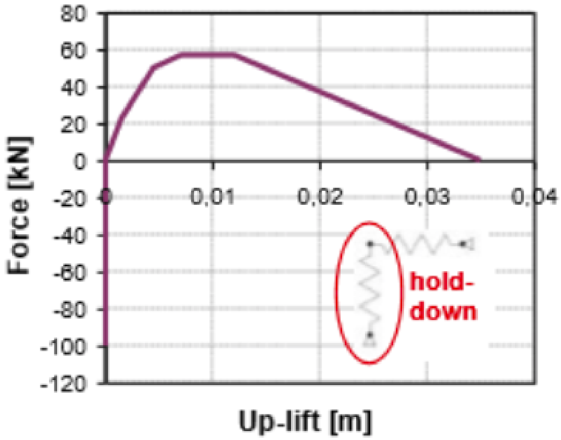


Fig.12.19 Hold down’s costitutive bond

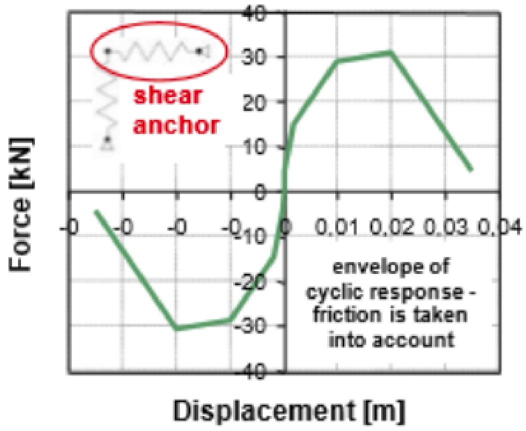


Fig.12.20 Brakets' costitutive bond

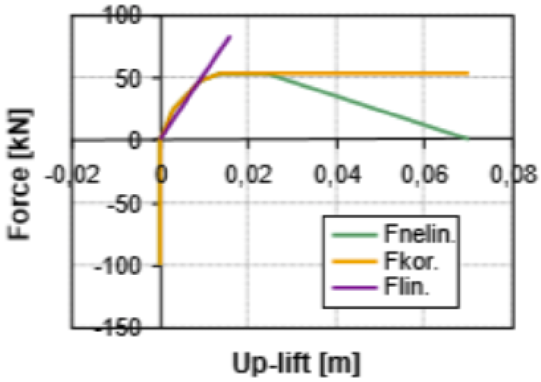


Fig.12.21 Costitutive bond

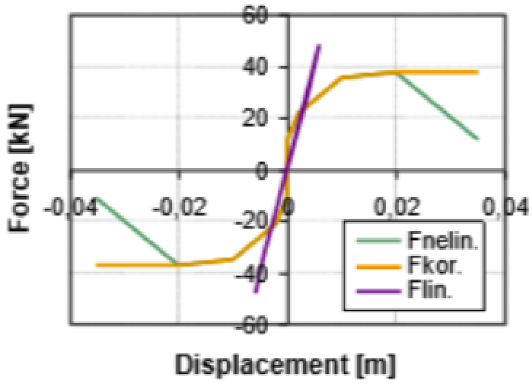
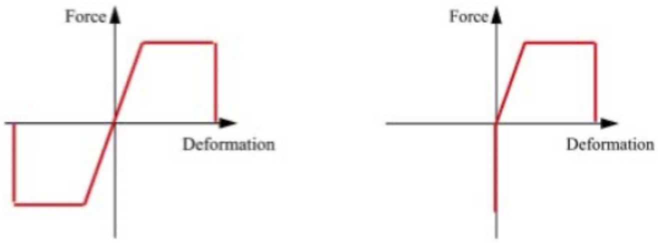


Fig.12.22 Simplify costitutive bond

A further simplification of these models that simulate hold-down and angular behaviour is suggested in Embury & Karacabey (2013) which assume an elastic-perfectly plastic bond (Fig.12.23). This link, even if it is characterized by a higher level of approximation, still provides satisfactory results both numerically when implemented in computing software.



(a) Shear connectors and (b) Hold-down

Fig.12.23 Elasto-plastic costitutive bond

Individual connection behavior analysis is the basis for investigating the behavior of X-lam panels with links to the base. Cyclical tests results on panels and buildings in full scale (Dujic et al., 2010) demonstrate good performance of behavior even under earthquakes characterized by high accelerations (even up to 0.8g).
Experimental tests carried out by Gavric et al. (2011, 2015b) point out that the cyclical behavior typical of an X-lam panel with mechanical connections at the base is characterized by large hysterical cycles with significant energy dissipation, pinching, resistance degradations and stiffness. At the end of the test the panel shows no signs of damage and all the energy dissipation has been concentrated

in the connections to the base. In addition, the flexion and shearing deformations of the panel can be considered to be completely negligible compared to the total deformation and that therefore the latter is almost all produced by the elastic and inelastic deformation of the links. Figure 12.24 represents a cyclical behavior of an X-lam panel.

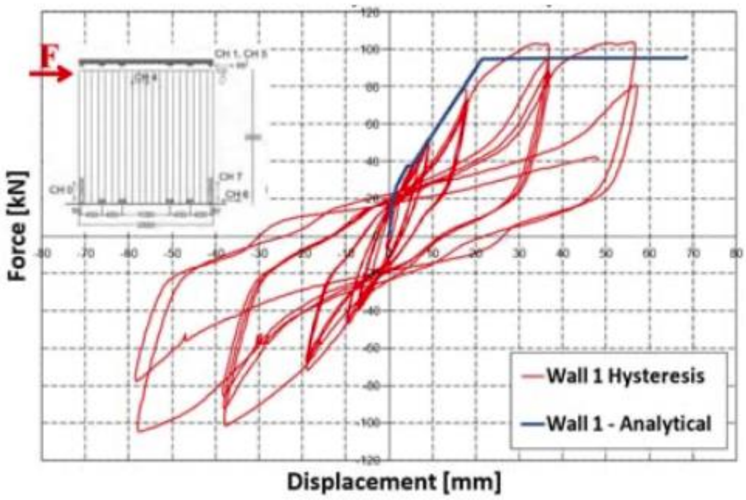


Fig.12.24 Xlam panel cyclical behavior

The panels' cyclical behavior is influenced by multiple factors such as the size of the vertical load, the geometry of the panel, and the type of connection. Some experimental tests on full-scale panels carried out by Gavric et al. (2013) demonstrate that the amount of vertical load slightly changes the shape of the hysterical cycle: the increase in assial load produces an increase in initial strength and stiffness without significantly affecting the ductility (Fig 12.24).

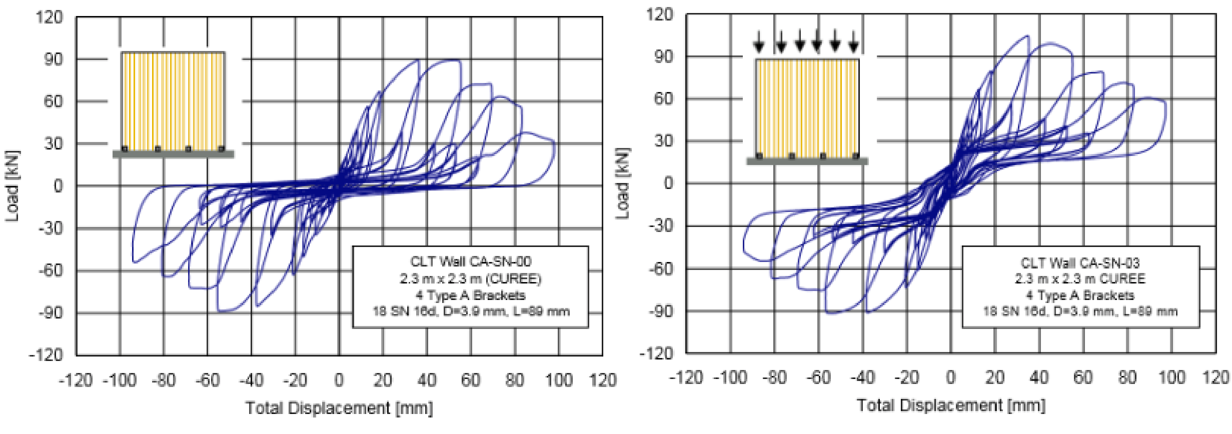
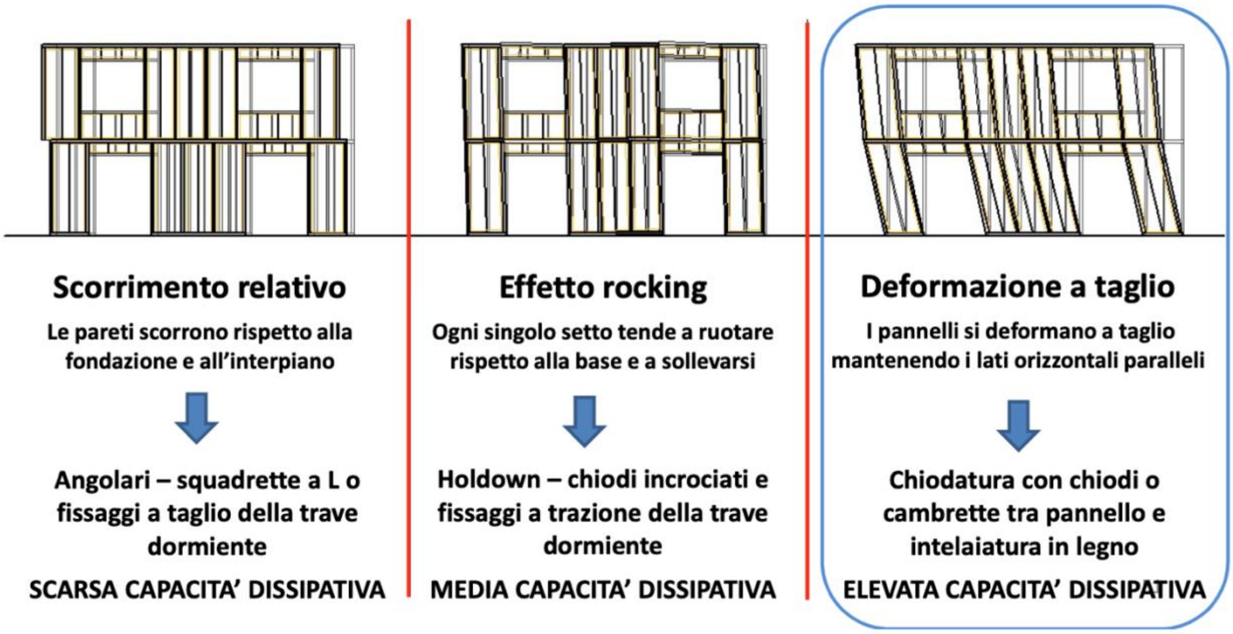


Fig.12.25 Vertical load influence on cyclical behavior

A different cyclical response is also observed depending on the type of predominant behavior that the X-lam panel can exhibit: rocking, rocking-sliding, sliding. In the case of rocking, which is activated for panels and with high axle loads, the crisis typically occurs by progressive unnerving of traction hold-downs(Figure 12. 25) and later also angular, accompanied bythe crushing of the wood in the compressed area; instead, thereare no deformations of the connectors in a horizontal direction, this is a testament to the absence of sliding of the panel.



In the case of rocking and sliding the connection crisis occurs due to an interaction between traction and cutting, while in the last case it occurs for translating only.
The prevailing rocking behavior is the one that guarantees better seismic behavior of the panels, as it is characterized by a re-centering capacity due to the vertical load and resistance of the connection; this means that in the event of a seismic event the panel is able to return to the unformed condition without significant residual displacements. The residual damage, however, will be all concentrated in the connection which will have undergone a process of cyclical plasticization, this was also demonstrated by Ceccotti (2008).

Fig.12.26 Pulling hold down crisis

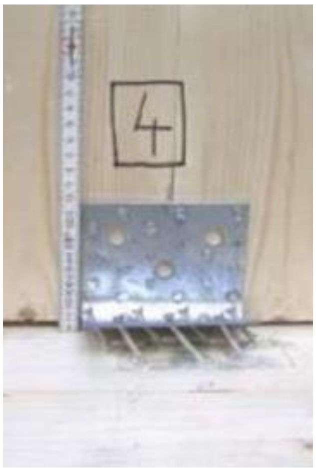


Fig.12.27 Pulling and sliding Bracket's crisis

This behaviour suggests that the design of the panels and more generally of the walls should be conducted by ensuring that plasticization focuses on the elements deputies to resist traction, while the angles, designed with appropriate over-resistance, they should always remain in the elastic field in order to avoid residual swiping in the presence of seismic actions.

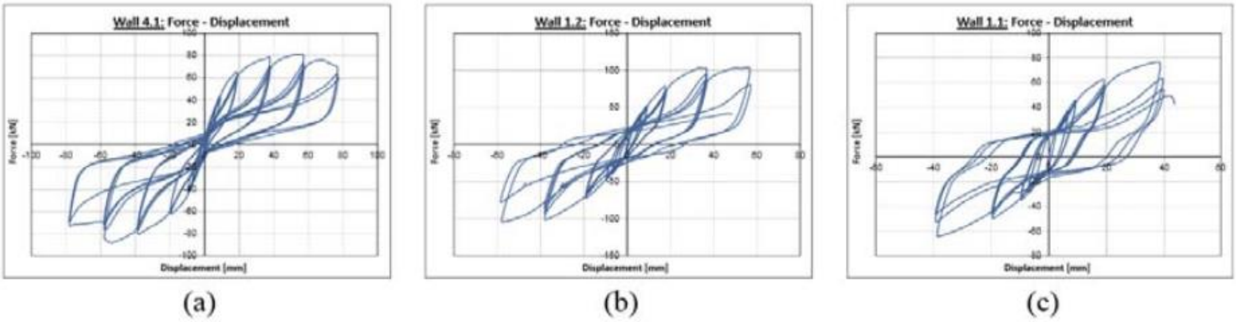


Fig.12.28 Xlam panels' cyclical behavior: a) rocking, b) rocking-sliding, c)sliding

Experimental tests carried out by Fragiacomo et al. (2011b) demonstrate that the ductile behavior of the connection is also influenced by the type of means of union used, in particular their infixion length within the panel. In fact, with reference to Fig. 3.17 it can be noted that, in the case of both hold-downs and angles, the use of nails with a lower infixation length (40mm) leads to lower resistance values and fragile behavior than those of length (60mm), which leads to higher strength and ductility values.

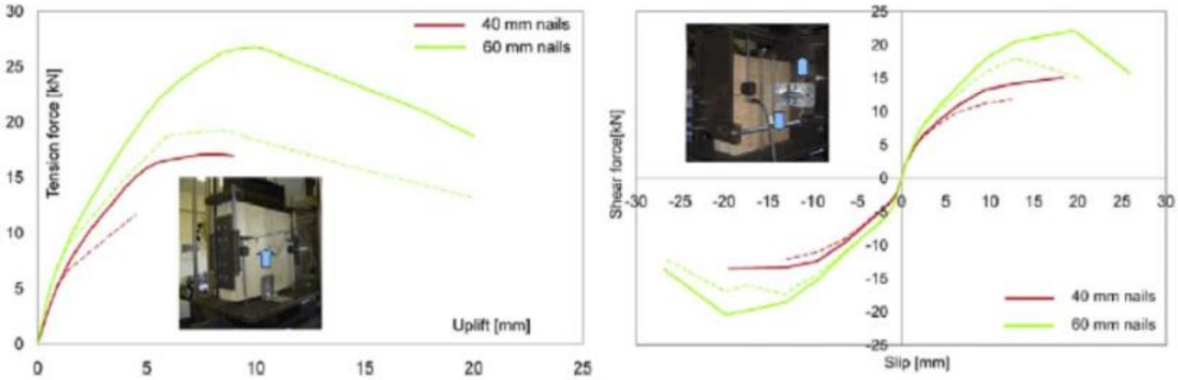


Fig.12.29 Brackets Behavior(left) and hold downs (right) as nail infixance length changes

All the results of experimental tests carried out on full-scale panels show that the crisis of the whole system is always attributable to the plasticization of mechanical unions and in particular the tense hold-downs, while the wooden panel - which undergoes negligible deformations of its own - assumes a rigid blocking behavior with respect to the connections. The location of damage in connection zones is generally a positive aspect of the seismic behaviour of these buildings because, as a result of the seismic event, it is possible to repair the damaged area.
How connections are crisisd also depends on how it was designed. In most experimental tests analysed, the break almost always occurs with an interaction between wood displacement and connector unyielding, while hold-downs and angles show some over-strength compared to connectors. In these cases, the union was designed by entrusting the wood-connector interaction-resistant mechanism, which therefore allows to develop even a non-binding
negligible ductile behavior. However, the local ductility can be concentrated in the metal saucers (hold-down and angular) while the interaction connector-wood is designed to remain in the elastic field, but always careful to avoid breakage (Latour et al., 2013).

12.4 Connections design

In the case in question were used screws between slab and panel below and small brackets to join slab and panel of the wall above and hold downs in the corners.

Two approaches were used for the connection design:

12.4.1 Approach by connections stiffness design

The multilayer XLAM structural elements were considered as a homogeneous orthotropic material.

The following connections stiffness based on scientific studies were used in the first analysis.
The values are as follows:

	Hold-down (staffe parete solaio)		Ancoraggio a taglio (parete -solaio)		Ancoraggio parete-parete
Piano	up-lift asse z (sollevamento) [kN]	up-lift asse x,y (sollevamento)[kN]	Shear (taglio) Kser [kN/m]	up-lift (sollevamento)[kN/m]	Rigidezza [kN/m]
1°	11250	5238,32	8000	5000	2292
2° e 3°	5250	5239,32	7500	5000	2292
4°,5°,6°	4375	5240,32	7200	5000	2292
7°,8°,9°	3500	4500	4500	3500	2292

We define the following sizes:
n= number of brackets per meter
Stiffness relative to the nails pull out= 12000kN/m
Stiffness related to nail shearing= 5238,32kN/m

Floor 1:
n=1,333 per metre for connections between wall and slab
n=5 per metre for connections between wall and ceiling in XLAM
n=8 per metre for wall-to-wall connections

- Shear connections between wall and slab:
  n*Kser= 1,333*8000=10666,4 kN/m^2

- Connections for lift between wall and slab:
  n*Kser= 1,333*5000=6666,5 kN/m^2

- Connecting wall and ceiling
  n*Kser= 5*2292=11460kN/m^2

- Connection between wall and wall
  n*Kser=8*2292=18336 kN/m^2

- Connection for lifting nails between ceiling-and-wall and wall-and-wall
  n*Kser =5*12000=60000 kN/m^2

- Torsion connections
  Lifting in z-axis: uz=11250*10= 112500 kN/m^2
  Shear in x and y axis: ux=uy= 5238,32*10=52383,2 kN/m^2

Piani 2 e 3 :
n=1,333 per metre for connections between wall and slab
n=5 per metre for connections between wall and ceiling in XLAM
n=8 per metre for wall-to-wall connections

- Shear connections between wall and slab:
  n*Kser= 1,333*7500=9999,75 kN/m^2

- Connections for lift between wall and slab:

n*Kser= 1,333*5000=6666,5 kN/m^2

- Connecting wall and ceiling
  n*Kser= 5*2292=11460 kN/m^2

- Connection between wall and wall
  n*Kser=8*2292=18336 kN/m^2

- Connection for lifting nails between ceiling-and-wall and wall-and-wall
  n*Kser =5*12000=60000 kN/m^2

- Connessioni per torsione
  Sollevamento lungo z: uz=5250*10= 52500 kN/m^2
  Shear in x and y axis: ux=uy= 5238,32*10=52383,2 kN/m^2

Piani 4,5 e 6 :

n=1 per metre for connections between wall and slab
n=4 per metre for connections between wall and ceiling in XLAM
n=8 per metre for wall-to-wall connections

- Shear connections between wall and slab:
  n*Kser= 1*7200=7200kN/m^2

- Connections for lift between wall and slab:
  n*Kser= 1*5000=5000 kN/m^2

- Connecting wall and ceiling
  n*Kser= 4*2292=9168 kN/m^2

- Connection between wall and wall
  n*Kser=8*2292=18336 kN/m^2

- Connection for lifting nails between ceiling-and-wall and wall-and-wall
  n*Kser =4*12000=48000 kN/m^2

- Torsion connections
  Lifting in z-axis: uz=5250*10= 52500 kN/m^2
  Shear in x and y axis: ux=uy= 5238,32*10=52383,2 kN/m^2

Piani 7,8 e 9:

n=1 per metre for connections between wall and slab
n=3 per metre for connections between wall and ceiling in XLAM
n=8 per metre for wall-to-wall connections

- Shear connections between wall and slab:
  n*Kser= 1*4500=4500kN/m^2

- Connections for lift between wall and slab:
  n*Kser= 1*3500=3500 kN/m$_2$

- Connecting wall and ceiling
  n*Kser= 3*2292=6876 kN/m$_2$

- Connection between wall and wall
  n*Kser=8*2292=18336 kN/m$_2$

- Connection for lifting nails between ceiling-and-wall and wall-and-wall
  n*Kser =3*12000=36000 kN/m$_2$

- Torsion connections
  Lifting in z-axis: uz=3500*10= 35000 kN/m$_2$
  Shear in x and y axis: ux=uy= 4500*10=45000 kN/m$_2$

This was the first approach used but then we used an exact procedure for a result's comparison.

Procedimento esatto

12.4.2 Approach accurate calculation of connections through Calculatis

In this case connections were calculated exactly through Calculatis.
To make such a calculation, the first 3 plans were considered as connected plans in the same way, the same applies to plans 4.5 and 6 and finally 7.8 and 9. Let us remember that the ground floor is the reinforced concrete foundation plan whose design is deferred to other courses.

RothOBLAAS VGS 9x240 screws were used for connections between adjacent walls and orthogonal walls between them, including a graphical representation:

Fig.12.30 ROTHOBLAAS VGS 9x240 screws

For the design of such connections we start from the model on RFEM in the most unfavourable seismic case, in this case is the one in which we consider an action equal to 30% long X and an action equal to 100% long Y.

You run the model and read the results related to the Vy effort (Long Y cut effort) as shown in the figure

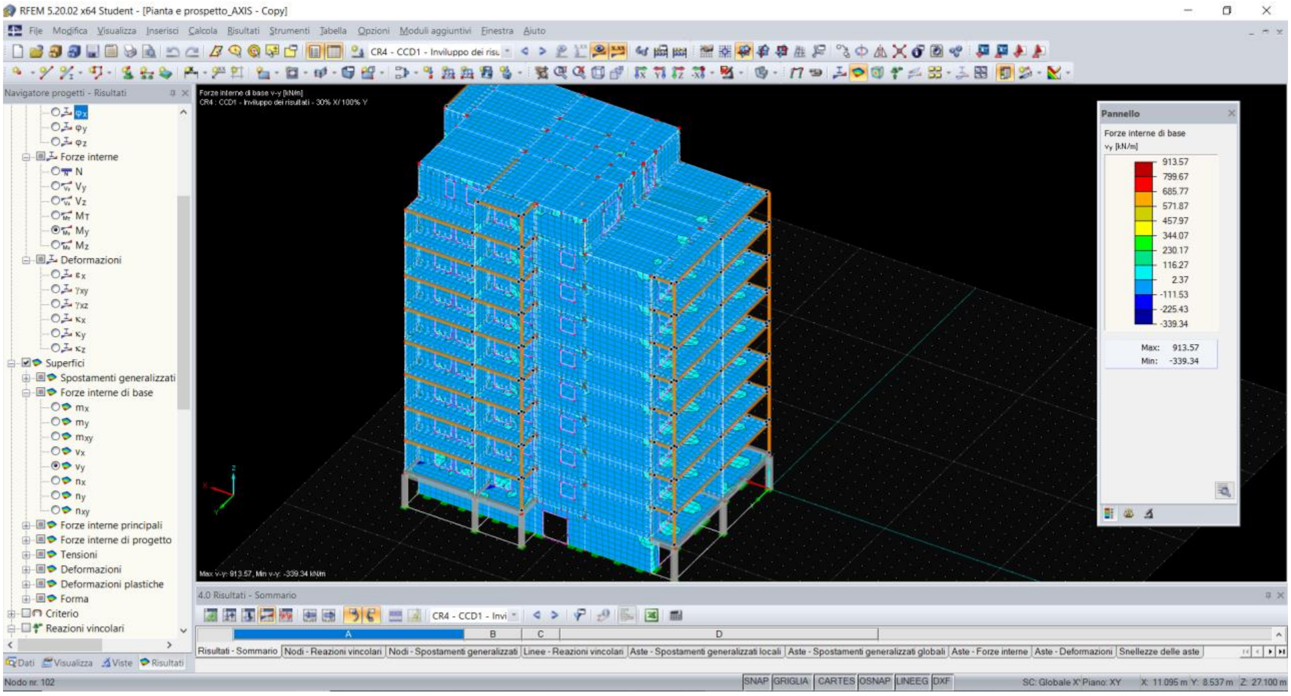


Fig.12.31 Vy values on RFEM model

At this point, once you read Vy's value, you report that value back to Calculatis.
On Calculatis you enter the value of Vy in the Fx pane, you place Fyo and Kmod .0.9; after that you insert the thickness of the panels considered and finally you reveal the screws to be used and you start the verification.
Below is a demonstration of the verification obtained with Calculatis for both connections between adjacent and otogol walls and a summary table of the results of all the walls.

- Design and verify screws between orthogonal and adjacent walls
- Orthogonal walls

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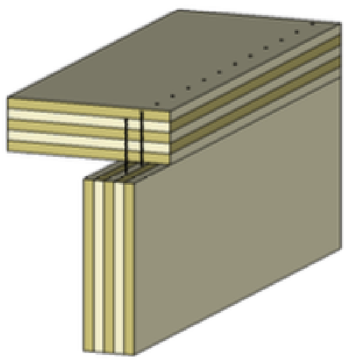
connection

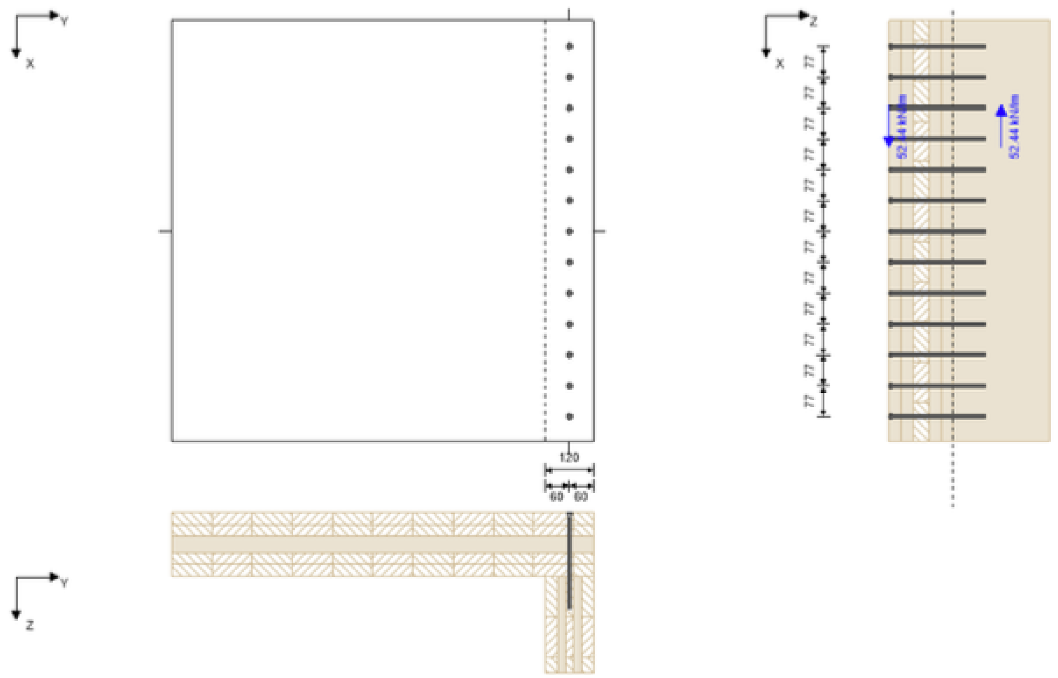


F_x	52.44	kN/lm
F_y	0	kN/lm
K_{mod}	0.9	-
material 1	C24 spruce ETA (2014)	
ρ_k	3.5	kN/m³
panel 1	CLT 160 L5s - 2	
orientation cover layer	X direction	
material 2	C24 spruce ETA (2014)	
ρ_k	3.5	kN/m³
panel 2	CLT 120 C5s	
orientation cover layer	✓	
connector type	Rothoblaas VGS	
connectors	9/240	
setup	vertical	
diameter	9	mm
head diameter	16	mm
length	240	mm
thread length	230	mm
connector positions	✗	
pre-drilled	✗	

analysis

analysis	existing	limit	unit	utilization
width 1	120	90	mm	75 %
width 2	120	90	mm	75 %
thickness 1	160	42	mm	26 %
thickness 2	80	42	mm	53 %
Fv	4033.88	4033.88	N	100 %
count	13	22.222	count / lm	58 %

system sketch



minimum spacing						
Name	$a_{1,min}$	$a_{2,min}$	$a_{3c,min}$	$a_{3t,min}$	$a_{4c,min}$	$a_{4t,min}$
	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
CLT top	45	23	54	54	36	54
CLT bottom	45	23	54	54	36	54

result in layers

element 1						
X	Dicke	Typ	α	l_{eff}	$l_{eff,v}$	$F_{ax,Rk}$
[mm]	[mm]		[°]	[mm]	[mm]	[N]
0	10	L	90	0	0	0
10	20	L	90	20	20	2106
30	30	L	90	30	30	3159
60	40	C	90	40	40	4212
100	30	L	90	30	30	3159
130	25	L	90	25	25	2633
155	5	L	90	0	0	0

results										
$b_{1,min}$	$b_{2,min}$	$f_{h,k,1}$	$f_{h,k,2}$	β	$t_{pen,1}$	$t_{pen,2}$	$l_{eff,1}$	$l_{eff,2}$	$t_{1,req}$	$t_{2,req}$
[mm]	[mm]	[N/mm²]	[N/mm²]	[-]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
90	90	26.12	26.12	1.00	160.00	80.00	145.00	65.00	42	42

results												
$M_{y,r,k}$	$F_{ax,Rk}$	$F_{head,Rk}$	$F_{tens,Rk}$	$F_{ki,Rk}$	$F_{v,Rk}$	$F_{v,Rd}$	$F_{v,Ed}$	$F_{ax,Rd}$	$F_{ax,Ed}$	Anz.	$Anz._{max}$	$a_{erf.}$
[Nmm]	[N]	[N]	[kN]	[kN]	[N]	[N]	[kN/lm]	[N]	[kN/lm]	[Stk/m]	[Stk/m]	[mm]
27244.13	6844.50	0.000	25.400	15.812	5826.72	4033.88	52.44	4738.50	0.00	13.00	22.22	77

reference documents for this analysis	
English title	description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-09/0322	GH Various Angle Brackets
ETA-11/0496	Rotho Blaas TITAN Angle Brackets

- Adjacent walls

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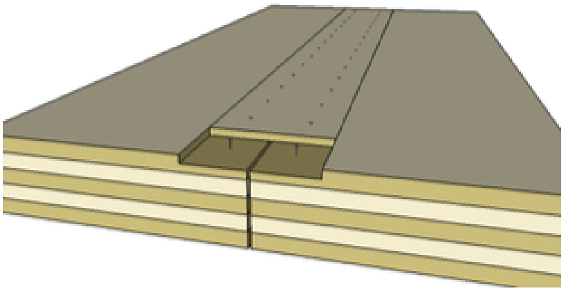
connection

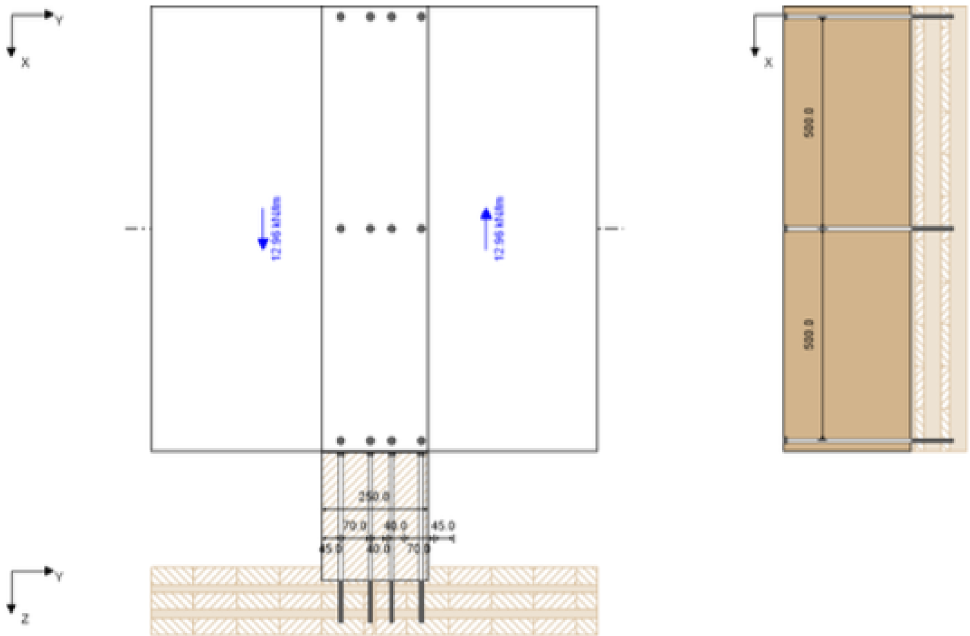


F_x	12.96	kN/lm
K_{mod}	0.9	-
material 1	C24 spruce ETA (2014)	
ρ_k	3.5	kN/m³
panel 1	CLT 160 L5s	
orientation cover layer	X direction	
connector type	Rothoblaas HBS	
connectors	10/400	
diameter	10	mm
head diameter	18.25	mm
length	400	mm
thread length	100	mm
splice plate	C24 softwood	
splice plate width	250	mm
splice plate thickness	300	mm
notch depth	30	mm
number of rows	2	
pre-drilled	×	

analysis

analysis	existing	limit	unit	utilization
plate width	250	190	mm	76 %
plate thickness	300	46	mm	15 %
CLT thickness	100	46	mm	46 %
Fv	3240	4030.552	N	80 %
count	3.215	40	count / lm	8 %

system sketch



minimum spacing

Name	$a_{1,min}$	$a_{2,min}$	$a_{3c,min}$	$a_{3t,min}$	$a_{4c,min}$	$a_{4t,min}$
	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
CLT	40	25	60	60	25	60
plate	50	40	40	80	30	30

result in layers

element 1						
X	Dicke	Typ	α	l_{eff}	$l_{eff,v}$	$F_{ax,Rk}$
[mm]	[mm]		[°]	[mm]	[mm]	[N]
30	5	L	90	0	0	0
35	5	L	90	5	5	585
40	20	C	90	20	20	2340
60	40	L	90	40	40	4680
100	20	C	90	20	20	2340
120	40	L	90	0	0	0

results										
$b_{1,min}$	$b_{2,min}$	$f_{h,k,1}$	$f_{h,k,2}$	β	$t_{pen,1}$	$t_{pen,2}$	$l_{eff,1}$	$l_{eff,2}$	$t_{1,req}$	$t_{2,req}$
[mm]	[mm]	[N/mm²]	[N/mm²]	[-]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
95	190	25.83	25.83	1.00	100.00	300.00	85.00	0.00	46	46

results												
$M_{y,r,k}$	$F_{ax,Rk}$	$F_{head,Rk}$	$F_{tens,Rk}$	$F_{ki,Rk}$	$F_{v,Rk}$	$F_{v,Rd}$	$F_{v,Ed}$	$F_{ax,Rd}$	$F_{ax,Ed}$	Anz.	$Anz._{max}$	$a_{erf.}$
[Nmm]	[N]	[N]	[kN]	[kN]	[N]	[N]	[kN/lm]	[N]	[kN/lm]	[Stk/m]	[Stk/m]	[mm]
35829.64	3497.163	497.156	0.000	0.000	5821.91	4030.55	12.96	2421.11	0.00	3.22	40.00	500

reference documents for this analysis	
English title	description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-09/0322	GH Various Angle Brackets
ETA-11/0496	Rotho Blaas TITAN Angle Brackets

The summary tables are as follows:

Floor	Wall 1		Wall 2	Thickness 1 [mm]	Thickness 2 [mm]	Vy [kN/m]	screw's type	n° of connectors per linear meter	n tot	i [mm]
	119 sx	perpendicular	219			52,44	ROTHOBLASS VGS 9/240	13	39	77
	119 dx	perpendicular	214			39,25	ROTHOBLASS VGS 9/240	10	30	103
	38	next	141			12,96	ROTHOBLASSHBS 10/400	3	9	500
	141	perpendicular	193			39,24	ROTHOBLASS VGS 9/240	10	30	103
	137	perpendicular	184			19,9	ROTHOBLASS VGS 9/240	5	15	203
	135	perpendicular	177			19,77	ROTHOBLASS VGS 9/240	5	15	204
	177	perpendicular	170			17,69	ROTHOBLASS VGS 9/240	5	15	228
	176	perpendicular	27			12,56	ROTHOBLASS VGS 9/240	4	12	321
	172	next	176			12,34	ROTHOBLASSHBS 10/400	3	9	500
	27	perpendicular	12			37,26	ROTHOBLASS VGS 9/240	10	30	108
	15	perpendicular	190			56,32	ROTHOBLASS VGS 9/240	14	42	72
	15	next	10			3,72	ROTHOBLASSHBS 10/400	3	9	500
1 2 3	10	perpendicular	215	160 L5s	160 L5s	99,1	ROTHOBLASS VGS 9/280	21	63	48
	8	perpendicular	221			37,12	ROTHOBLASS VGS 9/240	10	30	109
	221	perpendicular	2			47,37	ROTHOBLASS VGS 9/240	12	36	85
	219	perpendicular	159			14,17	ROTHOBLASS VGS 9/240	4	12	285
	210	perpendicular	159			10,57	ROTHOBLASS VGS 9/240	3	9	382
	209	perpendicular	162			19,08	ROTHOBLASS VGS 9/240	4	12	211
	165	perpendicular	215			113,68	ROTHOBLASS VGS 9/320	21	63	48
	144	perpendicular	198			8,95	ROTHOBLASS VGS 9/240	3	9	451
	152	perpendicular	198			7,96	ROTHOBLASS VGS 9/240	3	9	500
	202	perpendicular	155			28,26	ROTHOBLASS VGS 9/240	7	21	143
	201	perpendicular	166			38,52	ROTHOBLASS VGS 9/240	11	33	105
	190	perpendicular	164			17,6	ROTHOBLASS VGS 9/240	5	15	229
	188	perpendicular	157			5,28	ROTHOBLASS VGS 9/240	3	9	500
	185	perpendicular	146			84	ROTHOBLASS VGS 9/240	21	63	48

Floor	Wall 1		Wall 2	Thickness 1 [mm]	Thickness 2 [mm]	Vy [kN/m]	screw's type	n° of connectors per linear meter	n tot	i [mm]
4 5 6	509	perpendicular	596	140 L5s	140 L5s	12,47	ROTHOBLASS VGS 9/240	3	9	323
	474	perpendicular	591			12,5	ROTHOBLASS VGS 9/240	3	9	323
	474	next	518			11,96	ROTHOBLASSHBS 10/400	3	9	500
	514	perpendicular	561			13,34	ROTHOBLASS VGS 9/240	3	9	302
	512	perpendicular	554			12,44	ROTHOBLASS VGS 9/240	3	9	324
	547	perpendicular	553			11,29	ROTHOBLASS VGS 9/240	3	9	357
	553	next	549			11,37	ROTHOBLASSHBS 10/400	3	9	500
	463	perpendicular	549			11,27	ROTHOBLASS VGS 9/240	3	9	358
	463	perpendicular	567			11,3	ROTHOBLASS VGS 9/240	3	9	357
	461	perpendicular	458			33,83	ROTHOBLASS VGS 9/240	9	27	119
	461	next	456			10,13	ROTHOBLASSHBS 10/400	3	9	500
	456	perpendicular	592			97,68	ROTHOBLASS VGS 9/240	21	63	49
	545	perpendicular	598			32,66	ROTHOBLASS VGS 9/240	9	27	124
	598	perpendicular	450			40,05	ROTHOBLASS VGS 9/240	10	30	101
	596	perpendicular	536			99,93	ROTHOBLASS VGS 9/240	21	63	48
	591	perpendicular	521			130	ROTHOBLASS VGS 11/400	19	57	56
	589	perpendicular	529			30,61	ROTHOBLASS VGS 9/240	8	24	132
	587	perpendicular	536			9,17	ROTHOBLASS VGS 9/240	3	9	415
	539	perpendicular	586			20,15	ROTHOBLASS VGS 9/240	5	15	200
	542	perpendicular	586			40,24	ROTHOBLASS VGS 9/240	10	30	100
	542	perpendicular	578			34,86	ROTHOBLASS VGS 9/240	9	27	116
	575	perpendicular	539			31,46	ROTHOBLASS VGS 9/240	8	24	128
	529	perpendicular	579			37,28	ROTHOBLASS VGS 9/240	9	27	108
	575	perpendicular	532			28,51	ROTHOBLASS VGS 9/240	7	21	141
	526	perpendicular	575			75,51	ROTHOBLASS VGS 9/240	19	57	53
	572	perpendicular	526			32,78	ROTHOBLASS VGS 9/240	9	27	123
	570	perpendicular	523			34,08	ROTHOBLASS VGS 9/240	9	27	118
	523	perpendicular	561			96,14	ROTHOBLASS VGS 9/280	21	63	50
	562	perpendicular	534			81,33	ROTHOBLASS VGS 9/240	21	63	50
	541	perpendicular	567			47,36	ROTHOBLASS VGS 9/240	12	36	85

Floor	Wall 1		Wall 2	Thickness 1 [mm]	Thickness 2 [mm]	Vy [kN/m]	screw's type	n° of connectors per linear meter	n tot	i [mm]
7 8 9	658	perpendicular	745	120 L5s	120 L5s	29,89	ROTHOBLASS VGS 9/240	8	24	135
	740	perpendicular	623			11,89	ROTHOBLASS VGS 9/240	3	9	339
	623	next	667			14,73	ROTHOBLASSHBS 10/400	3	9	500
	667	perpendicular	719			8,73	ROTHOBLASS VGS 9/240	3	9	462
	663	perpendicular	710			29,77	ROTHOBLASS VGS 9/240	7	21	136
	661	perpendicular	703			12,53	ROTHOBLASS VGS 9/240	3	9	322
	696	perpendicular	702			16,27	ROTHOBLASS VGS 9/240	5	15	248
	702	next	698			10,25	ROTHOBLASSHBS 10/400	3	9	500
	698	perpendicular	613			12,72	ROTHOBLASS VGS 9/240	3	9	317
	608	perpendicular	613			23,04	ROTHOBLASS VGS 9/240	6	18	175
	611	perpendicular	716			44,16	ROTHOBLASS VGS 9/240	11	33	91
	611	next	606			14,51	ROTHOBLASSHBS 10/400	3	9	500
	604	perpendicular	741			53,37	ROTHOBLASS VGS 9/240	14	42	76
	604	perpendicular	747			20,08	ROTHOBLASS VGS 9/240	5	15	201
	747	perpendicular	600			60,16	ROTHOBLASS VGS 9/240	15	45	67
	685	perpendicular	745			51,1	ROTHOBLASS VGS 9/240	13	39	79
	740	perpendicular	670			76,57	ROTHOBLASS VGS 9/240	19	57	53
	738	perpendicular	678			42,3	ROTHOBLASS VGS 9/240	11	33	95
	736	perpendicular	688			40,85	ROTHOBLASS VGS 9/240	11	33	99
	735	perpendicular	685			12,47	ROTHOBLASS VGS 9/240	3	9	323
	741	perpendicular	691			32,24	ROTHOBLASS VGS 9/240	8	24	125
	727	perpendicular	692			50,08	ROTHOBLASS VGS 9/240	13	39	81
	688	perpendicular	727			16,83	ROTHOBLASS VGS 9/240	5	15	240
	728	perpendicular	681			19,42	ROTHOBLASS VGS 9/240	5	15	208
	678	perpendicular	724			38,64	ROTHOBLASS VGS 9/240	10	30	104
	675	perpendicular	721			10,35	ROTHOBLASS VGS 9/240	3	9	390
	672	perpendicular	719			34,22	ROTHOBLASS VGS 9/240	9	27	118
	710	perpendicular	672			76,21	ROTHOBLASS VGS 9/240	19	57	53
	711	perpendicular	683			29,04	ROTHOBLASS VGS 9/240	7	21	139
	714	perpendicular	683			52,47	ROTHOBLASS VGS 9/240	13	39	77
	690	perpendicular	714			40,95	ROTHOBLASS VGS 9/240	11	33	99
	721	perpendicular	675			41,32	ROTHOBLASS VGS 9/240	11	33	98
	721	perpendicular	681			19,95	ROTHOBLASS VGS 9/240	5	15	202
	728	perpendicular	678			14,99	ROTHOBLASS VGS 9/240	3	9	269

The analytical part of the tests carried out thanks to Calculatis is reported

- Shear checking merge elements

The project shear resistance of each individual panel that makes up a given wall is calculated according to the model proposed by the UNI EN 1995-1-1 standard in step 9.2.4.2 "Simplified analysis of diaphragm walls – Method A".
For a wall made of several panels, it is recommended that the design value of the capacity is calculated by:
where:

$$F_{v,Rd} = \sum_i F_{i,v,Rd}$$

$F_{i,v,R,d}$ represents the carrying capacity of the panel's design slab in accordance with the 9.2.4.2(4) and 9.2.4.2(5) points of the UNI EN 1995-1-1 standard.

Panels containing a door or window opening are not considered to contribute to the carrying capacity of the slab in accordance with paragraph 9.2.4.2 (6) of the UNI EN 1995-1-1 standard.

The following rules apply to panels with sheets on both sides:
- if the sheets and means of union are all of the same type and size, then the carrying capacity of total plate of the wall is assumed as the sum of the plate-carrying capacity of the individual sides
- If different types of sheets are used and similar means of merge are used (scrolling modules that do not vary more than 20% each other), then 75% of the plate-bearing capacity of the weaker side is taken into account.
- in other cases, 50% of the plate-carrying capacity of the weaker side is taken into account

- Connector Resistance

Resistance values are evaluated according to Johansen's theory in paragraph 8.2.2 of the UNI EN 1995-1-1 standard for the case of panel-wood connections to a cutting plane.
The characteristic supporting capacity for a single cutting plane and for a single means of union is assumed as the minimum value determined by the following expressions:

$$F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d$$

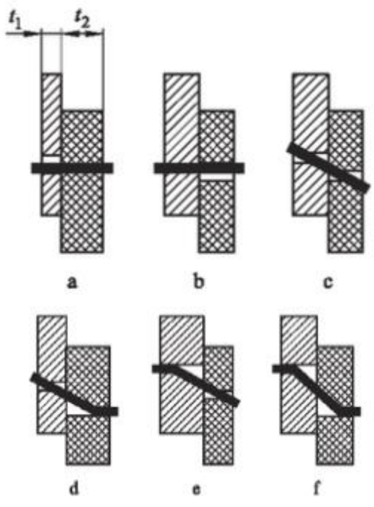
$$F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d$$

$$F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2\beta^2\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right] + \beta^3\left(\frac{t_2}{t_1}\right)^2} - \beta\left(1+\frac{t_2}{t_1}\right)\right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,d} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,e} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta} \cdot \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta\right] + \frac{F_{ax,Rk}}{4}$$

$$F_{v,Rk,f} = 1{,}15 \cdot \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$$



In the expressions shown, the first term represents the backbone according to the

Johansen theory, instead the second term $\frac{F_{ax,Rk}}{4}$ is the contribution due to the*rope effect.*

- Connector extraction resistance

Characteristic nail extraction capacity, $F_{ax,Rk}$, is assumed as the lowest value among those obtained by the following expressions:

- For smooth stem nails

$$F_{ax,Rk} = \begin{cases} f_{ax,k,punta}\, d\, t_{pen,telaio} \\ f_{ax,k,testa}\, d\, t + f_{head,k}\, d_h^2 \end{cases}$$

- For nails with improved grip

$$F_{ax,Rk} = \begin{cases} f_{ax,k,punta}\, d\, t_{pen,telaio} = f_{ax,k,350} \left(\frac{\rho_{k,tip}}{350}\right)^{0.8} d\, t_{pen,telaio} \\ f_{head,k}\, d_h^2 = f_{head,k,350} \left(\frac{\rho_{k,head}}{350}\right)^{0.8} d_h^2 \end{cases}$$

where:

$f_{ax,k,punta}$ is the characteristic resistance to extraction on the side of the tip

$f_{ax,k,testa}$ is the characteristic resistance to extraction on the side of the head

$f_{head,k}$ is the characteristic resistance of the crossing of the element by the head, on the side of the head of the nail

d is the nail's diameter

d_h is the nail's head diameter

$t_{pen,telaio}$ is the minimum value between the penetration length from the side of the tip and the length of the threaded part inserted into the element that receives the tip

t is the thickness of the element of the side of the head

$F_{ax,Rk}$ is the characteristic value of the connector extraction resistance

$F_{v,Rk}$ is the characteristic value of the carrying capacity of the means of union assessed considering both the Johansen contribution and the contribution due to the rope effect

Rope effect limit: represents the percentage limit of the contribution to lateral carrier capacity due to the rope effect

- Shear check on coating sheets

The project shear resistance of each individual panel that makes up a given wall is calculated according to the model proposed by the UNI EN 1995-1-1 standard in step 9.2.4.2 *"Simplified analysis of diaphragm walls – Method A".*

For a wall made of several panels, it is recommended that the project value of the carrier capacity be calculated using the expression:

$$F_{v,Rd} = \sum_i F_{i,v,Rd}$$

Where:

$F_{i,v,Rd}$ represents the project cut resistance of the shear resistance panel of each panel, $F_{i,v,Rd}$ is:

$$F_{i,j,v,Rd} = f_{j,v,d} \cdot b_i \cdot t_{i,j}$$

Where :

$F_{i,j,v,Rd}$ is the cutting resistance of the single sheet, in him the first subscript indicates the panel of belonging and the second the side (external or internal)

$f_{i,v,d}$	is the shearing resistance of the single coating sheet
b_i	is the panel width
$t_{i,j}$	is the thickness of the coating sheet

- Swallowing checks for shearing coating sheets

In accordance with paragraph 9.2.4.1 of the Eurocode EN1995-1-1, the swallowing of the coating sheets can be overlooked as all the walls used in the project verify the criterion

$$\frac{b_{net}}{t} \leq 100$$

where:

b_{net}	is the distance between the struts
t	is the thickness of the coating sheet

- Project screws between slab and wall below

For the screws between the slab and the wall design below it was proceeded by taking the values of Vx (long cut effort x) on the RFEM model and placing them inside Fx; again Fy is zero while Kmod is 0.9.

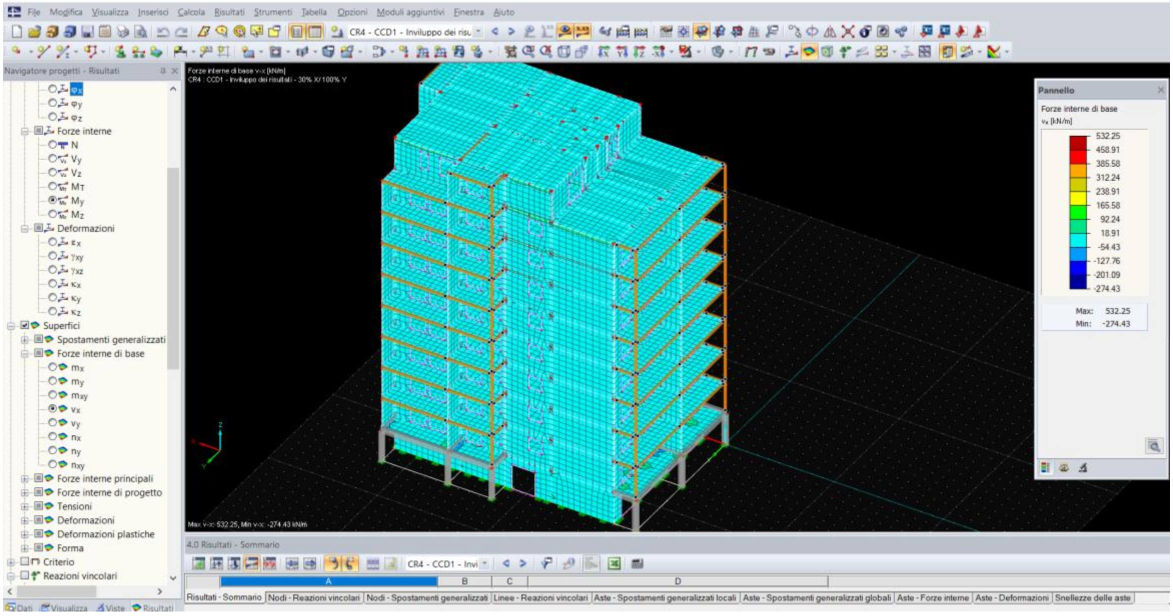


Fig.12.32 Vx values on RFEM model

In this case the screws must be necessarily tilted to avoid the phenomenon of lifting the slab.

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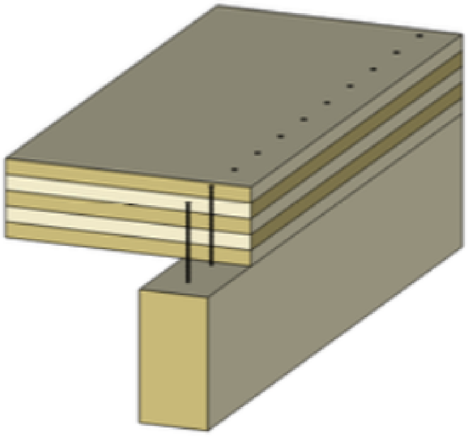
connection

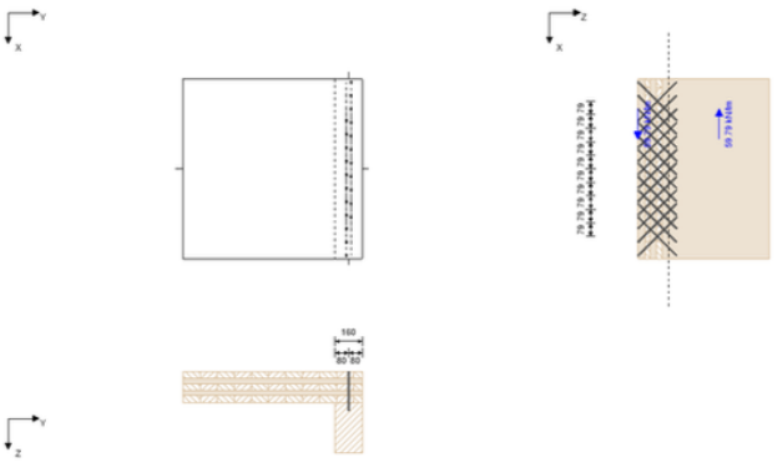


F_x	59.79	kN/lm
F_y	0	kN/lm
K_{mod}	0.9	-
material 1	C24 spruce ETA (2014)	
ρ_k	3.5	kN/m³
panel 1	CLT 180 L5s	
orientation cover layer	X direction	
material 2	C24 softwood	
ρ_k	3.5	kN/m³
connector type	Rothoblaas VGS	
connectors	9/320	
setup	crossed	
diameter	9	mm
head diameter	16	mm
length	320	mm
thread length	310	mm
pre-drilled	×	
timber beam width	160	mm
timber beam height	295	mm

analysis

analysis	existing	limit	unit	utilization
width 1	160	94	mm	59 %
width 2	160	94	mm	59 %
thickness 1	255	42	mm	17 %
thickness 2	65	42	mm	65 %
Fax	3342.9	3342.899	N	100 %
count	25.294	31.427	count / lm	80 %

system sketch



minimum spacing						
Name	$a_{1,min}$	$a_{2,min}$	$a_{3c,min}$	$a_{3t,min}$	$a_{4c,min}$	$a_{4t,min}$
	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
CLT top	45	14	54	54	36	54
timber beam bottom	45	36	36	80	27	27

result in layers

element 1						
X	Dicke	Typ	α	l_{eff}	$l_{eff,v}$	$F_{ax,Rk}$
[mm]	[mm]		[°]	[mm]	[mm]	[N]
0	7	L	90	0	0	0
7	33	L	90	46.6	32.9	4904
40	30	C	45	42.4	30	4061
70	40	L	90	56.6	40	5957
110	30	C	45	42.4	30	4061
140	36	L	90	51.6	36.5	5430
176	4	L	90	0	0	0

results										
$b_{1,min}$	$b_{2,min}$	$f_{h,k,1}$	$f_{h,k,2}$	β	$t_{pen,1}$	$t_{pen,2}$	$l_{eff,1}$	$l_{eff,2}$	$t_{1,req}$	$t_{2,req}$
[mm]	[mm]	[N/mm²]	[N/mm²]	[-]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
95	95	26.12	26.12	1.00	254.56	65.44	239.56	50.44	42	42

results												
$M_{y,r,k}$	$F_{ax,Rk}$	$F_{head,Rk}$	$F_{tens,Rk}$	$F_{ki,Rk}$	$F_{v,Rk}$	$F_{v,Rd}$	$F_{v,Ed}$	$F_{ax,Rd}$	$F_{ax,Ed}$	Anz.	$Anz._{max}$	$a_{erf.}$
[Nmm]	[N]	[N]	[kN]	[kN]	[N]	[N]	[kN/lm]	[N]	[kN/lm]	[Stk/m]	[Stk/m]	[mm]
27244.13	4828.63	0.000	25.400	15.812	5322.75	3684.98	0.00	3342.90	84.56	25.29	31.43	79

reference documents for this analysis	
English title	description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-09/0322	GH Various Angle Brackets
ETA-11/0496	Rotho Blaas TITAN Angle Brackets

The summary tables are also reported in this case:

Floor	Wall	Thickness [mm]	Vx for screws bettween slab and wall [kN/m]	wall length[m]	Vx for screws bettween slab and wall [kN]	screw's type: ROTHOBLASS VGS	n° of connectors per linear meter	n tot
1 2 3	119	160 L5s for wall and 165 for slab	14,03	5,75	80,67	9/320 crossed	6	35
	38		14,61	2	29,22	9/320 crossed	4	8
	141		14,35	2,07	29,70	9/320 crossed	4	8
	137		11,97	4,24	50,75	9/320 crossed	3	13
	135		12,68	6,06	76,84	9/320 crossed	3	18
	177		3,02	6,69	20,20	9/320 crossed	1	7
	176		6,48	7,1	46,01	9/320 crossed	2	14
	172		59,79	1,98	118,38	9/320 crossed	26	51
	27		12,17	6,06	73,75	9/320 crossed	5	30
	15		15,56	4,24	65,97	9/320 crossed	7	30
	10		3,59	4,3	15,44	9/320 crossed	2	9
	8		6,51	5,75	37,43	9/320 crossed	3	17
	221		18,04	7,1	128,08	9/320 crossed	6	43
	219		22,1	6,7	148,07	9/320 crossed	10	67
	2		10,13	1,98	20,06	9/320 crossed	5	10
	159		12,96	5,75	74,52	9/320 crossed	6	35
	162		14,82	2	29,64	9/320 crossed	6	12
	144		21,94	2	43,88	9/320 crossed	10	20
	149		14,58	2,07	30,18	9/320 crossed	7	14
	146		15	4,24	63,60	9/320 crossed	7	30
	152		18,31	2	36,62	9/320 crossed	8	16
	155		10,81	2,07	22,38	9/320 crossed	5	10
	157		15	4,24	63,60	9/320 crossed	7	30
	165		10,74	2	21,48	9/320 crossed	5	10
	166		3,5	2,07	7,25	9/320 crossed	2	4
	164		15,32	4,24	64,96	9/320 crossed	7	30
	184		11,3	3,5	39,55	9/320 crossed	5	18
	185		2,05	1,75	3,59	9/320 crossed	1	2
	189		5,3	3,3	17,49	9/320 crossed	3	10
	190		15,56	5,2	80,91	9/320 crossed	7	36
	12		6,2	1,98	12,28	9/320 crossed	3	6
	170		6,5	1,98	12,87	9/320 crossed	3	6
	193		12,7	3,5	44,45	9/320 crossed	6	21
	195		7,08	1,75	12,39	9/320 crossed	3	5
	214		15,05	3,5	52,68	9/320 crossed	7	25
	212		3,5	1,75	6,13	9/320 crossed	1	2
	210		14,82	1,44	21,34	9/320 crossed	7	10
	209		10	1,85	18,50	9/320 crossed	5	9
	215		17,82	5,2	92,66	9/320 crossed	8	42
	198		14,82	1,75	25,94	9/320 crossed	7	12
	202		11,28	1,44	16,24	9/320 crossed	5	7
	201		16,41	1,85	30,36	9/320 crossed	7	13

Floor	Wall	Thickness [mm]	Vx for screws bettween slab and wall [kN/m]	wall length[m]	Vx for screws bettween slab and wall [kN]	screw's type: ROTHOBLASS VGS	n° of connectors per linear meter	n tot
4 5 6	509	140 L5s for wall and 165 for slab	16,76	5,75	96,37	9/320 crossed	7	40
	474		4	2	8,00	9/320 crossed	2	4
	518		20,93	2,07	43,33	9/320 crossed	9	19
	514		21	4,24	89,04	9/320 crossed	9	38
	512		5,7	6,06	34,54	9/320 crossed	3	18
	554		5,57	6,69	37,26	9/320 crossed	3	20
	553		9,76	7,1	69,30	9/320 crossed	5	36
	549		2,78	1,98	5,50	9/320 crossed	2	4
	463		10,98	6,06	66,54	9/320 crossed	5	30
	461		18,76	4,24	79,54	9/320 crossed	8	34
	456		30,28	4,3	130,20	9/320 crossed	13	56
	454		20	5,75	115,00	9/320 crossed	9	52
	598		23,87	7,1	169,48	9/320 crossed	11	78
	596		26,59	6,7	178,15	9/320 crossed	12	80
	450		20	1,98	39,60	9/320 crossed	9	18
	536		26,88	5,75	154,56	9/320 crossed	12	69
	539		4	2	8,00	9/320 crossed	2	4
	521		26,88	2	53,76	9/320 crossed	12	24
	526		23,74	2,07	49,14	9/320 crossed	10	21
	523		13,15	4,24	55,76	9/320 crossed	6	25
	529		29,38	2	58,76	9/320 crossed	13	26
	532		12,59	2,07	26,06	9/320 crossed	6	12
	534		20,33	4,24	86,20	9/320 crossed	9	38
	542		9,36	2	18,72	9/320 crossed	4	8
	543		29,38	2,07	60,82	9/320 crossed	13	27
	541		16,03	4,24	67,97	9/320 crossed	7	30
	561		20,33	3,5	71,16	9/320 crossed	9	32
	562		18	1,75	31,50	9/320 crossed	8	14
	566		23,87	3,3	78,77	9/320 crossed	11	36
	567		25	5,2	130,00	9/320 crossed	11	57
	458		5,51	1,98	10,91	9/320 crossed	3	6
	547		9,8	1,98	19,40	9/320 crossed	5	10
	570		4,81	3,5	16,84	9/320 crossed	2	7
	572		13,15	1,75	23,01	9/320 crossed	6	11
	591		21,78	3,5	76,23	9/320 crossed	10	35
	589		19,36	1,75	33,88	9/320 crossed	9	16
	587		21,71	1,44	31,26	9/320 crossed	10	14
	586		19,77	1,85	36,57	9/320 crossed	9	17
	592		20,33	5,2	105,72	9/320 crossed	9	47
	575		39,19	1,75	68,58	9/320 crossed	16	28
	579		26,65	1,44	38,38	9/320 crossed	12	17
	578		13,73	1,85	25,40	9/320 crossed	6	11

Piani	Superior wall	spessore	Vx per chiodi tra parete superiore e inferiore [kN/m]	Lunghezza parete [m]	Vx per chiodi tra parete superiore e inferiore [kN]	tipo vite ROTHOBLASS VGS	n connettori al metro lineare	n tot
7 8 9	658	120 for wall and 165 for slab	10	5,75	57,50	9/320 crossed	5	29
	623		5,9	2	11,80	9/320 crossed	3	6
	667		3,28	2,07	6,79	9/320 crossed	2	4
	663		7,24	4,24	30,70	9/320 crossed	4	17
	661		2,54	6,06	15,39	9/320 crossed	2	12
	703		4,37	6,69	29,24	9/320 crossed	2	13
	702		14,18	7,1	100,68	9/320 crossed	6	43
	698		1,86	1,98	3,68	9/320 crossed	1	2
	613		7,75	6,06	46,97	9/320 crossed	4	24
	611		11,79	4,24	49,99	9/320 crossed	5	21
	606		17,71	4,3	76,15	9/320 crossed	8	34
	604		29	5,75	166,75	9/320 crossed	13	75
	747		16,28	7,1	115,59	9/320 crossed	7	50
	745		13,54	6,7	90,72	9/320 crossed	6	40
	600		16,28	1,98	32,23	9/320 crossed	7	14
	685		16,99	5,75	97,69	9/320 crossed	8	46
	688		11,95	2	23,90	9/320 crossed	5	10
	670		17,25	2	34,50	9/320 crossed	8	16
	675		16,44	2,07	34,03	9/320 crossed	7	14
	672		8,77	4,24	37,18	9/320 crossed	4	17
	678		13,74	2	27,48	9/320 crossed	6	12
	681		11,95	2,07	24,74	9/320 crossed	5	10
	683		19,02	4,24	80,64	9/320 crossed	8	34
	691		5	2	10,00	9/320 crossed	3	6
	692		19	2,07	39,33	9/320 crossed	8	17
	690		10,47	4,24	44,39	9/320 crossed	5	21
	710		13,36	3,5	46,76	9/320 crossed	6	21
	711		10,80	1,75	18,90	9/320 crossed	5	9
	715		15,09	3,3	49,80	9/320 crossed	7	23
	716		5,3	5,2	27,56	9/320 crossed	3	16
	608		6,09	1,98	12,06	9/320 crossed	3	6
	696		14,2	1,98	28,12	9/320 crossed	6	12
	719		17,51	3,5	61,29	9/320 crossed	8	28
	721		16	1,75	28,00	9/320 crossed	7	12
	740		14,04	3,5	49,14	9/320 crossed	6	21
	738		15,44	1,75	27,02	9/320 crossed	7	12
	736		10,27	1,44	14,79	9/320 crossed	5	7
	735		5,76	1,85	10,66	9/320 crossed	3	6
	741		5,38	5,2	27,98	9/320 crossed	3	16
	724		53,47	1,75	93,57	9/320 crossed	23	40
	728		36,94	1,44	53,19	9/320 crossed	16	23
	727		8	1,85	14,80	9/320 crossed	4	7

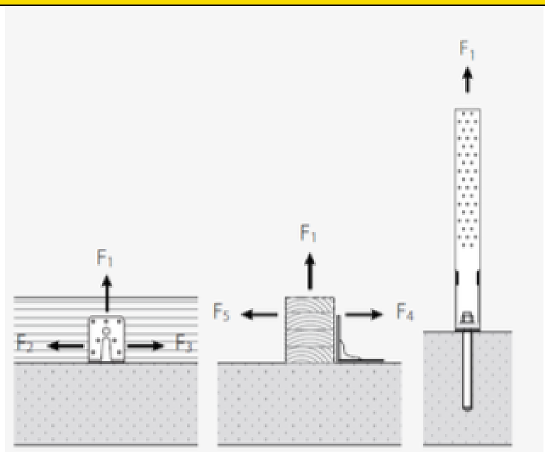
- Small brackets design and check
(connections between slab and top wall)

For the small brackets between the slab and the wall above design, it was proceeded by taking the Vx values (long cut effort x) on the RFEM model and placing them inside F23 while F1 is placed zero; Kmod is 0.9 as in the previous case.

An explanatory example of what has been done for each wall is:

Elisa Moretti	project element	Building brackets 137	page date	1 02.12.2019

connection



F_1	0	kN
F_{23}	19.52	kN
K_{mod}	0.9	-
connectors	TITAN F - TCF200, Hv > 90 mm	

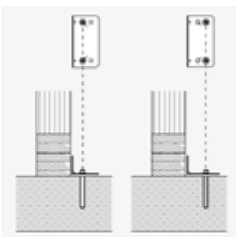
design F_{23}

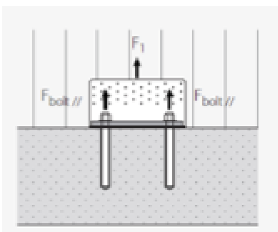
$F_{k,23}$ =	19.5	kN		$R_{k,23,Holz}$ =	42.5	kN	
				γ_m =	1.3	-	
				k_{mod} =	0.90	-	
$F_{d,23}$ =	19.5	kN	<	$R_{d,23}$ =	29.4	kN	✓
utilization ratio							66 %

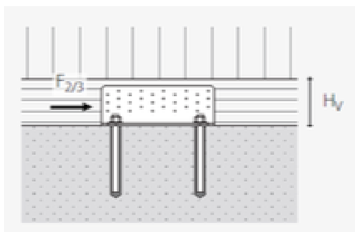
design forces for anchorage to concrete

design values, having "in" in the index refer to an inner anchor position
design values, having "out" in the index refer to an outer anchor position
see technical approvals and assessment documents

$F_{d,Bolt,23,in}$ = 14.64 [kN]
$F_{d,Bolt,23,out}$ = 18.7392 [kN]







reference documents for this analysis	
English title	description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-09/0322	GH Various Angle Brackets

The small brackets chosen are the TITAN F- TCF200, Hv>90 mm/Screw 5x50, an image for clarity:

Fig.12.33 Small brackets

The summary tables of the results obtained on Calculatis are as follows:

Floor	Superior wall	Slab	Vx tra parete e floor superiore [kN/m]	wall lenght [m]	Vx beetween wall and floor [kN]	small brackets	Utilization [%]	Rd [kN]	Vx < Rd
	119	50	16,91	5,75	97,23	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	38	60	15,05	2	30,10	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	141	62	25,77	2,07	53,34	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	137	66	4,65	4,24	19,72	TITAN F- TCF200, Hv>90 mm/Screw 5x50	66%	29,4	OK
	135	55	12,7	6,06	76,96	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	177	71	5,22	6,69	34,92	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	176	59	6,48	7,1	46,01	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	172	59	5,07	1,98	10,04	TITAN F- TCF200, Hv>90 mm/Screw 5x50	34%	29,4	OK
	27	59	8,66	6,06	52,48	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	15	68	15,56	4,24	65,97	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	10	61	7,5	4,3	32,25	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	8	54	33,72	5,75	193,89	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 7 small brackets	205,8	OK
	221	67	22,1	7,1	156,91	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	219	67	20,94	6,7	140,30	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	2	62	7,17	1,98	14,20	TITAN F- TCF200, Hv>90 mm/Screw 5x50	48%	29,4	OK
	159	60	22,1	5,75	127,08	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
	162	60	6,61	2	13,22	TITAN F- TCF200, Hv>90 mm/Screw 5x50	45%	29.4	OK
	144	60	18,31	2	36,62	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	149	62	14,58	2,07	30,18	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	146	63	14,82	4,24	62,84	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
1 2 3	152	60	11,28	2	22,56	TITAN F- TCF200, Hv>90 mm/Screw 5x50	77%	29,4	OK
	155	62	13,21	2,07	27,34	TITAN F- TCF200, Hv>90 mm/Screw 5x50	93%	29,4	OK
	157	66	13,21	4,24	56,01	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	165	61	17,01	2	34,02	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	166	62	16,41	2,07	33,97	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	164	63	10,19	4,24	43,21	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	184	66	11,27	3,5	39,45	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	185	58	3,05	1,75	5,34	TITAN F- TCF200, Hv>90 mm/Screw 5x50	18%	29,4	OK
	189	57	63,35	3,3	209,06	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 8 small brackets	235,2	OK
	190	59	15,56	5,2	80,91	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	12	68	1,42	1,98	2,81	TITAN F- TCF200, Hv>90 mm/Screw 5x50	10%	29,4	OK
	170	71	6,47	1,98	12,81	TITAN F- TCF200, Hv>90 mm/Screw 5x50	44%	29,4	OK
	193	65	18,31	3,5	64,09	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	195	65	1,5	1,75	2,63	TITAN F- TCF200, Hv>90 mm/Screw 5x50	9%	29,4	OK
	214	50	19,13	3,5	66,96	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	212	51	7,72	1,75	13,51	TITAN F- TCF200, Hv>90 mm/Screw 5x50	46%	29,4	OK
	210	52	23,96	1,44	34,50	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	209	52	7,8	1,85	14,43	TITAN F- TCF200, Hv>90 mm/Screw 5x50	49%	29,4	OK
	215	53	14,85	5,2	77,22	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	198	60	14,58	1,75	25,52	TITAN F- TCF200, Hv>90 mm/Screw 5x50	87%	29,4	OK
	202	60	11,28	1,44	16,24	TITAN F- TCF200, Hv>90 mm/Screw 5x50	55%	29,4	OK
	201	61	16,41	1,85	30,36	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK

Floor	Superior wall	Slab	Vx tra parete e floor superiore [kN/m]	wall lenght [m]	Vx beetween wall and floor [kN]	small brackets	Utilization [%]	Rd [kN]	Vx < Rd
	509	486	15,72	5,75	90,39	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	474	496	14,28	2	28,56	TITAN F- TCF200, Hv>90 mm/Screw 5x50	97%	29,4	OK
	518	498	13,62	2,07	28,19	TITAN F- TCF200, Hv>90 mm/Screw 5x50	96%	29,4	OK
	514	502	26,85	4,24	113,84	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	512	491	29,23	6,06	177,13	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 7 small brackets	205,8	OK
	554	505	6,45	6,69	43,15	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	553	494	5,8	7,1	41,18	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	549	504	4,93	1,98	9,76	TITAN F- TCF200, Hv>90 mm/Screw 5x50	91%	29,4	OK
	463	495	42,5	6,06	257,55	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 9 small brackets	264,6	OK
	461	504	15,39	4,24	65,25	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	456	498	27,35	4,3	117,61	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
	454	490	20,58	5,75	118,34	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
	598	490	22,38	7,1	158,90	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	596	486	25,11	6,7	168,24	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	450	490	18,04	1,98	35,72	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	536	488	22,59	5,75	129,89	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
	539	488	19,05	2	38,10	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	521	496	19,77	2	39,54	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	526	498	39,16	2,07	81,06	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	523	501	30,28	4,24	128,39	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
4 5 6	529	487	18,5	2	37,00	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	532	498	26,65	2,07	55,17	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	534	502	18,09	4,24	76,70	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	542	497	21,34	2	42,68	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	543	498	20,01	2,07	41,42	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	541	502	26,81	4,24	113,67	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	561	491	29,23	3,5	102,31	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	562	490	20,03	1,75	35,05	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	566	493	15,57	3,3	51,38	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	567	500	15,57	5,2	80,96	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	458	500	3,71	1,98	7,35	TITAN F- TCF200, Hv>90 mm/Screw 5x50	25%	294	OK
	547	505	17,49	1,98	34,63	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	570	501	26,88	3,5	94,08	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	572	501	1,69	1,75	2,96	TITAN F- TCF200, Hv>90 mm/Screw 5x50	10%	29,4	OK
	591	486	19,91	3,5	69,69	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	589	487	11,26	1,75	19,71	TITAN F- TCF200, Hv>90 mm/Screw 5x50	67%	29,4	OK
	587	488	9,23	1,44	13,29	TITAN F- TCF200, Hv>90 mm/Screw 5x50	45%	29,4	OK
	586	488	9,36	1,85	17,32	TITAN F- TCF200, Hv>90 mm/Screw 5x50	59%	29,4	OK
	592	488	44,72	5,2	232,54	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 8 small brackets	235,2	OK
	575	490	2	1,75	3,50	TITAN F- TCF200, Hv>90 mm/Screw 5x50	12%	29,4	OK
	579	490	0,22	1,44	0,32	TITAN F- TCF200, Hv>90 mm/Screw 5x50	1%	29,4	OK
	578	490	4,9	1,85	9,07	TITAN F- TCF200, Hv>90 mm/Screw 5x50	31%	29,4	OK

Floor	Superior wall	Slab	Vx tra parete e floor superiore [kN/m]	wall lenght [m]	Vx beetween wall and floor [kN]	small brackets	Utilization [%]	Rd [kN]	Vx < Rd
7 8 9	658	635	21,95	5,75	126,21	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 5 small brackets	147	OK
	623	645	20,07	2	40,14	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	667	647	19,22	2,07	39,79	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	663	650	38,22	4,24	162,05	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	661	640	41,8	6,06	253,31	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 9 small brackets	264,6	OK
	703	654	4,28	6,69	28,63	TITAN F- TCF200, Hv>90 mm/Screw 5x50	97%	29,4	OK
	702	643	3,63	7,1	25,77	TITAN F- TCF200, Hv>90 mm/Screw 5x50	88%	29,4	OK
	698	653	2,97	1,98	5,88	TITAN F- TCF200, Hv>90 mm/Screw 5x50	20%	29,4	OK
	613	644	24,64	6,06	149,32	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	611	649	26,39	4,24	111,89	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	606	647	12	4,3	51,60	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	604	639	62,9	5,75	361,68	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 13 small brackets	382,2	OK
	747	652	12,36	7,1	87,76	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	745	652	14,16	6,7	94,87	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	600	652	27,74	1,98	54,93	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	685	637	31,37	5,75	180,38	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 7 small brackets	205,8	OK
	688	645	13,44	2	26,88	TITAN F- TCF200, Hv>90 mm/Screw 5x50	91%	29,4	OK
	670	645	55,22	2	110,44	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	675	645	21,47	2,07	44,44	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	672	650	14,63	4,24	62,03	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 3 small brackets	88,2	OK
	678	645	23,97	2	47,94	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	681	647	16,76	2,07	34,69	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	683	651	25,39	4,24	107,65	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	691	646	29,2	2	58,40	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	692	647	9,98	2,07	20,66	TITAN F- TCF200, Hv>90 mm/Screw 5x50	70%	29,4	OK
	690	648	36,49	4,24	154,72	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 6 small brackets	176,4	OK
	710	640	16,48	3,5	57,68	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	711	641	10,11	1,75	17,69	TITAN F- TCF200, Hv>90 mm/Screw 5x50	60%	29,4	OK
	715	642	31,34	3,3	103,42	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 4 small brackets	117,6	OK
	716	649	60,07	5,2	312,36	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 11 small brackets	323,4	OK
	608	653	5,03	1,98	9,96	TITAN F- TCF200, Hv>90 mm/Screw 5x50	34%	29,4	OK
	696	654	3,9	1,98	7,72	TITAN F- TCF200, Hv>90 mm/Screw 5x50	26%	29,4	OK
	719	650	10,8	3,5	37,80	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	721	648	4,09	1,75	7,16	TITAN F- TCF200, Hv>90 mm/Screw 5x50	24%	29,4	OK
	740	636	16,06	3,5	56,21	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	738	636	11,94	1,75	20,90	TITAN F- TCF200, Hv>90 mm/Screw 5x50	71%	29,4	OK
	736	645	11,94	1,44	17,19	TITAN F- TCF200, Hv>90 mm/Screw 5x50	58%	29,4	OK
	735	637	8,25	1,85	15,26	TITAN F- TCF200, Hv>90 mm/Screw 5x50	52%	29,4	OK
	741	639	9,62	5,2	50,02	TITAN F- TCF200, Hv>90 mm/Screw 5x50	to check this wall we can use 2 small brackets	58,8	OK
	724	645	3	1,75	5,25	TITAN F- TCF200, Hv>90 mm/Screw 5x50	18%	29,4	OK
	728	645	7,88	1,44	11,35	TITAN F- TCF200, Hv>90 mm/Screw 5x50	39%	29,4	OK
	727	647	6,6	1,85	12,21	TITAN F- TCF200, Hv>90 mm/Screw 5x50	41%	29,4	OK

In this case, you should verify that the resistance of small brackets is greater than the agent action. In the event that only one angular (bracket) is not sufficient, more than one is proceeded alongside each other.

The analytical part of the tests carried out thanks to Calculatis is reported.

- Agent load

The agent shearing stresses on the individual angle are assessed by dividing the V_2 cut by the number of angles in the wall (taking into account the presence of angular on both sides of the structural element).

$$V_a = \frac{V_2}{n_{anc}}$$

V_2 is the agent project shearing stress on the wall considered
n_{anc} is the number of shear anchors in the wall

The agent cutting force on the most loaded anchor is calculated taking into account the additional moment due to the non-alignment between external forces agents on the vertical flange of the angular and the anchor itself by a coefficient, indicated with k_t.

$$V_p = V_a \cdot k_t$$

- Angular resistance

The design value of the angular shearing carrying capacity can be evaluated from the characteristic value using the following expression

$$R_{a,d} = \frac{k_{mod} \cdot \mathrm{R}_{\mathrm{a,k,dens}}}{\gamma_M}$$

where
$R_{a,k,dens}$ is the characteristic resistance of the correct nailing, for a density of the material used less than 350 kg/m3 according to the formula $R_{a,k,dens} = R_{c,k} \cdot \left(\frac{\rho k}{350}\right)^2$
The verification is carried out by comparing the agent strength with the break resistance.

$$V_{a,d} \leq R_{a,d}$$

- Hold-down design and check

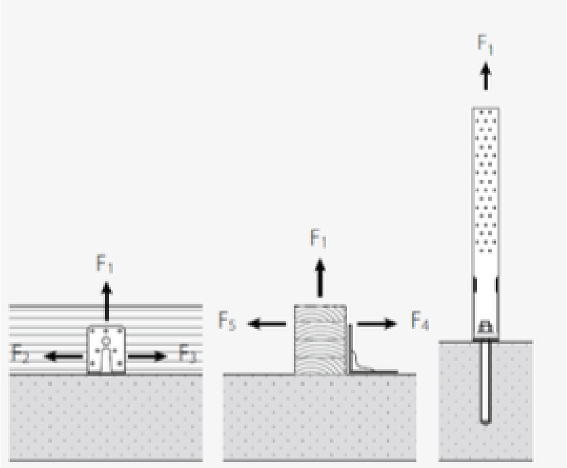
For the hold-downs placed only at the corners of the walls design, it was proceeded by taking the values of nx (normal effort) on the RFEM model and placing them within F1; F23 is zero while Kmod is 0.9.

The hold downs chosen are the WHT740-WHTBS130/ Screw 5 x 50, an image for clarity:

Fig.12.34 Hold down

The interface on Calculatis is the same as the small brackets:

connection			
	F_1	24.06	kN
	F_{23}	0	kN
	K_{mod}	0.9	-
	connectors	WHT740 + WHTBS130	

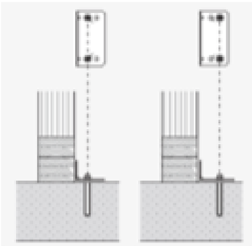


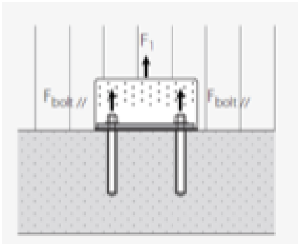
design F_1							
$F_{k,1}$ =	24.1	kN		$R_{k,1,Holz}$ =	144.8	kN	
				$R_{k,1,Stahl}$ =	158.6	kN	
				γ_m =	1.3	-	
				k_{mod} =	0.90	-	
$F_{d,1}$ =	24.1	kN	<	$R_{d,1}$ =	100.2	kN	✓
utilization ratio							24 %

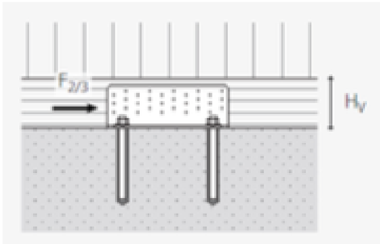
design forces for anchorage to concrete

design values, having "in" in the index refer to an inner anchor position
design values, having "out" in the index refer to an outer anchor position
see technical approvals and assessment documents

$F_{d,Bolt,//}$ = 24.06 [kN]







reference documents for this analysis

English title	description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
DM08	NTC2008 - Italian standards for structural design of buildings and constructions - D.M. 14 Gennaio 2008
CNR DT206	CNR-DT 206/2007: Reccomandations for the design and execution of timber structures
UNI EN 1995-1-1_NA	UNI EN 1995-1-1 - Italy - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-09/0322	GH Various Angle Brackets

The summary tables of the results obtained on Calculatis are as follows:

Floor	Wall	spessore	nx [kN/m] left	Hold down left	Utilization [%]	Rd [kN]	nx < Rd	nx [kN/m] right	Hold down right	Utilization [%]	Rd [kN]	nx < Rd
	119		24,06	WHT740+WHTBS130/ Screw 5 x 50	24%	100,2	OK	41,23	WHT740+WHTBS130/ Screw 5 x 50	41%	100,2	OK
	38		41,23	WHT740+WHTBS130/ Screw 5 x 50	41%	100,2	OK	19,82	WHT740+WHTBS130/ Screw 5 x 50	20%	100,2	OK
	141		3,71	WHT740+WHTBS130/ Screw 5 x 50	4%	100,2	OK	31,45	WHT740+WHTBS130/ Screw 5 x 50	32%	100,2	OK
	137		33,69	WHT740+WHTBS130/ Screw 5 x 50	34%	100,2	OK	42,6	WHT740+WHTBS130/ Screw 5 x 50	43%	100,2	OK
	135		37,17	WHT740+WHTBS130/ Screw 5 x 50	37%	100,2	OK	17,06	WHT740+WHTBS130/ Screw 5 x 50	17%	100,2	OK
	177		17,06	WHT740+WHTBS130/ Screw 5 x 50	17%	100,2	OK	14,09	WHT740+WHTBS130/ Screw 5 x 50	14%	100,2	OK
	176		18,35	WHT740+WHTBS130/ Screw 5 x 50	18%	100,2	OK	40,65	WHT740+WHTBS130/ Screw 5 x 50	41%	100,2	OK
	172		99,49	WHT740+WHTBS130/ Screw 5 x 50	99%	100,2	OK	31,13	WHT740+WHTBS130/ Screw 5 x 50	31%	100,2	OK
	27		62,85	WHT740+WHTBS130/ Screw 5 x 50	63%	100,2	OK	46,16	WHT740+WHTBS130/ Screw 5 x 50	46%	100,2	OK
	15		148,81	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	116,62	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	10		116,62	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	176,34	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	8		176,34	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	128,54	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	221		128,54	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	16,22	WHT740+WHTBS130/ Screw 5 x 50	16%	100,2	OK
	219		230,08	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	44,82	WHT740+WHTBS130/ Screw 5 x 50	45%	100,2	OK
	2		9,86	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK	74,75	WHT740+WHTBS130/ Screw 5 x 50	75%	100,2	OK
	159		10,89	WHT740+WHTBS130/ Screw 5 x 50	11%	100,2	OK	13,44	WHT740+WHTBS130/ Screw 5 x 50	14%	100,2	OK
	162		18,61	WHT740+WHTBS130/ Screw 5 x 50	19%	100,2	OK	246,99	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	144		34,75	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK	24,42	WHT740+WHTBS130/ Screw 5 x 50	25%	100,2	OK
	149		24,42	WHT740+WHTBS130/ Screw 5 x 50	25%	100,2	OK	9,37	WHT740+WHTBS130/ Screw 5 x 50	9%	100,2	OK
	146		13,33	WHT740+WHTBS130/ Screw 5 x 50	13%	100,2	OK	82,14	WHT740+WHTBS130/ Screw 5 x 50	82%	100,2	OK
1 2 3	152	160 L5s for wall and 165 for slab	7,94	WHT740+WHTBS130/ Screw 5 x 50	8%	100,2	OK	16,24	WHT740+WHTBS130/ Screw 5 x 50	16%	100,2	OK
	155		38,38	WHT740+WHTBS130/ Screw 5 x 50	39%	100,2	OK	49,86	WHT740+WHTBS130/ Screw 5 x 50	50%	100,2	OK
	157		2,41	WHT740+WHTBS130/ Screw 5 x 50	3%	100,2	OK	36,43	WHT740+WHTBS130/ Screw 5 x 50	37%	100,2	OK
	165		62,95	WHT740+WHTBS130/ Screw 5 x 50	63%	100,2	OK	222,14	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	166		10,86	WHT740+WHTBS130/ Screw 5 x 50	11%	100,2	OK	38,89	WHT740+WHTBS130/ Screw 5 x 50	39%	100,2	OK
	164		34,54	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK	2,61	WHT740+WHTBS130/ Screw 5 x 50	3%	100,2	OK
	184		20,22	WHT740+WHTBS130/ Screw 5 x 50	20%	100,2	OK	109,38	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	185		82,14	WHT740+WHTBS130/ Screw 5 x 50	83%	100,2	OK	74,23	WHT740+WHTBS130/ Screw 5 x 50	74%	100,2	OK
	189		25,96	WHT740+WHTBS130/ Screw 5 x 50	26%	100,2	OK	138,76	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	190		34,54	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK	14,96	WHT740+WHTBS130/ Screw 5 x 50	15%	100,2	OK
	12		46,16	WHT740+WHTBS130/ Screw 5 x 50	46%	100,2	OK	41,66	WHT740+WHTBS130/ Screw 5 x 50	42%	100,2	OK
	170		19,23	WHT740+WHTBS130/ Screw 5 x 50	19%	100,2	OK	14,09	WHT740+WHTBS130/ Screw 5 x 50	14%	100,2	OK
	193		31,45	WHT740+WHTBS130/ Screw 5 x 50	31%	100,2	OK	26,37	WHT740+WHTBS130/ Screw 5 x 50	27%	100,2	OK
	195		44,04	WHT740+WHTBS130/ Screw 5 x 50	44%	100,2	OK	112,45	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	214		296,24	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	41,23	WHT740+WHTBS130/ Screw 5 x 50	41%	100,2	OK
	212		98,8	WHT740+WHTBS130/ Screw 5 x 50	99%	100,2	OK	34,75	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK
	210		219,79	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	6,62	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK
	209		222,14	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	11,23	WHT740+WHTBS130/ Screw 5 x 50	11%	100,2	OK
	215		176,34	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	76,72	WHT740+WHTBS130/ Screw 5 x 50	77%	100,2	OK
	198		43,1	WHT740+WHTBS130/ Screw 5 x 50	43%	100,2	OK	49,86	WHT740+WHTBS130/ Screw 5 x 50	50%	100,2	OK
	202		87,41	WHT740+WHTBS130/ Screw 5 x 50	87%	100,2	OK	132,08	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	201		40,11	WHT740+WHTBS130/ Screw 5 x 50	40%	100,2	OK	164,12	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK

Floor	Superior wall	spessore	nx [kN/m] left	Hold down left	Utilization [%]	Rd [kN]	nx < Rd	nx [kN/m] right	Hold down right	Utilization [%]	Rd [kN]	nx < Rd
4 5 6	509	140 L5s	60,28	WHT740+WHTBS130/ Screw 5 x 50	60%	100,2	OK	20,57	WHT740+WHTBS130/ Screw 5 x 50	21%	100,2	OK
	474		28,74	WHT740+WHTBS130/ Screw 5 x 50	29%	100,2	OK	39,13	WHT740+WHTBS130/ Screw 5 x 50	39%	100,2	OK
	518		25,12	WHT740+WHTBS130/ Screw 5 x 50	25%	100,2	OK	27,6	WHT740+WHTBS130/ Screw 5 x 50	28%	100,2	OK
	514		6,69	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK	9,7	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK
	512		9,53	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK	9,42	WHT740+WHTBS130/ Screw 5 x 50	9%	100,2	OK
	554		9,61	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK	71,22	WHT740+WHTBS130/ Screw 5 x 50	71%	100,2	OK
	553		1,81	WHT740+WHTBS130/ Screw 5 x 50	2%	100,2	OK	35,06	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK
	549		59,09	WHT740+WHTBS130/ Screw 5 x 50	60%	100,2	OK	13,24	WHT740+WHTBS130/ Screw 5 x 50	13%	100,2	OK
	463		35,06	WHT740+WHTBS130/ Screw 5 x 50	35%	100,2	OK	52,26	WHT740+WHTBS130/ Screw 5 x 50	52%	100,2	OK
	461		104,69	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	61,55	WHT740+WHTBS130/ Screw 5 x 50	62%	100,2	OK
	456		73,1	WHT740+WHTBS130/ Screw 5 x 50	73%	100,2	OK	59,43	WHT740+WHTBS130/ Screw 5 x 50	59%	100,2	OK
	454		39,79	WHT740+WHTBS130/ Screw 5 x 50	40%	100,2	OK	21,42	WHT740+WHTBS130/ Screw 5 x 50	21%	100,2	OK
	598		86,35	WHT740+WHTBS130/ Screw 5 x 50	86%	100,2	OK	321,34	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK
	596		77,86	WHT740+WHTBS130/ Screw 5 x 50	78%	100,2	OK	294,8	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	450		7,12	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK	4,91	WHT740+WHTBS130/ Screw 5 x 50	5%	100,2	OK
	536		12,86	WHT740+WHTBS130/ Screw 5 x 50	13%	100,2	OK	15,69	WHT740+WHTBS130/ Screw 5 x 50	16%	100,2	OK
	539		14,98	WHT740+WHTBS130/ Screw 5 x 50	15%	100,2	OK	7,02	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK
	521		8,42	WHT740+WHTBS130/ Screw 5 x 50	9%	100,2	OK	138,66	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	526		39,72	WHT740+WHTBS130/ Screw 5 x 50	40%	100,2	OK	10,1	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK
	523		10,1	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK	21,57	WHT740+WHTBS130/ Screw 5 x 50	22%	100,2	OK
	529		198,56	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	198,89	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	532		121,14	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	120,82	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	534		61,19	WHT740+WHTBS130/ Screw 5 x 50	61%	100,2	OK	63,43	WHT740+WHTBS130/ Screw 5 x 50	63%	100,2	OK
	542		100,86	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	29,68	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK
	543		161,66	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	271,03	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	541		202,9	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	118,45	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	561		3,98	WHT740+WHTBS130/ Screw 5 x 50	4%	100,2	OK	46,94	WHT740+WHTBS130/ Screw 5 x 50	47%	100,2	OK
	562		475,3	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 5 hold down	501,00	OK	126,73	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	566		11,56	WHT740+WHTBS130/ Screw 5 x 50	12%	100,2	OK	78,95	WHT740+WHTBS130/ Screw 5 x 50	79%	100,2	OK
	567		78,95	WHT740+WHTBS130/ Screw 5 x 50	79%	100,2	OK	104,75	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	458		27,91	WHT740+WHTBS130/ Screw 5 x 50	28%	100,2	OK	52,26	WHT740+WHTBS130/ Screw 5 x 50	52%	100,2	OK
	547		7,89	WHT740+WHTBS130/ Screw 5 x 50	8%	100,2	OK	7,13	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK
	570		164,73	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	10,61	WHT740+WHTBS130/ Screw 5 x 50	11%	100,2	OK
	572		408,32	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 5 hold down	501,00	OK	121,14	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	591		184,27	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	365,96	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK
	589		231,33	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	163,8	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	587		349,32	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK	163,24	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	586		90,61	WHT740+WHTBS130/ Screw 5 x 50	91%	100,2	OK	261,74	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	592		294,74	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	628,52	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 7 hold down	701,4	OK
	575		580,59	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 6 hold down	601,2	OK	63,43	WHT740+WHTBS130/ Screw 5 x 50	63%	100,2	OK
	579		198,56	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	161,66	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	578		538,1	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 6 hold down	601,2	OK	116,8	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK

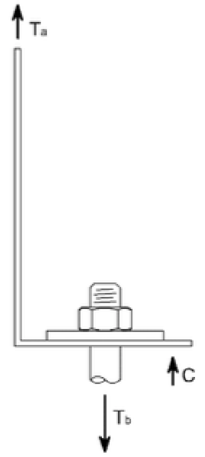
Floor	Superior wall	spessore	nx [kN/m] left	Hold down left	Utilization [%]	Rd [kN]	nx < Rd	nx [kN/m] right	Hold down right	Utilization [%]	Rd [kN]	nx < Rd
7 8 9	658	120 L5s	40,02	WHT740+WHTBS130/ Screw 5 x 50	40%	100,2	OK	29,41	WHT740+WHTBS130/ Screw 5 x 50	29%	100,2	OK
	623		23,67	WHT740+WHTBS130/ Screw 5 x 50	24%	100,2	OK	45,77	WHT740+WHTBS130/ Screw 5 x 50	46%	100,2	OK
	667		30,22	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK	55,39	WHT740+WHTBS130/ Screw 5 x 50	55%	100,2	OK
	663		91,9	WHT740+WHTBS130/ Screw 5 x 50	92%	100,2	OK	25,94	WHT740+WHTBS130/ Screw 5 x 50	26%	100,2	OK
	661		16,36	WHT740+WHTBS130/ Screw 5 x 50	16%	100,2	OK	11,92	WHT740+WHTBS130/ Screw 5 x 50	12%	100,2	OK
	703		200,25	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	75,07	WHT740+WHTBS130/ Screw 5 x 50	75%	100,2	OK
	702		44,26	WHT740+WHTBS130/ Screw 5 x 50	44%	100,2	OK	33,8	WHT740+WHTBS130/ Screw 5 x 50	34%	100,2	OK
	698		6,6	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK	6,61	WHT740+WHTBS130/ Screw 5 x 50	7%	100,2	OK
	613		271,93	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	92,78	WHT740+WHTBS130/ Screw 5 x 50	93%	100,2	OK
	611		239,95	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	173,88	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	606		215,66	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	99,76	WHT740+WHTBS130/ Screw 5 x 50	100%	100,2	OK
	604		50,13	WHT740+WHTBS130/ Screw 5 x 50	50%	100,2	OK	81,82	WHT740+WHTBS130/ Screw 5 x 50	82%	100,2	OK
	747		489,41	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 5 hold down	501,00	OK	572,85	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 6 hold down	601,2	OK
	745		447,21	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 5 hold down	501,00	OK	377,25	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK
	600		6,25	WHT740+WHTBS130/ Screw 5 x 50	6%	100,2	OK	149,46	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	685		9,68	WHT740+WHTBS130/ Screw 5 x 50	10%	100,2	OK	30,43	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK
	688		29,79	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK	95,25	WHT740+WHTBS130/ Screw 5 x 50	95%	100,2	OK
	670		116,72	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	91,37	WHT740+WHTBS130/ Screw 5 x 50	91%	100,2	OK
	675		89,84	WHT740+WHTBS130/ Screw 5 x 50	90%	100,2	OK	128,92	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	672		128,92	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	225,91	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	678		112,76	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	130,07	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	681		81,06	WHT740+WHTBS130/ Screw 5 x 50	81%	100,2	OK	99,19	WHT740+WHTBS130/ Screw 5 x 50	99%	100,2	OK
	683		100,05	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	150,06	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	691		29,79	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK	60,6	WHT740+WHTBS130/ Screw 5 x 50	61%	100,2	OK
	692		203,26	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	55,42	WHT740+WHTBS130/ Screw 5 x 50	55%	100,2	OK
	690		77,8	WHT740+WHTBS130/ Screw 5 x 50	78%	100,2	OK	84,58	WHT740+WHTBS130/ Screw 5 x 50	85%	100,2	OK
	710		25,94	WHT740+WHTBS130/ Screw 5 x 50	26%	100,2	OK	101,85	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	711		358,07	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK	112,32	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	715		67,99	WHT740+WHTBS130/ Screw 5 x 50	68%	100,2	OK	153,12	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	716		123,1	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK	35,53	WHT740+WHTBS130/ Screw 5 x 50	36%	100,2	OK
	608		11,23	WHT740+WHTBS130/ Screw 5 x 50	11%	100,2	OK	82,78	WHT740+WHTBS130/ Screw 5 x 50	83%	100,2	OK
	696		44,26	WHT740+WHTBS130/ Screw 5 x 50	44%	100,2	OK	17,92	WHT740+WHTBS130/ Screw 5 x 50	18%	100,2	OK
	719		91,9	WHT740+WHTBS130/ Screw 5 x 50	92%	100,2	OK	88,5	WHT740+WHTBS130/ Screw 5 x 50	88%	100,2	OK
	721		21,11	WHT740+WHTBS130/ Screw 5 x 50	21%	100,2	OK	167,28	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	740		213,3	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	88,71	WHT740+WHTBS130/ Screw 5 x 50	89%	100,2	OK
	738		365,32	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK	248,35	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK
	736		228,01	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	152,88	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	735		30,43	WHT740+WHTBS130/ Screw 5 x 50	30%	100,2	OK	145,7	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 2 hold down	200,4	OK
	741		240,26	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	374,69	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK
	724		261,76	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 3 hold down	300,6	OK	44,33	WHT740+WHTBS130/ Screw 5 x 50	44%	100,2	OK
	728		44,33	WHT740+WHTBS130/ Screw 5 x 50	44%	100,2	OK	95,25	WHT740+WHTBS130/ Screw 5 x 50	95%	100,2	OK
	727		301,3	WHT740+WHTBS130/ Screw 5 x 50	to check this wall we can use 4 hold down	400,8	OK	60,51	WHT740+WHTBS130/ Screw 5 x 50	61%	100,2	OK

Again, you need to verify that the hold down resistance is greater than the agent action. In the event that a single hold down is not sufficient, more than one.

Hold down Project resistance R_d degli is associated with the following break modes:

- breaking the nail
- steel-side break of the hold-down
- bolt break

- Agent load



The agent load project value on hold-downs was evaluated as shown in the "Description of the model" section.
The agent traction force on the bolt is calculated taking into account the additional moment due to the non-alignment between the external force agent on the vertical flange of the hold-down and the bolt itself by a coefficient of eccentricity, indicated with k_t.

$$T_b = T_a \cdot k_t$$

- Nail resistance

Nailing supporting capacity project value is given by the following expression

$$R_d = \frac{k_{mod} \cdot R_{c,k,dens}}{\gamma_M}$$

Where:

$R_{c,k,dens}$ is the characteristic resistance of the correct nailing, for a density of the material used less than 350 kg/m3 according to the formula$R_{c,k,dens} = R_{c,k} \cdot \left(\frac{\rho k}{350}\right)^2$

k_{mod} is the correction coefficient that takes into account the effects of load life and humidity

γ_M is the partial security coefficient for connections

- Hold down steel resistance

Angular design traction resistance can be evaluated according to the formula

$$\mathrm{R_{s,d}} = \frac{\mathrm{R_{s,u,k}}}{\gamma_{\mathrm{M2}}}$$

where:

$R_{s,k}$ is the characteristic value of angular resistance

γ_{M2} is the partial safety coefficient of the resistance of the tense sections

- Bolt traction resistance

The traction resistance was assessed as shown in Table 3.4 of the UNI EN 1993-1-8 standard using the following formula

$$\mathrm{R_{t,d}} = \frac{0.9 \cdot \mathrm{f_{ub}} \cdot \mathrm{A_s}}{\gamma_{\mathrm{M2}}}$$

Where:
fub is the last traction resistance of the anchor
As is the resistant area of the threaded part of the stem of the anchor
γ_{M2} is the safety coefficient

$$T_{a,d} \leq R_{a,d} = min.\left(R_{c,d}; R_{s,d}\right)$$

$$T_{b,d} \leq R_{b,d} = R_{t,d}$$

where:
$T_{a,d}$ is the project value of the agent stress on the hold-down
$T_{b,d}$ is the project value of the agent stress on the bolt
$R_{c,k}$ is the characteristic resistance of the wooden side connection
$R_{s,k}$ is the characteristic value of angular resistance
$R_{b,k}$ is the characteristic drag resistance to the connecting bolt
K_{mod} is the correction coefficient that takes into account the effects of load life and humidity
γ_M partial safety coefficient for the material, depending on the type of verification
$R_{a,d}$ hold-down resistance project value, assumed to be lowest among project resistance values of all rupture mechanisms associated with it
$R_{b,d}$ project value of bolt strength

13 Maximum building displacement check: Damage limit

The "damage limitation requirement" is considered satisfied if, as a result of a seismic action characterized by a greater probability of occurrence than the project seismic action corresponding to the "non-collapse requirement" in accordance with points **2.1(1)**P and **3.2.1(3)**, relative movements between planes are limited in accordance withpoint **4.4.3.2**. (eurocode 8)

Additional damage limitation may be required in cases of buildings that are important for civil protection or contain sensitive equipment.

- Restricting relative movement between floors

The following restrictions must be met unless otherwise specified

a) for buildings that have non-structural elements, consisting of fragile material, in solidarity with the structure:

$$d_r \cdot \nu \leq 0{,}005\ h$$

b) for buildings that have ductile non-structural elements:

$$d_r \cdot \nu \leq 0{,}0075\ h$$

c) buildings that have non-structural elements fixed so as not to interfere with deformations of the structure or without non-structural elements:

$$d_r \cdot \nu \leq 0{,}010\, h$$

where:

ν is the reduction coefficient that takes into account the lowest return period associated with the damage limitationrequirement.

Because we are in an importance II class ν=0,5

H it's the height

d_r is the project value of the relative movement between the planes

The value of d_s does not need to be greater than the elastic spectrum value.

In this case we use the expression for buildings that have non-structural elements, consisting of fragile material, in solidarity with the structure:

$$d_r \cdot \nu \leq 0{,}005\, h$$

- Case with displacement generated considering 100% of the agent forces in the x-direction and 30% in the y-direction

limitation of interstorey drift								
dr 1	dr2	Δdr	ν	dr*ν	⩽	h	0,005*h	verification
0,0937	0,088	0,0134	0,5	0,0067	⩽	3	0,015	ok
0,088	0,0852	0,0028	0,5	0,0014	⩽	3	0,0225	ok
0,0852	0,068	0,0172	0,5	0,0086	⩽	3	0,0225	ok
0,068	0,054	0,014	0,5	0,007	⩽	3	0,0225	ok
0,054	0,037	0,017	0,5	0,0085	⩽	3	0,0225	ok
0,037	0,0256	0,0114	0,5	0,0057	⩽	3	0,0225	ok
0,0256	0,015	0,0106	0,5	0,0053	⩽	3	0,0225	ok
0,015	0,008	0,007	0,5	0,0035	⩽	3	0,0225	ok

- Case with displacement generated considering 100% of the agent forces in the y-direction and 30% in the direction of x

limitation of interstorey drift								
dr 1	dr2	Δdr	ν	dr*ν	⩽	h	0,005*h	verification
0,123	0,11	0,0134	0,5	0,0067	⩽	3	0,015	ok
0,11	0,09	0,02	0,5	0,010	⩽	3	0,015	ok
0,09	0,08	0,01	0,5	0,005	⩽	3	0,015	ok
0,08	0,075	0,005	0,5	0,0025	⩽	3	0,015	ok
0,075	0,066	0,009	0,5	0,0045	⩽	3	0,015	ok
0,066	0,043	0,023	0,5	0,0115	⩽	3	0,015	ok
0,043	0,022	0,021	0,5	0,0105	⩽	3	0,015	ok
0,022	0,015	0,007	0,5	0,0035	⩽	3	0,015	ok

14 Conclusions

In Italy, awareness of the high seismic risk, post-seismic emergency situations and the need to reduce energy consumption have accelerated a process of change, already in place for some years, concerning a new way of conceiving Buildings. The wooden buildings with their lightness, speed of construction and good thermal behavior are the structural type that best meets all these requirements at the same time

14.1 CLT building examples in Italy

Below are some examples of constructions in CLT (Xlam) in Italy

- Varese building

Fig.13.1 Varese building

The house is located on the slopes of the Flower Field, in a magnificent position that embraces the lake of Lake Varese and from which you can push your gaze beyond the western shore of Lake Maggiore, towards the Alps- no, from Monte Rosa to Monviso. The building is arranged in the property in search of the sun and the best view.

Work tab:

- Location: Comerio (VA)
- Architectural project: Studio Ecoarcharchitetti Contavalli e Rivolta
- Concrete structure design: ing Andrea Meschini, Varese
- CLT structure design Calcolo strutture in legno: ing Roberto Belfiore, Varese
- Wood structures: Bianchi_Ferraro, Brebbia (VA)

- Work duration: 9 months (from excavations to key delivery)
- Covered surfaces: 172mq
- Total useful area: 120mq
- Type of building: single-family home
- Energy Classification: A+
- Construction System: X-LAM
- Transmittances:
  wall 0.17 W/(m2 K);
  coverage 0.21 W/(m2 K);
  floor on hornet's nest 0.17 W/(m2 K);
  windowed components from 0.87 to 1.16 W/(m2 K)
  Plants: Air/Water Heat Pump - 14 FV panels (4.5 Kwp)

- 9-storey building in Milan

(from which this project was inspired)

Fig.13.2 Milano Via Cenni

Work tab:

- Actuator subject:Polaris Investment Italia sgr spa
- Final design: Tekne s.p.a.
  Rossiprodi Associati S.R.L.: prof. arch. Fabrizio Rossi Prodi
  Borlini & Zanini SA: prof. ing. Andrea Bernasconi
- Timber structures design: Borlini & Zanini SA: prof. ing. Andrea Bernasconi
- Executive design: ETS Spa
- Directorate of Works and Safety: Tekne s.p.a.
- Artistic direction: Rossiprodi Associati S.R.L.: prof. arch. Fabrizio Rossi Prodi
- Contractors: Carron Cav. Angelo SpA-Service Legno s.r.l
- XLAM Panel Supplies (CLT): Production:StoraEnso

14.2 CLT (Xlam) constructions examples around the world

Below are some examples of constructions in CLT (Xlam) abroad

- Residential Tower Fortè living, Melbourne, Australia

Work card:

- Place: Melbourne, Australia
- Design and construction: Lend Lease (Millers Point, AU)
- Timber structures (technical consulting, production plan, logistics): KLH UK Ltd. (London, GB)
- XLAM Manufacturer:KLH Massivholz Gmbh (Katsch / Mur, AT)
- Projectdata:
  10-storeyresidentialtower
  23apartments
  Height:32.17meters
  Massive wood construction with XLAM panels
- Work duration: beginning February 2012
- Timber buildings: May 2012 to August 2012
- End of work: December 2012
- Sustainability: Savings of 1,400 tons of CO_2 compared to a steel or concrete construction
- Useful links: Zero Net Emissions strategy
- Completion December 2012

Fig.13.3 Fortè livingMelbourne, Australia

- Bridport House, Londra, Regno Unito

Work card:

- Building type: multi-storey residential building with 41 residential units
- Location: Bridport Place, Hackney, London
- Client: London Borough of Hackney
- Architecture: Karakusevic Carson Architects (Londra, UK)
- Enterprise: Willmott Dixon Ltd (Letchworth Garden City, Regno Unito)
- Structures: Peter Brett Associates
- Wood structures: EURBAN Ltd (Londra, UK)
- Amount of XLAM(CLT) used: 1.100 pannelli XLAM(CLT), circa 1.576 m^3, 30 consegne
- Supplier XLAM(CLT):Stora Enso Wood Products (Bad St. Leonhard, AT)
- Duration of work: 12 weeks, from October to November 2010-end of work 2011
- Technical data: 41 apartments, an 8-story complex and a five-story, 25.5-metre-high building, solid wood construction with large cross-layered laminated wood elements (XLAM), including stairwells and elevators
- Sustainability: 2,113 tonnes of CO_2 are stored in 1,576 m3 of wood


Fig.13.4 Bridport House, Londra, United Kingdom

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