SEISMIC DESIGN OF MULTI-STOREY CROSS LAMINATED TIMBER BUILDINGS ACCORDING TO EUROCODE 8

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SUMMARY – Cross Laminated Timber (CLT) structures are nowadays increasingly used worldwide and mostly in Europe where the system originated. However, in spite of this diffusion which led to the construction of a great number of multi-storey buildings all over Europe, still Eurocodes are almost completely missing provisions for CLT designers, especially regarding the seismic design. Nevertheless, Eurocode 8 requires in most cases, due to the regularity criteria being not fulfilled for most of the buildings, the use of the modal response spectrum analysis method, i.e. the linear dynamic analysis. This method requires the correct estimation of the lateral stiffness of the building in order to accurately calculate the design seismic forces in the building, which may be significantly underestimated or overestimated depending on the size of the building and the shape of the design spectrum. This can be done by modelling each connection with different methods that are often based on available test results, which are not easily accessible by a practicing engineer. This paper provides a design approach for dynamic linear modelling of CLT structures using SAP 2000. Equations are proposed based on available design codes and literature references, and used to design a 3-storey case study building. Further provisions for the seismic design of CLT buildings which are not included in Eurocode 8 are also given. Finally, the proposed design model is also compared with the results of the shaking table tests conducted in 2006 in Japan by CNR-IVALSA on a three-storey CLT building.

Key words: CLT structures, linear dynamic analysis, mechanical fasteners, shaking table test, friction.

1. **Introduction**

Cross Laminated Timber (CLT) structural systems, in spite of their relatively new diffusion, have been increasingly used over the last years for the construction of multi-storey timber buildings in many parts of the world (North America, Japan and Australasia) and mostly in Europe where the system originated about 20 years ago.

The reason for this success lies on the advantages over more traditional structural systems, such as the increased speed of erection, thanks to the quick and dry construction process, and the reduced environmental impact, in terms of CO2 emissions and energy consumption. Moreover, other additional benefits such as the increased fire resistance, the improved dimensional stability due to the cross-lamination process in the fabrication, and the possibility to use low grade timber in the production, make the CLT system an excellent alternative also to other traditional timber systems such as light-frame and glulam construction for the erection of low to medium rise buildings.

However, the actual lack of design rules in the Eurocodes referred to the CLT system requires some additional effort from the structural designer in order to find and explain the design choices and the calculation methods adopted to fully meet the safety requirements foreseen by the current building code. Especially the numerical modelling procedure of a CLT building may result very difficult to practicing engineers, who cannot easily access test results or specific literature references on the subject.

1.1. *Structural system and seismic behaviour*

Cross Laminated Timber (CLT) systems are structures in which walls and floors are composed of cross laminated timber panels, i.e. panels made of at least three laminations arranged alternatively at right angles and glued together. The laminations shall be made of one or two layers of parallel timber boards.

Unlike vertical actions, which affect only a portion of the structure and some of the structural elements, the seismic action is modelled via horizontal forces applied at the floor level that are transferred to the ground involving all the structural members, as shown schematically in Figure 1. Therefore, the continuity of the connections between the different parts of the structure is particularly important and must be effective both in tension and compression.

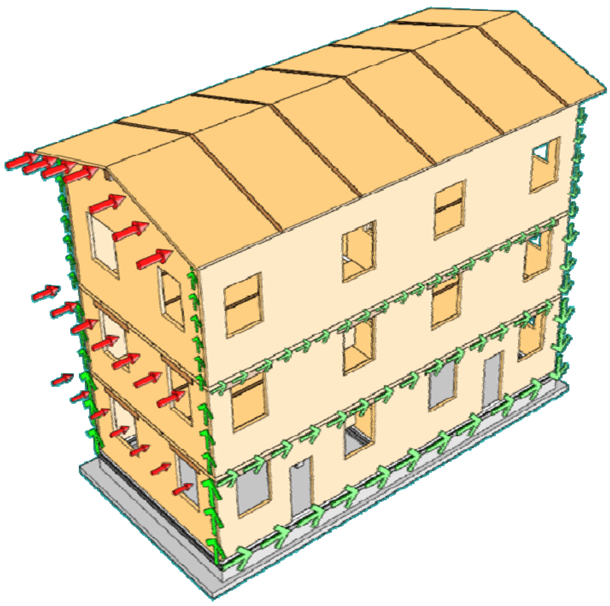


Figure 1. *Seismic forces acting on a CLT building*

With regard to the seismic response, a CLT building may be considered as a box-like structure in which the walls and floors are made of solid wood panels with high in-plane stiffness and strength, connected to each other by mechanical connections. The connections of the ground floor walls to the foundation must have a dual function as shown in Figure 2, as it must resist both overturning and sliding of the walls caused by the horizontal actions (wind or earthquake) acting in the plane of the wall and in general on the entire building.

The connection of the walls to the foundation or to the supporting floor panels is usually made with mechanical fasteners (hold-down anchors, steel brackets, anchoring bolts, nails and screws) and should adequately restrain the wall against uplift and sliding. Uplift-restrain connections are typically placed at wall ends and at opening ends, while sliding-restrain connections are uniformly distributed along the wall length. Figure 3 illustrates typical connections against uplift and sliding installed in a CLT building.

The erection process follows a platform type of construction with walls of height equal to the inter-storey height. Each wall assembly can be either made of a single CLT panel, provided that the length does not exceed the maximum transportable length of typically 16 m, or may be composed of a number of panels with a typical length of 2.5 m that renders the transportation more convenient and economical. In this case, the connections between wall panels are usually done with the interposition of a wood-based multi-layer panel that can be inserted in grooves cut inside the wall or on one of its faces. Sometimes the same connection is made with a half-lap joint. The connection is prescribed either with self-tapping screws with a diameter varying from 6 to 10 mm or with nails of 3 mm diameter and spacing depending on the seismic loads. Figure 4 illustrates some of the typical types of step joints used in construction. Walls in perpendicular directions shall be connected with mechanical fasteners (usually screws) to achieve a box-type behaviour.

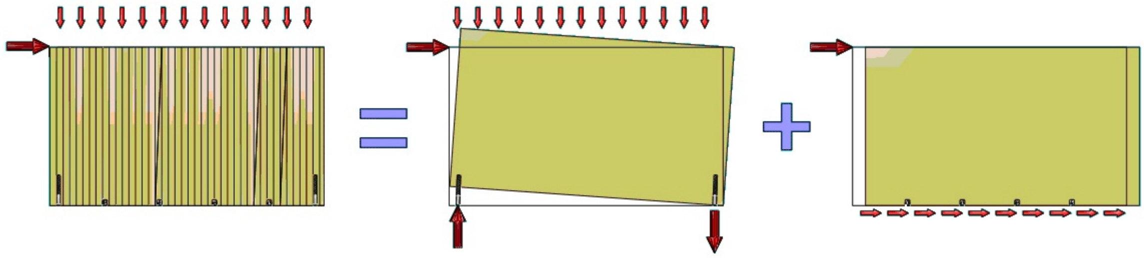


Figure 2. *Effects of the seismic forces acting on a wall and different function of the connection elements*

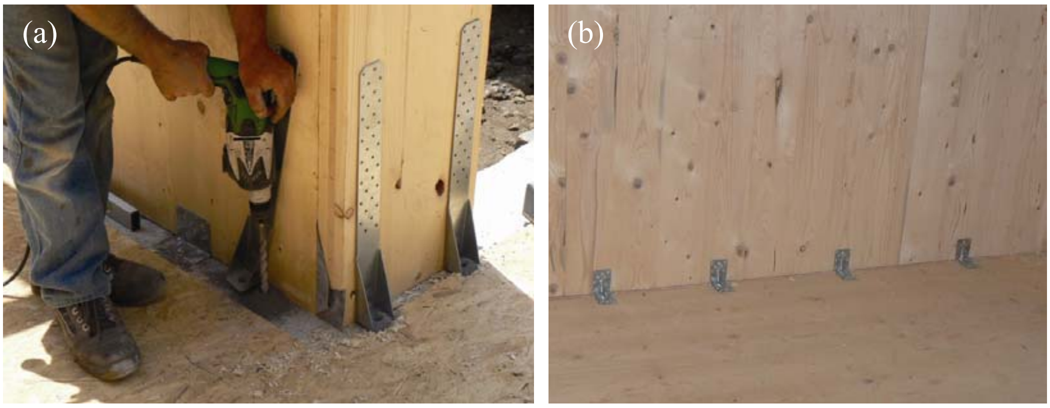
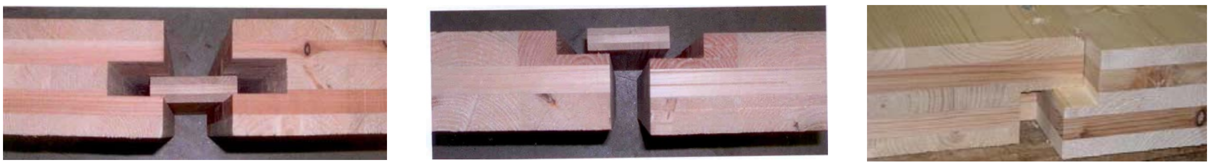


Figure 3. *(a) Hold-down placed at wall and at opening ends to prevent the uplifting of the wall, and (b) angle bracket connection between the wall and the floor panel to prevent the sliding of the wall*



(a) (b) (c)

Figure 4. *Three different step joints between vertical wall panels; (a) a cross-layer stripe inserted into internal grooves, (b) a cross-layer stripe inserted in grooves on the internal side of the wall, and (c) a half-lap joint*

The experience gained from the research carried out so far has shown that buildings with walls composed of panels with a maximum length of 2.5 m, if designed in full compliance with the capacity design method, show a greater level of ductility than buildings made of continuous walls and, therefore, a greater capacity to dissipate the energy induced by the earthquake [Ceccotti *et al.*, 2007].

Horizontal diaphragms are made of CLT timber panels connected through horizontal joints made with mechanical fasteners (screws or nails). The floor panels bear on the wall panels and are connected to them usually with self tapping screws. The upper walls bear on the floor panels, and are connected to the lower walls using metal connectors similar to those used for the wall-foundation connection. Figure 5 illustrates the typical connections between walls of a lower and an upper storey to the floor diaphragm.

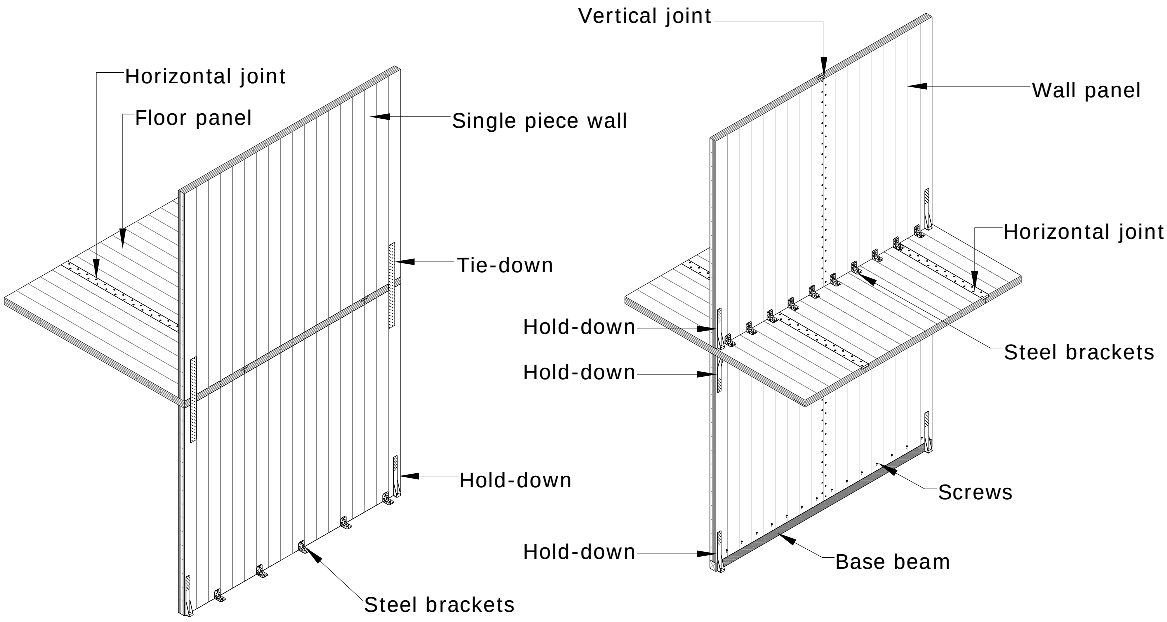


Figure 5. *Walls and floors in platform type CLT buildings (after Follesa et al., 2011)*

1.2. *Seismic design and capacity-based design criteria of CLT buildings*

The construction experience of CLT buildings and the research conducted over the last years on the static and seismic behaviour has led to the derivation of some design rules with regard to both the static and the seismic performance [Follesa *et al.*, 2011].

By adopting the assumption of rigid horizontal diaphragms, the in-plane seismic shear is divided among the walls underneath according to their stiffness taking into account an accidental eccentricity in addition to any eccentricity between the centre of mass and the centre of stiffness in the case of asymmetric distribution of stiffness.

To achieve the behaviour of a box-type structure it is important to ensure that local failures which may compromise the box-type behaviour will not occur. The connections devoted to the dissipative behaviour in a CLT building are:

- step joints between wall panels in case of walls composed of more than one element;
- connections against sliding between walls and floor below, and between walls and foundation; and
- anchoring connections against uplift placed at wall ends and at wall openings.

To ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength so as to avoid anticipated brittle failure. This overstrength requirement applies especially to connections shown in Figure 6 such as:

- connections between adjacent floor panels in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;
- connections between floors and walls underneath thus ensuring that at each storey there is a rigid floor to which the walls are well-connected; and

- connections between perpendicular walls, particularly at the building corners, so that the stability of the walls themselves and of the structural box is always assured.

Analyses carried out on the evaluation of the overstrength factor for the connections mentioned above have led to the conclusion that a conservative value of 1.6 shall be used [Jorissen and Fragiacomo 2011; Fragiacomo *et al.*, 2011; Follesa *et al.*, 2011].

The seismic resistance of shear walls should be higher at lower storeys and should decrease at higher storeys proportionally to the decrease of the storey seismic shear, thus leading to the simultaneous plasticization of the ductile connections in order to maximize the energy dissipation of the building. To achieve the desired ductile behaviour, each single connection should be detailed in order to avoid brittle failures. Special care should be used when designing the dissipative connections to ensure the attainment of a ductile failure mechanism characterized by the formation of one or two plastic hinges in the mechanical fastener. Brittle failures such as splitting, shear plug, tear out and tensile fracture of wood in the connection regions should be always avoided. Similarly, a brittle failure mechanism in the weaker section of the steel plate should be prevented in connections with steel brackets or hold-downs anchors connected to the wall panels with nails or screws.

1.3. *Seismic force modification factor*

Recent research work on the seismic behaviour of CLT buildings with pseudo-static tests on individual structural elements (CLT wall and floor panels), pseudo-dynamic and shake-table tests on full-scale CLT buildings, and numerical simulations [Ceccotti *et al.*, 2007; Ceccotti and Follesa, 2006] have shown that CLT buildings designed in accordance with the aforementioned capacity design method and built with walls composed of panels with a maximum length of 2.5 m can be designed with a value of the seismic behaviour factor q of at least 3. In the current version of Section 8 of Eurocode 8 [EC8, 2004] there is no provision for CLT buildings. However, for “Glued wall panels with glued diaphragms, connected with nails and bolts”, which is the type of structure more similar as description to the CLT system, the upper limit of the behaviour factor q is 2. Therefore, for compliance with the current building codes, this is the value which is currently used when designing CLT buildings.

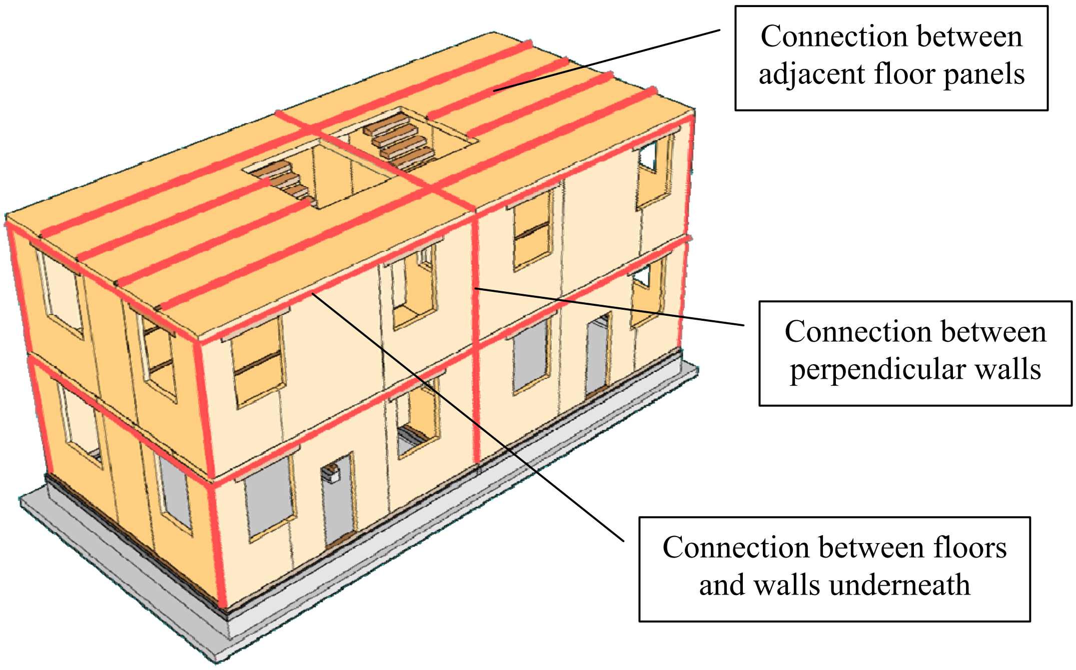


Figure 6. *Connections to be designed with overstrength in order to fulfill the capacity design criteria in Cross Laminated buildings (highlighted in red)*

2. **Numerical Modelling of CLT Buildings**

2.1. *General considerations*

Before conducting the seismic design of a multi-storey CLT building, the designer should follow the principles of conceptual design expressed in Section 4.2.1 of Eurocode 8, which are given for all types of buildings. These principles are particularly important for CLT structures, in order to achieve a good overall structural behaviour of the building, i.e. structural simplicity, uniformity, symmetry and redundancy, bi-directional strength and stiffness, diaphragmatic behaviour at storey level, and adequate foundation.

According to Eurocode 8 Part 1, depending on the fulfillment of the regularity criteria in plan and elevation, there are three possible choices for the seismic analysis of a numerical building model:

a) the lateral force method of analysis, i.e. the linear static analysis;
b) the modal response spectrum analysis, i.e. the linear dynamic analysis; and
c) non-linear methods, which could be either non-linear static (pushover) analysis or non-linear time-history analysis.

Method (a) is usually followed for low-rise and simple buildings which meet the regularity criteria in elevation. The horizontal forces may be calculated with an inverted triangular distribution based on the mass and the height of each storey. The natural vibration period may be calculated directly from the equation provided by the Eurocode 8 formula:

$$T_1 = C_T \cdot h^{3/4} \tag{1}$$

where C_T may be assumed equal to 0.05, h is the height of the building in meters, and T_1 is measured in seconds. If the building also meets the regularity criteria in plan the analysis may be performed using two planar models, one for each main horizontal direction.

Method (b) is used in cases where the building does not meet the criteria for regularity in elevation, which is the most common situation. The correct estimation of the natural vibration period of the building is in this case crucial, as it may lead to non-conservative design if underestimated or overestimated, depending on the type and size of structure and on the shape of the design spectrum [Sustersic and Dujic, 2012]. Moreover, it is important to estimate accurately the displacements of the building for the verification at the SLS and ULS, especially when the building is adjacent to other structures dynamically independent divided by a seismic separation. Therefore, it is crucial to model correctly the stiffness of the shear walls.

Both methods (a) and (b) are based on a linear elastic analysis, and the energy dissipation capacity of the building is implicitly taken into account by dividing the forces obtained from the linear elastic analysis by the behaviour factor q, which is associated with the relevant type of structure and ductility class, as defined in Table 8.1 of Eurocode 8. It should be mentioned however that no specific design rules and capacity design criteria are given for CLT buildings within the timber part (Section 8) of Eurocode 8, and that the value of the q factor given in Table 8.1 and currently adopted for CLT buildings is considered very conservative [Follesa *et al*., 2011].

Method (c) is more complex than method (a) and (b) and requires the knowledge of the non-linear monotonic (in case of static analysis) or cyclic (in case of time-history analysis) behaviour of the structural elements which are detailed and designed for energy dissipation, which in case of CLT buildings and of timber structures in general are mechanical fasteners, e.g. self-tapping screws, annular ringed nails, angle brackets and hold-down connectors. This is a more complex approach, which relies on the availability of experimental data on the same type of connectors and fasteners used in the design. In addition, a suitable finite element software should be used, which includes a proper non-linear monotonic or hysteretic model capable of accurately simulating the actual non-linear response of the mechanical fasteners.

However, designers usually do not have access to experimental data that can be used to calibrate the non-linear behaviour of the mechanical fasteners. Even when experimental data is available, it may refer to connections with different types, number and diameter of fasteners from those used in the actual design. Therefore, in most cases, the only possible way to perform a reliable design of a multi-storey building is to (i) formulate a proper finite element model of the CLT structure based on the available equations for the calculation of the connection slip modulus given in Eurocode 5 and (ii) analyze it through a linear dynamic analysis.

2.2. *Description of the numerical model*

The 3-dimensional numerical model of a CLT building proposed in this paper has been implemented in the widespread software package for structural analysis SAP2000 [CSI, 2007], which is utilized to perform the modal response-spectrum or the static analysis and to solve the associated equilibrium equations. A pre- and post-processing software specifically developed by Tecnisoft (Modest, Ver. 8.1, 2013) was used to aid in the implementation.

Three types of elements are utilized, namely:

- 4-noded, 24 DOFs, shell elements with membrane and bending capabilities for the CLT wall panels with a typical mesh of 0.5x0.5 meters;
- 2-noded, 6 DOFs, truss elements for the various mechanical connections between wall panels; and
- 2-noded, 12 DOFs, beam elements for the lintels connecting walls above openings.

Figure 7 illustrates a typical schematization of a pair of CLT wall panels and the connections at the base of the building as well as the connections with the upper floor walls.

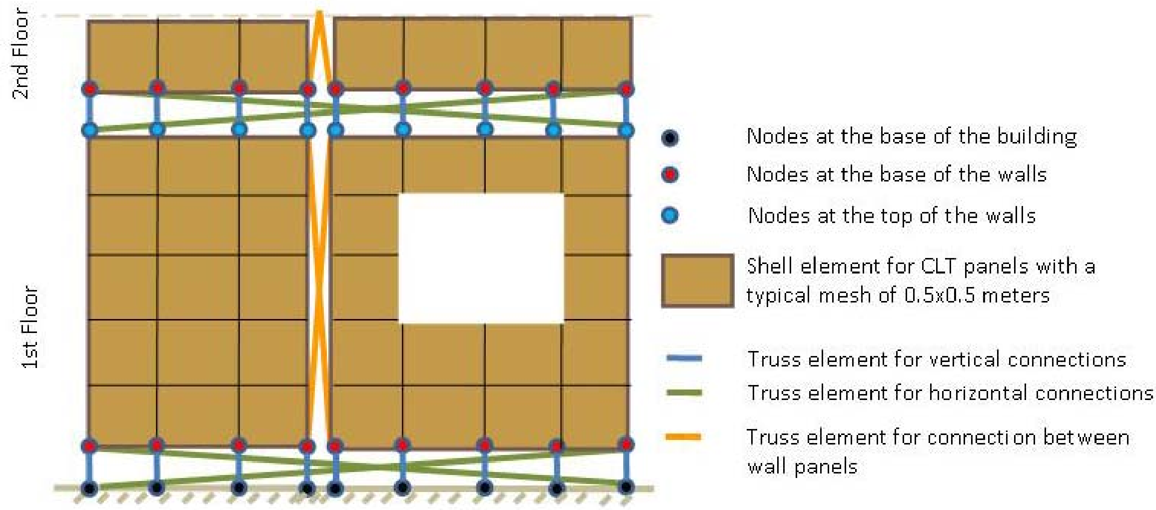


Figure 7. *Typical wall schematization*

With this representation, the in-plane shear forces transmitted from the walls to the walls underneath can be directly obtained from the axial forces of the horizontal truss elements, while the uplifting vertical forces are obtained from the tensile forces in the vertical truss elements. A rigid diaphragm constraint is used to constrain all nodes at the same level in Figure 7. It should be noticed that a separate constraint is used to constraint the nodes at the bottom of the wall and at the top of the wall underneath.

It is worth noting that the greater component of the inter-storey drift in a CLT building is attributed to the deformation of the mechanical joints connecting the walls with the floor diaphragms, which can be of the order of centimeters, while the shear and bending deformation of the CLT wall panel itself remains within the order of millimeters.

The model described above is based on some simplified assumptions:

- floor diaphragms are assumed to be in-plane rigid while the out-of-plane stiffness is not considered;
- the connection between perpendicular walls is assumed to be rigid;
- the connection between floors and supporting walls is assumed to be rigid; and
- hold-down connectors are not explicitly modelled.

2.2.1. *Section and material properties for shell elements*

Shell elements are defined with the same length, height and thickness as the associated CLT wall panels. Orthotropic material properties are defined based on: (i) the orthotropic properties of the wood class which the boards of the CLT walls are made of; and (ii) the number of layers and their associated thickness and grain direction.

For the orthotropic material properties, nine values have to be defined, namely the 3 moduli of elasticity in the 3 orthogonal directions E_x, E_y, E_z, the 3 Poisson's ratios ν_{xy}, ν_{yz}, ν_{zx}, and the 3 shear moduli G_{xy}, G_{yz}, G_{zx}. The x axis is assumed along the length of the wall (in the horizontal direction), the y axis in the direction perpendicular to the wall plane, and the z axis along the height of the wall (in the vertical direction), as shown in Figure 8. Out of these properties the most important ones are: (i) E_z that affects the vertical axial stiffness and the bending stiffness; and (ii) G_{zx} that affects the in-plane shear stiffness of the wall.

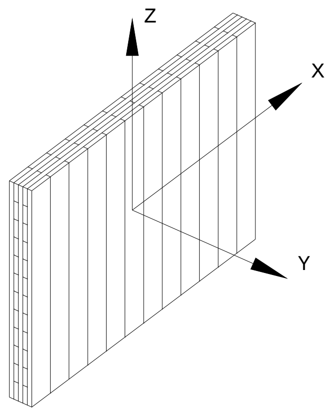


Figure 8. *Principal axes of a vertical wall element*

$E_z = E_{0,eq}$ is the equivalent modulus of elasticity of CLT wall panels in the direction parallel to the grain of the outer layers, calculated for a 5-layer CLT panel as suggested by Blaß and Fellmoser [2004]:

$$E_{0,eq} = \left[1 - \left(1 - \frac{E_{90,T}}{E_{0,L}}\right) \cdot \frac{a_3 - a_1}{a_5}\right] \cdot E_{0,L} \tag{2}$$

where $E_{0,L}$ is the modulus of elasticity parallel to the grain of the longitudinal layers, $E_{90,T}$ is the modulus of elasticity perpendicular to the grain of the transversal layers and a_1, a_3, a_5 are shown in Figure 9.

$E_x = E_{90,eq}$ is the equivalent modulus of elasticity of CLT wall panels in the direction perpendicular to the grain of the outer layers, calculated for a 5-layer CLT panel as suggested by Blaß and Fellmoser [2004]:

$$E_{90,eq} = \left[\frac{E_{90,L}}{E_{0,T}} + \left(1 - \frac{E_{90,L}}{E_{0,T}}\right) \cdot \frac{a_3 - a_1}{a_5}\right] \cdot E_{0,L} \tag{3}$$

Figure 9. *Definition of various thicknesses in a generic 5-layer CLT panel*

where $E_{0,T}$ is the modulus of elasticity parallel to the grain of the transversal layers, $E_{90,L}$ is the modulus of elasticity perpendicular to the grain of the longitudinal layers.

Furthermore, the equivalent modulus of elasticity to be used as E_y in the material properties of the numerical model (see Figure 8) is calculated as:

$$E_y = t_{tot} \cdot \left(\sum_{i=1,3,....,n} t_i / E_{90,L,i} + \sum_{j=2,4,....,n-1} t_j / E_{90,T,j} \right)^{-1} \tag{4}$$

where t_{tot} is the total thickness of the CLT panel, t_i and t_j are the layer thicknesses in the longitudinal and transversal directions, respectively, and n is the total number of layers.

The shear modulus G of the CLT panels is calculated as:

$$G = t_{tot} \cdot \left(\sum_{i=1,3,....,n} t_i / G_{L,i} + \sum_{j=2,4,....,n-1} t_j / G_{T,j} \right)^{-1} \tag{5}$$

where G_L and G_T are the shear moduli of the longitudinal and transversal layers, respectively.

2.2.2 *Section and material properties for horizontal and vertical truss elements*

A pair of horizontal cross truss elements is used to connect each wall to the foundation as well as to the wall of the upper floor where applicable. The section and material properties of the trusses are computed based on the horizontal stiffness of the connections used to transfer the shear force from the wall to the floor diaphragm below. Typically, these connections consist of angle brackets or a pair of inclined screws at a certain spacing. Connections between walls and upper diaphragms are considered rigid since they are typically over-sized, namely designed for the overstrength of the dissipative connectors, to satisfy capacity design requirements, but also because floor diaphragms are not explicitly modelled but just schematized using a kinematic constraint of rigid floor (option 'CONSTRAINT' in SAP 2000).

The horizontal stiffness K_H of the connection is computed based on the slip modulus at serviceability limit state of each fastener K_{con} used to connect the vertical metal plate of each connector to the wall panel and assuming the metal plate to concrete connection as rigid for the wall-foundation connection.

$$K_{con} = 2 \cdot \rho_m^{1.5} \cdot d / 23 \tag{6}$$

In Equation (6), ρ_m and d signify the mean density of timber in [kg/m^3] and the fastener diameter in [mm], respectively, with the values of K_{con} in [N/mm]. This equation is given in Table 7.1 of Eurocode 5 [EC5, 2009] for the calculation of the slip modulus under service load for dowels, bolts, screws and nails with pre-drilling used in steel to timber connections. Since the nails are usually banged, a different formula for nail without pre-drilling should be used. However, it was observed from the results of tests conducted on CLT structures [Ceccotti *et al.*, 2006] that Equation (6) provides a better match with the experimental values. A possible reason for that is the effect of the cross layers, which increases the stiffness of nails with respect to the calculated value for solid timber according to the formula proposed for nails without pre-drilling. Thus, for *n* nails, the horizontal stiffness of the wall-foundation connection is given by:

$$K_H = n \cdot K_{con} \tag{7}$$

For the wall to floor connection at upper storeys, the horizontal stiffness K_H is computed considering two horizontal springs in series: (i) the wall-metal plate connection, with stiffness K_{H1}; and (ii) the metal plate-floor connection, with stiffness K_{H2}.

$$K_H = \frac{1}{\frac{1}{K_{H1}} + \frac{1}{K_{H2}}} \tag{8}$$

Both stiffness values are computed by multiplying the slip modulus of a single fastener by the number of fasteners, as previously discussed. The axial stiffness $E_f A_H$ of each horizontal truss element is then calculated using Eq. (9):

$$E_f A_H = \frac{K_H \cdot L_W}{2} \tag{9}$$

where L_w is the length of the wall section, which can be made of a single panel or more adjacent panels connected with vertical step joints.

Similarly to the horizontal connection, the vertical step joints between wall panels are simulated with a pair of vertical cross truss elements which are used to connect each wall panel to the next one (see Figure 7). The section and material properties of the trusses are computed based on the stiffness of the connections used to transfer the shear force from adjacent wall panels. These connections are typically made of a cross-layered wood-based panel (usually cross banded LVL panels or plywood) inserted in an internal groove or in a groove cut on the internal side of the wall (case (a) and (b) of Figure 4, respectively) and the connection is usually made of self-tapping screws with a diameter varying from 6 to 10 mm or, sometimes, with 3 mm diameter nails and spacing depending on the seismic load.

The vertical stiffness K_V of the connection is computed again based on the slip modulus at serviceability limit state K_{con} of each fastener of the panel to timber connection given in Table 7.1 of Eurocode 5 [EC5, 2009] and in Equation (6) multiplied by the number of fasteners. This value should then, be divided by 2, considering the double panel to timber connection, and either multiplied by 2 for double shear connection like in the case of internal groove step joint, or by 1 for groove on the internal side of the panel. Similarly to the horizontal connections, the axial stiffness $E_f A_V$ of each vertical truss element is then calculated using Eq. (10):

$$E_f A_V = \frac{K_V \cdot H_W}{2} \tag{10}$$

where H_w is the height of the wall and K_V the stiffness of the step joint connection.

Vertical truss elements are also used to simulate the deformability of the floor diaphragms along their thickness, namely perpendicular to their plane, as the walls bear on the floor panels. Thus, the modulus of elasticity perpendicular to grain (E_{90} from Equation (4)) is selected for the isotropic material properties. Since the shell elements are meshed with a grid of 0.5 meters length, the cross sectional area of the vertical trusses is equal to 0.5 meters times the thickness of the wall above. At the foundation level, the modulus of elasticity of concrete is used for the isotropic material properties. Forces in the vertical truss elements of a wall are then utilized to calculate the tensile forces for the design of the hold-downs, typically installed at each end of the wall to resist overturning moments from horizontal seismic loads. It should be noted that although hold-down anchors play a major role in the actual lateral stiffness and strength of the wall, they are not explicitly simulated in the linear numerical model, mainly due to the nonlinear nature of their response that exhibits markedly different stiffness in compression (where there is contact between wall and floor panels) and in tension (where only the hold-downs resist).

3. **Design of a three-storey case study building**

The case study building considered in the design example is a 3-storey residential CLT structure which was built and tested on the unidirectional 14,5m x 15m NIED Shaking Table facility in Tsukuba, Japan in July 2006 by a joint group of Italian (CNR-IVALSA) and Japanese (NIED) researchers within the SOFIE Project, which was a research project on the seismic and fire behaviour of multi-storey CLT buildings funded by the Province of Trento, Italy, and coordinated by CNR-IVALSA (Italian National Research Council – Trees and Timber Institute) [Ceccotti and Follesa, 2006].

3.1. *Geometric and structural configuration*

The building has a square plan with dimensions of 7.0x7.0 m and a total height of 10.0 m with a double-pitched roof. The floor plan is symmetric in both directions and at each floor there is a central wall parallel to the E-W direction with a central door opening of 2.25x2.40 m. The two outer walls at the first floor parallel to the E-W direction have two openings; one of 2.25x2.20 m and one of 4.00x2.20 m. The outer walls parallel to the N-S direction incorporate two window openings of 1.20x1.20 m. The net storey height is 2.95 m for the first two floors and varies from 2.75 m at eaves to 3.80 m at the ridge for the third floor. Plan and elevation drawings are displayed in Figure 10 while Figure 11 shows the construction sequence of the building.

The walls are made of 5-layer (**17**-17-**17**-17-**17**)[1] CLT panels 85 mm thick with the primary orientation (orientation of the face grain) along the vertical direction. Each wall assembly consists of three panels of about 2.3 m width. The floor panels, which are 1.35 m or 2.30 m wide, are oriented along the E-W direction and span the full length of the structure with 6-layer (**27**-17-**27**-**27**-17-**27**) 142 mm thick CLT panels for the first two storeys and 5-layer (**17**-17-**17**-17-**17)** 85 mm thick CLT panels for the roof. The CLT panels are considered to be constructed of EN C24 timber class. The strength, stiffness and density properties are listed in Table 1.

Table 1. *Strength and density properties for EN C24 timber class*

Characteristic strength in bending	$f_{m,k}$ (MPa)	24.0
Characteristic strength in tension parallel to grain	$f_{t,0,k}$ (MPa)	14.0
Characteristic strength in tension perpendicular to grain	$f_{t,90,k}$ (MPa)	0.4
Characteristic strength in compression parallel to grain	$f_{c,0,k}$ (MPa)	21.0
Characteristic strength in compression perpendicular to grain	$f_{c,90,k}$ (MPa)	2.5
Characteristic strength in shear	$f_{v,0,k}$ (MPa)	4.0
Mean modulus of elasticity parallel to grain	$E_{0,mean}$ (MPa)	11000
Mean modulus of elasticity perpendicular to grain	$E_{90,mean}$ (MPa)	370
Mean shear modulus	G_{mean} (MPa)	690
5th percentile of modulus of elasticity parallel to grain	$E_{0,05}$ (MPa)	7400
Mean density	ρ_{mean} (kg/m^3)	420
Characteristic density	ρ_k (kg/m^3)	350

[1] Values indicate the individual layer thicknesses in mm and bold fonts designate the layers parallel to the primary orientation (face grain) of the CLT panel

Figure 10. *Plans and elevations of the case study building (dimensions in m)*

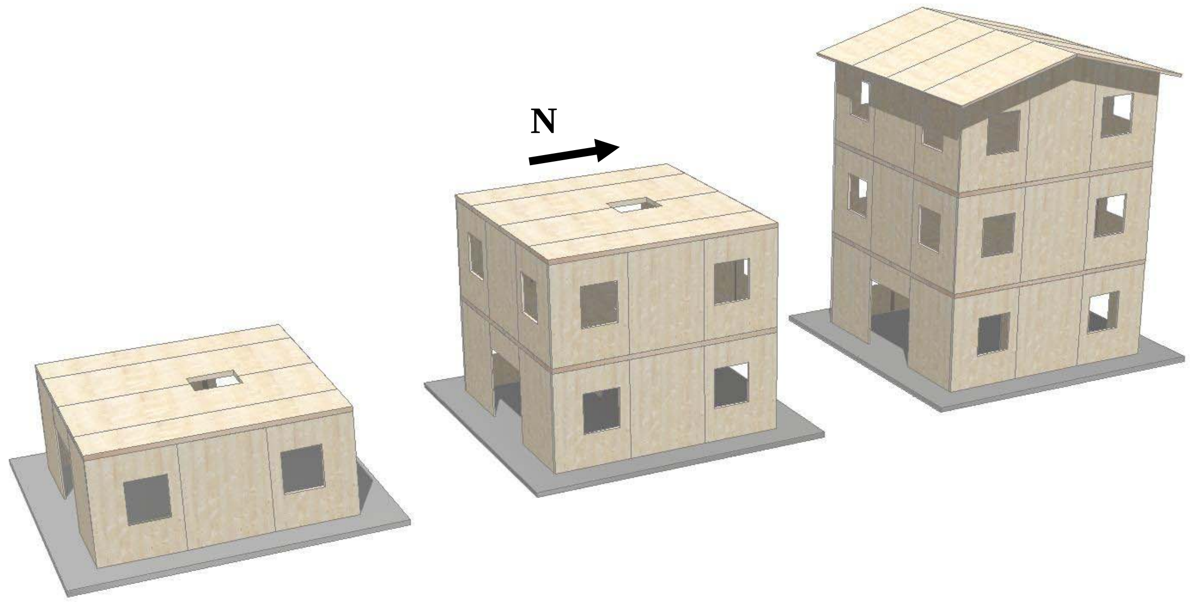


Figure 11. *Erection sequence of the case study building and direction of floor panels*

3.2. *Gravity loads and seismic weight*

Gravity loads for the seismic combination were estimated based on the structural and non-structural elements. The dead loads G of external and internal walls are 0.94 kPa and 0.79 kPa, respectively, while those of the floors and roof are 3.43 kPa and 1.28 kPa, respectively, where the roof loads refer to the inclined area. The live loads Q for the floors are 2.00 kPa for residential use and for the roof diaphragm no accidental load is considered for the seismic combination. Based on these gravity loads, Table 2 lists the total dead and live loads as well as the seismic weight of each floor of the building. The total seismic weight is W = 692.45 kN.

Table 2. *Total dead load, live loads and seismic weight for each level of the building*

Level	Dead load, G (kN)	Live load, Q (kN)	Seismic weight, G + 0.3Q (kN)
1	240.10	93.85	268.25
2	261.11	93.85	289.26
3	134.94	0.00	134.94
Sum	636.15	187.7	692.45

3.3 *Design spectrum*

The design response spectrum, illustrated in Figure 12, is defined from Section 3 of Eurocode 8, based on a Type 1 elastic response spectrum with the following parameters:

- design ground acceleration $\alpha_g = 0.35$ g;
- soil factor S = 1.2 for ground type B;
- lower limit of the period of constant spectral acceleration branch $T_B = 0.15$ s;
- upper limit of the period of constant spectral acceleration branch $T_C = 0.50$ s;
- value defining the beginning of the constant displacement response range of the spectrum $T_D = 2.00$ s;
- damping correction factor for 5% viscous damping $\eta = 1$; and
- behaviour factor q = 3.

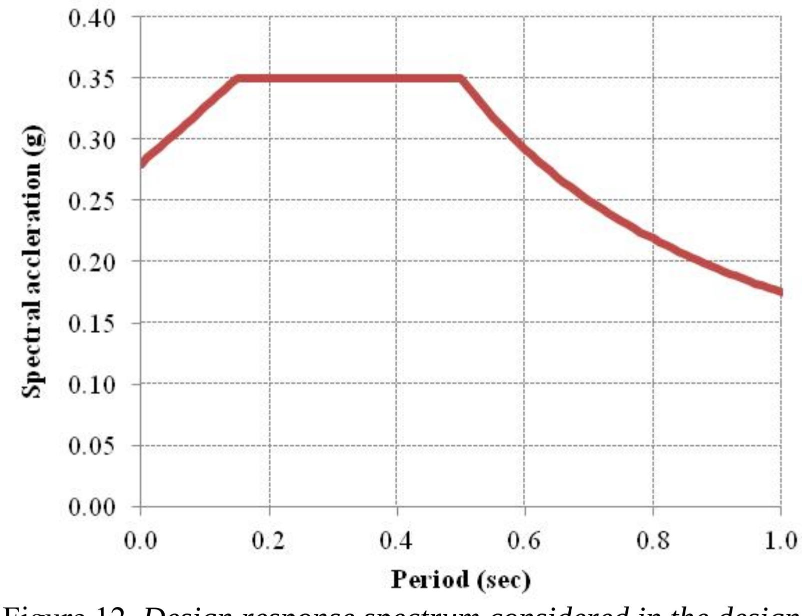


Figure 12. *Design response spectrum considered in the design example*

3.4 *Numerical model of the test building*

Based on the description provided in the previous section, a 3-dimensional numerical model has been developed for the 3-storey case study building. Floor and roof diaphragms are considered rigid in their plane and schematized by means of master-slave constraints applied to the reference nodes. Figure 13 shows the undeformed shape of the FE model and Figure 14 shows the Wall IDs of the building. The properties of the shell elements representing wall panels are computed according to the material properties shown in Table 1 and Equations (2-5) as:

$$
\begin{aligned}
E_x &= 4622\,MPa \\
E_y &= 370\,MPa \\
E_z &= 6748\,MPa \\
G_{xz} &= G_{xy} = G_{yz} = 690\,MPa
\end{aligned}
\tag{11}
$$

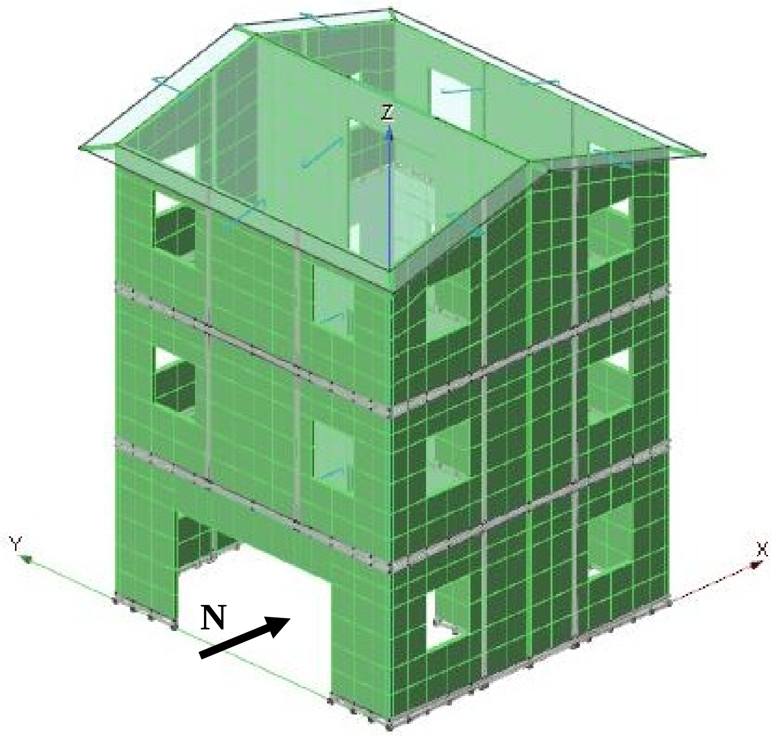


Figure 13. Undeformed shape of the building FE model

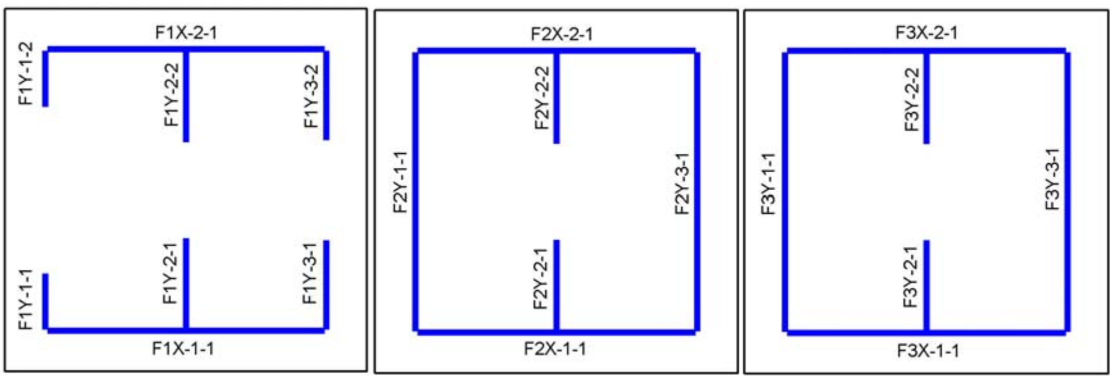


Figure 14. Wall IDs of structure for first (left), second (middle) and third (right) floor

3.5. *Preliminary analysis and design of the building*

Before the final design of the building, a preliminary design is conducted using the lateral force method of analysis. The natural vibration period of the structure is calculated from the code equation shown in Equation (1), which in this case gives:

$$T_1 = C_T \cdot h^{3/4} = 0.05 \cdot 10^{3/4} = 0.28\,\mathrm{s} \tag{12}$$

The purpose of the preliminary design is to provide all the necessary information needed to define the appropriate material and section properties of the elements used in the final finite element model. The lateral stiffness of the connections of the wall panels to the floor diaphragm below is assumed to be the same per linear meter for all wall assemblies of the structure. A value of 20 MN/m/m is used, based on engineering judgement, to calculate the required cross section of the pair of horizontal springs of each wall assembly according to Equation (9). Similarly, vertical step joints between panels of the same wall assembly are considered to provide the same generic stiffness per linear meter of 5 MN/m/m and the required cross section of each pair of vertical springs is computed according to Equation (10).

The results of the preliminary analysis in terms of shear per unit length for each wall and shear per unit height for each vertical step joint are presented in Table 3. The storey shear forces are 206.2 kN, 165.0 kN and 78.8 kN for the first, second and third storey, respectively, with a correction factor $\lambda = 0.85$ applied according to Section 4.3.3.2.2 of EC8.

Table 3. *Preliminary analysis and design results*

Wall ID	Length (m)	Height (m)	Shear per unit length (kN/m)	Vert. joint shear per unit height (kN/m)	No of ABs	AB stiffness (kN/m)	Vert. joint screw spacing (cm)	Vert. joint screw stiffness (kN/m)
F1X-1-1	6.94	2.95	16.82	9.44	11	362260.8	10.0	16242.5
F1X-2-1	6.94	2.95	16.82	9.44	11	362260.8	10.0	16242.5
F1Y-1-1	1.47	2.95	20.50	N/A	3	98798.4	N/A	N/A
F1Y-1-2	1.47	2.95	20.53	N/A	3	98798.4	N/A	N/A
F1Y-2-1	2.34	2.95	16.92	N/A	4	131731.2	N/A	N/A
F1Y-2-2	2.34	2.95	16.92	N/A	4	131731.2	N/A	N/A
F1Y-3-1	2.29	2.95	17.55	N/A	4	131731.2	N/A	N/A
F1Y-3-2	2.29	2.95	17.55	N/A	4	131731.2	N/A	N/A
F2X-1-1	6.94	2.95	12.85	10.94	16	191609.6	7.5	53615.7
F2X-2-1	6.94	2.95	12.85	10.94	16	191609.6	7.5	53615.7
F2Y-1-1	6.94	2.95	9.97	8.76	12	143707.2	10.0	16242.5
F2Y-2-1	2.34	2.95	8.81	N/A	4	47902.4	N/A	N/A
F2Y-2-2	2.34	2.95	8.81	N/A	4	47902.4	N/A	N/A
F2Y-3-1	6.94	2.95	9.92	8.11	12	143707.2	10.0	16242.5
F3X-1-1	6.94	3.40	6.14	8.19	8	95804.8	10.0	18338.3
F3X-2-1	6.94	3.40	6.14	8.19	8	95804.8	10.0	18338.3
F3Y-1-1	6.94	2.75	4.78	9.47	8	95804.8	10.0	15194.6
F3Y-2-1	2.34	3.70	4.15	N/A	4	47902.4	N/A	N/A
F3Y-2-2	2.34	3.70	4.15	N/A	4	47902.4	N/A	N/A
F3Y-3-1	6.94	2.75	4.76	8.15	8	95804.8	10.0	15194.6

Two types of angle brackets (AB) are used in the design of the building, which are manufactured by Rotho Blaas (2012): the WVS 90110 that is used for the shear-transferring connections of the 1st storey, and the WB90 that is used for the connections of the upper storeys. Table 4 summarizes the main properties of the angle brackets. The vertical step joints between panels of the same wall assembly are constructed with a single plywood strip 27x150 mm inserted into a groove in the internal side of the wall panels, as shown in Figure 4b, and nails ϕ2.8x80 that provide a design strength of 1.0 kN and a stiffness of 1047.9 kN/m per connector. Table 3 lists the required number of angle brackets and the required screw spacing in the vertical joints for each wall assembly as well as the corresponding stiffness for each type of connection.

Table 4. *Design strength and stiffness of angle brackets*

Angle bracket	Angle bracket to floor connection	Angle bracket to wall connection	Design strength (kN)	Stiffness (kN/m)
WVS 90110	1 ϕ12 steel rod class 4.6	11 ϕ4x60 anker nails	11.5	32932.8
WB 90	8 ϕ4x60 anker nails	8 ϕ4x60 anker nails	5.8	11975.6

3.6 *Final analysis and design of the building*

Based on the results of the preliminary design, the numerical model is updated with the correct stiffness contribution of the angle brackets and the vertical step joints to the horizontal and vertical pair of springs, respectively. The building is analyzed with the modal response spectrum analysis and Table 5 shows the fundamental periods and the mass participation factors for each mode shape of the structure. The first mode shape, with a period of 0.27 s that is similar to the 0.28 s computed with the code equation, is related to translation along the N-S direction of the building. The second mode shape, with a period of 0.24 s, is related to translation along the E-W direction of the building.

Table 5. *Modal analysis results*

Mode	Period (sec)	Mass participation for translation along N-S direction (%)	Mass participation for translation along E-W direction (%)	Mass participation for rotation along vertical direction (%)
1	0.27	83.91	0	0
2	0.24	0	84.27	1.35
3	0.16	0	0.83	84.76
4	0.09	12.59	0	0
5	0.08	0	12.59	0.03
6	0.06	2.88	0	0
7	0.06	0	0.11	10.99
8	0.05	0	1.61	0.09
9	0.04	0	0.02	1.94
Sum		99.38	99.43	99.16

The results of the response spectrum analysis in terms of shear per unit length, uplift force for each wall, and shear per unit height for each vertical step joint are presented in Table 6. The storey shear forces along the N-S direction are 205.6 kN, 165.0 kN and 80.4 kN for the first, second and third storey, respectively, while along the E-W direction are 207.2 kN, 163.6 kN and 79.2 kN. These values are very similar to the values obtained from the lateral force method of analysis.

Two types of hold-downs (HDs), the WHT 340 and the WHT 440, which are manufactured by Rotho Blaas (2012), are used to restrain the building uplift. In addition, a steel strap (SS) with dimensions of 100x1000x1.5 mm is used as a tie-down to restrain external wall assemblies of the second and third storey against uplift. Table 7 summarizes the main properties of the hold-downs and tie-downs. Self-tapping screws HBS ϕ8x300 and ϕ8x200 are prescribed for the connection of the floor and the roof panels, respectively, to the supporting wall panels. The design strength per screw connector is 4.56 kN and 4.24 kN for the ϕ8x300 and ϕ8x200, respectively. This type of connection is designed for a force equal to the shear force of the supporting wall multiplied by an overstrength factor of 1.6. Self-tapping screws HBS ϕ8x200, with a design strength of 4.24 kN, are also prescribed for the connection between perpendicular walls. This type of connection is designed for a force equal to the maximum shear force per unit length between the two walls multiplied by the height of the connection and further multiplied by an overstrength factor of 1.6. Table 6 shows the required number of ABs and the type of hold-downs/tie-downs for each wall assembly, as well as the required spacing for the screws in the vertical step joints, the wall-to-floor connections and the perpendicular wall connections.

Table 6. *Final analysis and design results*

Wall ID	Shear per unit length (kN/m)	Uplift (kN)	Vert. joint shear per unit height (kN/m)	No of ABs	Vert. Joint screw spacing (cm)	Type of hold-down/tie down	Wall-to-floor connection spacing (cm)	Perpendicular wall connection spacing (cm)
F1X-1-1	17.02	42.55	5.75	11	15.0	WHT440	15.0	10.0
F1X-2-1	17.02	42.55	5.75	11	15.0	WHT440	15.0	10.0
F1Y-1-1	22.52	34.84	N/A	3	N/A	WHT340	10.0	10.0
F1Y-1-2	22.56	34.84	N/A	3	N/A	WHT340	10.0	10.0
F1Y-2-1	17.52	34.01	N/A	4	N/A	WHT340	10.0	10.0
F1Y-2-2	17.52	34.02	N/A	4	N/A	WHT340	10.0	10.0
F1Y-3-1	15.90	46.61	N/A	4	N/A	WHT440	10.0	10.0
F1Y-3-2	15.90	46.61	N/A	4	N/A	WHT440	10.0	10.0
F2X-1-1	13.29	8.09	9.78	16	10.0	SS	20.0	15.0
F2X-2-1	13.29	8.09	9.78	16	10.0	SS	20.0	15.0
F2Y-1-1	10.41	24.25	5.67	13	15.0	SS	20.0	15.0
F2Y-2-1	8.42	4.65	N/A	4	N/A	WHT340	20.0	20.0
F2Y-2-2	8.42	4.65	N/A	4	N/A	WHT340	20.0	20.0
F2Y-3-1	9.29	14.65	4.69	12	15.0	SS	20.0	15.0
F3X-1-1	6.38	0.00	6.57	8	15.0	SS	20.0	20.0
F3X-2-1	6.38	0.00	6.57	8	15.0	SS	20.0	20.0
F3Y-1-1	4.83	0.00	8.19	8	10.0	SS	20.0	20.0
F3Y-2-1	5.22	7.97	N/A	4	N/A	WHT340	20.0	20.0
F3Y-2-2	5.22	7.97	N/A	4	N/A	WHT340	20.0	20.0
F3Y-3-1	4.18	0.00	6.39	8	15.0	SS	20.0	20.0

Table 7: *Design strength and stiffness of hold-downs and tie-downs*

Type of hold-down/tie-down	Connection to the floor for HDs or to the wall below for SSs	Connection to the wall	Design strength (kN)
WHT 340	1 ϕ16 steel rod class 4.6	14 ϕ4x60 anker nails	36.5
WHT 440	1 ϕ16 steel rod class 4.6	18 ϕ4x60 anker nails	46.9
SS 100x1000x1.5	15 ϕ4x60 anker nails	15 ϕ4x60 anker nails	29.5

4. **Validation of the Numerical Model**

In order to validate the numerical modelling approach described in Section 2.2, the experimental results from three low intensity shaking table tests of a three-storey full scale CLT building [Ceccotti and Follesa, 2006] are compared with the numerical predictions from linear dynamic analyses of a representative numerical model.

The test structure was tested with three different configurations, as shown in Figure 15, where the differences were the openings at the first floor along the W-E direction, the direction of shaking. The third configuration is identical to the structure considered in the case study building designed in the previous sections. The first configuration, considered in this case for the validation of the numerical model, incorporated a 1.2 m long door opening at the middle of the wall assemblies.

The ground motion records used for the shaking table tests are listed in Table 8. All three configurations were tested under increasing amplitudes of Peak Ground Acceleration (PGA) of 0.15g, 0.35g, 0.50g and higher. In order to validate the numerical model with a linear analysis, the low intensity tests (with PGA equal to 0.15g) conducted on the initial configuration, where the structure was not subjected to any strong shaking yet, were selected as the most appropriate data for the appraisal of the numerical approach.

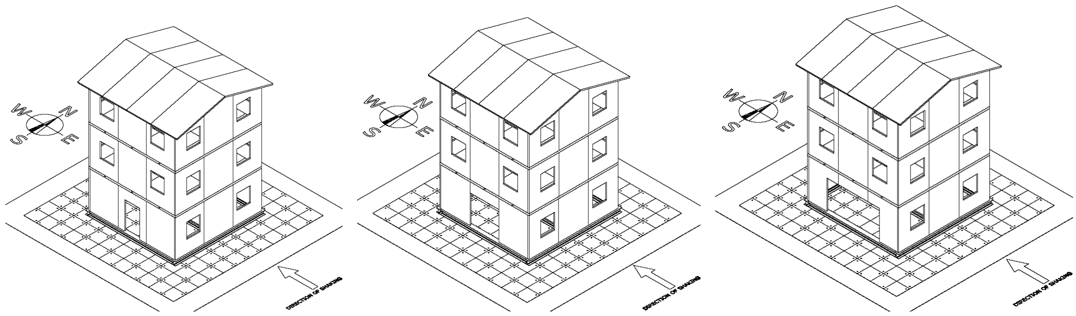


Figure 15. *First (left), second (middle) and third (right) configuration of the test structure*

Table 8. *Details of the original ground motions used in the shaking table tests*

Record Name	Country	Date	Station	Component	Duration (s)	PGA (g)
Kobe	Japan	1995/01/16	JMA	N-S	48.0	0.820
El Centro	California	1940/05/19	Imperial Valley	N-S	40.0	0.313
Nocera Umbra	Italy	1997/09/27	Nocera	E-W	13.7	0.500

4.1. *Test structure and numerical model*

The thickness and lay-up of wall and floor panels of the test structure were identical to the case study building described in Section 3.1 and the orthotropic material properties considered in the model are shown in Equation (11). The details of the shear-transferring connections and the vertical step joints of the test building, which are those schematized in the numerical model, are listed in Table 9, while Table 10 lists the corresponding stiffness values of the connections according to the procedure described in Section 2.2 and presented in Equations (6-10). Steel blocks were anchored on each floor diaphragm to augment the gravity and seismic load and account for permanent and live loads in seismic combination as prescribed by Eurocode 8. The dead and additional loads of the test structure are listed in Table 11. Note that the dead loads are calculated for each storey by considering the floor panels, half of the lower walls and half of the upper walls. The undeformed shape of the FE model is shown in Figure 16.

Table 9. *Details of the horizontal and vertical connections used in the test building*

Storey	Wall to floor horizontal connection for each floor	Vertical joint between wall panels
1	6 BMF 90x48x3,0x116 for each wall assembly, connected to the wall with 8 ϕ4x60 anker nails and to the foundation with one ϕ16 anchor bolt	Self tapping screws ϕ8x80 spaced at 450 mm c/c
2	4 BMF 105 for each wall assembly, connected to the wall and to the floor with 8 ϕ4x60 anker nails.	Self tapping screws ϕ8x80 spaced at 600 mm c/c
3	3 BMF 105 for each wall assembly, connected to	Self tapping screws ϕ8x80

	the wall and to the floor with 5 ϕ4x60 anker nails.	spaced at 900 mm c/c

Table 10. *Total stiffness calculated for the horizontal and vertical springs*

Storey	Total stiffness for angle bracket connections (kN/m)	Total stiffness for vertical step joints (kN/m)
1	K_1=197597 (wall length 6.935 m - 6 brackets); K_2=98798 (wall length 2.340 m - 3 brackets); K_3=98798 (wall length 2.868 m - 3 brackets)	K=10479 (wall height 2.95 m - 7 screws)
2	K_1=47902 (wall length 6.935 m - 4 brackets); K_2=23951 (wall length 2.34 m - 2 brackets)	K=7485 (wall height 2.95 m - 5 screws)
3	K_1=22454 (wall length 6.935 m - 3 brackets); K_2=14969 (wall length 2.340 m - 2 brackets); K_3=7485 (wall length 2.340 m - 1 bracket)	K=4491 (wall height 2.95 m - 3 screws)

Table 11. *List of dead and additional weights used in the test building*

Storey	Dead load (kN)	Additional load (kN)	Total load (kN)
1	69.3	150.0	219.3
2	69.3	150.0	219.3
3	47.4	0.0	47.4
Total			486.0

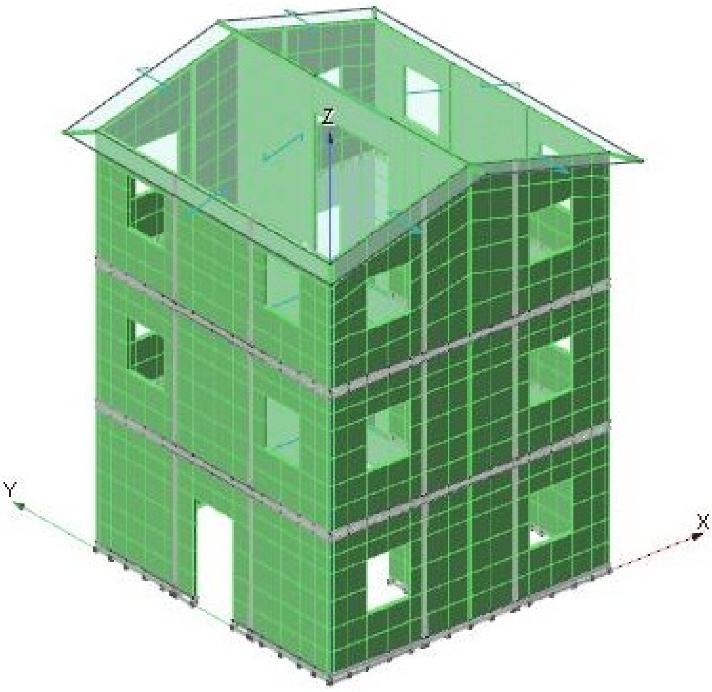


Figure 16. *Undeformed shape of the FE model*

4.2. *Comparison with the test results*

A modal linear analysis was carried out in SAP2000, with a modal damping of 5% constant for all modes, to analyze the numerical model under the three recorded shake table motions that were scaled to a peak acceleration of 0.15g. The comparison between the model and the test results is made in terms of natural period, maximum base shear and maximum displacements relative to the base measured at the centre of each storey. The experimental results and the numerical predictions are presented in Table 12.

The results indicate that the numerical model predicts a more flexible response than the actual test structure (natural vibration period of 0.195 s versus 0.166 s) and consequently the maximum displacements are overpredicted from 20% to as much as 65%. Maximum predicted base shear values are fairly well correlated to the experimental ones. This difference in the lateral stiffness of the structure is attributed to the friction between horizontal and vertical diaphragms, which is not accounted for in the numerical model but can nevertheless contribute in reducing both the natural period of the building and the measured displacements. A secondary effect that is not accounted for in the numerical model is the contribution of the out-of-plane bending stiffness of the CLT floor panels in reducing the rocking deformation of wall panels, although this effect can be more pronounced in the direction of the floor panels, which in this case were spanning perpendicular to the seismic direction.

Table 12. *Comparison between model and experimental results*

	Period (s)	Base shear (kN)	3rd storey (roof) displacement (mm)	2nd storey displacement (mm)	1st storey displacement (mm)
Kobe 0.15g					
Model	0.195	77.3	2.52	1.77	0.74
Experimental	0.166	87.6	1.54	1.29	0.49
Difference (%)	17.47%	-11.74%	64.01%	36.92%	52.53%
El Centro 0.15g					
Model	0.195	138.4	5.37	3.64	1.40
Experimental	0.166	129.1	3.90	2.80	1.15
Difference (%)	17.47%	7.22%	37.78%	30.20%	21.41%
Nocera Umbra 0.15g					
Model	0.195	193.90	7.17	4.94	1.93
Experimental	0.166	156.08	5.08	3.62	1.50
Difference (%)	17.47%	24.23%	41.20%	36.61%	28.26%

In order to quantify the effect of friction, a second numerical model is formulated that incorporates an increased stiffness in the horizontal springs. The additional stiffness provided by the friction was estimated first by selecting a coefficient of friction, which yields the lateral strength at the base of the wall due to friction when multiplied by the gravity axial load transferred by each wall. Then, a representative yield displacement is used to divide the lateral strength and yield the stiffness due to friction. According to literature references [Forest Products Laboratory, 2010], the coefficient of kinetic friction μ_k depends on the moisture content of wood and the roughness of the wood surface and of the second contact surface, ranging from 0.3 to 0.5. In this example, a mean value of 0.4 has been assumed. Consequently, considering the weight W_i at the base of each wall, the value of the friction force F_f is calculated as:

$$F_f = \mu_k \cdot W_i \tag{13}$$

The yield displacement is calculated by considering the yield displacement of the weakest fastener of the shear-transferring connection, i.e. the ϕ4 anker nails used in the connection of the angle brackets. This may be evaluated, in the absence of test results, by dividing the characteristic strength of the nail $R_{c,k}$, calculated according to EC5 with the embedment strength of wood taken from the equations provided by Uibel and Blaß [2006 and 2007], by the slip modulus under service load of Equation (6), i.e.:

$$u_y = \frac{R_{c,k}}{2 \cdot \rho_m^{1.5} \cdot d_f / 23} = \frac{2370}{2994} = 0.79\,mm \tag{14}$$

Therefore, the stiffness due to friction is evaluated as:

$$K_f = \frac{F_f}{u_y} \tag{15}$$

Adding the contribution of friction to the stiffness provided by the angle brackets, the total horizontal stiffness of each wall assembly was recalculated and the results are shown in Table 13.

Table 13. *Total stiffness calculated for the horizontal and vertical springs taking into account the friction contribution*

Storey	Total stiffness for angle bracket connection (kN/m)	Total stiffness for vertical step joint (kN/m)
1	K_1=197607 (wall length 6.935 m - 6 brackets); K_2=154189 (wall length 2.34 m - 3 brackets); K_3=124494 (wall length 2.868 m - 3 brackets)	K=10479 (wall height 2.95 m - 7 screws)
2	K_1=79252 (wall length 6.935 m - 4 brackets); K_2=55301 (wall length 2.34 m - 2 brackets)	K=7485 (wall height 2.95 m - 5 screws)
3	K_1=29771 (wall length 6.935 m - 3 brackets); K_2=22286 (wall length 2.34 m - 2 brackets); K_3=14801 (wall length 2.34 m - 1 bracket)	K=4491 (wall height 2.95 m - 3 screws)

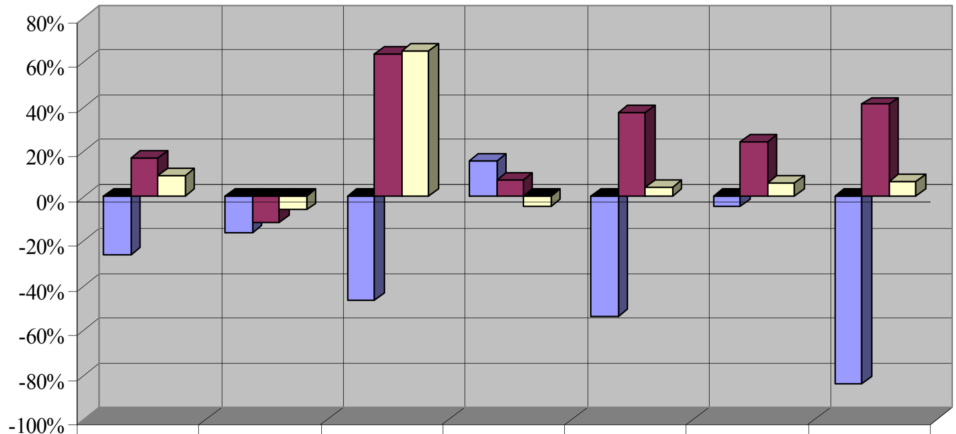
Table 14 shows the comparison with the updated numerical model with the friction contribution. The predicted natural period is still higher than the experimental one (0.182 s versus 0.166 s) but apart from the displacement predictions for Kobe earthquake, a very good correlation with the experimental results is obtained, thus leading to the conclusion that the friction contribution should be taken into account in the calculation of the horizontal stiffness.

The modelling approach described in Section 2, where all the horizontal wall-to-floor and vertical panel-to-panel connections are schematized with equivalent truss elements, may require a long input procedure especially for large and tall buildings. For this reason, sometimes simplified numerical models where all the joints between CLT elements are considered as rigid are used by designers. In order to quantify the effect of modelling the flexibility of the mechanical connections, a third numerical model was generated without any connections among the shell elements. Figure 17 illustrates the comparison between the experimental results and the numerical predictions from all three numerical models. As it can be observed the differences from the experimental values of the third model are larger, especially in terms of natural vibration periods and maximum displacements. This difference in the natural period can lead to significant differences in the calculation of the design seismic load, and confirms results of previous research [Fragiacomo et al. 2011].

Table 14. *Comparison between model with friction contribution and experimental results*

	Period (s)	Base shear (kN)	3rd storey (roof) displacement (mm)	2nd storey displacement (mm)	1st storey displacement (mm)
Kobe 0.15g					
Model	0.182	82.3	2.54	1.75	0.76
Experimental	0.166	87.6	1.54	1.29	0.49
Difference (%)	9.64%	-6.03%	65.32%	35.38%	56.65%
El Centro 0.15g					
Model	0.182	123.3	4.05	2.74	1.15
Experimental	0.166	129.1	3.90	2.80	1.15
Difference (%)	9.64%	-4.48%	3.91%	-2.00%	-0.27%

Nocera Umbra 0.15g					
Model	0.182	165.90	5.41	3.69	1.55
Experimental	0.166	156.08	5.08	3.62	1.50
Difference (%)	9.64%	6.29%	6.54%	2.04%	3.00%



	Period	Kobe Max. Base Shear	Kobe Max. Roof Displacemen	El Centro Max. Base Shear	El Centro Max. Roof Displacemen	Nocera Max. Base Shear	Nocera Max. Roof Displacemen
Rigid Connections	-25.90%	-16.19%	-46.63%	16.05%	-53.56%	-4.28%	-83.85%
Flexible Connections	17.47%	-11.74%	64.01%	7.22%	37.78%	24.23%	41.20%
Flexible Connections + Friction	9.64%	-6.03%	65.32%	-4.48%	3.91%	6.29%	6.54%

Figure 17. *Percentage difference between numerical models and experimental results*

5. **Conclusions**

Design provisions for the seismic design and a numerical procedure for the linear dynamic analysis of CLT buildings, which could be used by practicing engineers, are proposed in this paper and explained with a design example of a three storey case study building. The modelling procedure is based on code and literature references for the calculation of the mechanical properties of CLT panels and mechanical connections.

The proposed numerical model is made of shell elements for the CLT wall panels and truss elements for the mechanical connections between wall panels and between walls and floors. The analysis is carried out using SAP 2000. Hold-down connections are not explicitly modelled, but the uplifting vertical forces are obtained from the tensile forces of the vertical truss elements which are used to simulate the deformability of the floor diaphragms along their thickness.

The model has then been validated by comparing the analysis results of a sample three storey building tested on a unidirectional shaking table in Japan in 2006 with the test results. The comparison showed general good agreement for the natural vibration period and base shear but a general overestimation of maximum displacements. Results in terms of maximum displacements showed a much better correlation by taking into account the friction contribution.

Finally a comparison was made with a third model without any connections among the shell elements. The results in terms of differences from the experimental values of the third model are large, especially in terms of natural vibration period, which can lead to significant differences in the calculation of the design seismic load and maximum displacements.

Although the model showed a good agreement with the test results, further comparisons with other test results are needed in order to check and better calibrate the modelling procedure, which is the following step of this work. Further research is also needed to obtain prediction formulae for the evaluation of the slip modulus under service load of dowel type fasteners in CLT panels, in order to better estimate the correct linear behaviour of CLT buildings via numerical models.

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