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Cross-laminated timber buildings: numerical analysis of multi-storey structures

Presentata da: Ing. **Stefano Pacchioli**

Coordinatore Dottorato
Prof. **Luca Vittuari**

Supervisore
Prof. **Luca Pozza**

Co-Supervisore
Dr. Ing. **Andrea Polastri**

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Sommario

Scopo della tesi è quello di analizzare il comportamento sismico degli edifici multipiano con struttura portante a pannelli massicci composti da tavole incollate ed incrociate, comunemente conosciuti come pannelli Cross Laminated Timber (CLT). I principali argomenti trattati nel presente lavoro riguardano le diverse metodologie di modellazione utilizzate per la caratterizzazione del comportamento sismico e della progettazione per edifici in CLT e lo sviluppo di un sistema di connessione innovativo che consente l'ottimizzazione del numero di pareti sismo-resistenti e di connessioni permettendo ampi spazi privi di pareti in accordo con le esigenze architettoniche moderne.

Nella parte introduttiva si è effettuata una panoramica riguardante le principali tipologie di edifici multipiano in legno e si sono analizzati i principali motivi che hanno spinto il settore delle costruzioni a richiedere la realizzazione di edifici interamente in legno con sempre maggiore sviluppo in altezza. Particolare attenzione è stata posta nei confronti delle strutture in CLT, il cui comportamento sismico dipende in maniera sostanziale dagli elementi di connessione i quali sono stati analizzati in forma dettagliata. Nell'ultima parte dell'introduzione si sono elencati i principali vantaggi e le criticità legate a questa recente tipologia di costruzioni che ha visto uno sviluppo esponenziale nell'ultimo decennio. Con lo scopo di approfondire i temi ritenuti più significativi riguardanti gli edifici multipiano in CLT, si è deciso di dividere il presente lavoro nei due macro-argomenti riportati di seguito e trattati in forma indipendente.

Il primo argomento riguarda le diverse metodologie di modellazione impiegate per la caratterizzazione del comportamento sismico di edifici multipiano in CLT (sia del tipo lineare che non lineare). Per quanto riguarda l'ambito lineare sono state analizzate nel dettaglio le due diverse strategie di modellazione numerica utilizzate abitualmente nel mondo della ricerca (modellazione per componenti) e della progettazione (modellazione fenomenologica) con lo scopo di andare a definire un modello di calcolo semplificato utilizzabile dai professionisti per predire correttamente il comportamento globale delle strutture in CLT. Nella modellazione *per componenti* i pannelli in CLT sono stati modellati con elementi di tipo shell, mentre le connessioni sono state modellate con molle lineari a cui sono stati assegnati i legami costituivi desunti da test sperimentali. Nella modellazione *fenomenologica* le connessioni non vengono direttamente implementate, ma è possibile tener conto del loro comportamento andando a calibrare una rigidezza equivalente ridotta da assegnare ai pannelli CLT. Tale rigidezza è funzione di molteplici variabili, tra cui le più importanti sono la geometria della parete, il pattern di connessioni e la massa sismica. È stato quindi necessario definire un'analisi parametrica. Grazie a quest'ultima e a un algoritmo di ottimizzazione, in grado di minimizzare l'errore sui parametri ritenuti più significativi (periodo di vibrare, drift e azioni sollecitanti le connessioni), si sono ricavati degli abachi da fornire ai progettisti per il calcolo e la progettazione di edifici in CLT all'interno di una modellazione fenomenologica semplificata, compatibile con i tempi di progettazione degli studi tecnici e direttamente implementabile nei software di modellazione commerciali.
Passando alle analisi del tipo non lineare, si è investigato il periodo proprio fondamentale degli edifici multipiano in CLT grazie all'utilizzo di analisi time-history con input sinusoidale a frequenza variabile. Nel dettaglio si è studiata l'influenza del carico verticale stabilizzante sulla rigidezza globale della struttura. E' stato dimostrato come a causa dell'elevata rigidezza dei pannelli in CLT, il comportamento elastico degli edifici con struttura in CLT sia governato principalmente dalle proprietà meccaniche e dall'attivazione dei connettori utilizzati per collegare i pannelli tra loro e alla fondazione. Partendo dai risultati ottenuti da una campagna sperimentale, appositamente progettata e condotta al fine di studiare l'influenza del carico verticale stabilizzante sulla rigidezza globale della struttura, è stato validato un modello numerico agli elementi finiti il

quale è stato utilizzato per implementare un'estesa analisi parametrica. Grazie a quest'ultima è possibile affermare che l'attivazione del fenomeno di rocking delle pareti sismoresistenti e la conseguente attivazione dei connettori resistenti alle forze di trazione ricopre un ruolo fondamentale nella definizione della rigidezza globale degli edifici in CLT. Sono state proposte formulazioni utili alla valutazione del periodo proprio fondamentale, correlando quest'ultimo allo spostamento laterale dell'edificio.

Il secondo macro-argomento trattato nella tesi riguarda la definizione di una metodologia di progettazione che consente di ottimizzare il numero di pareti sismoresistenti ovvero di utilizzare un numero limitato di pareti resistenti alle forze orizzontali mentre le restanti sono sollecitate da soli carichi verticali. Tale metodologia permette ampi spazi privi di pareti e quindi grande libertà compositiva e di fruizione degli ambienti e facilita il controllo delle strutture in seguito ad un evento sismico. Si è dimostrato come le connessioni tradizionali impiegate per la realizzazione di edifici di altezza limitata (minore di 5 piani) non presentino proprietà meccaniche adeguate, sia in termini di resistenza che di rigidezza, per permettere una progettazione in grado di limitare il numero di pareti sismo-resistenti. Infatti, in edifici multipiano caratterizzati da un numero di pareti di controvento ridotto, le azioni sollecitanti nei connettori resistenti a trazione risultano particolarmente elevate in quanto l'effetto ribaltante generato dalle forze orizzontali tende a essere predominante rispetto all'effetto stabilizzante dei carichi verticali. Un ulteriore aspetto significativo riguarda la deformabilità delle strutture in CLT quando sollecitate da azioni orizzontali. Tale deformabilità è imputabile principalmente ai fenomeni di rocking che sollecitano le connessioni resistenti a trazione poste alla base dell'edificio e a livello di interpiano. La limitata rigidezza di tali connessioni amplifica gli spostamenti laterali della parete controventante in particolare quando le pareti di controvento sono snelle. Si è quindi sviluppato un sistema di connessione innovativo basato su piatti di acciaio continui lungo l'altezza della parete da disporre alle estremità di quest'ultime in grado di resistere alle elevate forze di trazione e di conferire sufficienza rigidezza alla struttura. Nell'ultima parte della tesi sono state proposte alcune soluzioni tecnologiche riguardanti il sistema innovativo appena descritto in grado di permetterne l'applicazione e portando particolare attenzione sia al tema strutturale che ai temi della durabilità e dell'efficienza energetica. Quest'ultimi due temi risultano particolarmente delicati nel nodo di collegamento tra pannello in CLT e fondazione in cemento armato, per il quale si sono proposte soluzioni in grado di mantenere la base del pannello sollevata rispetto al piano delle fondazioni e di garantire una continuità dell'isolazione posta sul lato esterno del pannello.

Parole chiave: Cross-Laminated Timber (CLT); progettazione sismica; strutture in legno; edifici multipiano; modellazione numerica; strategie di modellazione; metodi di progettazione; Tie-down; edifici alti in CLT.

Abstract

The aim of the thesis is to analyze the seismic behavior of Cross Laminated Timber (CLT) buildings. The main topics covered in this work concern the different modeling methodologies used for the characterization of seismic behavior and the design for CLT buildings and the development of an innovative connection system that permits the optimization of the number of seismic-resistant walls and of connections allowing large spaces without walls in accordance with modern architectural needs.

In the introduction, an overview of the main types of multi-storey wooden buildings was made analyzing the main reasons that led the construction sector to request the realization of multi-story full-timber buildings. Particular attention was paid to CLT structures whose seismic behavior substantially depends on the connection elements which were analyzed in detail. In the last part of the introduction the main advantages and criticalities related to this recent type of construction were listed. With the aim of developing the most significant themes concerning multi-storey CLT buildings, it was decided to divide this work into the two macro-topics listed below and treated independently.

The first topic concerns the different modeling methodologies used for the characterization of the seismic behavior of CLT multi-storey buildings (both linear and non-linear). As regards linear analysis, the two different numerical modeling strategies commonly used in research field (component approach) and in design field (phenomenological approach) were analyzed with the aim of defining a simplified calculation strategy that can be used by practitioners to correctly predict the behavior of CLT structures. In component model approach, CLT panels were modeled with shell-type elements, while the connections were modeled with linear springs assigning them constitutive laws obtained from experimental tests.

In phenomenological model approach connection elements are not directly implemented, but their behavior can be considered by calibrating a reduced equivalent stiffness to be assigned to the CLT panels. This stiffness is a function of multiple variables, the most important of which are the geometry of the wall, the pattern of connections and the seismic mass. It was therefore necessary to define a parametric analysis. Thanks to the latter and to least squares method, able to minimize the error on the parameters considered to be the most significant (principal elastic period, drift and forces acting the connections), conventional equivalent elastic stiffness to be assigned to CLT wall and capable of accounting for the deformability of the CLT panel and connections assembly were obtained. The equivalent stiffness can be used by engineers in the design phase in simplified phenomenological approach that is compatible with the design times of technical studies and directly implementable in commercial software.

Moving on to the non-linear analyses, the fundamental period of CLT multi-storey buildings was investigated thanks to the use of time-history analysis with sinusoidal variable frequency input. In detail, the influence of stabilizing vertical loads on the overall stiffness of the structure has been studied. It was demonstrated that due to the high in-plane stiffness of CLT panels, the elastic behaviour of CLT buildings is primarily governed by the mechanical properties of joints used to connect the panels to each other and to foundation. The results from an experimental campaign, specially designed and conducted in order to study the influence of the stabilizing vertical load on the overall stiffness of the structure, were adopted to validate a FE numerical model used to perform an extended parametric analysis. Results obtained from parametric analysis allows to affirm that the activation of the rocking phenomenon of the seismic-resistant walls and the consequent activation of the connectors resistant to traction forces play a fundamental role in the definition of the global stiffness of CLT buildings. Formulations useful for the evaluation of the fundamental proper period were proposed, correlating the latter to the lateral drift of the building.

The second macro-topic treated in the thesis concerns the definition of a design methodology that allows to optimize the number of seismic-resistant walls and therefore to use a limited number of walls resistant to horizontal forces while the rest have to withstand only to vertical loads. This methodology allows great freedom of composition and use of spaces and facilitates the control of the structures following a seismic event. It was demonstrated that traditional connections used for the construction of buildings of limited height (less than 5 floors) do not have adequate mechanical properties, both in terms of strength and stiffness, to allow a design capable of limiting the number of seismic-resistant walls. In fact, in multi-storey buildings characterized by a reduced number of bracing walls, forces acting in tensile resistant connectors are particularly high as the overturning effect generated by the horizontal forces is predominant compared to the stabilizing effect of vertical loads. A further significant aspect concerns the deformability of CLT structures when stressed by horizontal actions. This deformability is mainly attributable to rocking phenomena that involves tensile-resistant connections placed at the base of the building and at the inter-floor level. The limited stiffness of these connections amplifies the lateral displacements of the bracing wall in particular when the bracing walls are slender. An innovative connection system based on the use of vertical steel ties as alternative or in addition to traditional earthquake-resistant systems and able to withstand high seismic tensile forces and to limit inter-storey drifts was developed.
In the last part of the thesis, some technological solutions regarding the innovative system just described were studied paying attention to both the structural and the durability / energy efficiency issues. The latter two are particularly delicate in the connection node between the CLT panel and the reinforced concrete foundation. Solutions able to keep the base of the panel raised with respect to the plane of the foundations and to guarantee continuity of the insulation placed on the external side of the panel were proposed.

Keywords: Cross-Laminated Timber (CLT); seismic design, timber structures; multi-storey buildings; numerical modelling; modelling strategies; design strategies; Tie-down; high CLT buildings.

List of abbreviations

CLT	Cross-Laminated Timber
CM	Component model
DLS	Damage Limitation State
DOPHEM	Design Oriented PHEnomenological Modelling
EC	Eurocode
EC5	Eurocode 5
EC8	Eurocode 8
ENLID	Elastic non-linear incremental dynamic
ETA	European Technical Assessments
FE	Finite Element
FEM	Finite Element Method
FFT	Fast Fourier Transforms
FRFs	Frequency Response Functions
LFM	Lateral Force Method
LDA	Linear Dynamic Analyses
LLRS	Lateral Load Resisting System
LVL	Laminated Veneer Lumber
MRS	Modal Response Spectrum
OSB	Oriented Strand Boards
PGA	Peak Ground Acceleration
PM	Phenomenological model
RC	Reinforced Concrete
SFRS	Seismic Force Resisting Systems
SLS	Serviceability Limit State
ULS	Ultimate Limit State

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Introduction

Motivation and scope of the research

Environmental sustainability and optimization of energy consumption of buildings have led to strict guidelines, especially in urban areas [1]. For this reason, the construction industry is trying to develop modern technologies for new and existing buildings [2], based on minimizing carbon dioxide emissions and reducing soil consumption. In this scenario, wooden construction materials are experiencing significant enhancements. In particular, Cross-Laminated Timber (CLT) is a rapidly spreading construction system in current building practice and in recent years has acquired a valid alternative also in the construction of multi-storey buildings. Wood-based engineered CLT panels are characterized by high in-plane strength and stiffness of the timber panel conferred by the cross-lamination of solid-wood boards [3], which give the structure good resistance to vertical and lateral loads [4]. High structural performance, lightness, high level of prefabrication and rapid execution of the system are just some of the peculiarities that distinguish this structural type. For these reasons and due renewability and recyclability of the material, CLT buildings have become increasingly common over the last few years. Another reason that favours the success of CLT structure is represented by the good fire performance [5], contrary to what is thought as CLT is a combustible material. On the other hand, being CLT structures a relatively new technology, researchers are still working in order to investigate seismic behavior of CLT multi-story buildings through experimental tests [6,7] and numerical simulations methods [8,9]. Several studies have been conducted to determinate the behavior factor [10] and to define the mechanical proprieties of connection elements, which are assigned the function of connecting the panels together and to concrete foundation and dissipating energy during a seismic event [11,12]. It has been demonstrated that connections play a key role in the structural behavior of multi-story CLT buildings [13] and that it is necessary to consider the tension-shear interaction phenomenon of traditional hold-down and angle-bracket [14].

Using as references the above-mentioned studies, in **Part I** of this thesis two main modelling approaches adopted from researcher and practitioners for linear analyses of CLT buildings have been studied. The first method (component model) is research oriented and adopts elastic springs for connections and elastic elements for the CLT in order to investigate both the seismic behaviour and the limits of CLT structures [15,16]. The second method (simplified phenomenological model) is design oriented and the analyses are aimed to provide the necessary information for a simple and safe design of CLT buildings. In this method the behaviour of the system is reproduced by means of a conventional equivalent elastic stiffness assigned to the CLT wall and capable of accounting for the deformability of the CLT panel and connections assembly [17]. The aim of this part of thesis is to calibrate the correct equivalent stiffness to assign to CLT panels in a simplified phenomenological model in order to reproduce the seismic behaviour of the entire CLT building, that can be used by engineers in the design phase. In order to provide reliable estimation of the equivalent stiffness for the phenomenological model, different case study have been analysed varying the significant parameters such as, number of storeys, connection pattern, geometry and wall slenderness, that control the building response.

Another aspect that plays a key-role in the determination of dynamic properties of CLT multi-storey structures, is represented by the calculation of vibration period when earthquake motion is represented by an elastic ground acceleration response spectrum. The calculation of vibration periods and modes is in fact an essential step for the definition of seismic actions on the basis of linear-elastic behaviour of structures. Numerical finite element models are commonly adopted to adequately represent the distribution of stiffness and mass in case of modal response spectrum (MRS) analyses. Approximate expressions for the calculation of fundamental period as function of the total height of the buildings are generally reported in Standard Documents when Lateral Force Method (LFM) of analysis is implemented [18,19]. Due to the high in-plane stiffness of CLT panels, the elastic behaviour of CLT buildings is primarily governed by the mechanical properties of joints and mechanical connections used to connect the panels to each other and to foundation. The major deformation contribution is commonly related to rocking behaviour for CLT shearwalls with a relatively high height-to-length aspect ratio of CLT panels, whereas sliding is dominant in CLT panels with low aspect ratios [20,21]. In the analytical models proposed by Casagrande et al. [22,23], specific attention was paid to the stabilizing effect of vertical load on the elastic behaviour of single- and multi-panel CLT shearwalls, respectively. Two different states were identified: the former, which occurs when the stabilizing moment due to the vertical load is higher than the rocking moment due to lateral loads; the latter, which takes place when the rocking behaviour is activated because of relative-high lateral loads acting on the shearwall. Only in this latter case, the deformation contribution due to the uplift of the mechanical anchors should be considered in the calculation of the lateral stiffness of the CLT shearwalls. The importance of adequately taking into account the uplift of mechanical anchors in relation to the amplitude of the lateral displacement of CLT shearwalls was studied and it has been demonstrated that CLT structures may be characterized by a significant shift of the lateral stiffness and the natural period from the state where rocking does not occur (no-rocking state) to the condition when the rocking is activated (rocking state) [24]. Despite these researches the influence of the rocking behaviour of CLT shearwalls on the building dynamic properties is still incomplete especially on how the deformation contribution, due to the uplift of mechanical anchors, should be properly considered in modal analyses of CLT buildings in relation to the stabilizing effect of vertical loads. The objective of this thesis study is to establish a relationship between the fundamental period and the lateral drift in the global dynamic response of CLT buildings. The influence of rocking behaviour activation on building fundamental period was investigated by using experimental testing and numerical analyses. Modal tests were performed on a full-scale mock-up characterized by two single-storey CLT shearwalls to determine the lateral stiffness and natural period under different amplitudes of dynamic lateral response. The results from the experimental campaign were adopted to validate a FE numerical model used to perform parametric analyses on representative CLT building configurations. The shift of the fundamental period of a multi-storey CLT shear-wall from the no-rocking to the rocking state was investigated via elastic non-linear dynamic incremental analyses. Useful indications were found on the fundamental period values which should be adopted as reference values in linear-elastic RSM analyses of CLT buildings should be included. The effects of vertical load, stiffness of mechanical uplift and geometrical dimensions of CLT shearwalls are analyzed and discussed.

Scope of the **Part II** of the thesis is the development of an innovative earthquake-resistant system based on the use of vertical steel ties allowing for the realization of high-rise CLT buildings in high-seismicity areas. Previous studies available in the literature proved that the main issues of multi-storey CLT buildings subjected to seismic action are the limited lateral stiffness and the high tensile forces concentrated in the base hold-down connections, which represent the critical components of the building. Indeed, these nailed brackets were originally developed for the use in light-frame systems and were not originally conceived to be particularly strong and ductile [25]. The adoption of traditional hold-down connections in CLT structures has required to increase thickness, steel grade, number of nails and to add new stiffeners to reduce local deformations. Nevertheless, it has been demonstrated that stiffness and load-bearing capacity of such connections may not be sufficient to realize mid-rise CLT buildings in high-seismicity areas, due to the very high tensile forces at hold-downs and the excessive lateral flexibility of the structure [26,27]. The low permanent loads of timber buildings are the main cause of such high rocking contribution in the global lateral displacements of the structure. This contribution takes on greater importance with increasing seismic action, height and slenderness of the shear walls. Finally, the increasing architectural needs for internal free spaces are leading to optimize the number of shear walls in the building, increasing further the strength demand to connections. Various strategies have therefore been developed for tall and slender CLT buildings in high-seismicity areas [28,29]: for example hybrid timber-concrete [30] or timber-steel systems [31,32]; use of post-tensioning bars [33,34,35,36,37]; use of rocking coupled shear walls with vertical joints [38,39] and use of connections with high ductility and dissipative capacity [40,41,42,43,44,45,46]. All these strategies require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations. To overcome this limit, an original connection system based on the use of vertical steel ties as alternative or in addition to traditional earthquake-resistant systems (nailed plates or screwed connections) is proposed. According to this technology, high-strength vertical CLT cantilevers can be realized to brace the structure, optimizing the number of connections in the building, without the use of prestress, special connections or hybrid structures. The proposed system is therefore conceived to withstand high seismic tensile forces that arise in multi-story CLT buildings braced with a limited number of shear walls, limiting rocking, increasing the lateral strength and stiffness of the structure and therefore reducing inter-story drifts and damages to structural and non-structural components. The optimization of the number of connections and of shear walls in the building allows internal free spaces, that are required in modern architecture, and facilitates the control and possible replacement of connections after a seismic event.

In the last part of the thesis technological details for the original connection system based on the use of vertical steel ties have been studied with the aim of satisfy structural, durability and energy efficiency issues. The latter two are particularly delicate in the connection node between the CLT panel and the reinforced concrete foundation. The main problems correlated to this node were described and the technological solutions currently available in literature analyzed. A new solution for the connection of timber buildings with CLT shear walls to the reinforced concrete foundation is developed and described. Specifically, the proposed technological solution, consists of a linear concrete beam reinforced with steel elements that allows to keep the base of the panel raised with respect to the plane of the foundations, to guarantee the continuity of the insulation layer on the external side of the panel and to resist to the high tensile and shear forces typical of tall CLT buildings. Finally, additional technological details allowing for the use of vertical steel ties in high CLT buildings have been proposed.

Organization of the thesis

The thesis is organized in two independent parts, each subdivided in different chapters with the structure listed below:

<u>Part I</u>

- ***Chapter I.1*** A state-of-the-art about multi-story timber buildings is given, particular attention was paid on CLT structures that represents the central topic of the thesis. Different typologies typically used for the construction of timber buildings were presented, highlighting the fundamental role that connection elements play in the characterization of seismic behavior of CLT structures.
- ***Chapter I.2*** Two different types of linear analyses used to predict the behavior of CLT multistorey buildings were presented. *Component Model* (CM) approach and simplified *Phenomenological Model* (PM) approach, were described, highlighting their characteristics, advantages and issues. A numerical modelling approach, that can be used by practitioners for the prediction of the seismic response of multi-story CLT buildings, was proposed. This approach is based on a simplified phenomenological model where the behaviour of the CLT system is reproduced by means of an equivalent elastic modulus to be assigned to the CLT wall in order to account for both the connections and panel deformability. PM aims to reproduce correctly the global response of the structural system in terms of principal elastic period, internal forces in the connection elements and inter-storey drifts. This approach is easy to implement in commercial software and involves a low computation afford, for this reason it could be used by engineers in the design phase. To provide reliable estimation of the equivalent stiffness for the PM, different case study configurations were analyzed varying the significant parameters controlling the building response such as, number of storeys, connection pattern, geometry and wall slenderness.
- ***Chapter I.3*** The calculation of vibration periods of CLT structure was investigated. The determination of this parameter plays a key-factor in seismic design of structural systems when earthquake motion is represented by an elastic ground acceleration response spectrum. The calculation of vibration periods is in fact an essential step for the calculation of seismic actions on the basis of linear-elastic behaviour of structures. A relationship between the rocking behavior of CLT shear walls and the natural period of CLT structures was found. Modal tests on a full-scale timber mock-up and non-linear elastic incremental dynamic analyses were carried out to determine the natural period as function of the lateral response amplitude. The objective is to establish a relationship between the fundamental period of CLT buildings and the lateral drift of their global dynamic response. The influence of the activation of rocking behaviour on the fundamental period was investigated by using experimental modal testing and Finite Element (FE) numerical analyses. An analytical expression to predict the maximum range of natural period which a CLT shearwall may exhibit under different levels of the dynamic lateral response was reported as function of the vertical load and the equivalent rocking slenderness. This formula can be adopted to validate the natural period adopted in the design of CLT building using linear-elastic Response Spectrum Modal analyses.

<u>Part II</u>

- ***Chapter II.1*** A state-of-the-art of high CLT buildings erected in seismic areas was given highlighting the main issues of this construction typology correlated to the limited lateral stiffness and the high tensile forces concentrated in hold-down connections. The different strategies available in literature and developed to overcome these limits were presented. These strategies require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations.

- ***Chapter II.2*** An innovative earthquake-resistant system based on the use of vertical steel ties able to allow the realization of high CLT buildings in high-seismicity areas was proposed. The system is conceived to withstand high seismic tensile forces that arise in multi-storey CLT buildings braced with a limited number of shear walls, limiting rocking, increasing the lateral strength and stiffness of the structure and reducing inter-storey drifts and damages to structural and non-structural components. The optimization of the number of connections and of shear walls in the building allows internal free spaces, that are required in modern architecture, and facilitates the control and possible replacement of connections after a seismic event.
- ***Chapter II.3*** Technological details that allows for the use of the developed connection system based on vertical steel ties were studied with the aim of satisfy structural, durability and energy efficiency issues. A new solution for the connection of timber buildings with CLT shear walls to the reinforced concrete foundation and to connect CLT panel to tie down and panel to floor was proposed. Particular attention has been paid to the wall-to-concrete foundation node that represents one of the most critical issues in timber buildings.

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I

CLT multi-storey buildings: modelling strategies and calculation of principal period

I.1 State-of-the-art of multi-storey timber buildings

Abstract

In this section, a state-of-the-art about multi-storey timber buildings is given, describing different typologies typically used for the construction of timber buildings, particular attention is paid on connections elements. It has been demonstrated that joints, which are assigned the function of connecting the panels together and dissipating energy during a seismic event, play a key role in the structural behaviour of multi-storey buildings. The technology that has found most use in the construction sector in recent years is represented by CLT structures. Being CLT structures a relatively new technology, many aspects on its structural behavior still need to be investigated. An overview of this technology and of numerical approaches for the prediction of seismic behavior of CLT structures is provided, highlighting their characteristics, advantages and problems.

I.1.1 Overview on timber buildings

I.1.1.1 Modern timber structures

In the past, the use of wood as a structural material was relegated to the construction of small buildings, roofs and stiffening frames to be coupled to masonry structures or for replacement of degraded ancient wood elements. With the introduction of engineered wood materials, the use of wood in construction sector has completely changed. In particular with the development of glued laminated timber beams (glulam) and innovative timber panels (CLT), long-span roofs (swimming pools, hangars, sports halls…) and multi-storey buildings can be realized with wood. Wooden materials are nowadays considered as construction material particularly suitable for creation of seismic-resistant structures thanks to their low density and their mechanical proprieties. The strength on mass ratio of wood is similar to the steel one, and about six times the one of concrete. The negative aspect, however, lies in the intrinsic fragility of the material that could compromise its response to seismic actions. To overcome this limit, it is necessary to design properly connection elements in order to ensure the ductile behavior of timber building. A proper example of the excellent resistance of wooden structures to seismic events is represented by the Chinese pagodas, which have resisted seismic events for thousands of years (see example of Figure I.1.1).

Figure I.1-1 – Example of Chinese pagodas (*image credits*: The architecture designs website [47])

Other more recent examples of tall buildings and large timber structures can be found in Europe and in North America. In Figure I.1-2 is reported 8-storey office buildings built in Vancouver in 1905 and it is still in use today.

Figure I.1-2 – 8-storey office building built in Vancouver in 1905 using the brick-and-beam technique (*image credits*: Karen Magill website [48]

Despite the examples just mentioned, in the second half of the 20th it was observed a decline in the construction of high wooden buildings, mainly due to the restrictions imposed by many States that imposed limitations to the maximum height of timber buildings. The main reasons are linked to a lack of knowledge of seismic response and fire safety of tall wood buildings. However, the development of engineered wood-based products and connections elements, in addition to a substantial increase in knowledge of tall wood structure have led several States, i.e. Italy [49], to remove height limits on timber buildings that allowed a rapid growth of multi-storey timber buildings. Several examples of medium and high-rise timber structures can be found nowadays in the world. For example, the Forte building [50] in Australia and the Dalston Lane in London [51], both made with a bearing structure of CLT, see Figure I.1-3.

Figure I.1-3 – Examples of 10 storeys CLT buildings: a) Forte building in Australia b) Dalston Lane in London (*image credits:* The Possible website [52] and Waugh Thistleton Architects website [53])

The most important example of high-rise timber building is represented by Mjøstårnet, the tallest timber building in the world with its 18 storeys and a height of 81 m. Its structure is composed of a heavy glulam timber post-and-beam frame [54], as reported in Figure I.1-4.

Figure I.1-4 – Mjøstårnet, the highest timber building in the world (*image credits:* Moelven website [54])

Examples reported shows as multi-storeys building are gaining interest. It is possible distinguish different types of multi-storeys structure as a function of the technology used to transmit to foundation vertical and horizontal loads. The most common typologies are:

- Heavy Timber Frame construction
- Light Timber Frame wood construction
- Hybrid wood-concrete construction
- Cross Laminate Timber construction

A description of such structural system is reported in the following, analyzing the main characteristics and usages of each constructive typology.

I.1.1.2 Multi-storey building typologies

Different typologies of multi-storey timber buildings are described in the following, divided according to the technology used for withstanding to horizontal and vertical loads.

I.1.1.2.1 <u>Heavy timber frame construction</u>

Heavy timber frames are erected following the same structural concept as concrete and steel structures. They use a system of linear elements for beams and pillars, which are usually made of glulam or LVL, that are connected by metal elements, in analogy to steel structures. Heavy timber buildings were initially designed and used for industrial and storage purposes that requires large span length, and this use is very frequent even today: supermarkets, libraries, auditorium, gymnasium are the proof (see Figure I.1-5).

Figure I.1-5 – Examples of modern heavy timber frame constructions: retail space of Tanguay Ameublements in Trois-Rivieres (*image credits:* Nordic Structures website [55])

It is worth to underline how, despite many of heavy timber buildings are characterized by large area and single storey structures, nowadays, great potentialities of this technology allow to realize tall buildings, as reported in Figure I.1-6.

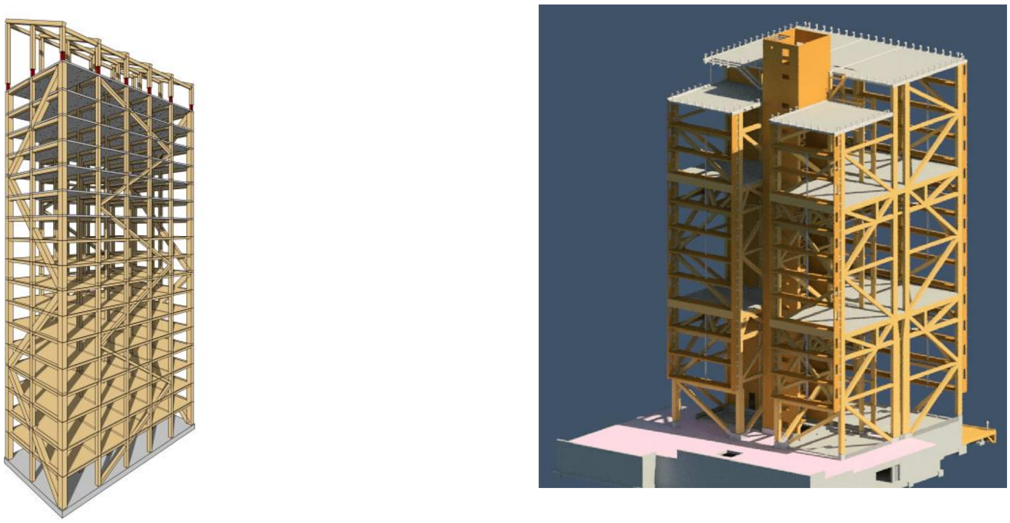


Figure I.1-6 – Post-and-beam frame of Mjøstårnet [54] and Treet [56]

It is important to underline how joints used to connect columns to beams play a key role in the definition of structural behaviour of multi-storey heavy frame buildings. These connections are generally realized using steel plates fixed by means of nails or dowels as reported in Figure I.1-7

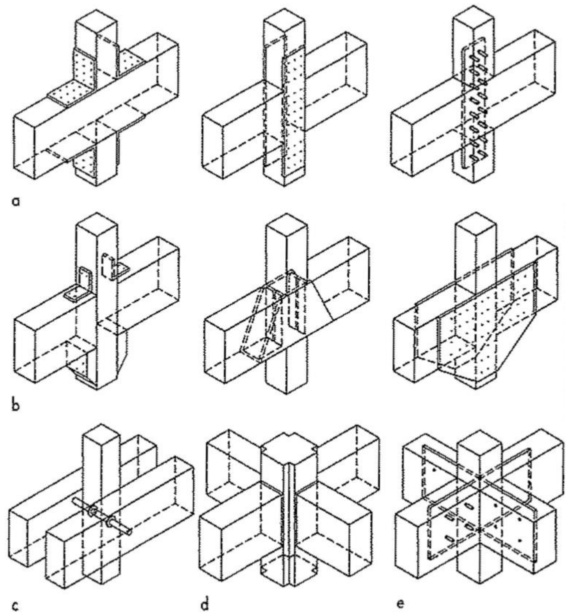


Figure I.1-7 – Examples of beam-column connections (*image credits:* Promolegno website [57])

In recent years innovative systems have been proposed for the connection between column and beam. Among the most interesting solutions one is based on post-tension techniques: in this case, the frames are generally made with full height columns to which the post-tensioned beams are connected by steel cables anchored to the column in the nodal area. Alternatively, the frame can also be made with continuous beam and post-tensioned columns interrupted at each level. In addition, metal devices can be used to ensure the dissipation of seismic energy. Post-tensioned techniques allow low-damage level of structural components and limited residual deformations. A schematic representation of post-tension technique is reported in Figure I.1-8 [58].

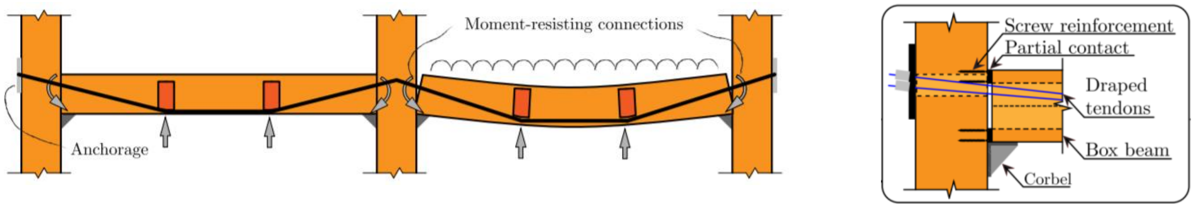


Figure I.1-8 – Schematic representation of post-tensioned timber frame (left) and example of column-beam joint detail (right) [58]

Despite example of heavy timber tall buildings reported previously, this technology system is quite exclusively used to realize single-storey large structures.

I.1.1.2.2 Light timber frame wood construction

Two different type of light timber frame buildings can be found: *platform frame* and *balloon frame. In balloon frame system* vertical elements are continuous for more than one story while in platform frame system floors are built one at a time as reported in Figure I.1-9

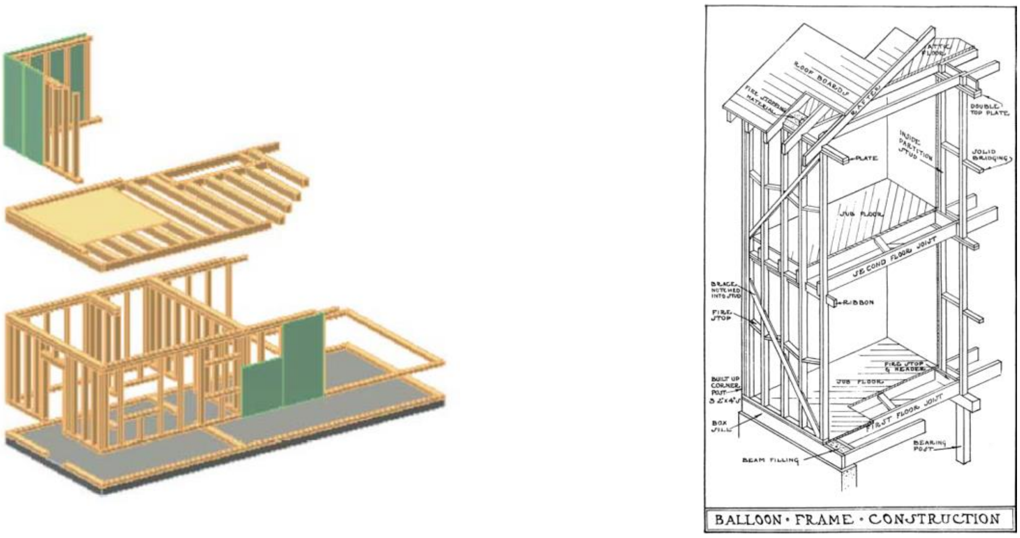


Figure I.1-9 – Platform frame (left) and balloon frame (right) (*image credits:* Docplayer website [59])

Light timber frame construction system is widely used for the construction of buildings for residential use, the construction technique originated in North America at the beginning of the 1800. The wall consists of a main wooden frame stiffened with panels, connected mechanically on one or both faces by means of nails, screws or metal clips. The minimum resistant unit, able to withstand both vertical and horizontal loads, is consisting of at least three columns, by a top and bottom crosspiece and a bracing panel. Typically, columns and beams are made with solid wood, lamellar or LVL, while the bracing panels can be in Oriented Strand Boards (OSB), chipboard, plywood or with other wood-based materials. Any openings for doors and windows must be surrounded by horizontal beams e vertical posts. Floors are made using framed panels with joists arranged at the same center distance of the uprights of the vertical walls and stiffened in its plane through wood-based panels on one or both sides of the frame. The panel-panel and panel-foundation connection is made by specific devices that prevent rocking and sliding respectively hold-down and angular brackets. These devices must be anchored directly on the wooden columns and not on the bracing panel, to avoid brittle failures. In Figure I.1-10 it is reported a scheme of the light timber frame system [60].

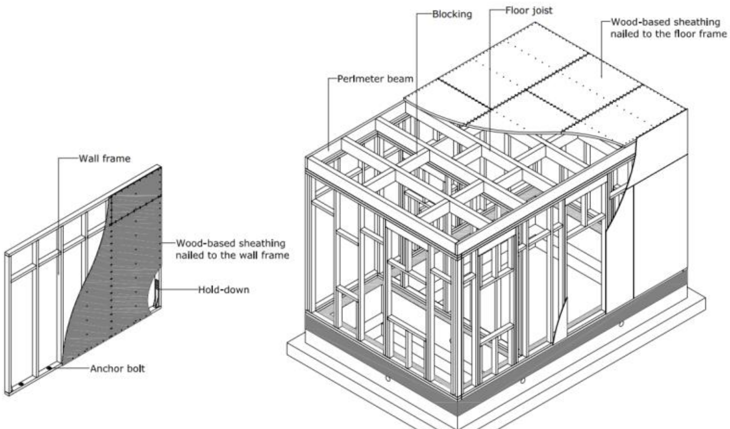


Figure I.1-10 – Light timber frame system [60]

Shaking table test conducted on full-scale light timber frame structures have certificated good seismic performance of this technology. Two of the most important are represented by SERIES project [61] and NEESWood [62]. Both these experimental campaigns demonstrated a high dissipative behavior mainly governed by sheathing-to-framing joints that allow timber frame structure to withstand to strong seismic events [63].

Despite the good earthquake-resistant characteristics, structural lightness of light timber frame technology makes this technology not suitable to realize high-rise building. For this reason and for low massiveness correlated to this system, light timber frame buildings have not spread in seismic areas with hot climates zones (i.e. Europe) where massive wall systems (i.e. CLT) or hybrid systems (i.e. wood-concrete structures) are preferred.

I.1.1.2.3 Hybrid construction

Wooden constructive systems can be coupled with other construction material such as reinforced concrete [64] or steel [31,32]. In this structural solution vertical load bearing system is generally constituted by a heavy timber frame, while concrete shearwalls or steel frames are used to resist to lateral loads. Shearwalls are positioned in order to maximize resistance to torsion under lateral loads. In Figure I.1-11 it is reported an example of wood-concrete hybrid building.

Figure I.1-11 – Wood-concrete hybrid multi-storey structure (*image credits:* Tree Source website [65])

Sometimes vertical load bearing system could be constituted by light timber frames or CLT walls, as reports the example of Figure I.1-12.

Figure I.1-12 – Hybrid CLT-concrete building [66]

It is important to highlight how in hybrid structure the connection between wood and the earthquake-resistant elements (concrete or steel) must properly be designed. Typically, this connection is made with steel plates that transfer seismic action form the floor diaphragm to the bracing system.

I.1.1.2.4 <u>CLT construction</u>

Cross Laminated Timber (CLT) structures are nowadays increasingly used worldwide and mostly in Europe where the system was developed in the early 1990s. In this technology, CLT wall panels have to withstand both to vertical and lateral loads. CLT panels consists in two-dimensional element constituted of several layers of boards, stacked crosswise and glued together on their wide faces and, sometimes, on the narrow faces. Boards have a variable thickness between 15 and 40-45 mm and a width between 80 and 240 mm. Number of layers varies, generally from three to seven layers. The production technique of the CLT panel follows the same steps of glulam, in fact all the boards used are preventively classified and assigned to a certain resistance class. Panel sizes vary by producers; typical widths are 1.2 m, 2.25 m, 2.45 m, 2.75 m and 2.95 m while length can be up to 16 m, as reported in Figure I.1-13.

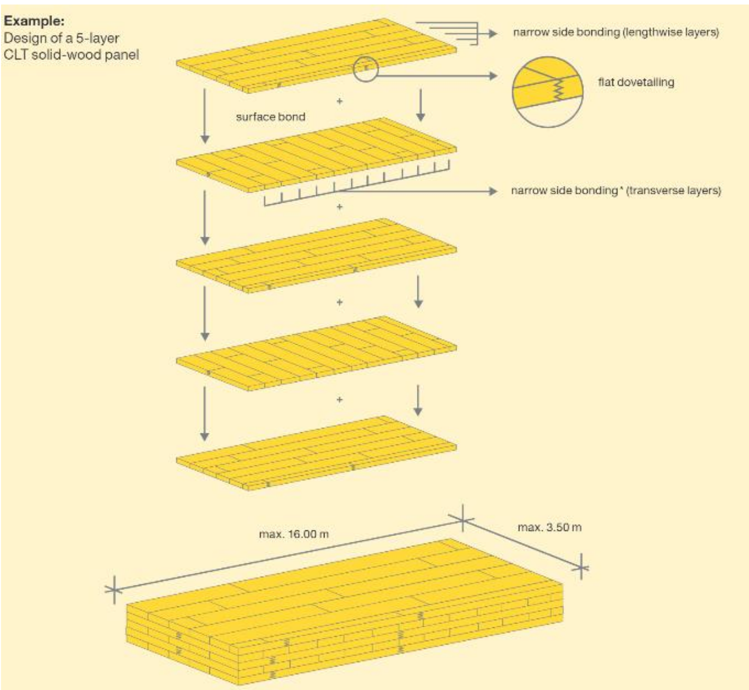
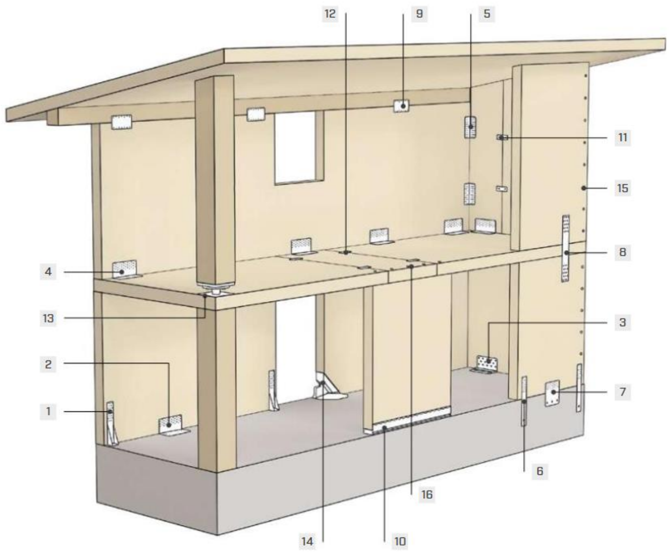


Figure I.1-13-Production technique and maximum dimensions of CLT panel (*image credits:* Storaenso website [67])

In recent years, CLT massive wood construction starts to be used in place of concrete, masonry and steel in the realization of commercial, industrial and residential buildings. The increasing use of CLT panels is due to the many advantages offered by this technique (dimensional stability, high in-plane and out-of-plane strength and stiffness properties, high level of prefabrication, reduction of CO_2 emission…). Several CLT multi-storey structures built around the world represents the evidence of the advantages that this product can offer. In addition of the examples reported in Figure I.I.1-3, other of the most important are reported in Figure I.1-14.

Figure I.1-14-CLT multi-storey buildings: Birdport house in London [57], CCG Yoker in Glasgow [68], Murray Groove in London [69]

In order to reach higher width and heights of buildings, it is necessary to connect CLT panels together and to foundation, as reported in Figure I.1-15.



1,6,8) Hold-down connection
2,3,4,7) Angle bracket connection
5,11,15) Wall to wall connection
9) Roof to wall connection
10) Continuous wall to foundation connection
12) Floor to floor connection
13) Column to floor connection
14) Screwed X-RAD connection

Figure I.1-15-Connection elements of a CLT structure (*image credits:* Rothoblaas website [70])

Traditional connectors, such as screws and nailed steel plates (hold-down and angular brackets), or innovative connection can be used. It has been demonstrated that connections play a key-role in the definition of structural behavior of CLT buildings. For this reason, many research activities have investigated their behavior and several scientific publications that studied the response to horizontal actions of CLT-connections wall systems can be found in literature [20,21]. Aforementioned researches, and the full-scale shaking table tests [7,71] have demonstrated the good seismic performance of CLT structures, that usually have a higher in-plane stiffness and a greater load-carrying capacity respect to light-frame buildings.
Despite these studies, it is important to highlight that seismic characterization of CLT multi-storey buildings is still a topic of study for many researchers [72,73], with the aim of filling lack present in codes, in design procedures and in numerical modelling strategies. An extensive discussion about the seismic behaviour of CLT multi-storey building will be reported in this Thesis, paying attention to numerical modelling strategies, evaluation of principal elastic period and design method for tall CLT buildings.

I.1.1.3 Connections elements in timber structures

Connections elements play a crucial role in the definition of seismic behavior of timber structures and influence the global stiffness and the dissipative capacity of the building. It is worth noting that in timber building the dissipative capacity is exclusively due to the fasteners because the wooden elements remain in elastic field during a seismic event. Therefore, the plastic behaviour of timber structures is primarily governed by the mechanical properties of mechanical connections used to connect the structural elements to each other and to foundation.
Connections are generally divided in two categories: carpentry joints and mechanical joints.

Carpentry joints are found mostly in ancient timber structures and are obtained through shaping of contact surfaces of timber structural elements. Nowadays these processes are performed with numerical control machines and loads are transferred through the connected elements by means of compression areas. Any kind of energy dissipation cam be offered by this typology of connections and therefore they cannot be used for buildings in seismic areas. Several studies have been conducted on carpentry joints in order to define their role in structural behavior of ancient timber structures [74,75,76]. Some examples of carpentry joints are reported in Figure I.1-16.

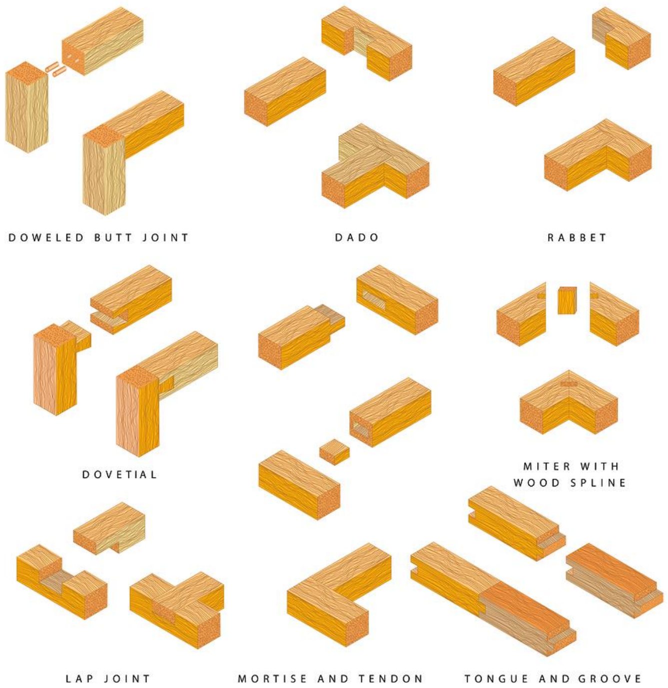


Figure I.1-16-Examples of carpentry joints [74]

Mechanical joints are made with steel devices able to connect two or more pieces of wood together (wood-wood joints) or wooden elements with metal plates (steel-wood joints). These connections can exhibit significant ductile behavior develops thanks to the interaction between yielding of the metal elements and bearing stress of wood. Dowel-type connectors (bolts, dowels, nails, screws…) or surface connectors (toothed, split rings) are generally employed to transfer forces through timber elements. In order to activate the ductile mechanisms, it is important to design connection respecting the distance given by standards between connectors and with respect to the edges. In this way it is possible to avoid dangerous brittle failures, such as splitting or group effects. In Figure I.1-17 are reported some examples of steel devices most used in mechanical joints.

Figure I.1-17-Mechanical joints for timber structure: bolts (left), screws (center), split ring (right) (*image credits*: Fastener and fixing website [77] and Quenneville et al. [78])

These types of connections, in particular dowel-type connections, are very popular in multi-storey timber buildings realized in seismic-prone areas. Structural proprieties (strength, stiffness, ductility…) of mechanical

joints are influenced from different parameters: such as type and diameter of the connector, the density of the wood and the angle of inclination between force and grain. Generally, joints with small diameter connectors have better dissipative capacity than those with large diameter, in fact connectors such as nails and screws allow to achieve a more accentuated ductile behavior compared to bolts or dowels. *Nailed joints, screwed connections* and *bolted and doweled connections* are described in the following.
As regard *nailed joints*, they are frequently used in timber structures because they are easy to use and cheap. Their limit is represented by low resistance and stiffness compared to other mechanical fasteners. In the last years there has been an increase in the use of annual-ringed nails that can guarantee a higher withdrawal resistance and strength respect to smooth nails [79]. Different types of nails are reported in Figure I-1-18.

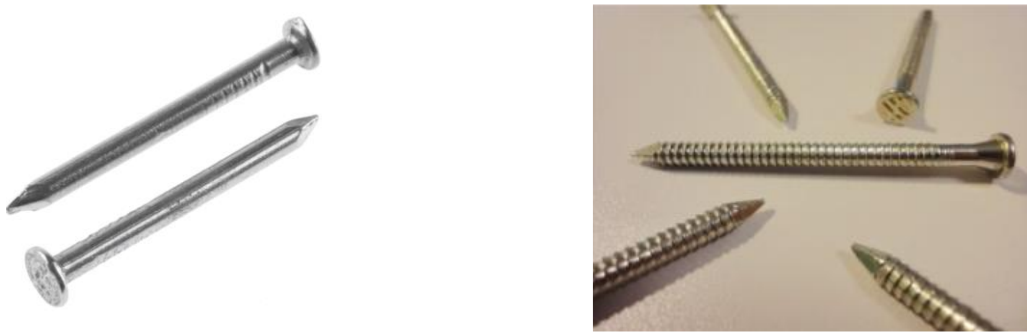


Figure I.1-18- Nailed mechanical connections: Round smooth shank nail (left) and ringed shank nail (right) [79]

Screwed connections are generally used in cases where both withdrawal and shear strength must be guaranteed. Several types of self-tapping screws are nowadays available in commerce with variable diameter (from 4 to 20 mm), length and threaded/smooth ratios. Usually, the diameter of the threaded part is 70% the diameter of the smooth part. Recently, screws arranged inclined with respect to wood grain have been often used: this configuration guarantees high levels of strength and stiffness, with a consequent reduction in ductility [80,81]. Screwed connections can be used to directly connect two or more wooden elements or to connect steel plates to timber members, as reported in Figure I.1-19.

Figure I.1-19 – Screwed joints: typologies of self-tapping screws (left); beam to beam connection (center), steel plate to timber connection (right) (*image credits*: Blaß & Sandhaas [82] and Rothoblaas website [70])

Bolted and dowelled connections are employed when high shear resistances are required. These types of joints require predrilled holes in timber elements, that are usually made by numerical control machines. If steel plates are used, it should be predrilled with a diameter larger (usually 1 mm of tolerance is adopted) than the shank one. Configurations with steel plates (one or more) interposed guarantees high resistance and stiffness and can be used to build high timber buildings. Diameter of bolts and dowels varies generally between 6 and 30 mm. In order to avoid brittle failures of wooden elements [83], some codes, such as Italian code [49] limit the maximum diameter of these type of connectors.

Figure I.1-20 – Bolted and dowelled joints: example of dowel and bolt (left); dowelled connection (center), bolted connection (right) (*image credits*: Rothoblaas website [70] and Wood works website [84])

I.1.1.4 Mechanical connections for CLT buildings

Connection elements play an essential role in the definition of seismic behavior of CLT structures. It is worth nothing that the structural efficiency of CLT multi-storey buildings is closely related to efficiency of the fastening systems used to connect CLT panels. Many tasks are assigned to the connections, including that of providing strength, stiffness, stability and ductility to the structure. Elastic and rigid behaviour of CLT panels means that the connections influence significantly the ductility [85] and the global stiffness of the structure [86] under lateral loads. The knowledge of the specific behaviour of the fasteners represents therefore a fundamental requisite to understand and to predict the response of CLT buildings under seismic actions. On the contrary, it must be remarked that for low-amplitude cyclic vibrations (i.e. low wind), the global behavior of the structure is non influenced by connections, since they require higher displacements to activate their resistance and forces may be transferred through friction and normal edge forces [87].
Nowadays, different types of joint details can be used to establish roof/wall, wall/wall, wall/floor, and inter-story connections in CLT structures. In the following, an overview of the most used connections for CLT assemblies will be given as a function of their location into CLT buildings (see Figure I-1-21) and paying attention on their seismic behaviour.

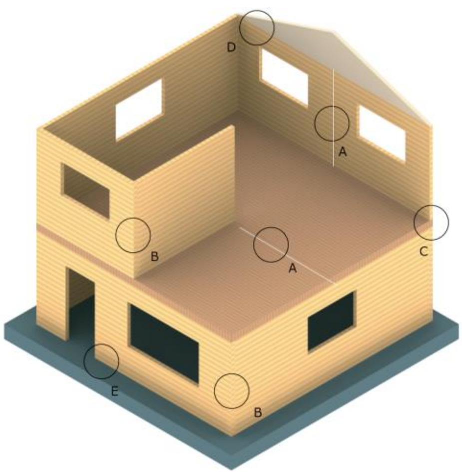


Figure I.1-21 – Location of principal connections in multi-storey CLT buildings [88]

I.1.1.4.1.1 Panel-to-panel connections (Detail A, Detail B in Figure I.1-21)

Panel-to-panel connections is the most used form of connection that is typically used to create larger structural elements (floors and walls) on site panels whose dimensions are limited by production necessities and transport limitations. In Figure I.1-22 are reported the most common panel-to-panel connections.

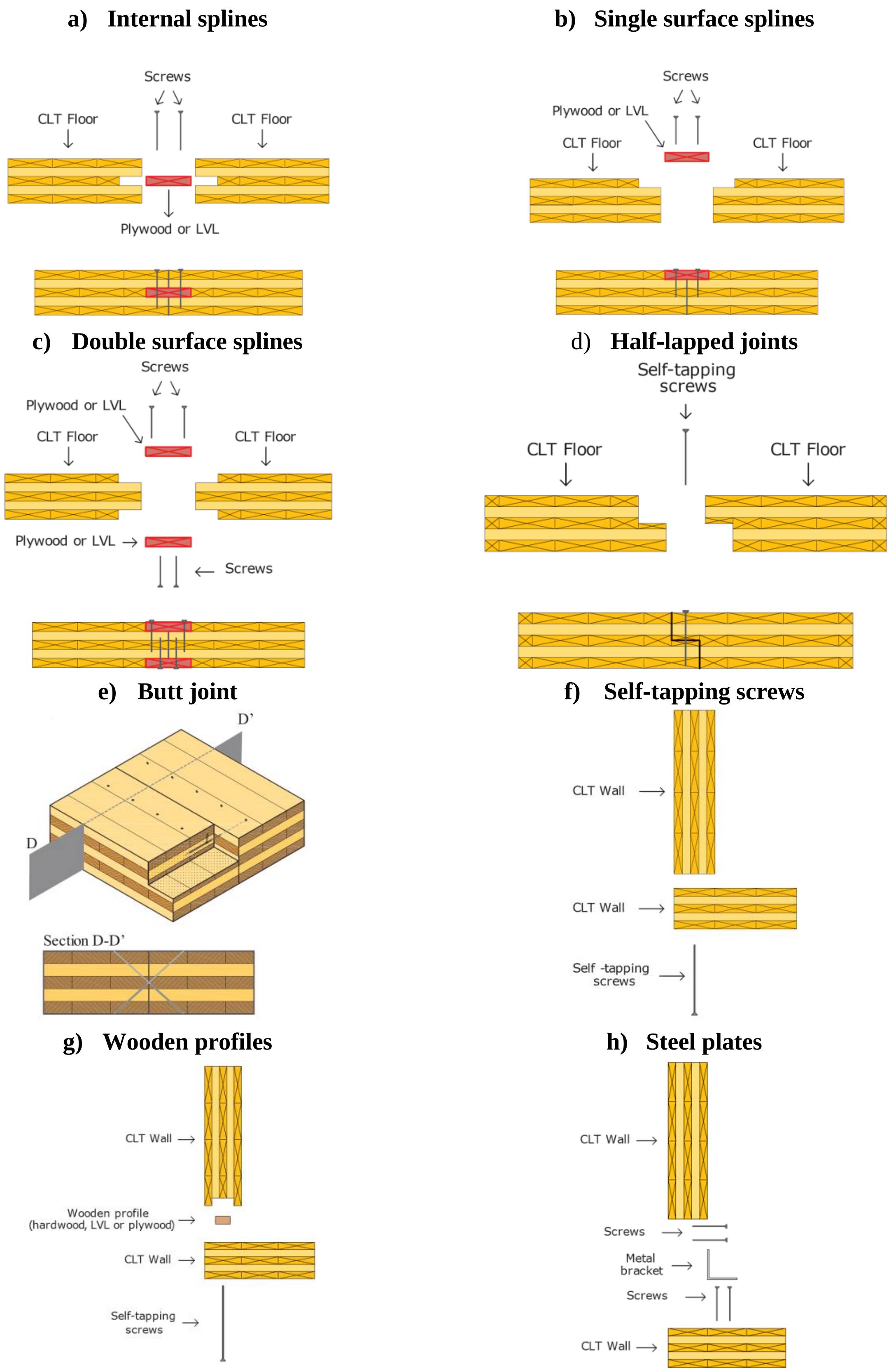


Figure I.1-22 – Panel-to-panel connections [88]

Configurations a), b), c), d), e) of Figure I.1-22 should be used to connect both wall to wall and floor to floor panels while solutions f), g) and h) are generally used to connect CLT walls arranged at 90°. Several experimental tests have been conducted on panel-to-panel connections with the aim of characterize their mechanical proprieties [89,90].

I.1.1.4.1.2 Wall-to-floor and wall-to-roof connection (Detail C, Detail D in Figure I.1-21)

This type of connection allows the transmission of forces from the upper to the lower floor in platform constructions. In Figure I.1-23 are reported the most common wall-to-floor connections, that can be replicated in wall-to-roof condition.

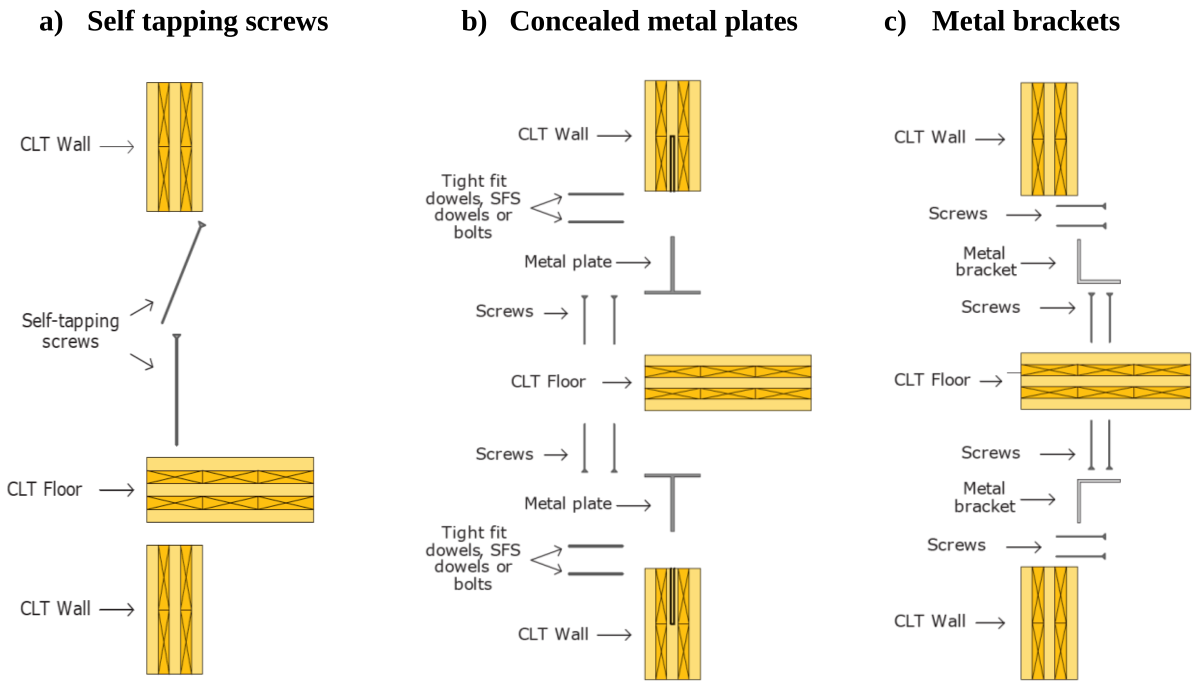


Figure I.1-23 – Wall-to-floor connections [88].

In *balloon constructions*, solutions reported in Figure I.1-24 are generally used.

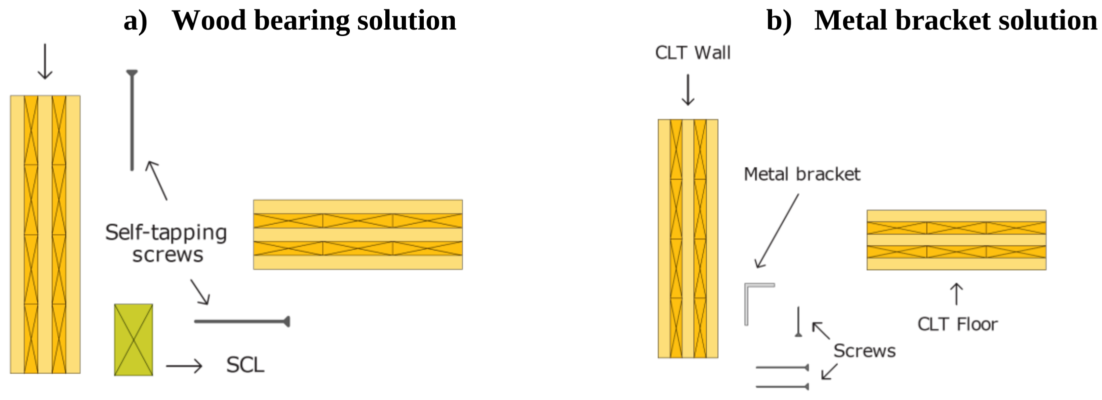


Figure I.1-24 – Wall-to-floor connections for balloon CLT constructions [88]

Mechanical proprieties of wall-to-floor and wall-to-roof connection have been studied by many authors that demonstrates the strongly influence of connections on the global seismic behaviour of CLT multi-storey buildings [91,92,93].

I.1.1.4.1.3 Wall-to-foundation connection (Detail E in Figure I.1-21)

The connection between CLT wall panels to foundations is generally made with external steel plates and brackets. Several fastening systems are available on the market. The main solutions are reported in Figure I.1-25.

a) Exterior metal plates **b) Concealed hardware**

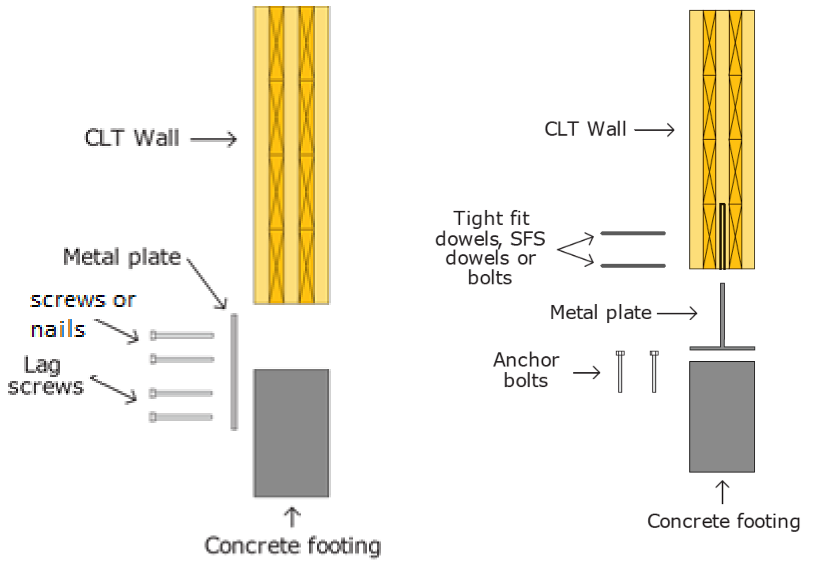


Figure I.1-25 – Wall-to-concrete connections [88]

In wall-to-foundation connection it is important to protect fasteners against corrosive environments and to avoid the direct contact between CLT panels and concrete to guarantee the durability of timber elements. Wooden profiles or metal profiles can be used for this purpose, as reported in Figure I..1-26.

a) Steel profile **b) Wooden profile**

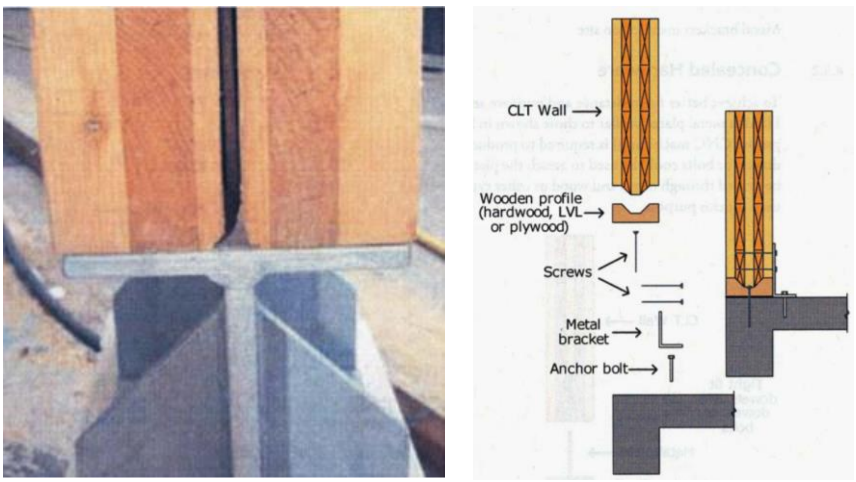


Figure I.1-26 – Wooden and metal profiles for wall-to-concrete connections [88]

I.1.1.4.2 Innovative connection systems for CLT buildings

In the last years, innovative types of connections have been devolved in order to confer to CLT walls higher mechanical proprieties in terms of ductility and energy dissipation, stiffness and strength. Several examples of innovative connection are available in literature, such as the X-RAD [94], the X-bracket [44], the XL-stub [95] and the dissipative connector proposed by Schmidt & Blaß [96] (see Figure I.1.27). All these examples are conceived to dissipate significant amount of energy avoiding brittle failures that can occur in traditional connections (hold-downs and angular brackets).

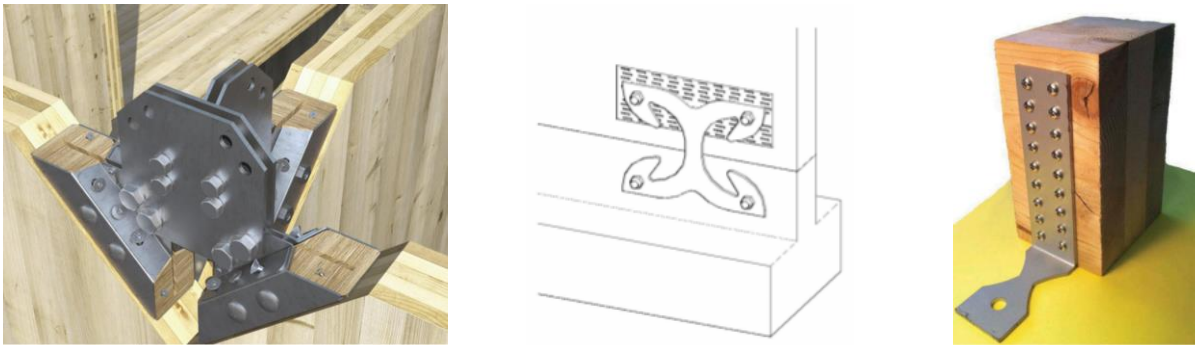


Figure I.1-27 – Examples of dissipative connectors [94,95,95]

Another innovative connection has been studied at the University of Canterbury, in New Zeeland, where the extension of posttensioned techniques to CLT walls has led to the development of new structural systems, referred as Pres-Lam (prestressed laminated timber) [46,97]. Pres-Lam walls consist of a rocking timber element with unbonded post-tensioned tendons running through the length and attached to the foundation, which provides a centring force to the wall, while energy dissipation is supplied by either internal or external mild steel dissipaters, as reported in Figure I.1-28.

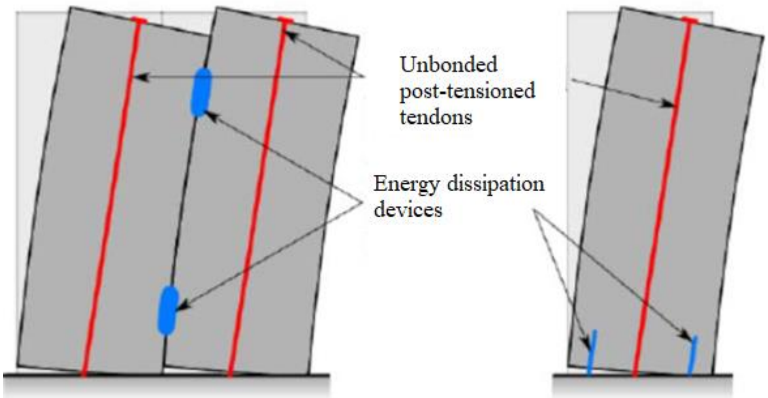


Figure I.1-28 – Posttensioned technique with energy dissipation devices

The experimental results showed excellent performance of post-tensioned timber wall systems, which provide a high level of dissipation while showing negligible residual displacements and negligible damage to the wall element.

I.1.2 Role of connections in CLT buildings

It the lasts years, researchers have investigated, through experimental tests and numerical simulations methods, the response of CLT structures under seismic actions. One of the most important experimental investigation on seismic behaviour of CLT buildings was carried out by CNR-IVALSA, Italy, within the SOFIE Project [6] [7]. Other important experimental campaigns and numerical studies have been conducted at the University of Trento, Italy [98], and at The University of Ljubljana, Slovenia [99], with the aim of characterizing the behaviour of CLT structures. Important tests to determine the seismic resistance of CLT shear walls and small-scale 3-D structures were conducted by FPInnovations in Canada [100]. Several studies have been conducted on connection elements, which are assigned the function of connecting the panels together and to concrete foundation and dissipating energy during a seismic event. It has been demonstrated that connections play a crucial role in the structural behaviour of multi-storey CLT buildings [13]. For this reason, a correct definition of mechanical proprieties and a proper implementation of connections in finite elements software are necessary. Results of experimental tests that investigated the cyclic behavior of connections, such as hold-down and angular brackets, are nowadays available in literature. On the basis of these results it was possible to calibrate theoretical models useful to simulate the behaviour of single connection, CLT walls and CLT structures. An accurate evaluation of the principal mechanical proprieties of connections, such as strength, stiffness, energy dissipation, impairment of strength and ductility, was conducted at CNR-IVALSA by Gavric et al. [101] [102], that performs monotonic and cyclic tests in shear and tension respectively on hold-downs and angle brackets. In Figure I.1-29 and I.1-30 are reported force-displacement curve obtained respectively for hold-down and angle brackets.

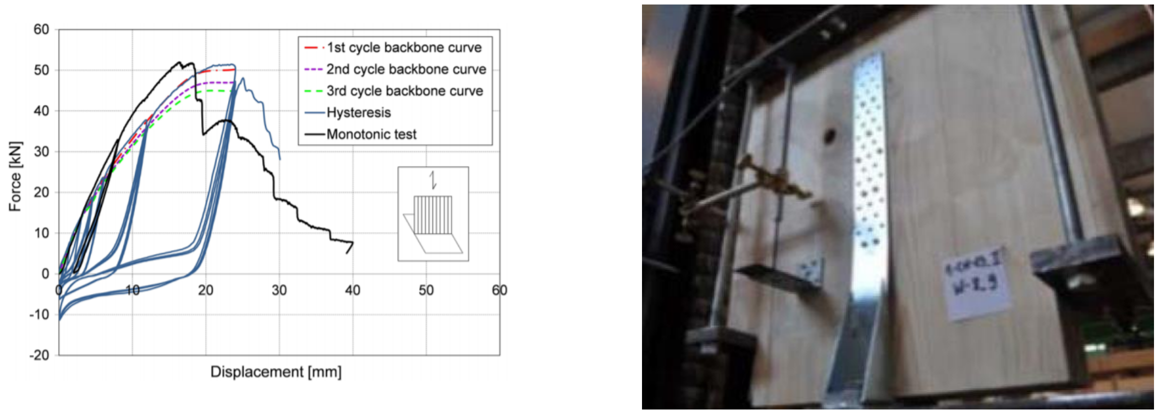


Figure I.1-29 – Force-displacement curve for hold-down load in tension

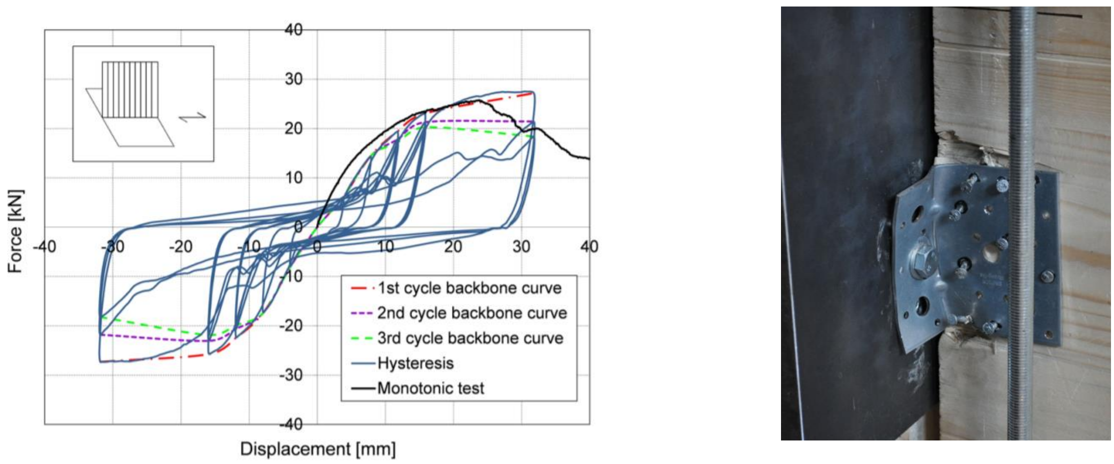
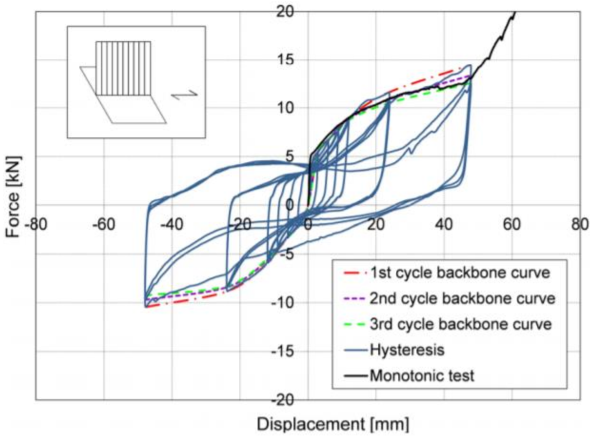


Figure I.1-30 – Force-displacement curve for angle bracket load in shear

The hysteretic cycle of these connections is characterized by significant energy dissipation, pinching and strength and stiffness degradation. Hold-downs and angle brackets were also tested in secondary direction (shear direction for hold-downs and tension direction for angle brackets) in order to evaluate their bi-axial behaviour. Results obtained are reported in Figure I-1-31 and in Figure I.1-32.



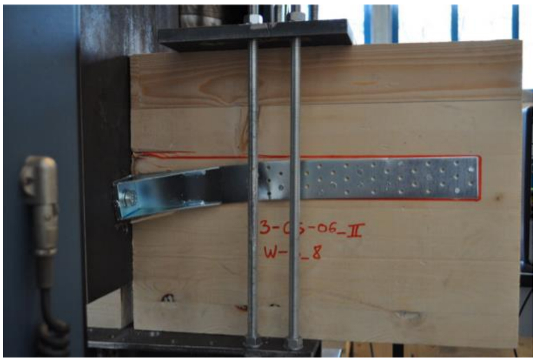
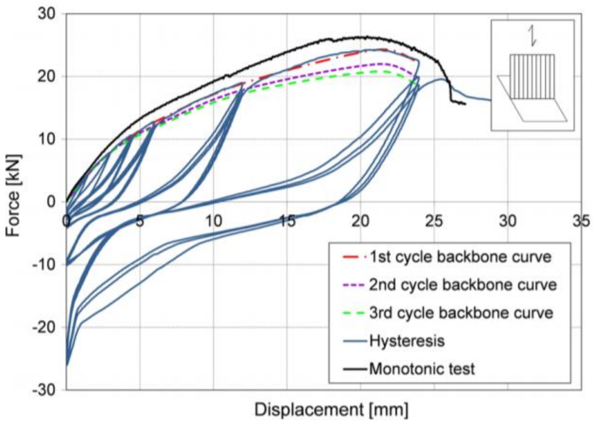


Figure I.1-31 – Force-displacement curve for hold-down load in shear



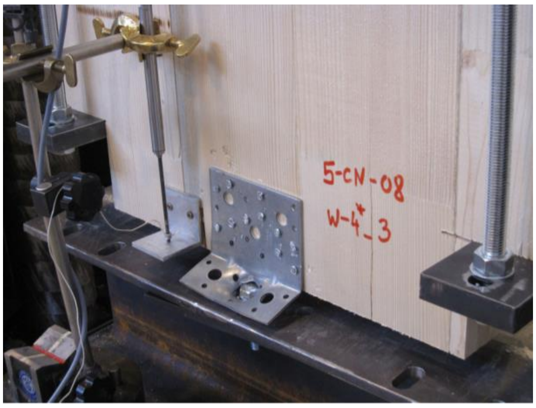


Figure I.1-32 – Force-displacement curve for angle bracket load in tension

As can be seen from Figure I.1.31, hold-downs are not able to reach significant strength and stiffness in shear direction because of the buckling of their metal part. On the other hand, as reported in Figure I.1-32, angle brackets can reach significant strength and stiffness capacity in tension direction too.
Other significant experimental campaigns were conducted by Pozza et al. [103] and Liu et al. [104], where the axial-shear interaction in typical hold-down connections was investigated. Setup adopted by authors are reported in Figure I.1-33.

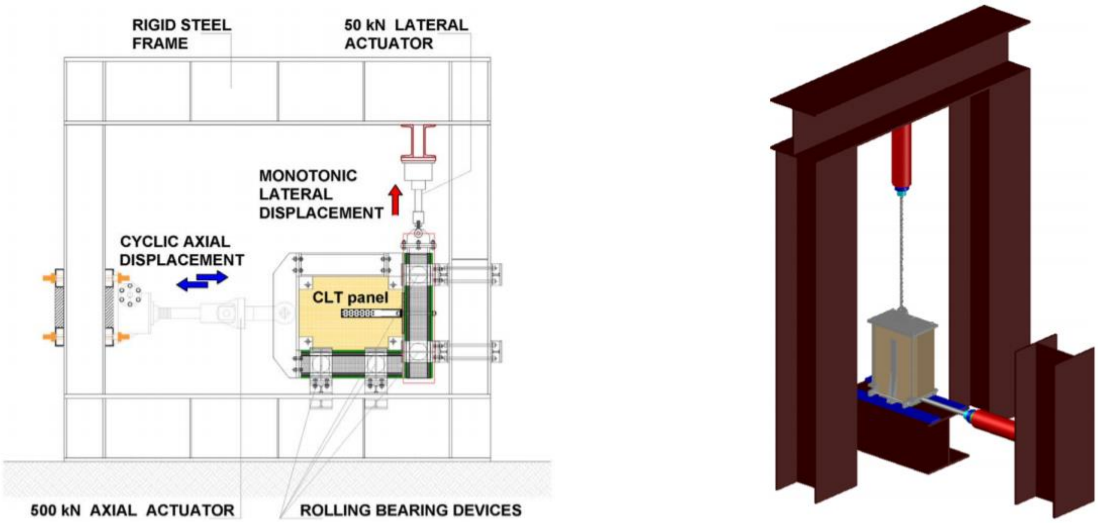


Figure I.1-33 – Setup adopted by Pozza et al. [103] (left) and by Liu et al. [104] (right)

Figure I.1-34 reports an example of failure of hold-down connection subjected at the same time to axial and shear loads.

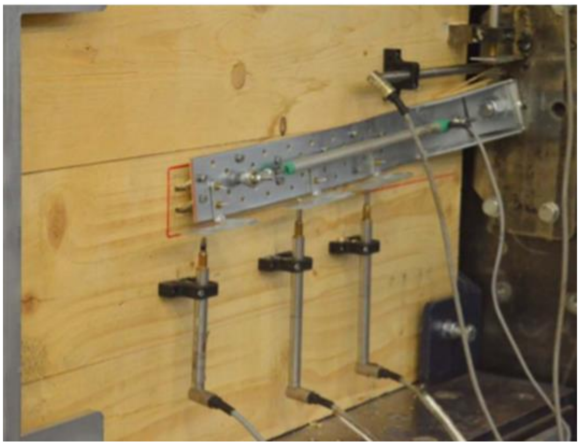


Figure I.1-34 – Hold-down failure [103].

The failure mode observed at the end of experimental tests principally involved the nails used to connect the hold-down steel plate to the CLT panel. For specimens subjected to a large value of imposed lateral displacement, the hold-down deformation exhibited a rigid rotation of the base ribbed portion of the steel connection. Axial force vs axial displacement curves, for different levels of lateral displacements imposed, have been obtained by Pozza et al. [103] and are reported in Figure I.1-35.

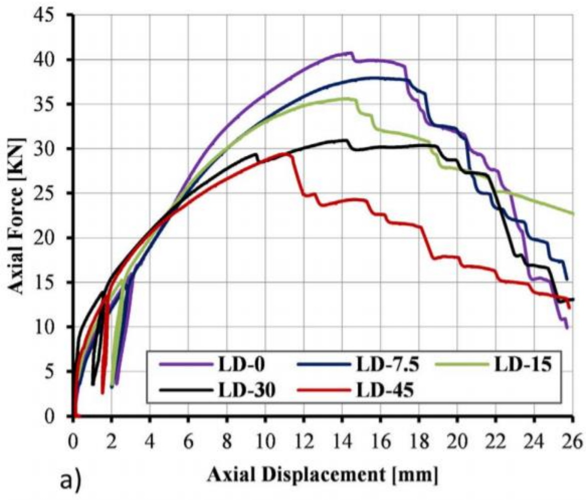


Figure I.1-35 – Axial force vs axial displacement curves obtained for different levels of lateral displacements (LD) [103].

Results show that axial-shear interaction is quite small up to the value of 7.5 mm of lateral displacement (i.e., when angle brackets supporting the shear of a shear wall system belong to the elastic range), while for higher values of lateral displacement, the maximum axial force decrease, with respect to the uniaxial configuration, becomes significant: about 50%. Similar reduction can be observed for the yielding force, that decreases from 15 to 35% according to the lateral displacement imposed. Experimental results demonstrate that the load-carrying capacity of the configuration with imposed lateral displacement cannot be predicted with the standard approach (i.e. Johansen theory [105]) and then specifically developed models are necessary.
Results of the experimental tests permit to develop models capable of describing the behavior of the connections. The first model is proposed by Rinaldin et al. [106] and use a back-bone curve able to consider pinching, degradation of strength and stiffness and post-peak softening to define the cyclic behavior of non-linear springs that simulate the behavior of the connections. In Figure I.1-36 and in Figure I.1-37 are reported the backbone curves proposed respectively for hold-downs and angle brackets.

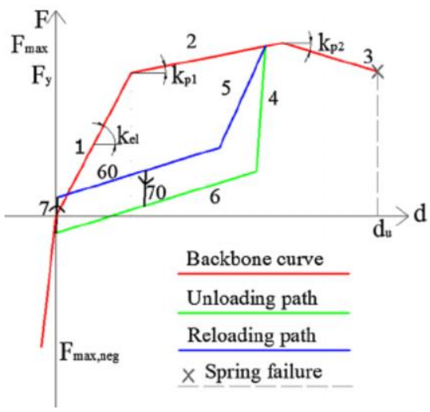


Figure I.1-36 – Backbone curve of hold-down elements

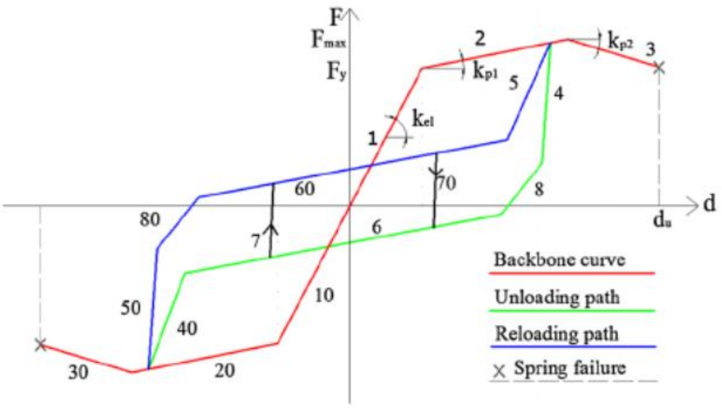


Figure I.1-37 – Backbone curve of angle bracket elements

An example of simplified curves used to simulate the behavior of hold-down and angular with the use of elastic-perfectly plastic curves is reported in Figure I.1-38 [107].

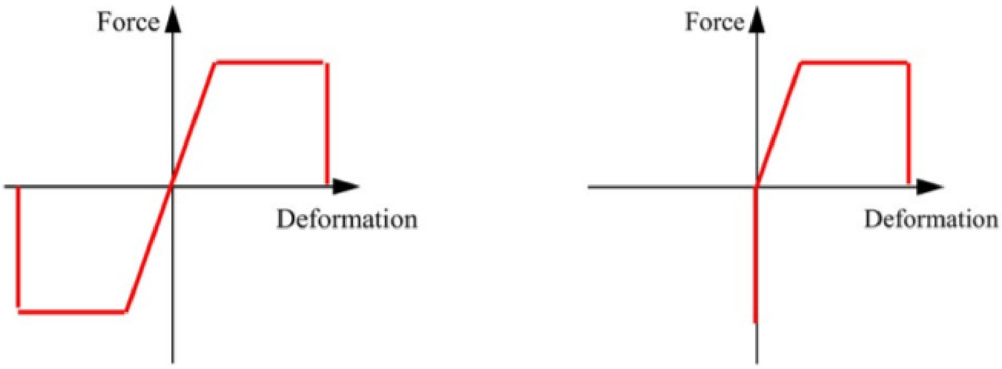


Figure I.1-38 – Simplified backbone curves of angle bracket (left) and hold-down (right)

These curves, even if characterized by a greater level of approximation, provides good results from a numerical point of view when implemented in finite elements calculation software.
The second model was recently developed by Pozza et al. and consist of a novel hybrid force-displacement based coupling method able to take into account for coupling phenomena of hold-down [108] and angle-bracket [109] connections. The model takes into account for the modification of the constitutive law in the considered direction due to displacements in the secondary direction (see Figure I.1-39).

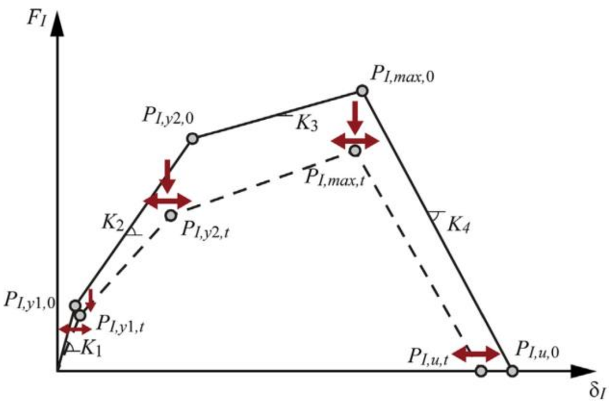


Figure I.1-39 – Example of variation of the backbone envelope curve of axially-loaded hold-downs due to displacements in the secondary direction [108]

The model can predict both monotonic and cyclic behavior of connections also accounting for degradation of strength and stiffness. Authors provided formulation of the coupling model for axially-loaded hold-downs subjected to lateral displacement and laterally-loaded angle brackets subjected to axial displacement.
The analysis of the behavior of the single connection represents the basis for the definition of the seismic behaviour of CLT walls. Several tests have been conducted on CLT walls connected to foundation with hold-downs and angle brackets [110]. All tests demonstrate large hysteretic cycles with significant energy dissipation, pinching, degradation of strength and stiffness, as reports the examples of Figure I.1-40.

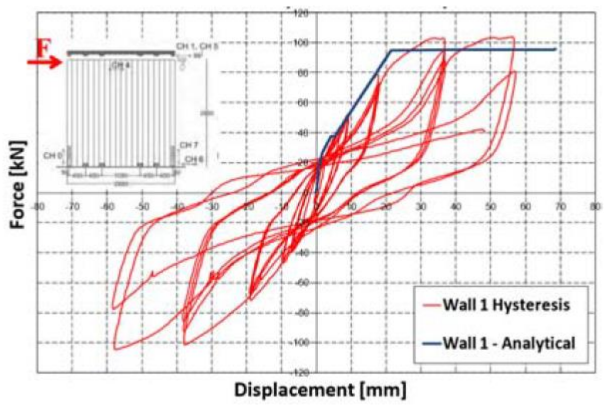


Figure I.1-40 – Test of entire CLT wall connected to foundation with hold-downs and angle brackets

At the end of the tests CLT panel doesn’t show any sign of damage and all energy dissipation is concentrated in the connection elements. Furthermore, the bending and shear deformations of the panel can be considered negligible compared to the total deformation and therefore the latter is almost produced by the elastic and inelastic deformation of the connections. It was also observed that the crisis of the entire system is always due to the plasticization of the connections and in particular hold-downs loaded in tension, while the wooden panel shows a rigid and elastic behaviour. It is worth nothing that the localization of the damage in the connection areas constitutes a positive aspect of the seismic behavior of CLT buildings since, after of the seismic event, it is possible to repair the damaged area.
Several numerical analyses developed with the aim of characterizing the seismic behaviour of CLT shear walls are available in literature. Izzi et al. [111] proposed a numerical model capable of predicting the mechanical behavior and failure mechanisms of CLT wall systems.
In this case, the mechanical connections are simulated as two-node hysteretic springs with three degrees of freedom (two displacement components simulate the behavior in the axial and shear directions, whereas the third one reproduces the out-of-plane response) where the hysteretic behaviour of connectors under shear and tension load is implemented according to Rinaldin et al. [106] (see Figure I.1-41).

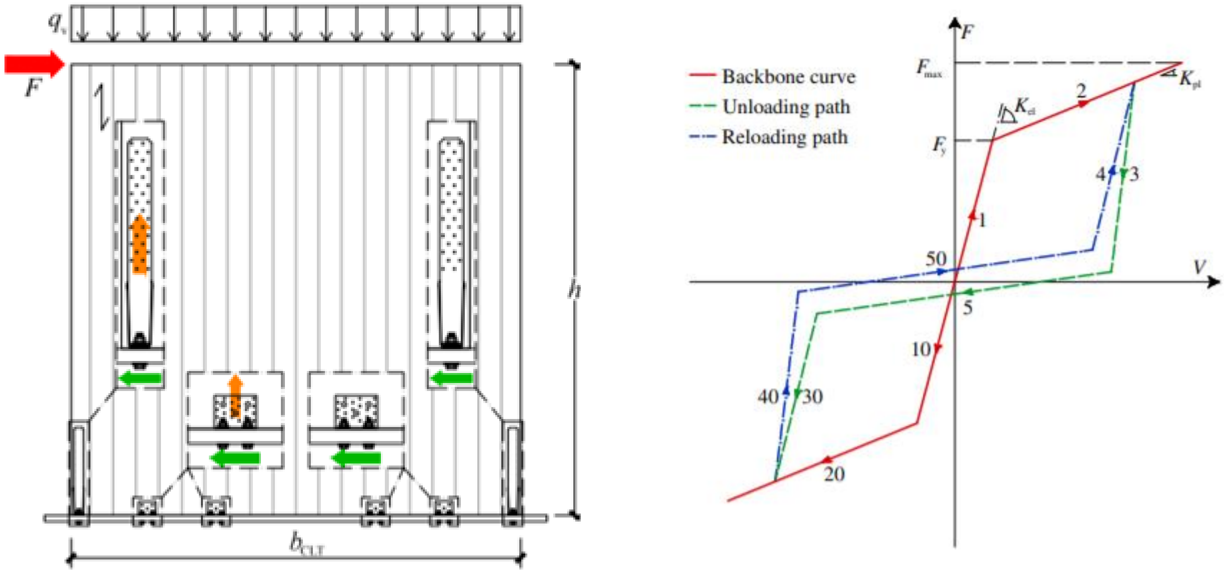


Figure I.1-41 – Schematics of a CLT wall system subjected to a lateral load when a biaxial loading condition is considered (left) and piecewise-linear law of hysteretic spring (right) [111]

Authors reproduced experimental tests conducted at CNR IVALSA by Gavric et al. [110] and reported in Figure I.1-42.

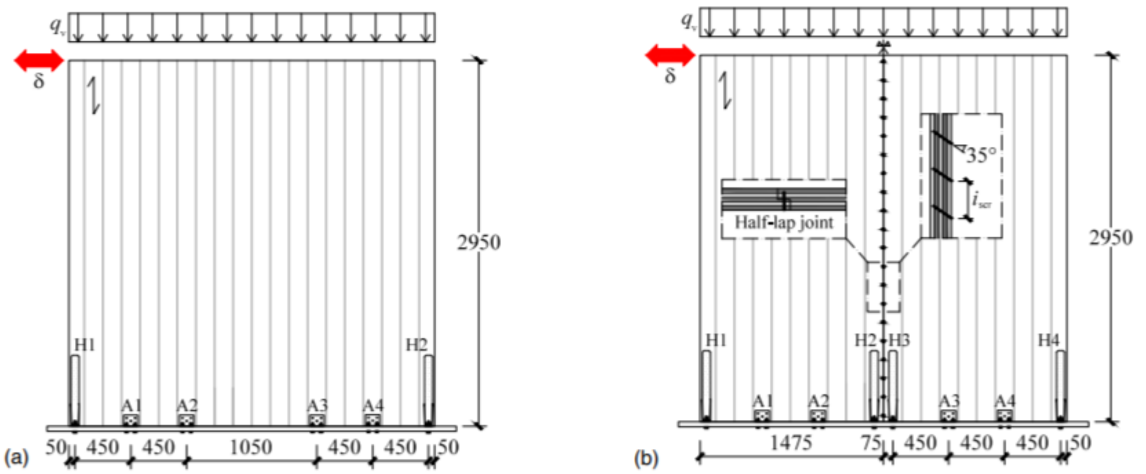


Figure I.1-42 – CLT wall systems tested by Gavric et al. and reproduced in the numerical simulations [111]

Four different methods are considered, implementing connections elements as reported in the following. In Method A connections resist both shear and tension loads, and the coupled shear-tension interaction is accounted with the following quadratic strength domain:

$$\left(\frac{F_{ax,i}}{F_{ax,y}}\right)^2 + \left(\frac{F_{sh,i}}{F_{sh,y}}\right)^2 \leq 1$$

where $F_{ax,y}$ and $F_{sh,y}$ represents the yield loads in the axial and shear direction and $F_{ax,i}$ and $F_{sh,i}$ are the axial and shear loads. Method B is derived from Method A by neglecting the interaction between tensile and shear components. Method C is a further simplification of Method B because hold-downs are assumed to resist only tension whereas angle brackets still have both components (uncoupled). Finally, Method D assumes the hold-downs resist only tension and the angle brackets only shear.

Results obtained by authors highlighted that Method A provides a correct identification of the mechanical behavior of the system. Methods B and C lead to slightly less accurate predictions of the maximum load-carrying capacity. In addition, method C confirmed that the shear component of hold-downs provides a minor contribution to the lateral resistance a CLT wall system. Finally, because of the several simplifications introduced in the analysis, Method D, that is usually adopted by engineers in the design phase, leads to a maximum load-carrying capacity approximately 25% lower than Method A.
All the results above mentioned highlights how the global stiffness of a CLT shear wall, and more in general of a CLT structure, depends strongly on the deformability of connection elements. For this reason, in order to predict the seismic behaviour of CLT building it is crucial to consider the correct stiffness of each connections used to assemble the structure. An iterative design procedure suitable to define the best connection arrangement among the shear walls and therefore to reproduce the reliable estimation of the building global stiffness has been proposed by Polastri and Pozza [16]. The iterative process starts with a preliminary design of connection systems based on code provisions both for principal elastic period value and connection's parameters. It is important to highlight how the preliminary calculation does not involve the definition of T_1 accounting for effects of connection stiffness but refers to a priori definition of the periods referring just to the number of storey and to the building typology. Once static forces on each CLT wall panel are defined connection capacities can be designed to be compatible with external static forces. This allows estimation of the connection elastic stiffness and therefore realistic preliminary estimation of T_1 using a more precise Natural Frequency analyses in a specifically developed building numerical model. Then T_1 can be used in modal analyses to calculate the effective forces induced in connections by earthquakes. Obtained connection forces may or may not be compatible with the connection strength, and if not, it is necessary to redesign connections. Finally, it is necessary to verify the compatibility of the bi-directional action and displacement of the connection with the relative interaction domain. The calculation process just described it schematized in Figure I.1-43.

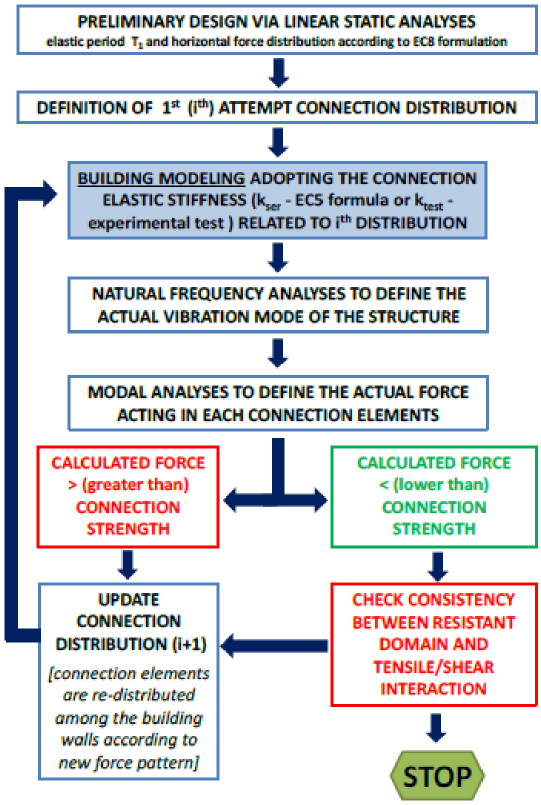


Figure I.1-43 – Calculation process for design of connections proposed by Polastri and Pozza [16]

It is necessary to underline how procedure proposed by authors is suitable both for Linear Dynamic Analyses (i.e. Modal Response Analyses) and Nonlinear Analyses.
In conclusion, in this Section it has been demonstrated the crucial role that connections play in the definition of seismic response of CLT structures and it has been described how an iterative process for the design of connection elements is necessary. It has also been demonstrated that it is fundamental to consider the bi-axial behaviour of connections in order to obtain realistic predictions of the load-carrying capacity of CLT walls. For this reason, in numerical analyses presented in this Thesis, particular attention has been paid to connection elements, that are considered resist both to shear and tension loads and designed according to the iterative procedure.

I.2 Design Oriented Phenomenological Modelling approach for seismic design of multi-storey CLT buildings

Abstract

The widespread diffusion of CLT buildings requires the development of numerical strategies able to reproduce the seismic response of CLT structures easy to implement in commercial software and characterized by a low computation afford that will be used by engineers in the design phase. In this Section a numerical modelling approach for the prediction of the seismic response of multi-storey CLT buildings is proposed. The approach is based on a simplified phenomenological model where the behaviour of the CLT system is reproduced by means of an equivalent elastic modulus to be assigned to the CLT wall in order to account for both the connections and panel deformability. Phenomenological modelling aims to reproduce correctly the global response of the structural system in terms of principal elastic period, internal forces in the connection elements and inter-storey drifts. Results of a parametric analyses carried out on different configurations of CLT walls, designed considering increasing level of seismic intensity, is presented and critically discussed.

I.2.1 Modelling strategies for CLT buildings

CLT is a rapidly spreading construction system in current building practice and in recent years has acquired a valid alternative also in the construction of multi-storey buildings. Lightness, high structural performance, high level of prefabrication and rapid execution of the system are just some of the peculiarities that distinguish this structural type. For these reasons and due renewability and recyclability of the material, CLT buildings have become increasingly common over the last few years. On the other hand, being CLT structures a relatively new technology, the prediction of the seismic response CLT constructions has become a key topic in the last twenty years. Different modelling strategies are available in literature, both for linear and nonlinear analyses. It is possible to affirm that the different modelling approaches do not allow to draw reliable outcomes, especially for the case of linear analyses of buildings.

As reported in Section I.1.2, in recent years, researchers have investigated, through experimental tests and numerical simulations methods, the response of CLT structures under seismic actions demonstrating that connections play a key role in the structural behaviour of multi-storey CLT buildings [13].

Using as references experimental tests described in previous Section, different numerical models has been implemented with the aim of reproducing faithfully the response of single connection elements [106] or of entire CLT structure [8,112] in both linear and nonlinear field. In this regard, two main modelling approaches have been proposed for linear analyses of CLT buildings and are typically adopted from researcher and practitioners. The former approach (known as component-level modelling approach) is mainly research oriented and adopts springs for connections and elastic elements for the CLT in order to investigate both the seismic behaviour and the limits of CLT structures [15,16]. The second method (known as simplified phenomenological modelling approach) is mainly design oriented: the analyses are aimed to provide the necessary information for a simple and safe design of CLT buildings. According to this approach, the behaviour of the system is reproduced by means of a conventional equivalent elastic stiffness assigned to the CLT wall and capable of accounting for the deformability of the CLT panel and connections assembly [17].

In the following a simplified modelling strategy is proposed: the CLT wall systems are implemented with elastic membrane elements with an equivalent stiffness able to represent the deformability of both timber panel and connections. The proper calibration of equivalent stiffness to be assigned to the membrane elements is a crucial aspect: in order to provide reliable estimation of the wall equivalent stiffness for the phenomenological model, different case study configurations have been analysed varying the significant parameters that control the building response such as number of storeys, connection pattern, geometry and wall slenderness. A minimization algorithm, through an ad-hoc procedure is developed in order to obtain the equivalent stiffness of the phenomenological model capable of representing the stiffness of the refined numerical model. The results of those analyses are used to propose a design abacus that permits an easy implementation of the proposed modelling approach by practitioners. Finally, the proposed methodology is applied to a CLT building selected as a case study to demonstrate the suitability of the proposed model.

I.2.1.1 Deformative mechanism of a CLT shear-wall

The seismic elastic behaviour of a CLT shear wall is described in the following. The elastic top horizontal displacement Δ of a CLT shear wall subjected to a horizontal external force can be evaluated considering three main deformation contributions: the rigid body translation (Δt), the rigid body rotation (Δr) and the CLT panel deformations (Δp). In Figure I.2.1 are schematized the three contribution above mentioned.

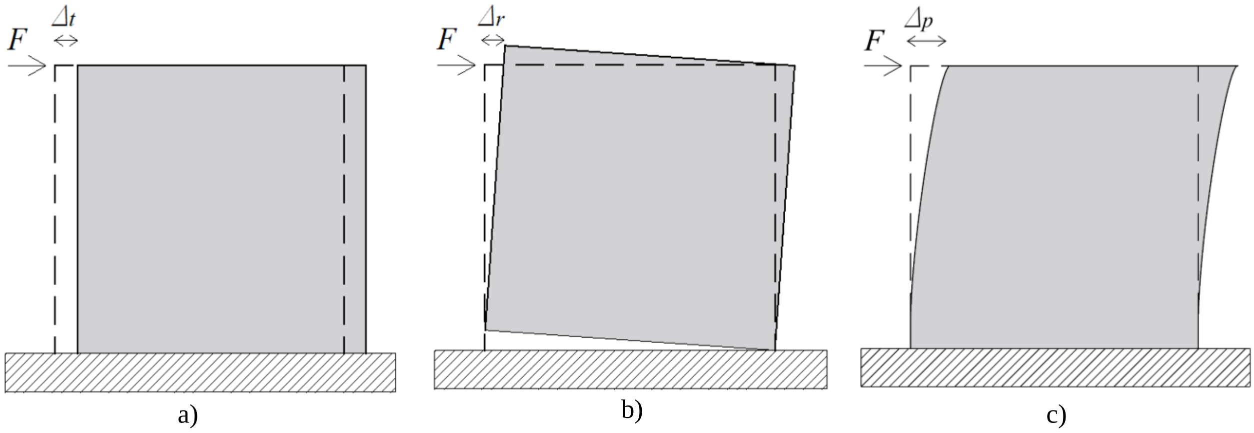


Figure I.2-1 – Deformation contributions of a CLT shear wall subjected to horizontal loads: rigid body translation (a), the rigid body rotation (b) and the CLT panel deformations (c)

The rigid body translation Δt is attributable to the deformation of angle-brackets used to resist to shear forces. The rigid body rotation Δr contribution accounts for the tensile deformation of the hold-downs nailed at each corner and the angle brackets of the wall used to prevent the wall rotation caused by the acting overturning moment, lastly the horizontal displacement of the CLT panel Δp is the elastic top displacement due to the shear and the bending deformability of the CLT panel itself. It is worth nothing that due to the high in-plane stiffness of CLT panels, the elastic behaviour of CLT buildings is primarily governed by the mechanical properties of connections used to fasten the panels to foundation and the panel deformation plays a minor role in the whole deformation of the CLT shear-wall system.

Figure I.2.2 reports a schematic representation of the two modelling approaches, CM and PM, that can predict the global behaviour of a CLT shear-wall subjected to seismic forces.

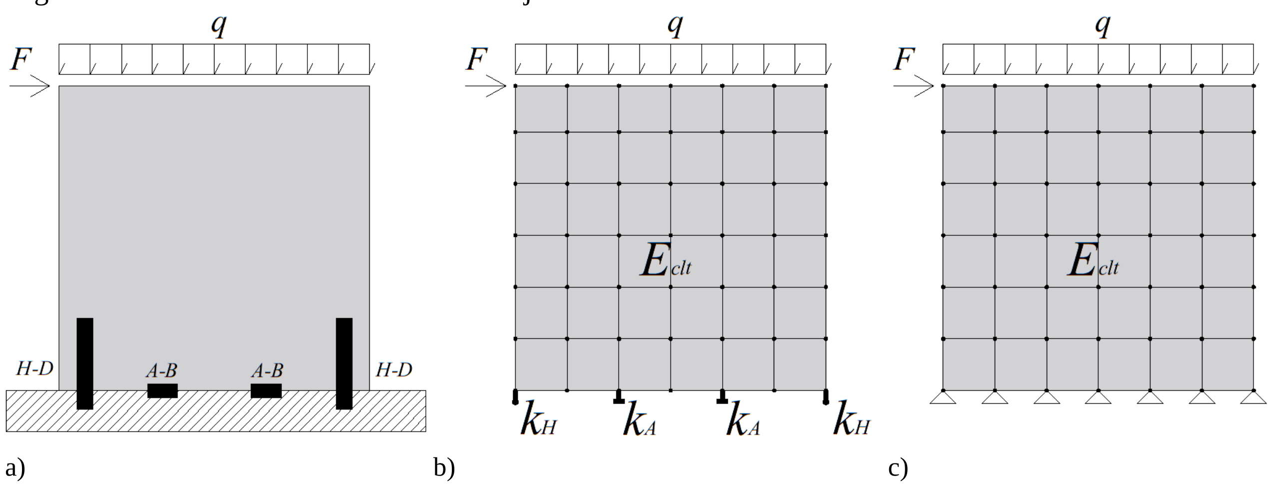


Figure I.2-2 – Different modelling approaches for the prediction of the global seismic behaviour of a CLT shear wall: real configuration (a), CM approach (b) and PM approach (c)

I.2.1.2 Component level modelling approach

The component-level modelling approach [106,112] requires the implementation of all the single components (i.e. connectors and CLT panels) of the structural system in order to investigate the seismic behaviour of the whole system. Concerning linear analyses, the CM approach is based on the use of linear multi-spring element to represent the behaviour of connections (hold-downs and angle brackets), while CLT panels are schematized with linear elastic shell elements (Figure I.2-2b): walls are usually modelled as equivalent isotropic or orthotropic material. In the first case, an equivalent modulus of elasticity obtained by the weighted mean values of elastic modulus in the parallel and perpendicular direction to the grain is used [116]. In the second case, an orthotropic material model considering the effective mechanical cross-section properties, that are derived from the basic material properties of the boards, and the specific layer configuration of each type of panel is used [8].

Finally, it is necessary to calibrate the elastic stiffness of connections to be assigned to multi-spring elements in FE models. This calibration can be performed using two different methods. The first method is based on the study of the force-displacement curve derived from experimental tests on the single connection elements. After the linearization of the above-mentioned curve, it is possible to obtain the elastic stiffness of the connector [26,113]. The second method is based on the calculation of the sliding modulus (k_{ser}) of a nailed steel-timber connection provided in Eurocode 5 [114]. It is worth nothing that this formulation considers only the deformability of nails disregarding the deformation of metal plate and base anchor. Consequently, calculated stiffness can overestimate the elastic stiffness, defined by experimental tests, of connections with deformable steel plate [13]. In both cases, in order to consider the actual behaviour of each connection element, it is necessary to evaluate the interaction and coupling effects between shear and tension forces acting into the connection element [115].

Completed the calibration and implementation of all the single components of the structural system, that involves a high computation afford, CM approach allows to simulate faithfully the structural behaviour of any buildings under seismic actions.

I.2.1.3 Phenomenological modelling approach

PM approach is typically adopted by practitioners since its simplicity and its direct and straightforward implementation in commercial software. Furthermore, the lower computational afford related to PM approach leads to a time saving both in the implementation and resolution phase, making the project timing compatible with those of practitioners.

PM approach available in literature aims to reproduce the global response of the structure by means of elastic stiffness (E_{CLT}) assigned to the CLT wall and disregards the contribution to the structural response given by each element of the system (i.e. connectors). In detail, steel connections are not directly included in FE model and CLT panels are modelled assigning a modulus of elasticity obtained by the weighted mean values of modulus in the parallel and perpendicular direction to the grain can be used [116], as reported in Figure I.2-2c.

It is necessary to underline how CM approach, considering only the deformation of CLT panels and completely neglecting the deformation contribution linked to the connection systems, leads to evaluate the behaviour of the structures in an inaccurate way. Several studies demonstrate the fundamental key role that connection elements play in the characterization of CLT structures, influencing the main parameters such as the fundamental period T_1 and displacements. To overcome this problem, a simplified PM approach based on the use of a reduced equivalent elastic stiffness have been proposed and discussed in the following paragraphs.

I.2.2 Design oriented phenomenological modelling approach

Design **O**riented **PHE**nomenological **M**odelling (DOPHEM) approach proposed is based on the use of a specifically calibrated equivalent elastic modulus of elasticity (E_{eq}) that has to be assigned to building shear-walls and that is able to account for the deformability contributions of both connections and CLT panel. DOPHEM approach is suitable for linear seismic analyses and requires the calibration of E_{eq} for the different shear wall configuration of a real CLT building.

It is worth nothing that in a real CLT building the shar-walls are characterized by different geometry, applied vertical loads and pattern of connections that significantly affect the building response under seismic action. In order to provide a reliable definition of the equivalent elastic modulus of elasticity (E_{eq}) for the DOPHEM approach, an ad-hoc iterative procedure has been implemented. This procedure has been used to perform a multi-parametric analysis on selected shear-wall configurations whose results are presented and discussed in this section.

I.2.2.1 Methodology

The calibration of a phenomenological model can be performed using experimental tests on representative wall samples or implementing refined numerical models capable of faithfully reproduce the actual response of the investigated system [17]. It is worth nothing that the experimental-based approach results effective when a limited number of configurations characterized by limited geometrical dimension have to be tested. Otherwise, the numerical-based approach results more profitable even if an accurate model implementation is required (i.e. according to CM approach).

In this work, because of the significant number of wall configuration to be study, the numerical-based approach has been used according the methodology schematized in Figure I.2-3.

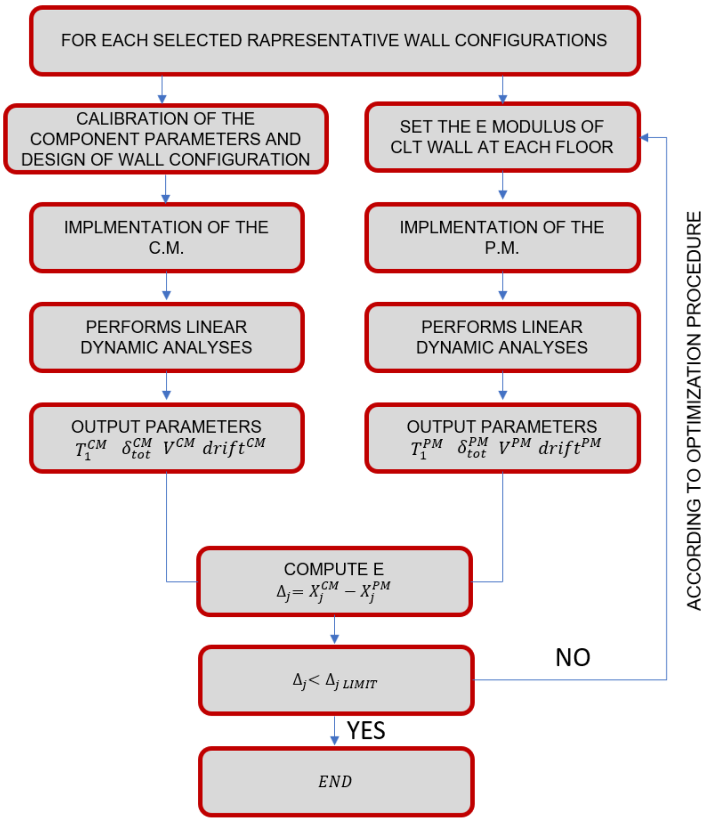


Figure I.2-3 – Conceptual process for calibration of E_{eq}

The proposed procedure starts selecting the representative shear-wall configuration that is characterized by a certain geometry (length and height of the wall, thickness and layering of CLT panel) and a certain level of seismic intensity. Then the reference CM is implemented according to the procedure by Polastri and Pozza [16] and described in Section I.1.2 which allows for a balanced definition of the connection pattern satisfying the shear-wall strength and deformability requirement. Implemented CM is therefore used to perform a Linear Dynamic Analyses in order to define the following reference parameters characterizing the shear-wall seismic response: principal vibration period T_1^{CM}; base Shear V^{CM}; top displacement δ_{tot}^{CM} and inter-storey drifts $drift^{CM}$.

The same shear-wall configuration is then implemented according to DOPHEM approach, evaluating E_{eq} in order to minimize an objective function that is representative of the error between the two modelling strategies on significant parameters. Specifically, the following objective function based on weighted least-squares is used:

$$f\left(E_{eq,0}, \dots, E_{eq,n}\right) = \sqrt{\gamma_T (T_1^{CM} - T_1^{PM})^2 + \gamma_\delta (\delta_{tot}^{CM} - \delta_{tot}^{PM})^2 + \gamma_V (V^{CM} - V^{PM})^2 + \gamma_d (dr^{CM} - dr^{PM})^2}$$

where γ_T, γ_δ, γ_V and γ_d are respectively the weights for first period, top displacement, base shear and inter-story drift. The values of the parameters $E_{eq,0}$ .. $E_{eq,n}$, i.e. the equivalent elastic modulus for each floor, are obtained by minimizing the objective function proposed with an iterative algorithm.

In this work, the wight of the different control parameter has been assumed equal to 1 since they are characterized by the same relevance in describing of the shearwall seismic response.

Furthermore, the following constraints are considered for the solution of the optimization problem:

$$E_{eq,0} \geq E_{eq,1} \geq \cdots \geq E_{eq,n} > 0$$

that represents the physical meaning of having positive young modulus and decreasing young modulus with increasing height of the CLT shear wall. It is worth nothing that in the optimization procedure only the Elastic moduli was varied since the Poisson coefficient was always set equal to 0.35.

In order to calculate the function *f* the generation of a FE model and the solution of a response spectrum analysis is required, therefore the optimization procedure is characterized by a high computational effort.

The proposed methodology has been applied to a multi-parametric study of CLT shear-wall configurations in order to define a suitable set of E_{eq}. This set of values, allows to define a phenomenological model of the shear-wall that gives structural results comparable with the component one results. The main advantage of the PM is that it is extremely easier to implement since it disregards the definition of the connection parameters that are difficult to calibrate for a practitioner.

I.2.2.2 Assessment of the parametric study

The proposed procedure was applied to a parametric analysis with the aim of study the correlation between E_{eq} and the proprieties of many wall configurations. In detail the influence of the most important variables on the determination of E_{eq} to be assigned to CLT panels in a simplified PM approach were investigated. The first variable considered is represented by the slenderness of the CLT shear-wall. In detail, walls with 3, 5, and 7 storeys were considered, with different length equal to L=2, 4 and 6 m and an inter-storey height of 3 m (see Figure I.2-3).

Figure I.2-3 – Different geometries of CLT walls

Two different levels of mass were computed according to the seismic combination of EN 1990 [117], assuming floor dead loads equal to 2.5 kN/m^2 (low mass) or 4.0 kN/m^2 (high mass), and floor live loads equal to 2 kN/m^2. A halved mass was assumed on the roof and a tributary area equal to 6xL m^2 for each wall was considered. The seismic storey mass is listed in Table I.2.1.

Table I.2-1 – Seismic mass per storey of the shear wall

	L=2 m		**L=4 m**		**L=6m**	
	Medium Mass	**High Mass**	**Medium Mass**	**High Mass**	**Medium Mass**	**High Mass**
Floor mass	3.8 t	5.6 t	7.6 t	11.3 t	11.4 t	16.9 t
Roof mass	1.9 t	2.8 t	3.8 t	5.65 t	5.7 t	8.45 t

Two increasing levels of seismic intensity, represented by PGA=0.25 g and PGA=0.35g, were analysed and characterised by the seismic frequency spectra in Figure I.2-4, according to Italian Standard [118], considering a ground type C and a topography category T1. A behaviour factor equal to 1.6 ($q = q_0 \times k_r$) has been assumed, considering an initial reduction factor q_0 of 2 to the elastic spectra, according to [18]. The k_r coefficient has been taken equal to 0.8, as for non-regular structures. Fig. 4 also shows the variation ranges of the principal elastic period T_1 for 3-5-7 storey buildings obtained using the CM approach.

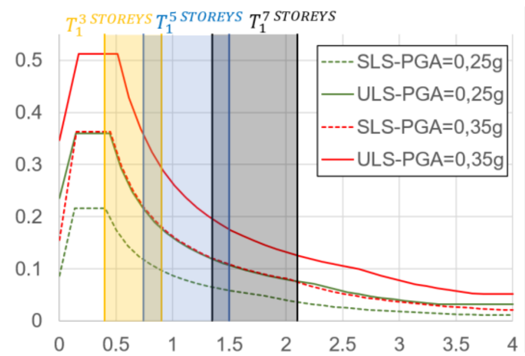


Figure I.2-4 – ULS-SLS design spectra for the different seismicity level considered

Considering the four variables described, the parametric analysis leads to 32 different case studies, which are summarized in Table I.2-2. The different configurations were named according to the following nomenclature: total wall height (3 storeys = 9m → 9; 5 storeys=15m → 15m; 7 storeys = 21m →21), wall width (2; 4; 6 m), mass (2.5 kN/m^2 → low L; 4.0 kN/m^2 → high H), seismic intensity (PGA=0.25g weak → W, PGA=0.35g strong → S). Due to the excessive slenderness of the wall, the case of a 2 m wall was not analysed for the 7-storey configuration.

Table I.2-2 – Analysed configuration of the shear walls

Case study	**Number of storey**	**Lenghth of wall [m]**	**Seismic Mass**	**PGA [g]**
9/2 LW(S)	3	2	LM	0.25 (0.35)
9/4 LW(S)	3	4	LM	0.25 (0.35)
9/6 LW(S)	3	6	LM	0.25 (0.35)
9/2 HW(S)	3	2	HM	0.25 (0.35)
9/4 HW(S)	3	4	HM	0.25 (0.35)
9/6 HW(S)	3	6	HM	0.25 (0.35)
15/2 LW(S)	5	2	LM	0.25 (0.35)
15/4 LW(S)	5	4	LM	0.25 (0.35)
15/6 LW(S)	5	6	LM	0.25 (0.35)
15/2 HW(S)	5	2	HM	0.25 (0.35)

15/4 HW(S)	5	4	HM	0.25 (0.35)
15/6 HW(S)	5	6	HM	0.25 (0.35)
21/4 LW(S)	7	4	LM	0.25 (0.35)
21/6 LW(S)	7	6	LM	0.25 (0.35)
21/4 HW(S)	7	4	HM	0.25 (0.35)
21/6 HW(S)	7	6	HM	0.25 (0.35)

According to the proposed methodology, the different shear-wall configuration has been modelled using the Finite Element code SAP2000 [119].

The reference CM has been implemented using the following finite elements: two-dimensional elastic shell elements to model the CLT panel and linear spring elements to simulate the connections. As an example, Figure I.2.5 shows the models of a 3 storeys shear walls.

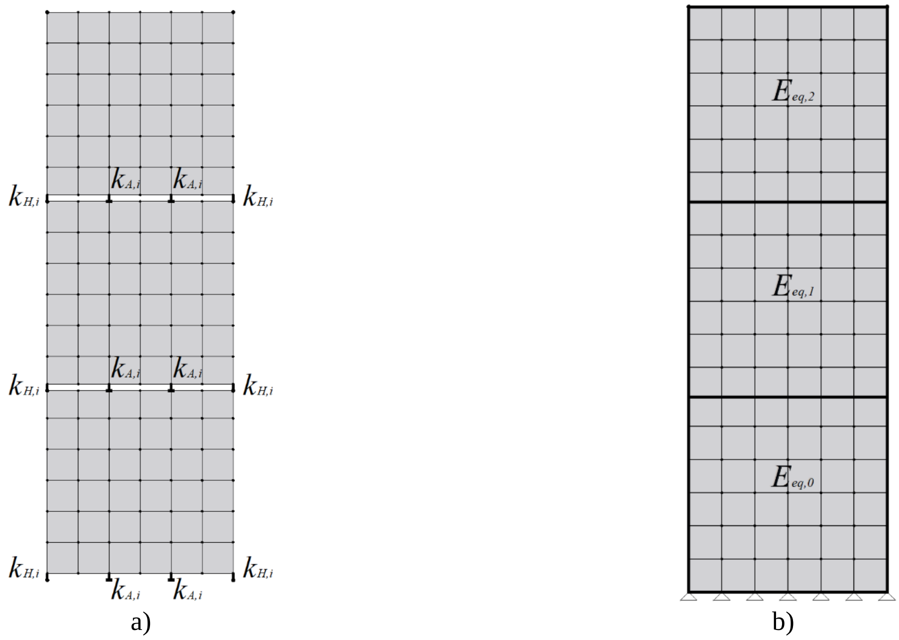


Figure I.2-5 – Scheme of the 3 storeys shear walls: a) CM approach; b) DOPHEM approach

The elastic values adopted for shell elements in CM approach are equal to E=6600 MPa and v=0.35. These mechanical proprieties were obtained as an average between the longitudinal and transversal direction proprieties of CLT. Connection elements are linear springs characterized by uniaxial behaviour of hold-down connection ($k_{H,i}$) and by biaxial behaviour of angle-brackets ($k_{A,i}$) in order to consider the typical tension-shear interaction phenomenon of high capacity brackets [14]. Connections implemented in the different case studies have been designed with the iterative linear static analysis described in Ref. [16] and reported in Section I.1-2. The strength and the stiffness of nailed connections were obtained from experimental tests [26] and are reported in Table I.2.3.

Table I.2-3 – Strength and stiffness of the earthquake-resistant connections

Metal connector	WHT340 hold-down [26]	WHT620 hold-down [26]	TTF200 angle bracket [26]	TCF200 angle bracket [26]
Elastic siffness (kN/mm)	5.70	13.25	8.21	8.48
Design force (kN)	46.20	93.70	39.10	39.10

Connection elements have been designed verifying both the resistance and the deformability of the structure. Particular attention has been paid to respecting the displacement limits imposed by the standards [118]. To evaluate the inter-story drift, the contribution related to the rigid rotation of the underlying floors was deducted on the upper floors. This is because a rigid rotation does not directly damage the structure. As regard the disposition of angle brackets, it has been hypothesized to have a minimum number of connectors equal to 2, 3 and 4 respectively for walls with length 2m, 4m and 6m. The arrangement of connections is reported for all the considered configurations in Table I.2-4. The hold-down number shown in Table I.2-4 is the total number present at each level on the wall, to trace the number of hold-downs present at each end, it must be halved.

Table I.2-4 – Type and number of connections at each level for each configuration

LEVEL	0		1			2			3			4		5		6	
Conn. type / Wall ID	WHT620	TCF200	WHT340	WHT620	TTF200	WHT340	WHT620	TTF200	WHT340	WHT620	TTF200	WHT340	TTF200	WHT340	TTF200	WHT340	TTF200
9/2 LW	2	2	2	-	2	2	-	2									
9/4 LW	2	3	2	-	3	2	-	3									
9/6 LW	2	5	4	-	4	2	-	4									
9/2 HW	2	2	2	-	2	2	-	2									
9/4 HW	4	3	4	-	3	2	-	3									
9/6 HW	4	5	4	-	4	2	-	4									
9/2 LS	4	3	4	-	3	2	-	2									
9/4 LS	4	3	4	-	3	2	-	3									
9/6 LS	4	6	4	-	5	2	-	4									
9/2 HS	6	3	4	-	3	2	-	2									
9/4 HS	6	5	4	-	4	4	-	3									
9/6 HS	8	6	8	-	5	2	-	4									
15/2 LW	4	2	4	-	2	4	-	2	2	-	2	2	2				
15/4 LW	4	3	4	-	3	4	-	3	3	-	2	2	3				
15/6 LW	4	4	4	-	4	4	-	4	2	-	4	2	4				
15/2 HW	4	2	4	-	2	4	-	2	2	-	2	2	2				
15/4 HW	4	3	4	-	3	4	-	3	2	-	3	2	3				
15/6 HW	4	5	4	-	4	4	-	4	2	-	4	2	4				
15/2 LS	4	3	6	-	3	4	-	2	2	-	2	2	2				
15/4 LS	8	4	-	4	3	4	-	3	4	-	3	2	3				
15/6 LS	8	6	-	4	5	4	-	4	4	-	4	2	4				
15/2 HS	6	3	-	4	3	4	-	2	2	-	2	2	2				
15/4 HS	8	4	-	4	3	8	-	3	4	-	3	2	3				
15/6 HS	10	7	-	6	5	8	-	4	4	-	4	2	4				
21/4 LW	4	3	4	-	3	4	-	3	2	-	3	2	3	2	3	2	3
21/6 LW	4	4	4	-	4	4	-	4	4	-	4	2	4	2	4	2	4
21/4 HW	8	4	-	6	3	-	4	3	8	-	3	6	3	4	3	2	3
21/6 HW	5	4	4	-	4	4	-	4	4	-	4	2	4	2	4	2	4

21/4 LS	6	3	4	-	3	4	-	3	2	-	3	2	3	2	3	2	3
21/6 LS	10	6	-	8	5	-	6	4	-	4	4	6	4	4	4	2	4
21/4 HS	12	4	-	8	3	-	6	3	-	4	3	6	3	4	3	2	3
21/6 HS	14	7	-	8	5	-	6	4	-	4	4	6	4	4	4	2	4

It is worth noting that, for increasing number of storeys and for increasing level of seismic intensity it is necessary to increase the number of connectors to make the structure verified. However, the increase of the connectors arrangement is not linear because an increasing of the number of storeys and seismic mass leads to longer period of vibration (T_1) and therefore the structure moves away from the spectrum plateau, with a consequent decrease in the seismic force.

For 3 significant configurations (5 storeys, PGA=0.35g, low mass, L=2-4-6 m), Figure I.2-6 shows the trend of the forces and the wall resistance at each level. Shear strength was calculated considering tension-shear interaction domain [14].

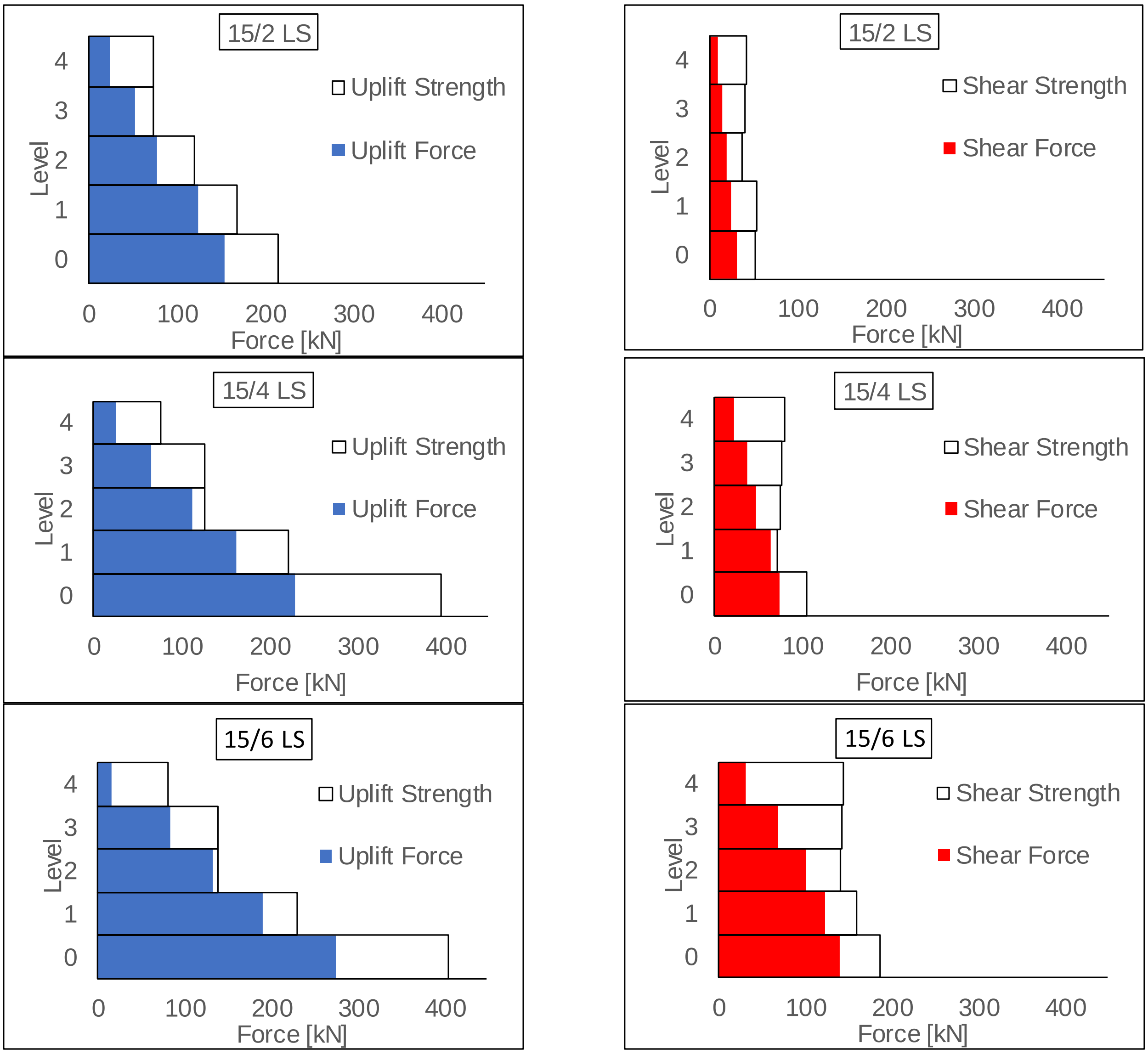


Figure I.2-6 – Acting force and connection strength at each level for 15/2 LS, 15/4 LS and 15/6 LS configurations

I.2.2.3 Results

Results obtained from the CM approach for all the considered configurations are reported in terms of fundamental elastic period (T_1), peak base shear forces at the Ultimate Limit State (ULS), maximum inter-story drift and top displacements at the Serviceability Limit State (SLS). These results are used as reference values for the calibration of the DOPHEM approach according to the proposed calibration procedure.

Figure I.2.7 reports for all the examined shear-wall configurations the values of the principal elastic periods T_1. The values of the period of the limit of the plateau zone T_C of the design spectra is also reported for the strong and weak seism intensity (T_C^S and T_C^W marked with black and red line respectively).

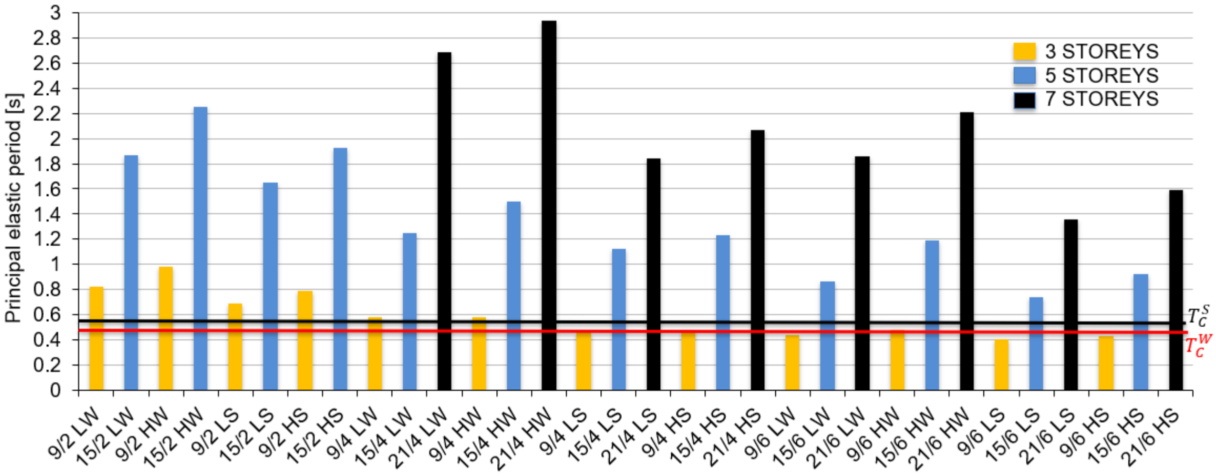


Figure I.2-7 – Principal elastic periods T1 for each configuration

As can be seen from Figure I.2.7, wall configurations with short walls and with connection patterns related to low PGA, have high principal elastic periods T_1. The results in terms of principal elastic period T_1 show how connections play a fundamental role in the definition of the global response of a CLT structure subjected to a seismic force. In fact, there is a marked difference between the wall configuration linked to low PGA, with a lower number of connections, and those with high PGA where the number of connections increases. Another parameter that greatly influences the period is represented by the slenderness of the wall. In fact, walls with high slenderness show greater periods of vibration than walls with less slenderness.

Figure I.2.8 reports for each configuration the peak base shear forces on angle brackets at the ULS.

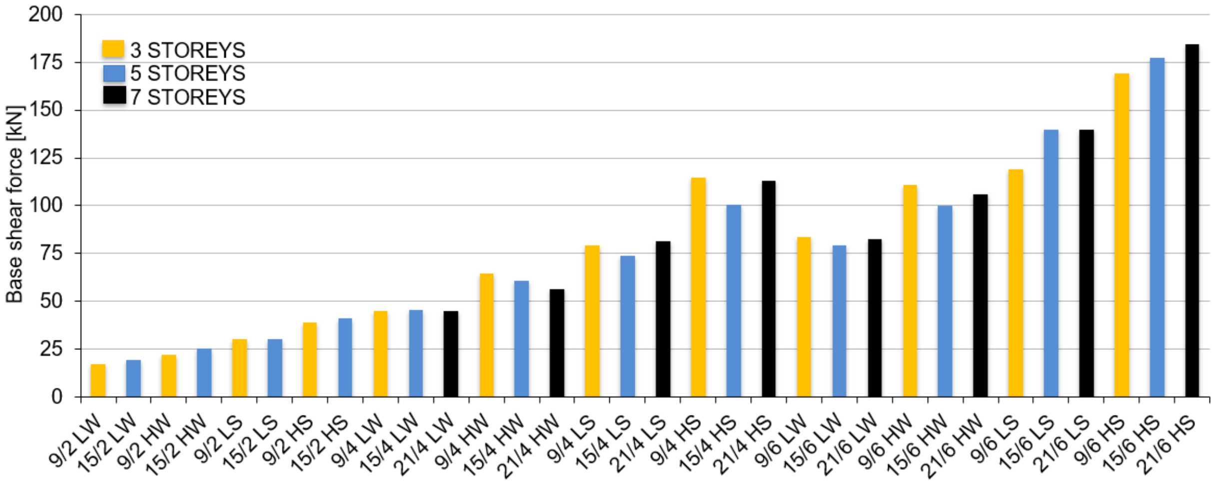


Figure I.2-8 – Base shear force for each configuration

It is possible to observe that in several configurations, as the number of floors increases, the base shear forces at the base varies slightly. This is because the principal elastic period (T_1) of the structure increases, as can be seen in Figure I.2-7, and a decrease in seismic force follows. On the other hand, an increase in the height of the building leads to high top displacements (Figure I.2-9), that requires the use of many traditional hold-downs, as can be noted in Table I.2-4, in order to stiffen the structure and to reduce lateral displacements and inter-storey drifts. Figure I.2-9 and in Figure I.2-10 reports top displacements and maximum inter-story drift at the SLS.

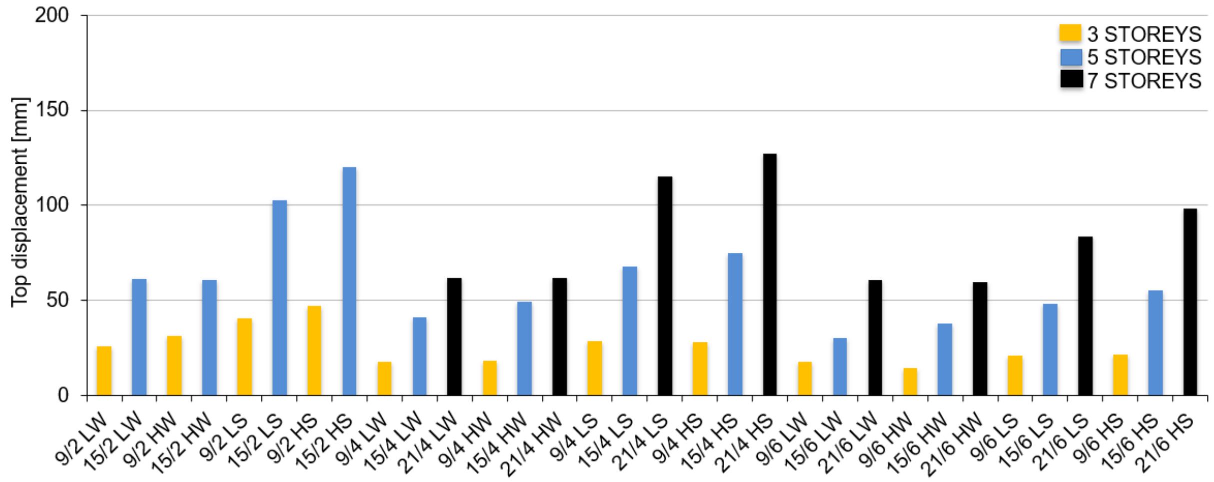


Figure I.2-9 – Top displacement for each configuration

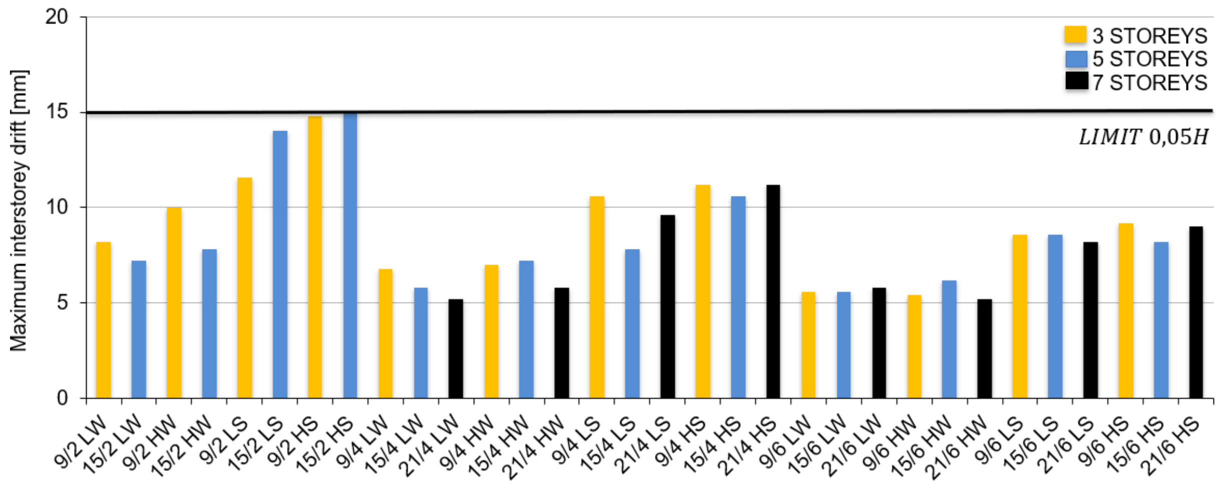


Figure I.2-10 – Maximum interstorey drift for each configuration (at ground level)

Results in terms of maximum interstorey drifts shows as the limit imposed by the Italian regulation [118], equal to 15 mm (0.05*H), is respected for all configurations studied.
Results assessed using the CM approach, on parametric wall configurations (results reported from Figure I.2-7 to Figure I.2-10), have been used in order to calibrate a set of E_{eq} following the optimization procedure defined in previous section. The model schematized in Figure I.2-5 for the case of a 3 storey CLT shear-wall has been used to iteratively defined the E_{eq} of the DOPHEM approach.

In order to define a set of results independent from the CLT wall thickness, the obtained values of the equivalent modulus of elasticity E_{eq} are reported in Table I.2-5a as E_{eq} multiplied by the wall thickness (E_{eq} t [N/mm]). For a sake of clearness, in the typical 100 mm thick CLT wall, E_{0eq} to assign to CLT wall in case 9/2 LW is equal to 810 MPa (=8.1 E+04/100). In Table I.2.5b there are reported values of ratio obtained dividing E_{eq} to the averaged elastic values of CLT panels. In Table I.2.5b it was chosen to report in red values of E_{eq} that are greater than 10% of the averaged elastic values of CLT panels and in green other values of equivalent stiffness.

Table I.2-5a– Equivalent elastic modulus for walls of PM for each configuration

E_{eq} * t [N/mm]	LEVEL						
Wall ID	0	1	2	3	4	5	6
9/2 LW	8.1E+04	5.2E+04	4.9E+04	-	-	-	-
9/4 LW	5.0E+04	3.1E+04	3.0E+04	-	-	-	-
9/6 LW	4.8E+04	3.4E+04	3.3E+04	-	-	-	-
9/2 HW	8.5E+04	5.5E+0.4	5.2E+04	-	-	-	-
9/4 HW	6.8E+04	5.5E+04	5.4E+04	-	-	-	-
9/6 HW	6.4E+04	3.5E+04	3.3E+04	-	-	-	-
9/2 LS	1.17E+05	6.9E+04	6.7E+04	-	-	-	-
9/4 LS	7.7E+04	4.7E+04	4.6E+04	-	-	-	-
9/6 LS	6.9E+04	3.5E+04	3.3E+04	-	-	-	-
9/2 HS	1.24E+05	8.7E+04	8.5E+04	-	-	-	-
9/4 HS	9.1E+04	7.6E+04	7.5E+04	-	-	-	-
9/6 HS	8.2E+04	5.3E+04	4.6E+04	-	-	-	-
15/2 LW	7.8E+04	5.2E+04	4.9E+04	4.7E+04	4.5E+04	-	-
15/4 LW	7.5E+04	5.0E+04	4.9E+04	4.8E+04	4.7E+04	-	-
15/6 LW	6.2E+04	3.5E+04	3.4E+04	3.3E+04	3.2E+04	-	-
15/2 HW	8.4E+04	4.4E+04	4.0E+04	3.8E+04	3.6E+04	-	-
15/4 HW	7.5E+04	5.0E+04	4.7E+04	4.6E+04	4.5E+04	-	-
15/6 HW	6.4E+04	3.5E+04	3.4E+04	3.3E+04	3.2E+04	-	-
15/2 LS	1.25E+05	6.2E+04	5.8E+04	5.5E+04	5.3E+04	-	-
15/4 LS	1.08E+05	5.5E+04	4.5E+04	4.4E+04	4.3E+04	-	-
15/6 LS	9.6E+04	5.8E+04	5.7E+04	5.6E+04	5.5E+04	-	-
15/2 HS	1.35E+05	7.0E+04	6.4E+04	6.0E+04	5.9E+04	-	-
15/4 HS	1.15E+05	7.9E+04	7.2E+04	7.0E+04	6.8E+04	-	-
15/6 HS	1.05E+05	7.7E+04	4.8E+04	4.7E+04	4.6E+04	-	-
21/4 LW	8.1E+04	6.6E+04	6.3E+04	6.2E+04	6.1E+04	6.0E+04	5.9E+04
21/6 LW	5.6E+04	4.2E+04	4.1E+04	4.0E+04	3.9E+04	3.8E+04	3.7E+04
21/4 HW	9.0E+04	7.0E+04	6.5E+04	6.4E+04	6.3E+04	6.1E+04	6.0E+04
21/6 HW	6.3E+04	4.7E+04	4.5E+04	4.3E+04	4.1E+04	4.0E+04	3.9E+04
21/4 LS	1.10E+05	9.0E+04	8.9E+04	8.8E+04	8.5E+04	8.4E+04	8.3E+04
21/6 LS	9.7E+04	9.2E+04	8.5E+04	7.3E+04	6.2E+04	5.2E+04	4.5E+04
21/4 HS	1.33E+05	1.1E+05	9.7E+04	9.0E+04	8.6E+04	8.2E+04	7.8E+04
21/6 HS	1.1E+05	7.1E+04	6.8E+04	6.6E+04	5.7E+04	5.4E+04	5.2E+04

Table I.2-5b– Ratio obtained dividing Equivalent elastic modulus for walls of PM to the averaged elastic values of CLT panels

E_{eq}/E_{CLT}[N/mm]	LEVEL						
Wall ID	0	1	2	3	4	5	6
9/2 LW	12%	8%	7%				
9/4 LW	8%	5%	5%				
9/6 LW	7%	5%	5%				
9/2 HW	13%	8%	8%				
9/4 HW	10%	8%	8%				
9/6 HW	10%	5%	5%				
9/2 LS	18%	10%	10%				
9/4 LS	12%	7%	7%				
9/6 LS	10%	5%	5%				
9/2 HS	19%	13%	13%				
9/4 HS	14%	12%	11%				
9/6 HS	12%	8%	7%				

15/2 LW	12%	8%	7%	7%	7%		
15/4 LW	11%	8%	7%	7%	7%		
15/6 LW	9%	5%	5%	5%	5%		
15/2 HW	13%	7%	6%	6%	5%		
15/4 HW	11%	8%	7%	7%	7%		
15/6 HW	10%	5%	5%	5%	5%		
15/2 LS	19%	9%	9%	8%	8%		
15/4 LS	16%	8%	7%	7%	7%		
15/6 LS	15%	9%	9%	8%	8%		
15/2 HS	20%	11%	10%	9%	9%		
15/4 HS	17%	12%	11%	11%	10%		
15/6 HS	16%	12%	7%	7%	7%		
21/4 LW	12%	10%	10%	9%	9%	9%	9%
21/6 LW	8%	6%	6%	6%	6%	6%	6%
21/4 HW	14%	11%	10%	10%	10%	9%	9%
21/6 HW	10%	7%	7%	7%	6%	6%	6%
21/4 LS	17%	14%	13%	13%	13%	13%	13%
21/6 LS	15%	14%	13%	11%	9%	8%	7%
21/4 HS	20%	17%	15%	14%	13%	12%	12%
21/6 HS	17%	11%	10%	10%	9%	8%	8%

Results reported in Table I.2.5a demonstrate that the equivalent elastic modulus of elasticity to be assigned to the wall increase with the slenderness of the wall. It is also possible to notice a knee adjustment in the stiffness values. Indeed, the trend presents an initial rapid increase which gradually decreases with the height of the wall. Range variation of equivalent elastic modulus is between 5-20% respect to the averaged elastic values of CLT panels (E_{CLT}=6600 MPa). This demonstrates that connection elements greatly influence the overall stiffness of CLT shear walls. This statement is confirmed by the fact that, green values, that as anticipated previously are less than 10% respect to the averaged elastic values of CLT panels, are predominant in Table I.2-5b.

In Table I.2-6, Table I.2-7, Table I.2-8 and Table I.2-9 are reported respectively top displacements, maximum inter-storey drifts, principal elastic periods (T_1) and base shear forces for each configuration obtained from CM and PM strategies with the relative errors in percentage terms.

Table I.2-6 – Top displacement for each configuration studied obtained from CM and PM approaches

Top displacement (mm)								
Wall ID	**9/2 LW**	**15/2 LW**	**9/2 HW**	**15/2 HW**	**9/2 LS**	**15/2 LS**	**9/2 HS**	**15/2 HS**
CM	25.9	61.2	31.2	61.0	40.6	102.7	47.0	120.3
PM	26.7	61.8	31.1	62.2	40.7	101.1	47.0	118.5
ERR[%]	3%	1%	0%	2%	0%	2%	0%	2%
Wall ID	**9/4 LW**	**15/4 LW**	**21/4 LW**	**9/4 HW**	**15/4 HW**	**21/4 HW**	**9/4 LS**	**15/4 LS**
CM	17.9	40.9	61.6	18.0	49.1	61.9	28.5	67.7
PM	18.3	40.9	60.4	17.7	49.6	62.4	28.5	69.9
ERR[%]	2%	0%	2%	2%	1%	1%	0%	3%
Wall ID	**21/4 LS**	**9/4 HS**	**15/4 HS**	**21/4 HS**	**9/6 LW**	**15/6 LW**	**21/6 LW**	**9/6 HW**
CM	115	27.8	74.9	127.1	17.9	30.0	60.5	14.6
PM	117.1	28.5	74.6	125.5	17.9	32.1	60.3	14.5
ERR[%]	2%	3%	0%	1%	0%	7%	0%	0%
Wall ID	**15/6 HW**	**21/6 HW**	**9/6 LS**	**15/6 LS**	**21/6 LS**	**9/6 HS**	**15/6 HS**	**21/6 HS**
CM	38.0	60.5	21.1	48.4	83.5	21.4	55.1	98.4
PM	39.0	61.4	21.2	48.7	83.5	21.5	55.1	103.2
ERR[%]	3%	2%	0%	0%	0%	0%	0%	5%

Table I.2-7 – Maximum interstorey drift for each configuration studied obtained from CM and PM approaches

Maximum interstorey drift (mm)								
Wall ID	**9/2 LW**	**15/2 LW**	**9/2 HW**	**15/2 HW**	**9/2 LS**	**15/2 LS**	**9/2 HS**	**15/2 HS**
CM	8.2	7.2	10	7.8	11.6	14.0	14.8	15.0
PM	7.9	7.1	10.1	7.8	11.2	19.7	15.0	14.9
ERR[%]	3%	1%	1%	0%	4%	2%	1%	1%
Wall ID	**9/4 LW**	**15/4 LW**	**21/4 LW**	**9/4 HW**	**15/4 HW**	**21/4 HW**	**9/4 LS**	**15/4 LS**
CM	6.8	5.8	5.2	7	7.2	5.8	10.6	7.8
PM	6.6	5.8	5.3	7.3	7.1	5.7	10.8	7.6
ERR[%]	3%	0%	2%	4%	1%	2%	2%	3%
Wall ID	**21/4 LS**	**9/4 HS**	**15/4 HS**	**21/4 HS**	**9/6 LW**	**15/6 LW**	**21/6 LW**	**9/6 HW**
CM	9.6	11.2	10.6	11.2	5.6	5.6	5.8	5.4
PM	9.6	11.6	10.6	11.1	5.5	5.5	5.8	5.6
ERR[%]	0%	4%	0%	1%	1%	2%	0%	3%
Wall ID	**15/6 HW**	**21/6 HW**	**9/6 LS**	**15/6 LS**	**21/6 LS**	**9/6 HS**	**15/6 HS**	**21/6 HS**
CM	6.2	5.2	8.6	8.6	8.2	9.2	8.2	9
PM	6.3	5.0	8.7	8.8	8.2	9.1	8.1	9.2
ERR[%]	1%	4%	1%	2%	0%	1%	1%	2%

Table I.2-8 – Principal elastic period for each configuration studied obtained from CM and PM approaches

T1(s)								
Wall ID	**9/2 LW**	**15/2 LW**	**9/2 HW**	**15/2 HW**	**9/2 LS**	**15/2 LS**	**9/2 HS**	**15/2 HS**
CM	0.82	1.87	0.98	2.25	0.69	1.65	0.79	1.93
PM	0.84	1.89	0.97	2.22	0.69	1.68	0.79	1.92
ERR[%]	2%	1%	1%	1%	0%	2%	0%	1%
Wall ID	**9/4 LW**	**15/4 LW**	**21/4 LW**	**9/4 HW**	**15/4 HW**	**21/4 HW**	**9/4 LS**	**15/4 LS**
CM	0.58	1.25	2.69	0.58	1.5	2.94	0.46	1.12
PM	0.59	1.25	2.74	0.58	1.52	2.88	0.48	1.15
ERR[%]	2%	0%	2%	0%	1%	2%	4%	3%
Wall ID	**21/4 LS**	**9/4 HS**	**15/4 HS**	**21/4 HS**	**9/6 LW**	**15/6 LW**	**21/6 LW**	**9/6 HW**
CM	1.84	0.48	1.23	2.07	0.44	0.86	1.86	0.48
PM	1.88	0.49	1.23	2.15	0.44	0.90	1.88	0.48
ERR[%]	2%	2%	0%	4%	0%	5%	1%	0%
Wall ID	**15/6 HW**	**21/6 HW**	**9/6 LS**	**15/6 LS**	**21/6 LS**	**9/6 HS**	**15/6 HS**	**21/6 HS**
CM	1.19	2.21	0.4	0.74	1.36	0.43	0.92	1.59
PM	1.22	2.25	0.41	0.78	1.36	0.42	0.92	1.65
ERR[%]	2%	2%	2%	5%	0%	2%	0%	4%

Table I.2-9 – Base shear forces for each configuration studied obtained from CM and PM approaches

V (kN)								
Wall ID	**9/2 LW**	**15/2 LW**	**9/2 HW**	**15/2 HW**	**9/2 LS**	**15/2 LS**	**9/2 HS**	**15/2 HS**
CM	17	19.6	21.8	25.2	30.1	30.4	38.7	41.2
PM	16.4	19.2	21.7	25.2	29.6	30	38.5	42.2
ERR[%]	4%	2%	0%	0%	2%	1%	1%	2%
Wall ID	**9/4 LW**	**15/4 LW**	**21/4 LW**	**9/4 HW**	**15/4 HW**	**21/4 HW**	**9/4 LS**	**15/4 LS**
CM	44.8	45.4	44.7	64.7	60.6	56.2	79.3	74
PM	47.5	44.8	43.6	61.3	59.4	58	76.3	74.4
ERR[%]	6%	1%	2%	5%	2%	3%	4%	1%
Wall ID	**21/4 LS**	**9/4 HS**	**15/4 HS**	**21/4 HS**	**9/6 LW**	**15/6 LW**	**21/6 LW**	**9/6 HW**
CM	81.5	114.5	100.7	113	83.6	79.4	82.4	110.7
PM	80.4	110.4	100.9	109.3	82.8	77.1	82.4	115.6
ERR[%]	1%	4%	0%	3%	1%	3%	0%	4%
Wall ID	**15/6 HW**	**21/6 HW**	**9/6 LS**	**15/6 LS**	**21/6 LS**	**9/6 HS**	**15/6 HS**	**21/6 HS**
CM	99.7	105.8	119.1	139.7	139.5	169.2	177.5	184.4
PM	98.7	102.7	116.4	136.2	139.7	172.4	178.1	189
ERR[%]	1%	3%	2%	3%	0%	2%	0%	2%

It can be observed that errors on the significant control parameters reported in Table I.2-6, Table I.2-7, Table I.2-8 and Table I.2- 9 are always equal within a maximum of 5% confirming the effectiveness of the adopted calibration procedure.

Table I.2-10 and Table I.2-11 report a further comparison between displacements and shear forces acting at each level obtained from CM and PM strategies with the relative errors in percentage terms. The comparison is made on three significant cases: 15/2LS, 15/4LS and 15/6 LS (same cases reported in Figure I.2-6).

Table I.2-10 – Displacements at each level for 15/2LS, 15/4LS and 15/6LS configurations obtained from CM and PM approaches

Δ [mm]		LEVEL				
Wall ID		0	1	2	3	4
15/2 LS	CM	7	21.9	44.8	72.5	102.7
	PM	6.8	21.6	43.5	70.9	101.1
	ERR[%]	3%	1%	3%	2%	2%
15/4 LS	CM	3.9	14.9	30.6	48.9	67.7
	PM	3.9	14.2	30.2	49.5	69.9
	ERR[%]	0%	5%	1%	1%	3%
15/6 LS	CM	4.3	12.9	24.4	36.9	48.4
	PM	4.1	12.2	23.2	35.8	48.7
	ERR[%]	5%	5%	5%	3%	0%

Table I.2-11 – Shear forces at each level for 15/2LS, 15/4LS and 15/6LS configurations studied obtained from CM and PM approaches

Shear force [kN]*		LEVEL				
Wall ID		0	1	2	3	4
15/2 LS	CM	30.4	25.2	19.5	15.0	9.7
	PM	29.5	24.7	20.3	15.2	9.3
	ERR[%]	3%	2%	4%	1%	4%
15/4 LS	CM	74.0	63.3	47.9	37.8	21.6
	PM	73.3	61.4	46.7	38.0	22.9
	ERR[%]	1%	3%	3%	2%	6%
15/6 LS	CM	139.7	123.5	100.3	69.3	31.4
	PM	136.9	118.6	104.6	67.2	29.9
	ERR[%]	2%	4%	5%	3%	5%
** shear forces are obtained by integrating the internal stresses of shell elements*						

The results reported in Tables I.2-10 and I.2-11 validate the accuracy of the proposed procedure that allows to capture, with limited errors, also displacements and shear forces acting at each level.

This demonstrates that although the calibration was performed on global parameters (top displacements, base shear forces, principal elastic periods and interstorey drifts), the E_{eq} of the DOPHEM approach allows for the correct evaluation of principal parameters at each level as well. This represents a key issue since a correct prediction of local forces and displacements plays a fundamental role in the design of joints and more in general in the design of multistorey CLT building.

I.2.3 Analyses of results and definition of a practical design procedure

In this Section, results obtained from the parametric study in terms of storey's equivalent elastic modulus of elasticity $E_{eq,i}$ are analysed and implemented in a practical design procedure that allow an immediate application of the DOPHEM approach for the seismic analyses of a CLT building.

Moreover, a practical example of application of the propose design procedure is reported considering a selected CLT multi-storey building case study.

I.2.3.1 Analyses of results

Analyses of obtained results is based on the definition of a direct correlation between the values of the $E_{eq,i}$ and the different parameters considered in the parametric study exposed in the previous section.

In detail, a preliminary analysis of the results demonstrates a potential correlation between the equivalent stiffness of the wall at the first level (E_{eq0}) and the geometric proprieties of the shear-wall exposed in terms of slenderness (λ) that is defined as the ratio between the hight (H) and the base length (L) of the wall λ=H/L.

The aforementioned linear correlation, for the two previously defined level of mass and level of seismic intensity considered in the parametric study, is shown in Figure I.2-11.

As it can be noted, the parameter that has less influence on the correlation is seismic mass, for this reason, it has been chosen to merge the curves with different masses in a single curve (continuous lines). This assumption facilitates the reading of the abacus by practitioners, who usually use the simplified model in order to reproduce the seismic behaviour of an entire CLT buildings.

Furthermore, Figure I.2-11 shows the linear correlation between $E_{eq,0}$ and λ as confirmed by the coefficient of determination R^2. It is worth nothing that the correlation of Figure I.2-11 is related to single CLT walls. Values of the equivalent modulus of elasticity E_0 are reported in Figure I.2-11 as E_0 times the wall thickness (E_0 t [N/mm]) in order to define a set of results independently from the thickness of the adopted CLT wall.

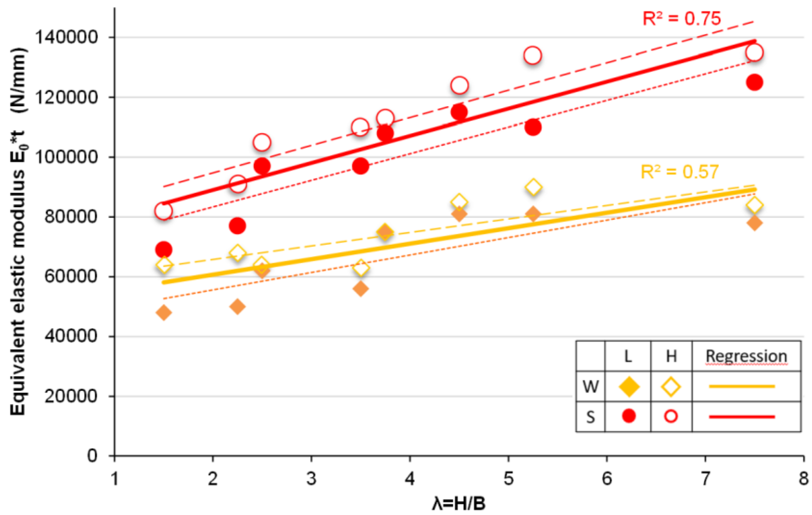


Figure I.2-11 – Correlation between equivalent elastic modulus E_{eq0} and slenderness of wall for different PGA

Equations of the linear regression to the red and the orange lines of Figure I.2-11 are reported in the following. The equivalent stiffness E_{eq0} assign to the wall of the level zero increases as the slenderness increases and is define as:

$$E_{eq0} \cdot t = a \cdot \lambda + b \qquad (1)$$

where a=9044 and b=70987 for strong seismicity (continuous red line of Figure I.2-11) and a=5184 and b=50325 for weak seismicity (continuous orange line of Figure I.2-11).

It is also observed that the stiffness E_{eq0} is higher in cases of high seismic actions. This is because as the seismicity increases, the connection pattern increases and consequently the wall system has greater stiffness with respect to horizontal loads. For values of PGA different than those studied, it is possible to obtain E_{eq0} by linearly interpolating with respect to the values reported in Figure I.2-11.

Once E_{eq0} is correlated to the characteristics of the structure, in order to define a methodology that can be useful for practitioners, a quick method to obtain equivalent stiffnesses of the upper floors of the structure ($E_{eq,i}$) has been defined: the next step of the analyses of the results in fact consists in the definition of a relation between E_{eq0} and $E_{eq,i}$. In this work, based on the observed results, the following function is proposed:

$$E_{eq,i} = E_{eq0}(1 - 0{,}31 \cdot i^{0.12}) \quad (2)$$

where *i* represent the floor number. In Figure I.2-12 are reported the mean and predicted values of equivalent stiffness of the upper floors ($E_{eq,i}$) scaled with respect to those of the ground floor ($E_{eq,0}$), at each level. Mean values of equivalent stiffness reported are obtained relating stiffness summarized in Table I.2-5. It is possible to notice a knee adjustment in the stiffness values that can be predicted precisely thanks to Equation 2.

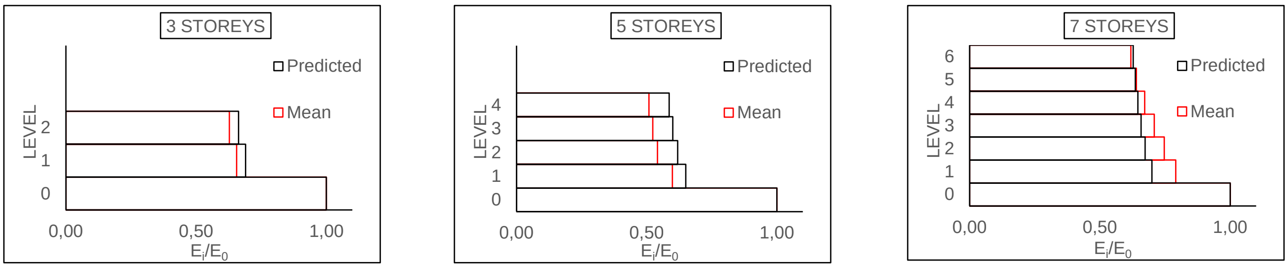


Figure I.2-12 – Equivalent stiffness trend and prediction for 3, 5 and 7 storey's structures

I.2.3.2 Definition of a practical design procedure and application to a selected case study

PM approach, that aims to characterize the seismic behavior of multi-storey CLT building thanks to the use of an equivalent elastic modulus of elasticity $E_{eq,i}$ to be assigned to CLT panels, follows the methodology schematized in Figure I.2-13.

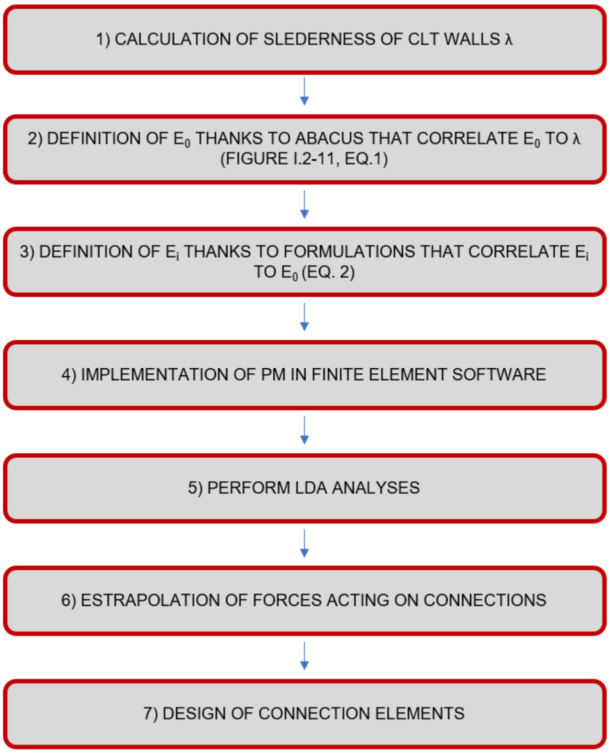


Figure I.2-13 – DOPHEM procedure

In the following it is reported an example of PM approach application to characterize the seismic behaviour of a multi-storey CLT building, selected as a case study, to show the suitability of the proposed procedure reported in Figure I.2-13. Plan dimensions of examined case-study building are reported in Figure I.2-14, storey height is equal to 3m in all cases. 3-5 and 7 storeys configuration were analysed. The numbering of the walls will be useful later to assign to each wall the equivalent stiffness based on walls slenderness.

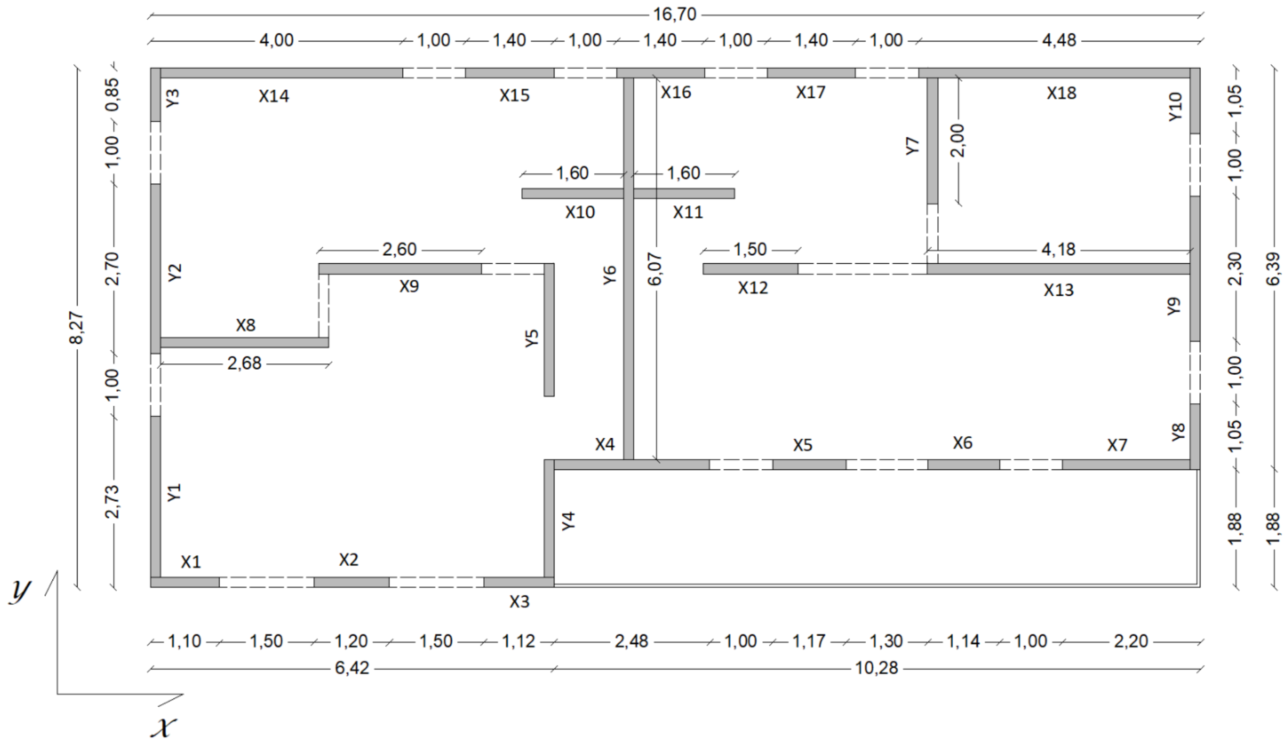


Figure I.2-14 – Plan dimensions of case study

Response spectrum analyses were conducted calculating the earthquake action according to Italian regulations [118] (ground type C, topography category T1 and PGA equal to 0.35 g). A behaviour factor *q* equal to 1.6 has been applied. Interaction between perpendicular walls were neglected since in the current design practice the vertical “L” and “T” joints are fastened with mounting screws that have no structural significance in terms of stiffness and strength. It is worth nothing that when the corner joints are specifically designed to withstand loads, they must be considered in the global analyses of the building system. Floor dead loads was assumed equal to 2.5 kN/m^2 and floor live loads equal to 2 kN/m^2. A halved mass was assumed on the roof and the seismic mass was calculated according to the seismic combination of EN 1990 [117] and reported in Table I.2-12.

Table I.2-12 – Seismic mass per storey of the case study

	Seismic mass
Floor mass	37.6 t
Roof mass	18.8 t

The equivalent elastic modulus to be assign at CLT walls of the first level was calculated according to abacus of Figure I.2-11 (E_{eq0}) and are reported in Table I.2-13. It is underlined how the slenderness of the single wall, necessary for estimate E_0 from the abacus above mentioned, must be calculated as the ratio between the total height of the wall (building height) and its width.

Table I.2-13 – E_{eq0} for each wall of case studies analysed

X DIRECTION							Y DIRECTION						
ID	3 STOREYS		5 STOREYS		7 STOREYS		ID	3 STOREYS		5 STOREYS		7 STOREYS	
	λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]		λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]
X1	8.2	164087	13.6	234243	19.1	304399	Y1	3.5	103982	5.8	134068	8.2	164155
X2	7.5	155317	12.5	219627	17.5	283937	Y2	3.3	101725	5.6	130308	7.8	158890
X3	8.0	162207	13.4	231111	18.8	300015	Y3	10.7	195844	17.8	287172	24.9	378499
X4	3.6	105623	6.1	136804	8.5	167984	Y4	5.0	123162	8.3	166035	11.7	208909
X5	7.7	157790	12.8	223749	17.9	289708	Y5	4.3	113584	7.1	150072	9.9	186560
X6	7.9	160394	13.2	228089	18.4	295784	Y6	1.5	77923	2.5	90636	3.5	103350
X7	4.4	115736	7.4	153658	10.3	191580	Y7	4.5	116731	7.5	155317	10.5	193903
X8	3.4	102126	5.6	130975	7.9	159825	Y8	8.6	169625	14.4	243474	20.1	317323
X9	3.6	104971	6.0	135717	8.4	166462	Y9	3.9	109182	6.5	142735	9.1	176288
X10	5.6	130751	9.3	178684	13.0	226617	Y10	8.6	169625	14.4	243474	20.1	317323
X11	5.6	130751	9.3	178684	13.0	226617							
X12	6.0	136024	10.0	187472	14.0	238920							
X13	2.2	86545	3.6	105008	5.0	123470							
X14	2.3	88880	3.9	108899	5.4	128917							
X15	6.4	141536	10.7	196659	15.0	251782							
X16	6.4	141536	10.7	196659	15.0	251782							
X17	6.4	141536	10.7	196659	15.0	251782							
X18	2.1	85617	3.5	103460	4.9	121303							

The values of the stiffnesses to be assigned to the walls of the upper floors of the structure ($E_{eq,i}$) was calculated according to the linear regression function reported in Equation (2).
Horizontal slabs elements in floor and roof diaphragms were assumed to be rigid in-plane. Results obtained from numerical models, in terms of principal elastic period T_1, base shear force and displacements are reported in Table I.2-14.

Table I.2-14 – Results of numerical analyses implemented in PM approach

	3 Storeys		5 Storeys		7 Storeys	
	X direction	Y direction	X direction	Y direction	X direction	Y direction
Principal elastic period T_1 [s]	0.44	0.52	0.77	1.03	1.44	2.00
Base shear force [kN]	704.5	621.1	752.1	519.6	924.1	653.7
Top displacement [mm]	25.2	31.3	54.3	59.0	100.6	115.2

In order to verify the reliability of obtained results, the same case-studies (3-5 and 7 storeys), have been analysed with CM approach. The elastic values adopted for shell elements are equal to E=6600 MPa and v=0.35. These connections implemented (hold-down with an axial stiffness, angle-brackets with axial and shear stiffness) have been designed with the iterative linear static analysis described in Ref. [18]. The strength and the stiffness of nailed connections were obtained from experimental tests (Table I.2-3). The example of 5 storeys case study implemented with PM and CM approaches is reported in Figure I.2-15.

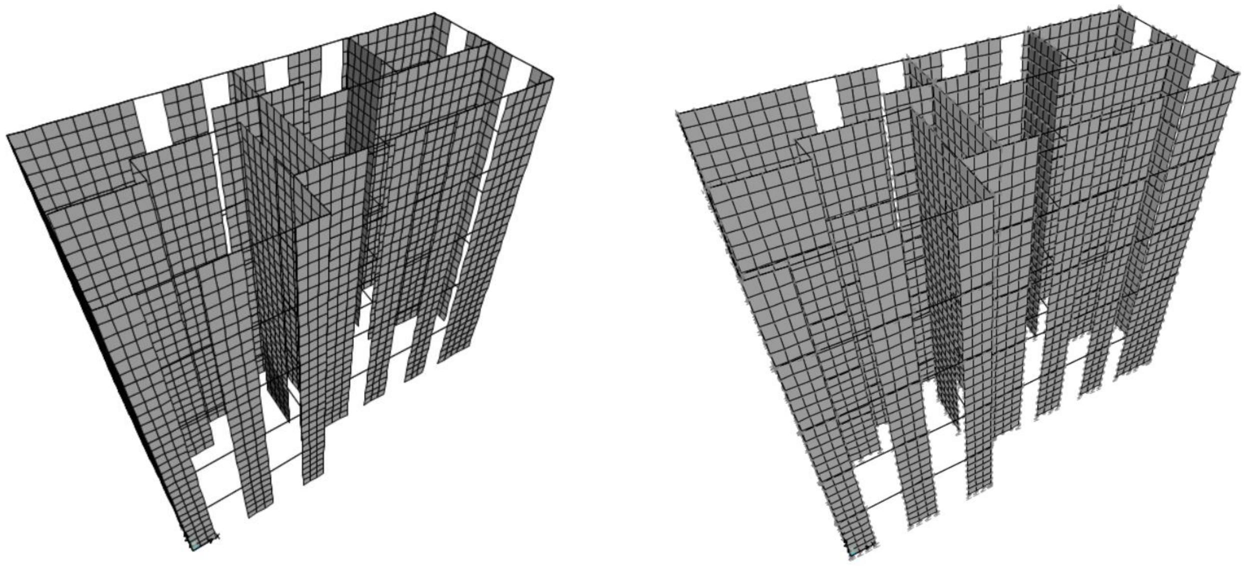


Figure I.2-15 – 3D view of 5 storeys FE model implemented with PM (left) and CM (right) approaches

Table I.2-15 reports the results obtained from CM approach and the variations (in brackets) respect to the simplified PM approach on the 3 parameters deemed significant.

Table I.2-15 – Results of numerical analyses implemented in CM approach and variations respect to PM approach

	3 Storeys		**5 Storeys**		**7 Storeys**	
	X direction	Y direction	X direction	Y direction	X direction	Y direction
Principal elastic period T_1 [s]	0.41 (9%)	0.49 (6%)	0.71 (8%)	0.91 (10%)	1.40 (3%)	1.95 (3%)
Base shear force [kN]	681.2 (3%)	612.2 (2%)	762.1 (1%)	563.7 (8%)	920.6 (0%)	643.6 (2%)
Top displacement [mm]	23.1 (9%)	27.1 (13%)	49.1 (10%)	50.9 (13%)	94.4 (6%)	108.5 (6%)

As can be seen from the table above, the variations on results between the two types of modelling strategies are less than 15%.

I.2.3.3 Implementation of the case study in commercial software Sismicad

The same case study presented in Section I.2.3.2 was implemented in commercial software Sismicad [120] in order to compare results obtained using the research-oriented software Sap2000 [119]. PM methodology of Figure I.2-13 was applied to characterize the seismic behavior of 3,5 and 7 story's CLT buildings. Response spectrum analyses were conducted assigning to CLT walls the equivalent elastic modulus reported in Table I.2-13. In Figure I.2-16 is reported a 3D view of 3-5 and 7 story's CLT buildings implemented in Sismicad.

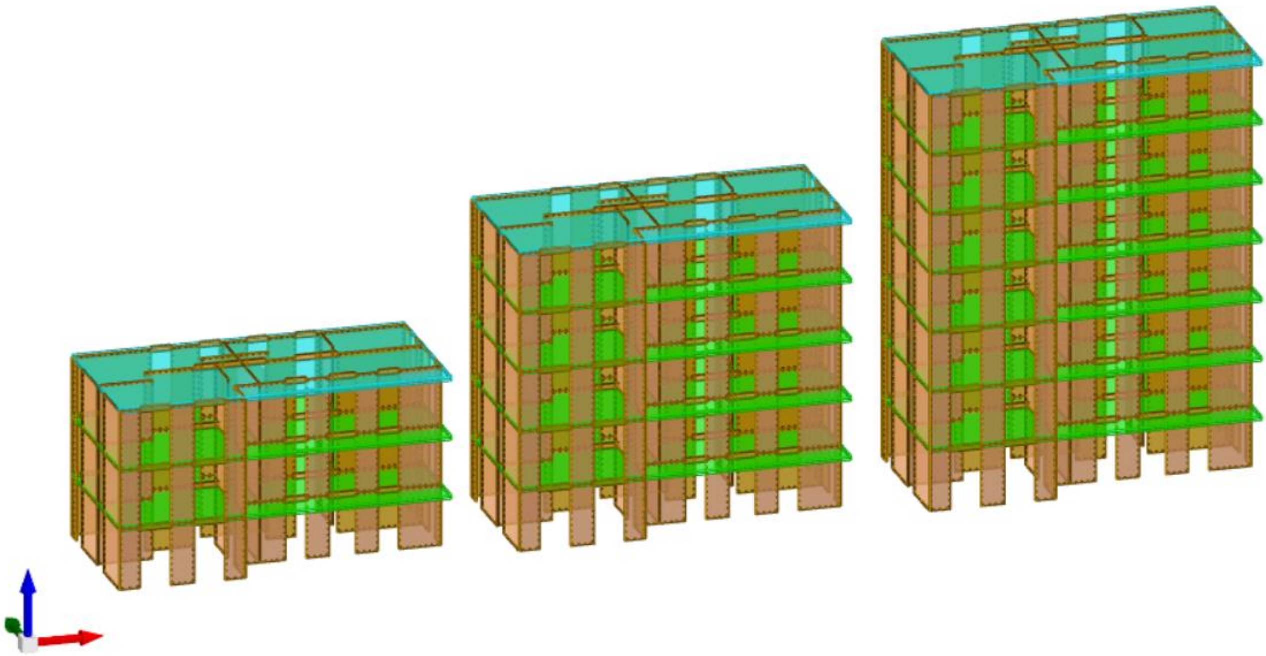


Figure I.2-16 – 3D view of 3-5-7 Storey's CLT buildings implemented with PM approach in FE commercial software Sismicad

Table I.2-16 reports the results obtained from response spectrum analyses implemented in FE software Sismicad and the variations (in brackets) respect to the Sap2000 software on the 3 parameters deemed significant.

Table I.2-16 – Results of numerical analyses implemented in FE software Sismicad and variations respect to Sap2000 software

	3 Storeys		**5 Storeys**		**7 Storeys**	
	X direction	Y direction	X direction	Y direction	X direction	Y direction
Principal elastic period T1 [s]	0.44 (0%)	0.52 (0%)	0.77 (0%)	1.02 (1%)	1.44	1.99 (0%)
Base shear force [kN]	702.1 (1%)	709.5 (1%)	738.9 (2%)	528.1 (2%)	915.1 (1%)	650.8 (0%)
Top displacement [mm]	25.1 (0%)	31.0 (1%)	53.8 (1%)	58.0 (2%)	99.9 (1%)	116.1 (0%)

As can be seen from the table above, the variations on results between the two different types of modelling software (commercial software Sismicad and research software Sap 2000) are equal or less than 2%. It is important to highlight how commercial software are generally preferred by practitioners thanks to their simplicity and their lower computational afford. These leads to a time saving both in the implementation and resolution phase, making the project timing compatible with those of practitioners. In order to further reduce the implementation phase, it is possible to divide CLT walls into different categories according to their slenderness, assigning an average slenderness to each category. In the following it is reported the example of the case study of Figure I.2-14 studied dividing CLT walls in 6 categories according to their length and consequently their slenderness. The hypothesized categories with the relative equivalent stiffness E_{eq0} to assign to CLT walls are reported in Table I.2-17.

Table I.2-17 – CLT wall categories with relative E_{eq1} of case studies analysed.

CLT WALL CATEGORY	Length range [m]	Length considered [m]	3 STOREYS		5 STOREYS		7 STOREYS	
			λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]	λ	E_{eq0}*t [N/mm]
TYPE 1	L≤1.5	L=1 m	9.00	174610	15.00	251782	21.00	328954
TYPE 2	1.5<L≤2.5	L=2 m	4.50	116731	7.50	155317	10.50	193903
TYPE 3	2.5<L≤3.5	L=3 m	3.00	97438	5.00	123162	7.00	148886
TYPE 4	3.5<L≤4.5	L=4 m	2.25	87792	3.75	107085	5.25	126378
TYPE 5	4.5<L≤5.5	L=5 m	1.80	82004	3.00	97438	4.20	112872
TYPE 6	L>5.5	L=6 m	1.50	78145	2.50	91007	3.50	103869

The values of the stiffnesses to be assigned to the walls of the upper floors of the structure ($E_{eq,n}$) was calculated according to Equation (1).

In Figure I.2-17 are highlighted with different colours the different categories of CLT shear walls considered in the analyses.

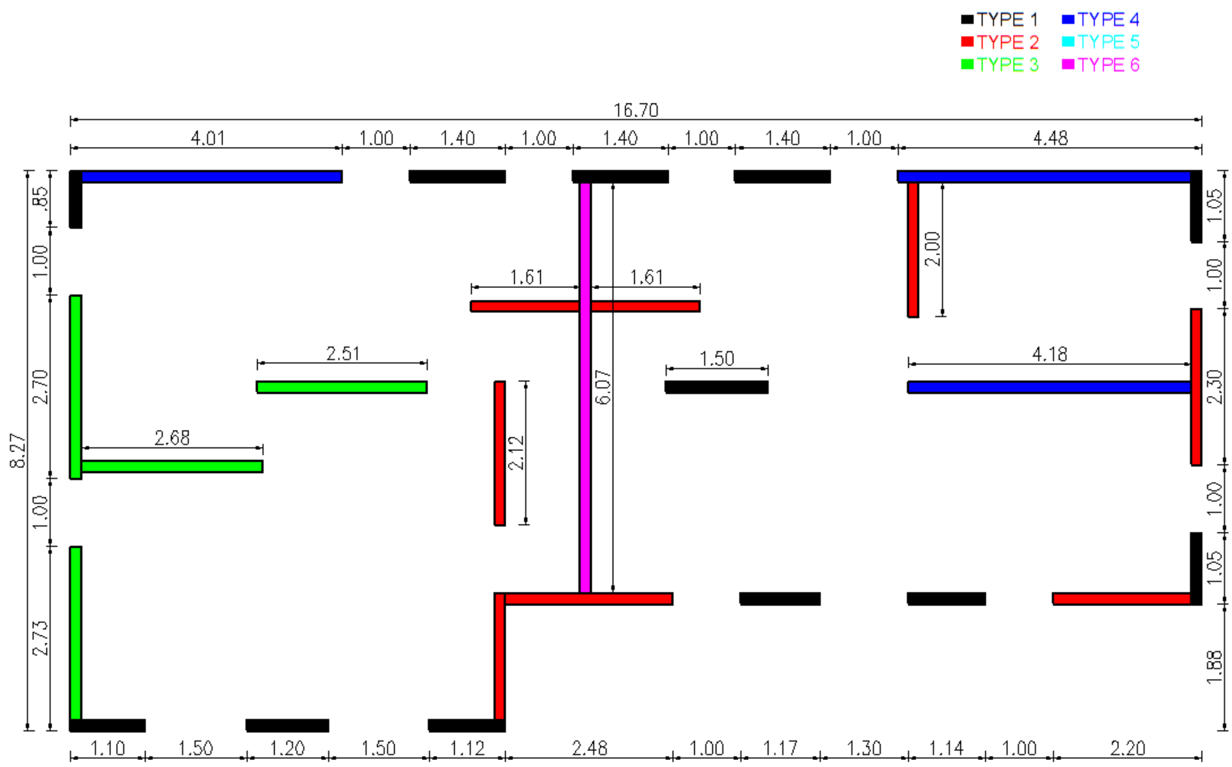


Figure I.2-17 – Different type of CLT shear walls considered

In Table I.2-18 are reported the results obtained from PM approach implemented considering the 6 categories of CLT walls reported in Table I.2-18 and the variations (in brackets) respect to PM approach implemented considering each wall with its relative equivalent stiffness.

Table I.2-18 – Results of numerical analyses implemented in PM approach considering 6 categories of CLT walls and and variations respect to PM approach implemented considering each wall with its relative equivalent stiffness

	3 Storeys		**5 Storeys**		**7 Storeys**	
	X direction	Y direction	X direction	Y direction	X direction	Y direction
Principal elastic period T_1 [s]	0.46 (5%)	0.50 (4%)	0.81 (5%)	1.00 (3%)	1.45 (0%)	2.01 (0%)
Base shear force [kN]	685.2 (3%)	680.1 (5%)	738.0 (2%)	504.4 (3%)	901.6 (3%)	625.1 (4%)
Top displacement [mm]	25.5 (1%)	30.7 (3%)	57.2 (7%)	56.0 (6%)	102.9 (2%)	120.3 (5%)

As can be seen from Table I.2-18, the variations on results is limited confirming the reliability of the proposed design procedure which results useful for structures characterized by a high number of floors and by complex plant.

I.2.4 Conclusions on design oriented phenomenological modelling approach for seismic design of multi-storey CLT buildings

In the section, two common modelling approaches for linear analyses of CLT buildings were analysed. Component level approach, typically used in research field, and simplified phenomenological approach, typically adopted by practitioners were investigated in detail. Results demonstrates that it is possible to reproduce the global seismic behaviour of a CLT building, in terms of principal elastic period T_1, base shear force and displacements, implementing a simplified phenomenological approach. For this purpose, it is essential to assign to CLT walls an equivalent stiffness able to consider both the connections and panel deformability. The calibration phase of equivalent stiffness demonstrates that it depends from different parameters (number of storeys, connection pattern, geometry and wall slenderness). A correlation between the equivalent elastic stiffness and the geometric proprieties of the structure was found thanks to least-squares method that minimizes the error on the fundamental parameters. Results have been included in an abacus, to correlate the equivalent stiffness of the wall at level zero (E_0) to the geometric proprieties of the wall, and in a formulation to correlate E_0 to the values of all the stiffnesses to be assigned to the walls of the upper floors of the structure (E_n). Phenomenological approach, thanks to its direct implementation in commercial software and its low computational afford, will be preferred by practitioners in the design phase over component level approach. The last is more accurate but requires detailed information about every component and involves high implementation times and a large computational afford.
Future analyses will be implemented with the aim of extending the research to cases where a vertical load is placed on the walls. The stabilizing contribution of the vertical load may be enough to prevent the rotation of the walls. In these cases, the deformation contribution linked to the tensile resistant connectors must be neglected, thus leading to more rigid systems and higher equivalent stiffnesses to be assigned to CLT walls.

I.3 Influence of the rocking behaviour of shearwalls on the fundamental period of CLT structures

Abstract

The comparison of results obtained from on-site modal tests and numerical analyses presented in recent studies showed that Cross Laminated Timber (CLT) buildings may exhibit a significant shift of fundamental period from the condition when rocking does not occur to the condition which the shearwalls rocking is activated for. The objective of the current study is to establish a relationship between the fundamental period of CLT structures and the lateral drift of their global dynamic response. The influence of the activation of rocking behaviour on the fundamental period was investigated by using experimental modal testing and Finite Element (FE) numerical analyses. The results from an experimental campaign, specially designed and conducted in order to study the influence of the stabilizing vertical load on the overall stiffness of the structure, were adopted to validate a FE numerical model used to perform an extended parametric analysis. The shift of the fundamental period from the value of period representing the condition for which the rocking of shearwall does not occur for is investigated via elastic non-linear incremental dynamic analyses. The effects of vertical load, stiffness and yield displacement of mechanical anchors and geometrical dimensions of CLT shearwalls are analysed and discussed.

An analytical expression to predict the maximum range of fundamental period which a CLT shearwall may exhibit under different levels of the dynamic lateral response is reported as function of the vertical load and the equivalent rocking slenderness. This formula can be adopted to validate the fundamental period adopted for the design a CLT building using linear-elastic Response Spectrum Modal analyses.

I.3.1 Introduction and literature review

The determination of dynamic properties is a key-factor in seismic design of structural systems when earthquake motion is represented by an elastic ground acceleration response spectrum. The calculation of vibration periods and modes is in fact an essential step for the calculation of seismic actions on the basis of linear-elastic behaviour of structures.

Numerical finite element (FE) models are commonly adopted to adequately represent the distribution of stiffness and mass in case of modal response spectrum (MRS) analyses. For structures whose dynamic response is not significantly affected by higher vibration modes, the lateral force method (LFM) of analysis can alternatively be used. Approximate expressions for the calculation of fundamental period as function of the total height of the buildings are generally reported in Standard Documents when the LFM of analysis is implemented [18,19].

The exploration of seismic dynamic response of Cross Laminated Timber (CLT) buildings has been the objective of several studies in recent years [121,122,123]. Experimental tests and numerical modelling have been conducted at wall- [10,20,124] and building-level [27,63,125] in order to identify the major deformation contributions for the dynamic response of CLT structures subjected to seismic loads.

CLT panels with no openings are characterized by high in-plane stiffness; as a result, the mechanical behaviour of CLT buildings is primarily governed by the mechanical properties of joints and mechanical connections used to connect the panels to each other and to foundation. The major deformation contribution is commonly related to rocking behaviour for CLT shearwalls with a relatively high height-to-length aspect ratio of CLT panels, whereas sliding governs in CLT panels with low aspect ratios [20,22].

Recently, Shahnewaz et al. [126] estimated the seismic fragility of CLT buildings through incremental dynamic analyses (IDA) by using a non-linear numerical model whereas Van de Lindt et al. [127] determined the seismic performance factors for CLT buildings in platform type according to the FEMA P695 methodology. A review of current challenges and emerging trends for the seismic design of timber buildings was presented by Pei et al., [121] and by Izzi et al. [73].

Several calculation approaches and FE modelling strategies have been proposed to adequately represent the lateral stiffness of CLT shearwalls [128]. A general agreement has been achieved for the calculation of panel deformation and wall sliding. Different proposals have been conversely presented for determining the rocking behaviour of single- and multi-panel CLT shearwalls. The major differences are attributed to angle brackets response, i.e. uni- vs bi-axial behaviour, and the definition of CLT panel contact area which governs the lever arm of rocking behaviour of shearwalls [20,129,130].

In the analytical models proposed by Casagrande *et al.* [22,23], specific attention was paid to the stabilizing effect of the vertical load on the elastic behaviour of single- and multi-panel CLT shearwalls. Two different states were identified: the former, which occurs when the stabilizing moment due to the vertical load is higher than the rocking moment due to lateral loads; the latter, which takes place when the rocking behaviour is activated because of a relative-high lateral load acting on the shearwall. Only in this latter case, the deformation contribution of the mechanical anchors due to the uplift of the shearwalls should be taken into account in the calculation of the wall lateral stiffness.

The importance of adequately taking into account the deformation contribution of the mechanical anchors due to the uplift of the shearwalls in relation to the amplitude of the lateral displacement of CLT shearwalls was emphasised by Aloisio *et al.* [24]. Comparing the fundamental frequencies obtained from ambient-vibration modal tests on a 8-storey CLT building with those calculated from a numerical model, it was observed that the deformation contribution related to the rocking behaviour of shearwalls should be neglected in case of low-amplitude dynamic response. Similar outcomes were found by Reynolds et al. [87]: a good match was observed between the fundamental frequencies obtained from an ambient-vibration low-amplitude modal test on a 5-storey CLT building and those obtained from a FE model where the contribution of the mechanical anchors was not implemented and the flexibility of CLT shearwalls was only represented by the panel deformation.

Despite the evidence that CLT structures may be characterized by a significant shift of the lateral stiffness and the fundamental period from the state where rocking does not occur (no-rocking state) to the condition when the rocking is activated (rocking state), research dealing with the influence of the rocking behaviour of CLT shearwalls on the building dynamic properties is still incomplete. Limited information are available on how the deformation contribution of the mechanical anchors due to the uplift of shearwalls should be properly considered in modal analyses of CLT buildings in relation to the stabilizing effect of vertical loads.

The objective of the current study is to establish a relationship between the fundamental period and the lateral drift in the global dynamic response of CLT structures. The influence of the activation of the rocking behaviour on the fundamental period was investigated by using experimental testing and numerical analyses. Modal tests were performed on a full-scale mock-up characterized by two single-storey CLT shearwalls to determine the lateral stiffness and fundamental period under different amplitudes of the dynamic lateral response.

The results from the experimental campaign were adopted to validate a FE numerical model used to perform parametric analyses on representative CLT shearwall configurations. The shift of the fundamental period of a multi-storey CLT shearwall from the "no-rocking" to the "rocking" state was investigated via elastic non-linear dynamic incremental analyses. Reference values of fundamental period which should be adopted in linear-elastic RSM analyses of CLT buildings are presented. The effects of vertical load, stiffness of mechanical anchors and geometrical dimensions of CLT shearwalls are analysed and discussed.

I.3.2 Modal tests

In order to investigate the influence of the rocking behaviour activation on the lateral stiffness and the fundamental period of CLT structures, modal tests were carried out on two parallel full-scale single-storey CLT shearwalls at the CIRI-EC laboratory of the University of Bologna (Italy). Test set-up, procedure and results are presented and discussed in this section.

I.3.2.1 Test set-up

Two parallel full-scale single-panel CLT shearwalls with length and height equal to 1.6 m and 2.8 m, respectively, were used as lateral load resisting system (LLRS) in the longitudinal direction of a 6.00x2.42x3.12 m timber mock-up, see Figure I.3-1.

The floor, made with a 160 mm thick CLT panel of length and width equal to 2.1 m and 6.0 m, respectively, was supported by the two CLT shearwalls and a timber frame made with 160x320 mm glulam columns and 160x160 mm glulam beams, see Figure I.3-2.

Figure I.3-1 – timber mock-up for modal tests

The CLT shearwalls consisted of five-ply 100 mm thick CLT panels, manufactured using spruce timber boards. Four steel plates were used to anchor each CLT shearwall to the foundation structures made with 320x160 mm glulam beams constrained to the strong floor of the laboratory by means of 30 mm diameter bars, see Figure I.3-2b.

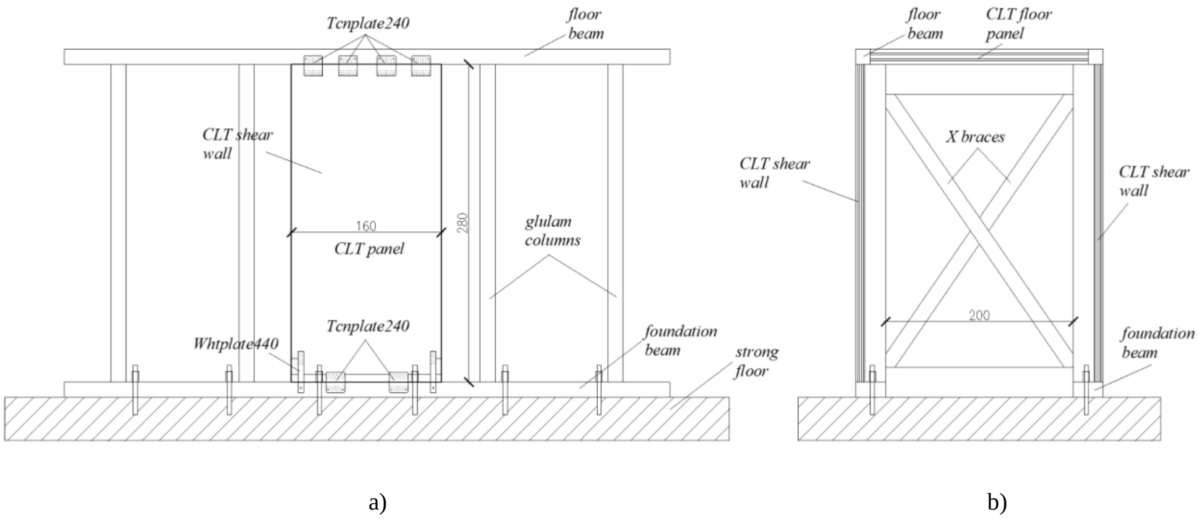


Figure I.3-2 – CLT wall and timber frame as support of the CLT floor (a); connections between the CLT walls and foundation/floor elements (b)

Two Whtplate440 steel plates were connected at each bottom corner of the walls by means of twenty-four 4x60 mm annular-ringed shank nails. The same number and typology of nails was used to connect two Tcnplate240 steel plates located along the base of the walls. 12 mm and 16 mm bolts were used to anchor the Whtplate440 and Tcnplate240 steel plates to the foundation beams, respectively. Four Tcnplate240 steel plates were used to transfer the horizontal actions from the floor element to the two CLT shearwalls.

The LLRS in the transversal direction consists of two concentric X-braces with section of glulam timber diagonals equal to 160x160 mm, see Figure I.3-3. Two15 kN masses and a 10 kN shaker were located on the floor panel, see Figure 3, for a total additional mass on the mock-up equal to 40 kN.

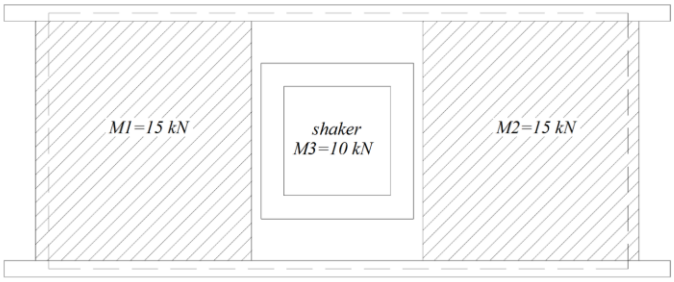


Figure I.3-3 – Layout of additional masses

I.3.2.2 Test procedure and instrumentation

Fifteen quick-release (also known as pull-back) tests were performed in order to characterize the dynamic properties of the mock-up in the direction parallel to the CLT shearwalls for different levels of the lateral response amplitude.

Firstly, a quasi-static pull force was applied at the floor level by means of a hydraulic jack connected to the floor structure through a steel wire, see Figure I.3-4. A quick–release, obtained from the instantaneous cut of the wire, was then applied in order to obtain a free vibration of the structure.

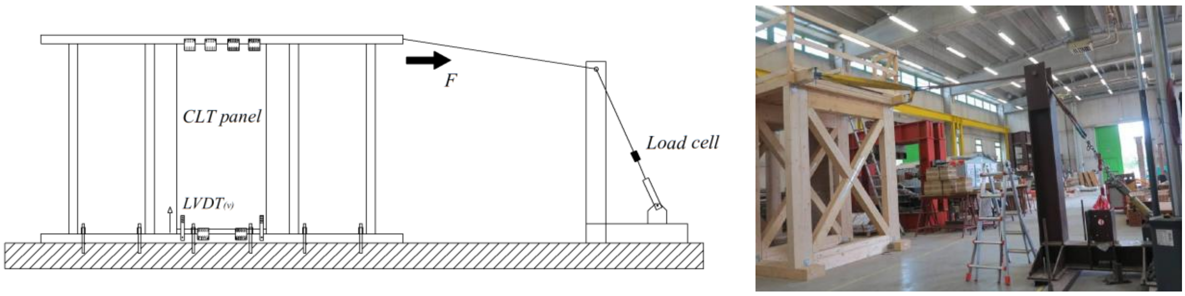


Figure I.3-4 – Quick-release tests

Different values of the lateral force *F* were applied to impose fifteen different values (between 1 and 8 mm with increasing step of 0.5 mm) of the initial lateral displacement *s* on the mock-up. An additional test was performed to determine the dynamic properties of the timber frame only.

Two LVDTs (A and B) and four accelerometers (acc0, acc2, acc3 and acc4) were used to measure the horizontal displacements and the absolute accelerations of the structure at the floor level along the longitudinal direction (parallel to the two CLT shearwalls), respectively, see Figure I.3-5. Two accelerometers (acc1 and acc4) were used along the transversal direction as well. A vertical LVDT was used to measure the uplift of the bottom corner of each CLT shearwall due to the activation of the rocking behaviour. A 100 kN load cell was adopted for the horizontal load imposed by the hydraulic jack.

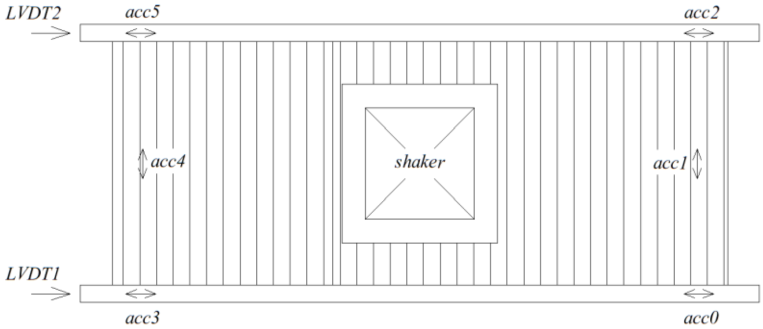


Figure I.3-5 – Layout of instrumentation – LVDTs and accelerometers

After the quick-release tests, two shaker tests with stepped-sine technique were also performed. A 1-ton shaker was anchored to the floor panel by means of four M24 threaded bars, Figure I.3-6. The tests were conducted by imposing two different values of the phase shift α between the two rotating masses, namely equal to 172°and 176°. The excitation frequency was stepped between 2 Hz and 8.5 Hz with incremental steps of 0.1 Hz (steps equal to 0.2 Hz were used in the range of frequencies far from resonance condition).

In order to avoid any structural damage of the mock-up, the exciting dynamic force, see Equation 1, was limited in order to impose a lateral displacement lower than 4 mm.

$$F = 1026\, f^2 \cos\left(\frac{\alpha}{2}\right) \;\; [N] \qquad (1)$$

where *f* represents the excitation frequency. The same layout of accelerometers adopted for quick-release tests was used for the two shaker tests.

Figure I.3-6 – 1-ton shaker on the CLT flor panel

I.3.2.3 Results and discussion

The lateral stiffness *K* of the structural system along the longitudinal direction was calculated from the pull phase of the quick-release tests as the ratio between the horizontal pull load *F* and the controlled initial lateral displacement *s* at the top of the mock-up, as reported in Table I.3-1. The initial uplift *v* of the bottom corner of each CLT shearwall due to the activation of the rocking behaviour is reported as well.
The results show a decrease of the lateral stiffness with the increase of the amplitude of the lateral displacement due to the activation of the rocking behaviour of the CLT shearwalls, as shown by the corresponding non-null

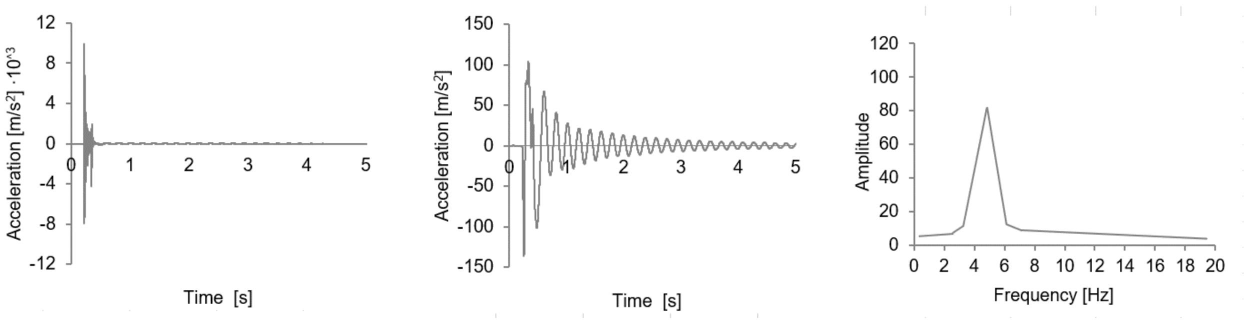
values of *v*. An almost constant value of the lateral stiffness, consistent with a linear-elastic behaviour of the structure, is observed for values of *s* higher than 5 mm.

Table I.3-1 – Lateral Stiffness obtained from the pull phase of the quick-release tests

Structure	*s*	*F*	*K*	*v*
	[mm]	[kN]	[kN/mm]	[mm]
Mock-up	1.0	5.94	5.94	0.00
	1.5	7.42	4.95	0.04
	2	8.61	4.31	0.09
	2.5	9.66	3.86	0.15
	3	10.40	3.47	0.23
	3.5	11.14	3.18	0.32
	4	12.18	3.05	0.43
	4.5	13.22	2.94	0.53
	5	14.11	2.82	0.65
	5.5	14.85	2.70	0.83
	6	16.34	2.72	1.07
	6.5	17.82	2.74	1.23
	7	17.82	2.55	1.46
	7.5	21.53	2.87	1.68
	8	22.28	2.79	1.84
Timber-frame	5	4.00	0.80	-

The acceleration signals measured during the release phase were post-processed to determine the fundamental frequency and period of the mock-up. The signals were filtered between 0 and 20 Hz and Fast Fourier Transforms (FFT) were used to convert the signal from time- to frequency-domain, see Figure I.3-7.

As for the lateral stiffness, a shift in frequency from values of initial lateral displacement lower than 4 mm to values higher than 6 mm was observed, as shown in Figure I.3-8. As expected, the activation of the rocking behaviour of shearwalls caused a reduction of fundamental frequency of the system from 4.8 Hz (i.e 0.21 s), for values of the lateral displacement lower than 4 mm, to 3.6 Hz (i.e. 0.28 s) for values of the lateral displacement higher than 6 mm.



a) b) c)

Figure I.3-7 – Post-processing of acceleration signals: measured a); filtered b) and FFT c) signals

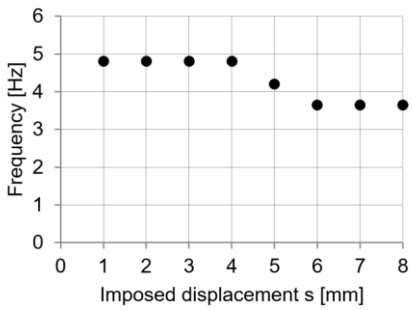
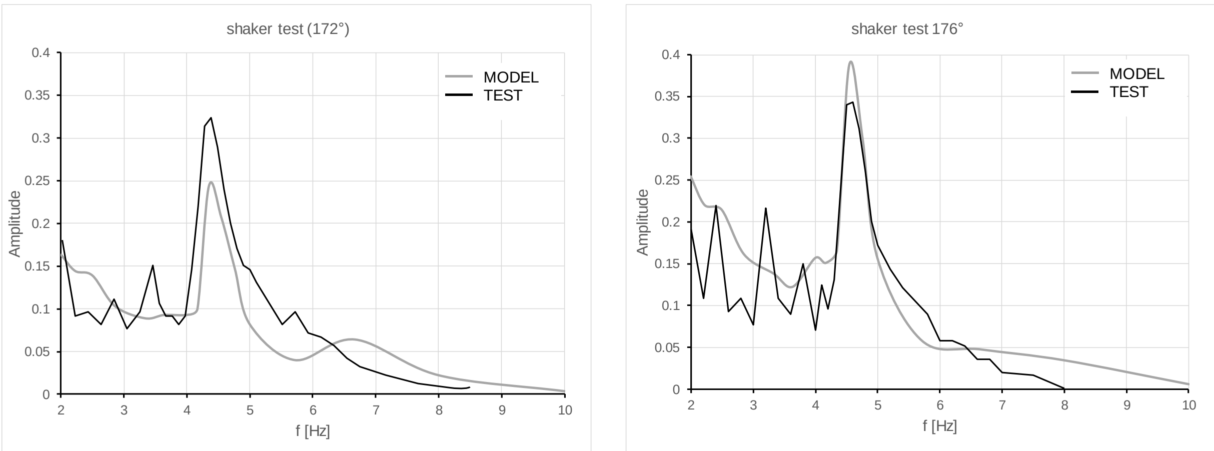


Figure I.3-8 – frequency values obtained from experimental tests

A fundamental frequency equal to 4.3 Hz (i.e. 0.23 s) and 4.6 Hz (i.e. 0.22 s) was detected from the Frequency Response Functions (FRFs) of the two shaker tests, see Figure I.3-9. The lateral displacement output signals for shaker tests were obtained by means of a double-integration of the acceleration signals.

The values of the measured fundamental frequency are consistent with those obtained from the quick-release tests with an initial lateral displacement lower than 4 mm due to the non-activation of the rocking behaviour of the two CLT shearwalls. In correspondence of the resonance frequency a maximum amplitude of the lateral displacement equal to 3.1 mm was in fact measured during the shaker tests.

The results obtained from the free (quick-release) and forced (shaker) modals tests showed hence a clear influence of the rocking behaviour of CLT shearwalls both on lateral stiffness and fundamental frequency.



a) b)

Figure I.3-9 – Fundamental frequencies obtained from shaker tests and numerical analyses for angle phase of rotational masses equal to 172° (a) and 176° (b)

I.3.3 Numerical Modeling

I.3.3.1 Model description

A FE numerical model was implemented in the commercial software Sap 2000 [119] to reproduce the dynamic behaviour of the tested structural system and presented in the previous section.

CLT panels were modelled by using four-node quadrilateral orthotropic homogenous 100 mm thick area elements, see Figure I.3-10. Effective moduli of elasticity $E_{eff,x} = 6600\ MPa$, $E_{eff,z} = 4400\ MPa$ along the vertical and horizontal direction, respectively, were calculated according to Mestar *et al.* [131] assuming for the CLT laminations a modulus of elasticity parallel to grain E_0 equal to 11.000 MPa. An effective in-plane shear modulus $G_{ef} = 500\ MPa$ was assigned to the area elements in order to consider the effective shear and torsional deformation of laminations according to the formula proposed by [131], considering an in-plane shear modulus $G_{0,mean}$ of laminations equal to 690 MPa.

Elastic-linear link (spring) elements were adopted to model the steel plates used to connect the CLT shearwalls to the foundation and the timber frame elements. A uni-axial behaviour along the vertical z-direction of link element (H) was assumed for Whtplate440 steel plate with a vertical-tensile stiffness $k_{v.z}$ equal to 6115 kN/m. For Tcnplate240 steel plates (A) a bi-axial behaviour was considered with the same value of stiffness $k_{a.x}$ and $k_{a.z}$ along the horizontal x- and vertical-tensile z-direction equal to equal to 6220 kN/m. The values of stiffness assumed for Whtplate440 and Tcnplate240 were determined by multiplying the number of nails by the value of stiffness of a single nail obtained from the experimental tests presented by Casagrande et al.[21] on similar mechanical anchors, namely Wht620 (characterized by 52 nails and a vertical-tensile stiffness equal to 13250 kN/m) and Tcf240 (connected by 30 nails and a shear-horizontal stiffness equal to 8480 kN/m), respectively. vertical uni-axial rigid gap elements (G) were used to simulate the contact between the CLT panel and the surrounding elements.

Frame elements were adopted for columns and beams. Flexible rotational springs with stiffness k_{rot} equal to 1850 kNm/rad were used to connect the column to the beam frame elements. The dead load of the CLT floor panel and the additional masses were implemented in the model as uniform distributed vertical load q on the two longitudinal frame elements representing the top beams. A diaphragm constrain was applied at the joints of the floor level.

Due to the non-linear response of the gap elements, elastic non-linear analyses were performed in order to adequately take into account the activation of rocking behaviour and the stabilizing effect of vertical load.

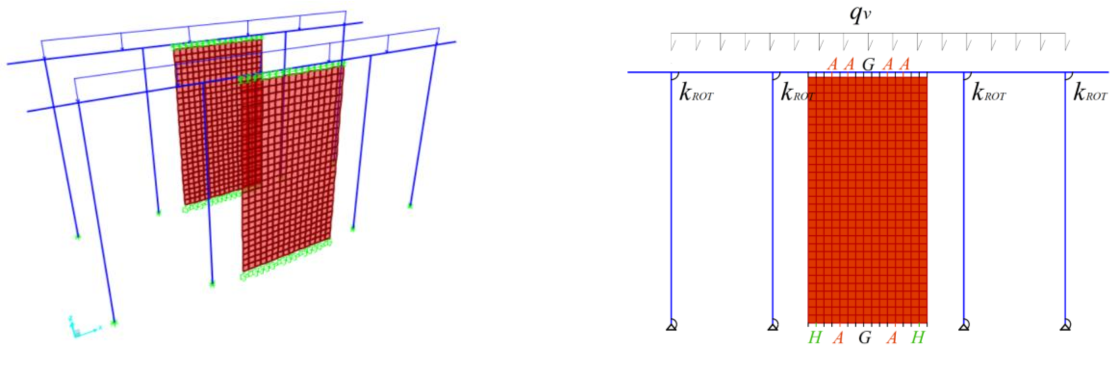
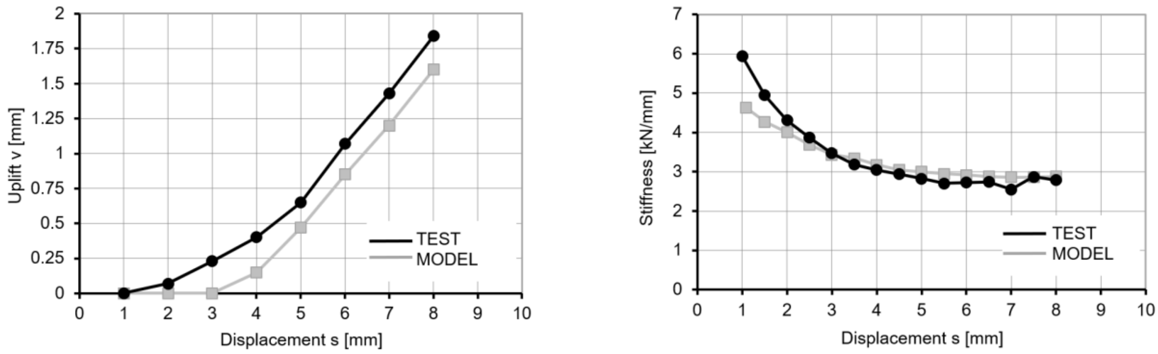


Figure I.3-10 – Finite Element numerical model of the mock-up

I.3.3.2 Model validation

Displacement-controlled elastic non-linear static analyses were performed to determine the lateral stiffness of the model. The same values of the initial displacement imposed in the pull phase of the quick-release tests were applied as static horizontal displacements to the floor diaphragm along the longitudinal direction. The lateral stiffness was measured as the ratio between the longitudinal base shear and the imposed lateral displacement.

A good match from the values obtained from the experimental tests and the numerical model was observed in terms of lateral stiffness and vertical uplift of the shearwall as shown in Figure I.3-11a and b, respectively. A small discrepancy was detected for lateral displacements lower than 3 mm due to the contribution of friction and other secondary elements not directly implemented in the model.



a) b)

Figure I.3-11 – Comparison between experimental tests and FE numerical model: lateral stiffness (a) and uplift of shearwalls (b)

Load-controlled elastic non-linear dynamic analyses were performed to determine the fundamental frequency and period of the model. A sinusoidal dynamic horizontal force with amplitude equal to that adopted for the shaker tests, see Equation 1, was applied at the center of the floor. The load frequency was stepped between 2 Hz and 8.5 Hz.

A fundamental frequency equal to 4.5 Hz (i.e. 0.22 s) was detected from the FRFs of the model. A good match was observed between the shaker tests and the numerical analyses as shown in Figure I.3-9. The lateral displacement corresponding to the resonance condition was equal to 3 mm.

In order to investigate the variation of fundamental frequency and period at different levels of amplitude of the lateral displacement response, the amplitude of the sinusoidal dynamic force was later increased by a factor equal to 2.5. A fundamental frequency equal to 3.7 Hz (i.e. 0.27 s) and a lateral displacement corresponding to model resonance equal to 8 mm were measured in this case.

A summary of the results obtained in terms of fundamental frequency and period from quick-release tests, shaker tests and FE analyses is reported in Table I.3-2. The results showed a reasonable match between the FE model and the experimental tests in both states of CLT shearwalls related to the non-activation and activation of the rocking behavior of CLT shearwall, respectively.

Table I.3-2 – Comparison between fundamental frequencies obtained from quick-release tests, shaker tests and FE analyses

Displacement response [mm]	Rocking behavior	Fundamental frequency [Hz] (fundamental period [s])		
-	-	Quick-release tests	Shaker tests	FE analyses
3	No rocking	4.8 (0.21)	4.3-4.6 (0.22-0.23)	4.5 (0.22)
8	Rocking	3.6 (0.28)	-	3.7 (0.27)

I.3.4 Elastic non-linear incremental dynamic analyses

Elastic non-linear incremental dynamic (ENLID) analyses were performed to evaluate the shift of the fundamental period of multi-storey CLT shearwalls from "no-rocking" to "rocking" state. Geometrical dimensions, magnitude of vertical load and stiffness of the mechanical anchors used to prevent the uplift of the shearwalls were used as input parameters in the numerical analyses. All analyses were carried out using the software Sap2000 [119].

As for the model used to reproduce the results of the experimental modal tests, elastic non-linear analyses were performed in order to adequately take into account the activation of the rocking behaviour of CLT shearwalls. Eigenvector Analyses are in fact always linear and, for this reason, are not able to consider the non-linear response of the gap elements unless that the stiffness matrix, obtained at the end of a non-linear analysis, is adopted to calculate the dynamic properties under different lateral load conditions.

I.3.4.1 Methodology

A sinusoidal horizontal input signal was used as ground acceleration for the excitation of a CLT multi-storey shearwall as expressed by Equation 2:

$$a_g(t) = A_g \cdot \sin\left(2 \cdot \pi \cdot f_g \cdot t\right) \qquad (2)$$

where A_g and f_g are amplitude and frequency of the input signal, respectively. As in a shaker test, the frequency of the input signal f_g was stepped in the range close to the fundamental frequency f of the structure.

The dynamic response was represented in this study by the steady-state output signal of the horizontal displacement at the top the shearwall, as expressed by Equation 3:

$$d(t) = D \cdot \sin\left(2 \cdot \pi \cdot f_g \cdot t\right) \qquad (3)$$

where D is the amplitude of output signal.

The Frequency Response Function $H(f_g)$ was calculated as the ratio between amplitude of output and input signal in the frequency domain as function of the excitation frequency f_g, as expressed by Equation 4. The peak value of teach FRFs was used to identify the fundamental frequency f of the shearwall.

$$H(f_g) = \frac{D(f_g)}{A_g(f_g)} \qquad (4)$$

In order to establish a relationship between the fundamental frequency f (or fundamental period T) with the amplitude of the lateral response d due to the activation of the rocking behavior, the amplitude of the input signal A_g was scaled from zero to the value for which the hold-down at the ground floor reaches its yield strength. For each value of A_g the FRF and the fundamental frequency of the shearwall were determined, see Figure I.3-12.

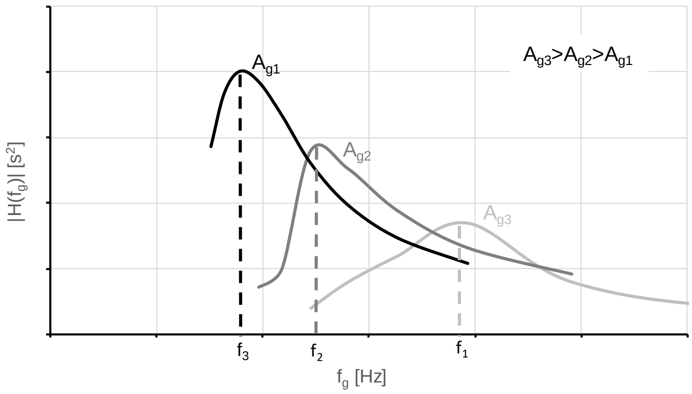


Figure I.3-12 – FRFs for different values of the amplitude of the input signal

For each shearwall, the fundamental period T, defined as the inverse of the fundamental frequency, was plotted as function of the amplitude of shearwall lateral drift $\vartheta = \frac{d}{h}$, where h is the total height of the shearwall, as shown in Figure I.3-13a.

Due to the activation of the rocking behaviour of the wall, an increase of the fundamental period was observed with the increase of lateral drift ϑ. In particular, for non-null values of vertical loads, three regions can be distinguished:

1) A "no-rocking" region characterized by a constant value of the fundamental period T equal to T_{nr}. For values of the lateral drift lower than ϑ_{nr} the rocking behaviour is not activated, see Figure I.3-14a.

2) A "full-rocking" region with a constant value of the fundamental period T equal to T_{fr}. For values of the lateral drift higher than ϑ_{fr} the rocking behaviour is activated at all storeys along the height of the shearwall, see Figure I.3-14c.

3) A "partial-rocking" region where a shift of the fundamental period from the lower value T_{nr} to the upper value T_r is observed due to a partial activation of the rocking behaviour which occurs only at the lower storeys of the shearwall, see Figure I.3-14b.

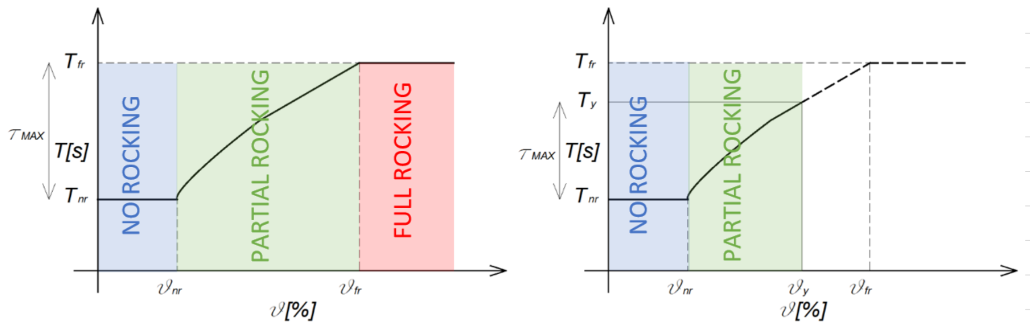


Figure I.3-13 – period vs lateral drift curves where full-rocking behavior is achieved (a) or limited by yielding of hold-down (b)

It is noteworthy to mention that the full activation of the rocking behaviour along the height of the shearwall may be limited by the yielding of the hold-down at the ground level. The activation of the rocking behaviour at upper storeys may require, in fact, values of the lateral drift ϑ higher than those which the hold-down at the ground storey yields for. For these cases, since the evaluation of fundamental period is to be performed by assuming an elastic behaviour of structures, a "full rocking" behaviour along the entire height of the wall might not be achieved. The upper value of the fundamental period is defines in this case as $T_y < T_{fr}$. The corresponding value of the lateral drift which at least one of hold-down yields for is defined as ϑ_y, see Figure I.3-13b.

Introducing a normalized period $\tau(\vartheta)$ defined as $\frac{T(\vartheta)}{T_{nr}}$, we get $\tau(\vartheta) = 1$ for the "no-rocking" region and $\tau(\vartheta) > 1$ for "full-" and "partial-rocking" regions. The upper value of the dimensionless period, τ_{up}, represents the maximum shift from the no-rocking state of the fundamental period which a certain CLT shearwall may exhibit in relation to the lateral drift amplitude.

τ_{up} is defined as the ratio between the upper and lower value of the fundamental period as expressed by Equation 5, and estimates the range of the fundamental periods in which the reference value of period adopted in linear-elastic RSM analyses should be included.

$$\begin{aligned} \tau_{up} &= \tau_{fr} = \frac{T_{fr}}{T_{nr}} \quad \text{when} \quad \vartheta_{fr} \leq \vartheta_y \\ \tau_{up} &= \tau_y = \frac{T_y}{T_{nr}} \quad \text{when} \quad \vartheta_{fr} > \vartheta_y \end{aligned} \tag{5}$$

For null values of the vertical loads, since no stabilizing effect exists, the rocking behaviour is always fully activated, even for low-amplitude values of the lateral response. As a result, the fundamental period is constant and equal to T_{fr}.

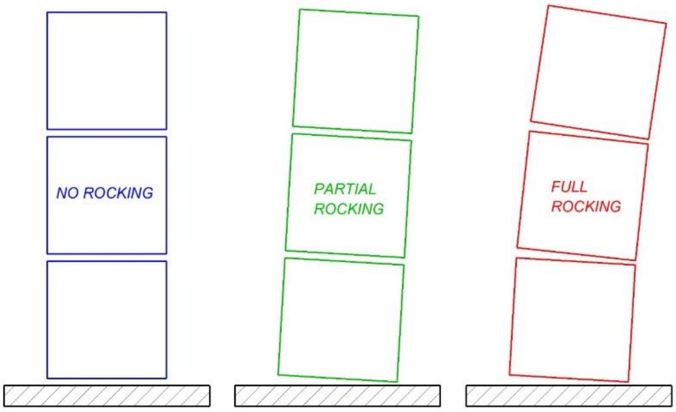


Figure I.3-14 – no-rocking, partial rocking and full rocking states of a CLT multi-storey shearwall

I.3.4.2 Model description and parametric analyses

Parametric numerical analyses were performed to evaluate the influence of geometrical dimensions, magnitude of the vertical load and stiffness of the mechanical anchors on the variation of the fundamental period due to the rocking behaviour of CLT shearwalls. A total of 87 case studies were investigated.

Single-, three- and five-story single-panel 100 mm thick CLT shearwalls were analyzed with an inter-story height h_s equal to 3 m. Three different values of the wall length l were chosen, namely equal to [2; 4; 6] m.

The CLT shearwalls were modelled in SAP2000, see Figure I.3-15, using the same strategy adopted for the modelling of mock-up experimentally tested. Four-node quadrilateral orthotropic homogenous area elements with effective moduli of elasticity $E_{eff,z} = 6600\ MPa$, $E_{eff,x} = 4400\ MPa$ along the vertical and horizontal direction and an effective in-plane shear modulus $G_{ef} = 500\ MPa$ were used.

Uni-axial elastic-linear link (spring) elements were used to model the mechanical anchors. The hold-downs, adopted to prevent the uplift of the wall, were considered effective along the tensile-vertical direction and were located on the two corners of each CLT panel. The angle brackets used to prevent the sliding of the shearwall were considered acting only along the horizontal shear direction and were uniformly distributed along the CLT panels. The number of angle brackets n_A was assumed equal to 2, 4 and 6 for values of shearwall length equal to 2, 4 and 6 m respectively.

Three different values of the vertical-tensile stiffness of the hold-down k_H were selected, namely equal to [5000; 10000;15000] kN/m, in the common range of values reported in literature [21]. Since the evaluation of the fundamental period is to be performed by assuming an elastic behaviour of mechanical anchors, the analyses were carried out for an amplitude of the input signal A_g from zero to the value such as the hold-down at the ground floor reaches its yield displacement $v_{H,y}$, assumed in this study equal to $v_{H,y}$ = 10 mm.

The value of the horizontal stiffness k_A of the link representing the angle brackets was set equal to 5000 kN/m. A gap element with compression stiffness equal to 15000 kN/m was used to simulate the contact between the panels with the foundation/the panels below.

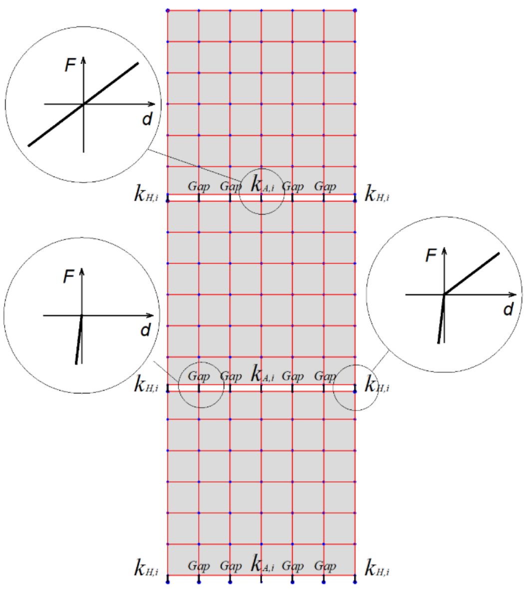


Figure I.3-15 – FE model of multi-storey shearwall for parametric analyses

A dead and live load per unit area of the floor equal to 3.3 kN/m^2 and 2 kN/m^2, respectively were adopted for the calculation of the vertical load acting on the shearwall. Three different values of the tributary length of floor elements equal to 0.0 m, 2.5 m and 5.0 m were selected, giving a uniform distributed vertical load q acting at each storey equal to 0.00 kN/m, 9.75 kN/m and 19.5 kN/m. The total vertical load per unit length acting at the ground level normalized on the axial compressive strength per unit length q_R of the CLT panes was introduced as reported in Equation (6):

$$\tilde{q} = \frac{q \cdot n}{q_R} \quad (6)$$

where n is the number of the storeys. q_R was calculated as the product of the design compressive strength parallel to the grain, assumed equal to 8690 kN/m^2 and the total thickness along the vertical direction of the CLT wall equal to 60 mm (three 20 mm thick vertical layers), getting $q_R = 521\ kN/m$. A summary of the values of input parameters is reported in Table I.3-3.

Table I.3-3 – Comparison between fundamental frequencies obtained from quick-release tests, shaker tests and FE analyses

Parameter		Input values
Number of storeys	n	[1;3;5]
Length of shearwall	l	[2;4;6] m
Inter-storey height	h_s	3 m
Total height of the CLT shearwall	h	[3;6;9] m
Number of hold-down per each corner per panel	n_H	1
Number of angle brackets per panel	N_A	1 when l =2 m 2 when l =4 m 3 when l =6 m
Vertical tensile stiffness of hold-down	k_H	[5000;10000;15000] kN/m
Vertical yield displacement of hold-down	$v_{y,H}$	10 mm
Horizontal shear stiffness of angle-bracket	k_A	5000 kN/m
Floor dead load per unit area	G	3.3 kN/m^2
Floor live load per unit area	Q	2 kN/m^2
Tributary length of floor elements	l_T	[0;2.5;5] m
Vertical load per unit length at each storey	q	[0.00;9.75;19.50] kN/m
Normalized vertical load per unit length at ground level	$\tilde{q}$	[0.0;1.9;3.7] % for n=1 [0.0;5.6;11.2] % for n=3 [0.0;9.3;18.7] % for n=5

I.3.4.3 Results and discussion

The results obtained from parametric analyses and reported in Tables I.3-4,I.3-5 and I.3-6 and shown in Figure I.3-16 in terms of lateral drift ϑ and normalized period $\tau(\vartheta)$ for non-null (i.e. $\tilde{q} > 0$) values of the normalized vertical load, respectively, showing that:

- No- rocking behaviour is shown for values of the lateral drift smaller than 0.07%, 0.17% and 0.18% for 1-, 3- and 5 –storey respectively.

- a full-rocking behaviour is achieved for 1–storey shear walls while for most of 3- and 5-storey shearwalls the full activation of the rocking behaviour is limited by the yielding of the hold-down at the ground level, i.e. $\vartheta_y < \vartheta_{fr}$. For this reason, the fundamental period related to a full rocking behaviour seems to be inadequate to represent the dynamic behaviour of a multi-storey CLT shearwall.

- the value of the lateral drift ϑ_{nr} related to the no rocking region is almost independent on the length of the wall, while the values of lateral drift $\vartheta_{fr}, \vartheta_y$ and the corresponding normalized period τ_{fr}, τ_y related to the full rocking behaviour and the yielding of hold-down at the ground level, respectively, decrease with l, see Figure I.3-16a.

- increasing the vertical load, the value of the normalized period decreases. The value of the lateral drift ϑ_{nr} which limits the no-rocking region decreases as well. From Figure I.3-16b it can be observed that the two curves related to two different values of $\tilde{q}$ are almost parallel. In case of a null value of vertical load, only the full rocking behaviour is observed, with a constant value of the normalized period equal to τ_{nr}.

- increasing the vertical tensile stiffness of the hold-down k_H, the values of lateral drift $\vartheta_{nr,}\vartheta_{fr}$ and ϑ_y are almost constant whereas the normalized period τ decreases as shown in Figure 16c.

- the higher the shearwall, the higher the values of the lateral drift $\vartheta_{nr,}\vartheta_{fr}$ and ϑ_y and the normalized period τ_y , τ_{fr}, As shown in Figure I.3-16d the curves related to different number of storeys almost overlap for low values of the lateral drift.

In Table I.3-7, the normalized fundamental period τ_{fr} is reported for null values (i.e. $\tilde{q} = 0$) of the vertical loads; a full-rocking behaviour is achieved for any value of the lateral drift in this case. As expected, with the increase of the length of shearwall and the increase of hold-down stiffness, the values of τ_{fr} decrease. A maximum value equal to 2.30 is observed for a shearwall of length equal to 4 m and hold-down stiffness equal to 5000 kN/m.

In order to take into account the influence of the global rocking stiffness of the wall on the maximum fundamental period shift τ_{up}, the equivalent rocking slenderness λ_{eq} is defined as expressed by Equation 7:

$$\lambda_{eq} = \frac{\lambda \cdot G \cdot t}{k_H} \quad (7)$$

where $\lambda = h/l$ is the geometrical slenderness of CLT wall, G is the shear modulus of CLT panel and t is the total thickness of CLT wall.

The maximum shift of the fundamental period $\tau_{up,}$ which a certain CLT shearwall may exhibit in relation to the lateral drift amplitude, is plotted as function of λ_{eq} for different values of $\tilde{q}$ as shown in Figure I.3-17. It is noteworthy to mention that τ_{up} does not represent the normalized period of the structure but it measures the upper value of the fundamental period for a CLT shearwalls behaving elastically under different lateral response amplitude as reported in Equation 5.

Table I.3-4 – Results of parametric analyses for non-null values of the vertical load – 1 storey

l	[m]	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6
k_H	kN/m	5E3	5E3	5E3	5E3	5E3	5E3	10E3	10E3	10E3	10E3	10E3	10E3	15E3	15E3	15E3	15E3	15E3	15E3
$\tilde{q}$	[%]	1.90	1.90	1.90	3.70	3.70	3.70	1.90	1.90	1.90	3.70	3.70	3.70	1.90	1.90	1.90	3.70	3.70	3.70
ϑ_{nr}	[%]	0.03	0.04	0.05	0.06	0.07	0.07	0.03	0.04	0.05	0.06	0.07	0.07	0.03	0.04	0.05	0.06	0.07	0.07
ϑ_y	[%]	-	0.43	-	0.73	-	-	-	-	-	-	0.57	-	-	-	-	-	-	-
τ_y	[-]	-	1.62	-	1.81	-	-	-	-	-	-	1.32	-	-	-	-	-	-	-
ϑ_{fr}	[%]	0.56	-	0.32	-	0.88	0.88	0.33	0.40	0.21	0.54	-	0.36	0.22	0.23	0.17	0.37	0.37	0.27
τ_{fr}	[-]	1.91	-	1.46	-	1.65	1.46	1.52	1.34	1.26	1.52	-	1.26	1.37	1.24	1.17	1.37	1.24	1.17

Table I.3-5 – Results of parametric analyses for non-null values of the vertical load – 3 storeys

l	[m]	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6
k_H	kN/m	5E3	5E3	5E3	5E3	5E3	5E3	10E3	10E3	10E3	10E3	10E3	10E3	15E3	15E3	15E3	15E3	15E3	15E3
$\tilde{q}$	[%]	5.60	5.60	5.60	11.2	11.2	11.2	5.60	5.60	5.60	11.2	11.2	11.2	5.60	5.60	5.60	11.2	11.2	11.2
ϑ_{nr}	[%]	0.08	0.08	0.10	0.17	0.17	0.12	0.08	0.08	0.10	0.17	0.17	0.12	0.08	0.08	0.10	0.17	0.17	0.12
ϑ_y	[%]	1.03	0.53	0.40	1.00	0.60	0.47	1.08	0.58	0.51	1.11	0.62	0.44	-	0.56	0.49	1.00	0.59	0.42
τ_y	[-]	1.95	1.59	1.43	1.90	1.46	1.29	1.66	1.45	1.41	1.60	1.30	1.20	-	1.39	1.28	1.40	1.28	1.15
ϑ_{fr}	[%]	-	-	-	-	-	-	-	-	-	-	-	-	0.92	-	-	-	-	-
τ_{fr}	[-]	-	-	-	-	-	-	-	-	-	-	-	-	1.51	-	-	-	-	-

Table I.3-6 – Results of parametric analyses for non-null values of the vertical load – 5 storeys

l	[m]	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6	2	4	6
k_H	kN/m	5E3	5E3	5E3	5E3	5E3	5E3	10E3	10E3	10E3	10E3	10E3	10E3	15E3	15E3	15E3	15E3	15E3	15E3
$\tilde{q}$	[%]	9.30	9.30	9.30	18.7	18.7	18.7	9.30	9.30	9.30	18.7	18.7	18.7	9.30	9.30	9.30	18.7	18.7	18.7
ϑ_{nr}	[%]	0.09	0.12	0.07	0.13	0.18	0.15	0.09	0.12	0.07	0.13	0.18	0.15	0.09	0.12	0.07	0.13	0.18	0.15
ϑ_y	[%]	1.27	0.64	0.47	1.30	0.69	0.51	-	0.67	0.49	1.37	0.72	0.54	-	0.63	0.47	1.30	0.66	0.51
τ_y	[-]	2.11	1.72	1.47	1.90	1.58	1.27	-	1.57	1.42	1.62	1.45	1.29	-	1.42	1.33	1.50	1.32	1.24
ϑ_{fr}	[%]	-	-	-	-	-	-	1.29	-	-	-	-	-	1.70	-	-	-	-	-
τ_{fr}	[-]	-	-	-	-	-	-	1.70	-	-	-	-	-	1.55	-	-	-	-	-

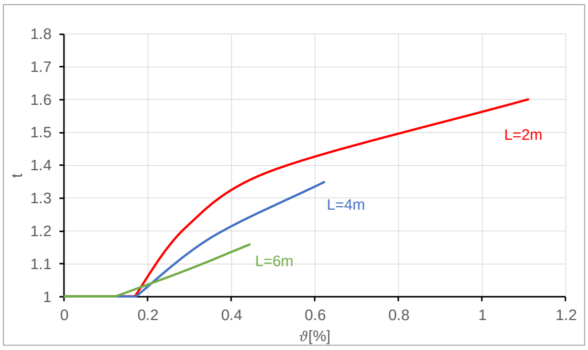
Table I.3-7 – Results of parametric analyses for non-null values of the vertical load

l	k_H	τ_{fr}		
		$n = 1$	$n = 3$	$n = 5$
[m]	[kN/m]	[-]	[-]	[-]
2	5000	1.91	2.22	2.25
4	5000	1.65	2.08	2.30
6	5000	1.46	1.91	2.20
2	10000	1.52	1.71	1.70
4	10000	1.34	1.63	1.77
6	10000	1.26	1.53	1.70
2	15000	1.37	1.51	1.55
4	15000	1.24	1.43	1.50
6	15000	1.17	1.37	1.50

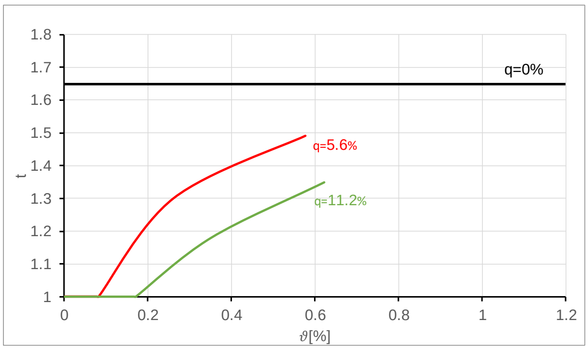
For 1-storey shearwall, maximum values of τ_{up} between 1.8 and 1.9 are observed for values of the equivalent rocking slenderness equal to 15. A negligible influence of the normalized vertical load is shown, especially for values of λ_{eq} lower than 10.

Values of τ_{up} between 1.4 and 2.2 are obtained in case of no vertical load for values of λ_{eq} equal to 5 and 45 respectively in 3-storey walls. A decrease of τ_{up} is observed with the increasing of the vertical load with values of τ_{up} equal to 1.9 and 1.8 for $\tilde{q}$ equal to 5.6% and 11.2%, respectively, and $\lambda_{eq} = 45$.

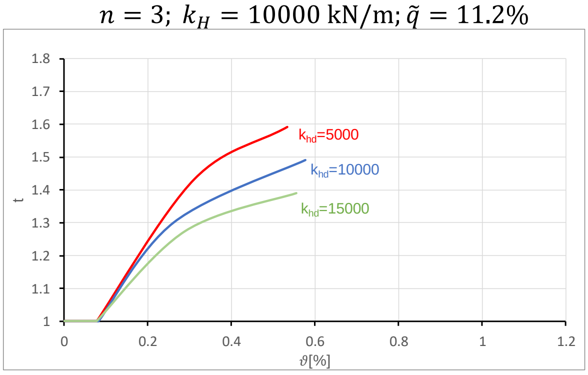
For 5-storey shearwalls the values of τ_{up} are between 1.5 and 2.3 for values of λ_{eq} equal to 8 and 75, respectively. Not great difference is observed in terms of values and trend of curves between 3- and 5-storey walls. In case of a normalized vertical load $\tilde{q}$ equal to 18.7% the values of τ_{up} are between 1.25 and 1.95.



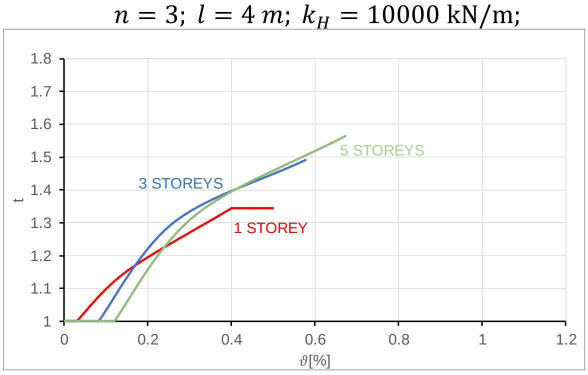
$n = 3;\ k_H = 10000$ kN/m; $\tilde{q} = 11.2\%$



$n = 3;\ l = 4\ m;\ k_H = 10000$ kN/m;



$n = 3;\ l = 4\ m\ ;\tilde{q} = 5.6\%$



$l = 4\ m;\ k_H = 10000$ kN/m; $\tilde{q} = 5.6\%$

Figure I.3-16 – normalized period vs lateral drift curves

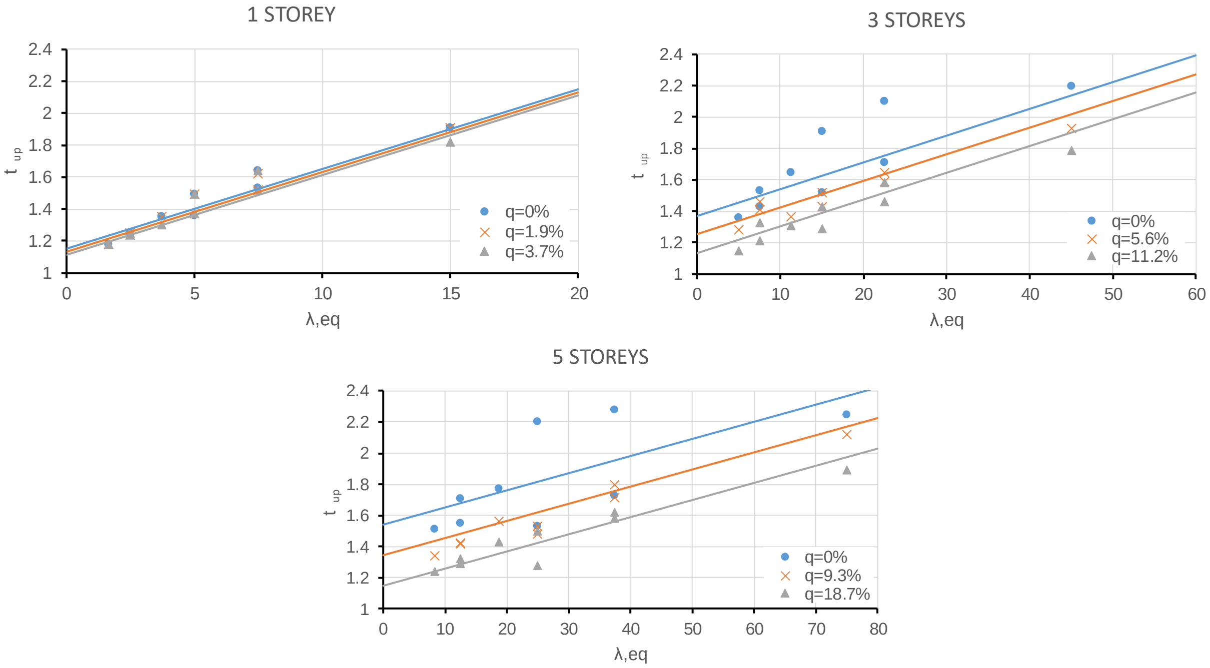


Figure I.3-17 – τ_up vs λ_eq curves for 1-, 3- and 5- storey shearwalls

An analytical expression, based on a linear regression of results reported in Figure I.3-17, is proposed for the prediction of the maximum shift of the fundamental period τ_{up} in Equation 8.

$$\boldsymbol{\tau_{up}} = a \cdot \lambda_{eq} + b \cdot \tilde{q} + c \qquad (8)$$

where *a*, *b* and *c* are defined in Table 8 as function of the number of storeys *n*.

Table I.3-8 – Coefficients for Equation 8

n	*a*	*b*	*c*
1	0.050	-1.00	1.15
3	0.017	-2.10	1.37
5	0.011	-2.10	1.54

The absolute values of discrepancies between the values of τ_{up} of Figure I.3-17 and those obtained from the Equation 5 are reported in Table I.3-9, showing a reasonable agreement. Discrepancies not higher than 10% are observed for all cases with non-null values of the vertical load with the only exception for the 5-storey shearwall with length equal 6 m, hold-down stiffness equal to 5000 kN/m and $\tilde{q}$ = 18.7% with a discrepancy equal to 12%. In case of null values of vertical load, discrepancies smaller than 10% are obtained for 1-store shearwalls whereas the maximum absolute value is equal to 18% for the same 5-storey characterized by the highest error in case of non-null values of $\tilde{q}$.

Table I.3-9 – Discrepancies between the values of τ_up of Figure I.3-17 and those obtained from the Equation 8

l	k_H	$n = 1$			$n = 3$			$n = 5$		
		$\tilde{q} = 0$	$\tilde{q} = 1.9\%$	$\tilde{q} = 3.7\%$	$\tilde{q} = 0$	$\tilde{q} = 5.6\%$	$\tilde{q} = 11.2\%$	$\tilde{q} = 0$	$\tilde{q} = 9.3\%$	$\tilde{q} = 18.7\%$
[m]	[kN/m]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]
2	5000	1	2	3	4	3	0	5	3	4
4	5000	8	7	10	6	3	4	15	2	1
6	5000	4	5	7	15	5	8	18	10	12
2	10000	0	1	2	2	2	5	15	3	4
4	10000	0	2	1	4	0	2	1	1	7
6	10000	1	0	2	2	2	5	1	4	0
2	15000	2	1	1	8	0	1	17	4	5
4	15000	3	1	0	5	1	1	8	4	3
6	15000	5	4	2	6	4	6	9	8	0

I.3.5 Conclusions

In this section a relationship between the rocking behavior of CLT shearwalls and the fundamental period of CLT structures was investigated. Modal tests on a full-scale timber mock-up and non-linear elastic incremental dynamic analyses were carried out to determine the fundamental period as function of the lateral response amplitude.

Due to the stabilizing effect of vertical load, three significant regions were observed in the fundamental period vs global drift curves, namely “no rocking”, “partial rocking” and “full rocking regions”. For 3- and 5-storey walls, however, the full activation of the rocking behaviour was limited by the yielding of the hold-down at the ground level; for this reason partial- rather than full-rocking behaviour was observed. No rocking was conversely detected for values of the lateral drift lower than 0.07%, 0.17% and 0.18% for 1-, 3- and 5 –storey, respectively.

A significant dependency of the fundamental period on the magnitude of vertical load as well as the stiffness of the hold-down was shown, while changing the length of walls and number of story no significant differences of the fundamental period were obtained for the same value of the lateral drift.

The upper value of the normalized fundamental period τ_{up} which a certain CLT shearwall may exhibit in relation to the lateral drift amplitude, was plotted as function of the equivalent rocking slenderness. A linear expression was proposed for the prediction of τ_{up} as well showing a good match with results obtained from the analyses.

Maximum values of the fundamental period between 1.2 and 1.9, 1.4 and 2.2 and 1.5 and 2.5 times the fundamental period T_{nr} obtained in case of non-activation of the rocking behaviour was detected for null values of the vertical load. The ranges decrease between 1.2 and 1.8, 1.2 and 1.7, 1.2 and 1.8 for values of the normalized vertical load $\tilde{q}$ equal to 3.7%, 11.2% and 18.7 for 1-, 3- and 5-storey shearwalls, respectively.

The observed values of τ_{up} can be used as upper values of the fundamental period which should be adopted in linear-elastic RSM analyses in seismic design of CLT buildings to adequately take into account either the partial or full activation of rocking behaviour.

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II

An innovative earthquake-resistant system for the realization of high CLT buildings

II.1 State-of-the-art of high CLT buildings placed in seismic areas

Abstract

In the first part of the Thesis, it has been demonstrated via numerical and experimental analyses that the global stiffness of CLT structures is strongly influenced by connections elements. In order to permit the construction of high rise CLT buildings in seismic areas, connections able to reduce lateral displacements and inter-storey drifts of tall CLT buildings and to withstand high tensile forces are necessary. Scope of this Section is to give an overview on the state-of-the-art of high CLT buildings located in seismic areas highlighting the main issues of this construction typology correlated to the limited lateral stiffness and the high tensile forces concentrated in hold-down connections. It has been demonstrated that traditional nailed brackets, such as hold-downs and angle brackets do not present stiffness and load-bearing capacity enough to realize CLT buildings taller than five or six storeys in high-seismicity areas. The different strategies available in literature and developed to overcome these limits are presented. These strategies generally require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations.

II.1.1 Overview on solutions developed for multi-storey CLT buildings

Wood-based engineered products like Cross-Laminated Timber (CLT) are characterized by high in-plane strength and stiffness of the timber panel conferred by the cross-lamination of solid-wood boards [3], which give the multi-storey structure good resistance to vertical and lateral loads [4].
As anticipated in previous sections, a key role in the mechanical behaviour of CLT structures subjected to lateral loads (wind or earthquake) is assumed by connections, generally hold-downs and angle brackets. It has been demonstrated that the nailed brackets above mentioned do not present enough stiffness and load-bearing capacity to realize tall CLT buildings. Indeed, the rocking contribution of seismic resistant shear walls that takes great importance with increasing seismic action, height and slenderness of the walls generate high tensile forces at hold-downs and the excessive lateral flexibility of the structure [7,26]. In order to permit the realization of tall CLT buildings in high-seismicity areas different strategies have been developed in recent years. Some of the most important solutions available in literature are:

1. Hybrid timber-concrete or timber-steel systems;
2. Use of post-tensioning bars, to reduce tensile forces at connections and assuring recentering of shear walls;
3. Rocking coupled shear walls with vertical joints;
4. Reduction of inertial forces using connections with high ductility and dissipative capacity, employing the post-elastic hysteretic behaviour of steel or the friction to dissipate energy during a strong earthquake, preserving the structure from damages.

A description of such structural system is reported in the following.

II.1.1.1.1 <u>Hybrid timber concrete or timber steel systems</u>

The use of hybrid structure is one of the common methods that can be used to increase the height of timber buildings. Hybrid material can be integrated at component levels (hybrid beams or coloumns, hybrid slab, hybrid walls) or in building system levels (vertical mixed system, hybrid shear wall system). An example of hybrid stell-timber system is reported in Tesfamariam et al [31] where a steel moment resisting frame and CLT infill walls is proposed. This system consists of combining the steel moment resisting frame with CLT infill walls, as reported in Figure II.1-1:

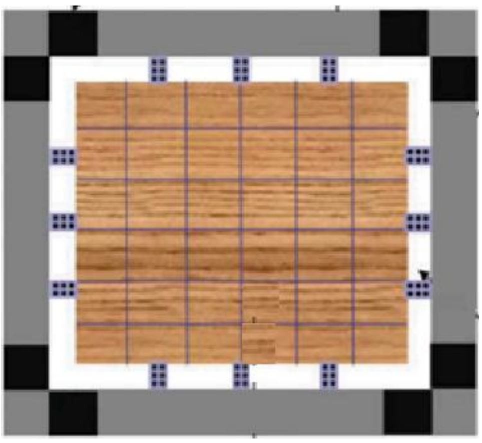


Figure II.1-1 – Steel timber system proposed in Tesfamariam et al [31]

As can be seen in Figure II.1-1, CLT panels are connected to the steel frame with nailed brackets on all side. Results of static pushover analysis implemented by authors on buildings with dimensions reported in Figure

I.2-2a and varying heights (3,6 and 9 storeys) and series of CLT infill wall configurations (one-bay infilled, two-bay infilled and fully infilled, show respectively Figure II.1-2b, Figure II.1-2c, Figure II.1-2d) shows that the CLT-steel hybrid proposed represents a valid alternative to increase the seismic demand and lateral stiffness of tall CLT buildings.

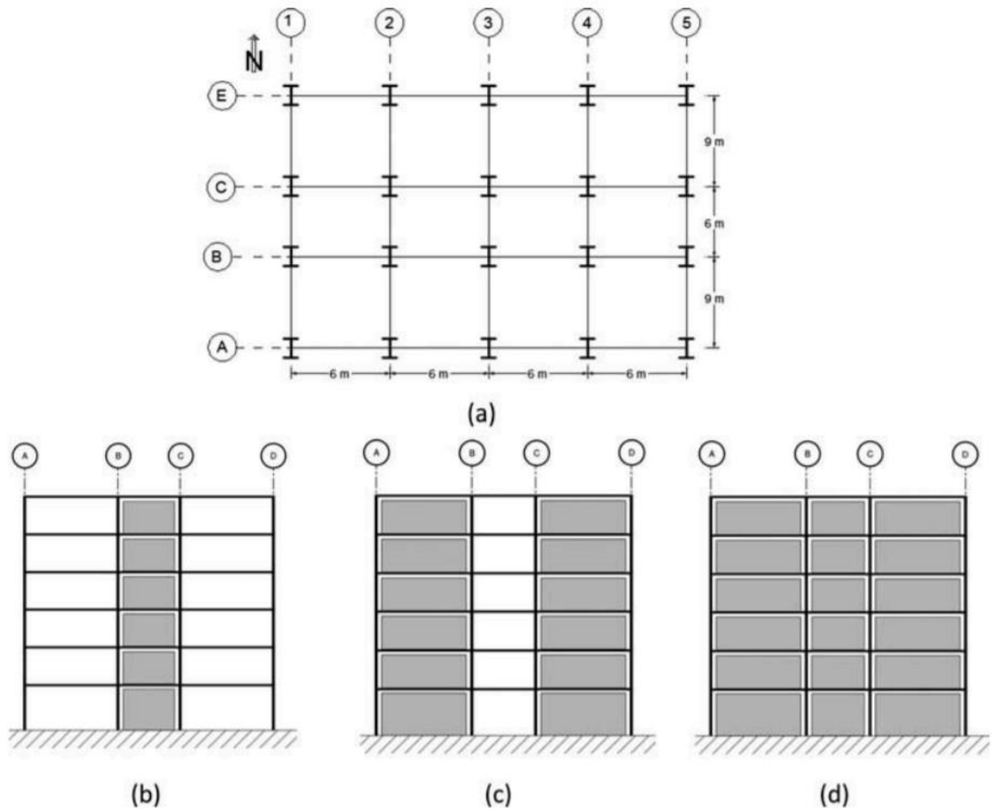
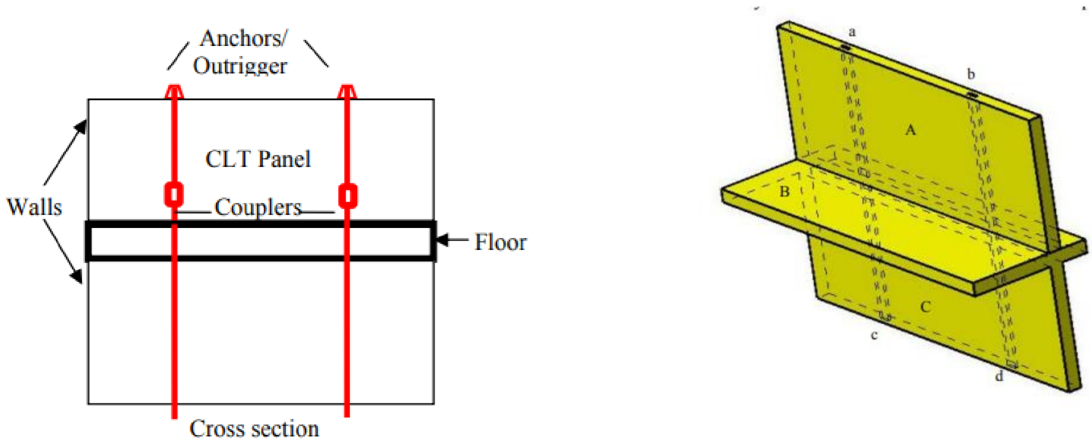


Figure II.1-2– Details of the 6-story steel moment resisting frame: (a) base building floor plan; (b) one-bay infilled configuration (c two-bay infilled configuration; and (d) fully infilled configuration [31]

Another example of hybrid system is represented by timber-concrete structure. An example of the latter is reported in Van de Kuilen et al. [132] where authors demonstrates that thanks to the combination of CLT and concrete, skyscrapers can be designed using a system of outriggers and tensile elements in the façade to increase the flexural capacity of the building and concrete core to resist to shear forces, as reported in Figure II.1-3.



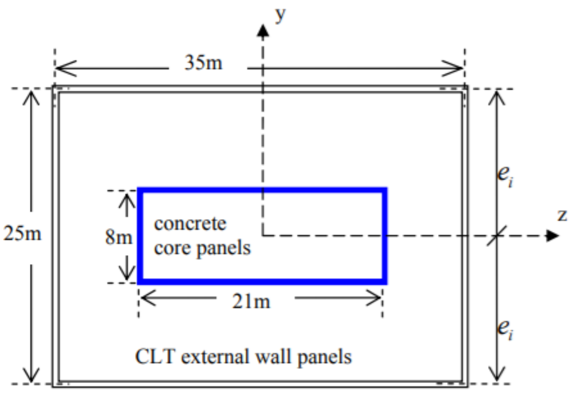


Figure II.1-3– 2D and 3 D view of outrigger system obtained coupling steel tensile bars inside CLT panels (a,b); example of internal concrete core and CLT external walls [132]

Authors demonstrate that structure with outriggers will have 30 to 40 percent less overturning moment in the core compared to a free cantilever. In addition, the decreased lateral displacement and overturning moment of the building will also decrease the cost of its columns and its foundation. The connections between timber building and outriggers and core plays a crucial role in the design process indeed their locations and stiffnesses determine the force flow inside the building and the overall stiffness of the building.

II.1.1.1.2 Post-tensioned systems

Post-tensioned Pres-Lam mass timber represents a valid alternative for the realization of timber buildings in areas with high wind loads and high seismic activity. Thanks to this technology, that was introduced in Section I.1.1.4.2 of the present work, it is possible to stiffen timber structures against lateral loads and limit damage in case of large seismic events. Several experimental tests have been conducted on post-tensioned systems, one of the most important was performed by Moroder et al. [34] where CLT Pres-Lam walls were tested under quasi-static seismic loads. In detail two different configurations where investigated by authors. In the first configuration CLT wall panels were connected in the corners with screws, while in the second configuration CLT wall panels were connected to the steel columns, that were introduced at the corners, with dissipative plates (see Figure II.1-4).

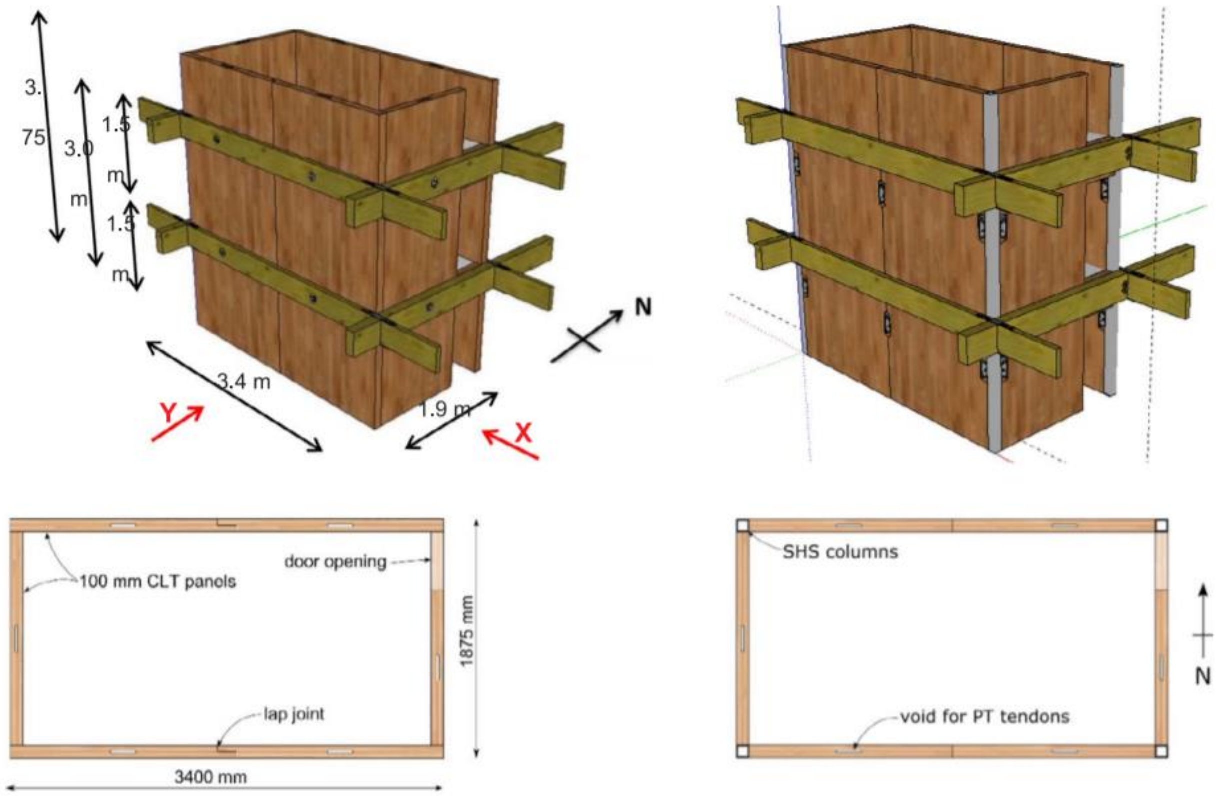


Figure II.1-4– Setup of the CLT core tested: fully threaded screws configuration (left) and dissipative plates configuration (right) [34].

As reported in Figure II.1-4, each core specimen was rectangular in shape with overall outside dimensions of 3.4 m long and 1.87 m wide with a total wall height of 3.75 m. Post-tensioning strands were placed inside cavities created by leaving a space between boards during manufacture in order to apply a post-tensioning force in each shear wall. The core wall structure was tested under quasi-static cyclic loads in the two main directions applying different level of post-tensioning forces. Force-drift curves obtained are reported in Figure II.2-5 (first configuration with screws) and in Figure II.2-6 (second configuration with dissipative plates).

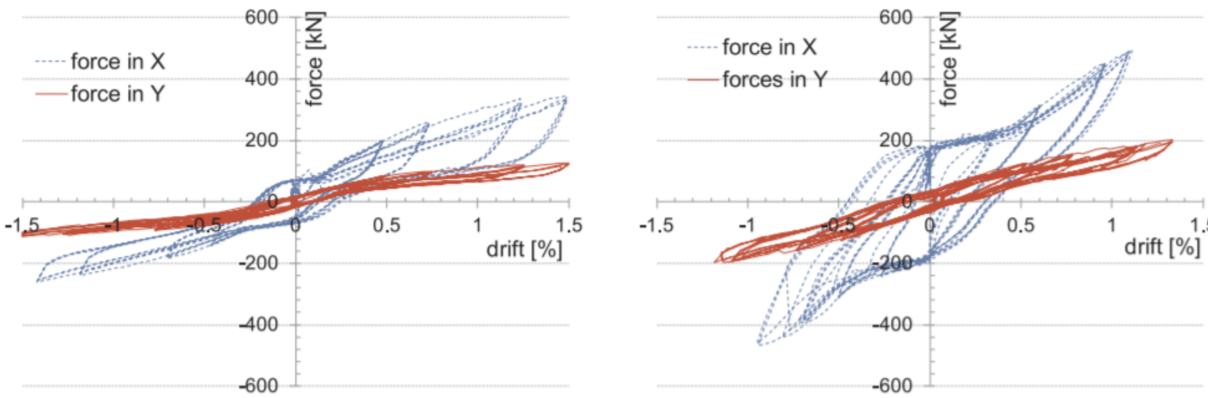


Figure II.1-5– Force-drift curves for first configuration: low post-tension force (left), high post-tension force (right) [34]

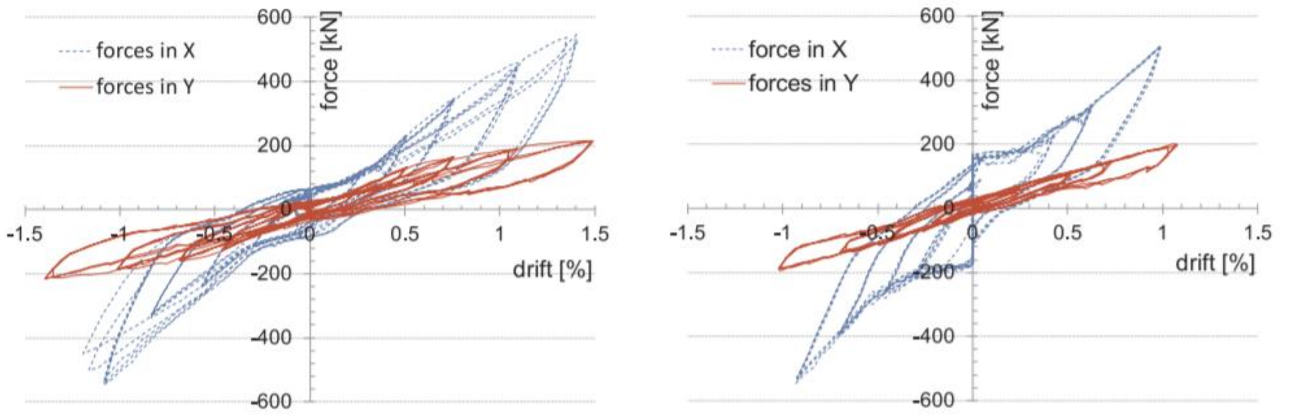


Figure II.1-6– Force-drift curves for second configuration: low post-tension force (left), high post-tension force (right) [34]

As can be seen from Figure II.1-6, stable hysteretic loops with minimal stiffness and strength degradation was observed with little to no damage. The increase in the post-tensioning force increased the core wall strength and stiffness by reaching its maximum capacity of 0.5% drift sooner than the lower post-tensioning option. For the reasons above mentioned it is possible to affirm that Pres-Lam concept represents a valid structural solution for the realization of high CLT buildings with core walls around stairwells and elevator shafts.

II.1.1.1.3 Rocking coupled shear walls with vertical joints;

Rocking coupled shear walls with vertical joints are generally used to provide a perfectly elastoplastic behaviour and a stable hysteretic response of CLT multi-storey buildings. Hasemi at al [38] have studied a hybrid damage avoidant steel-timber wall system using the innovative resilient slip friction joint reported in Figure II.1-7.

Figure II.1-7– Slip friction joint proposed by Hasemi et al. [38]

This system includes coupled timber walls and boundary steel column as the main lateral load resisting members (see Figure II.1-8).

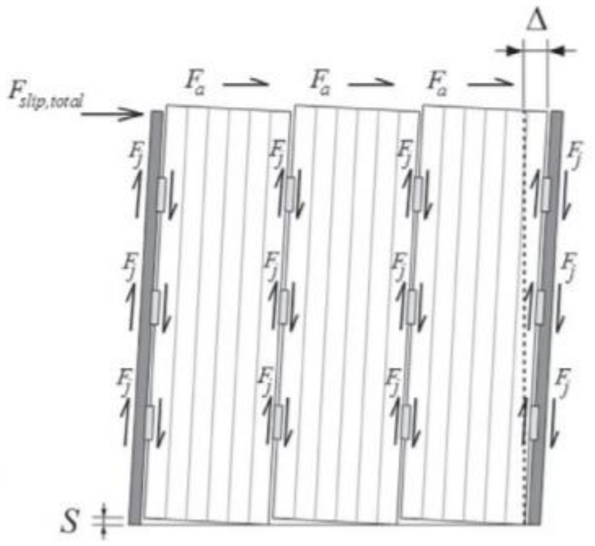


Figure II.1-8– CLT coupled walls with slip friction joints proposed by Hasemi et al. [38]

Experimental tests confirmed that this system is able to provide a selfcentring behaviour in addition to a significant rate of seismic energy dissipation through friction.
Another example of system able to dissipate seismic energy is represented by CLT shear walls with coupling beams [39]. Under seismic load, bending of the shear walls causes deformations at the end of the coupling beams. The forces of the beams constrain the coupled walls in turn decreasing the internal forces and deformation of the shear walls. Coupling beams system proposed by Liu et al. [39] is reported in Figure II.1-9, where steel plates were adopted to connect CLT walls and dampers were introduced in the centre of the coupling beams in order to dissipate energy.

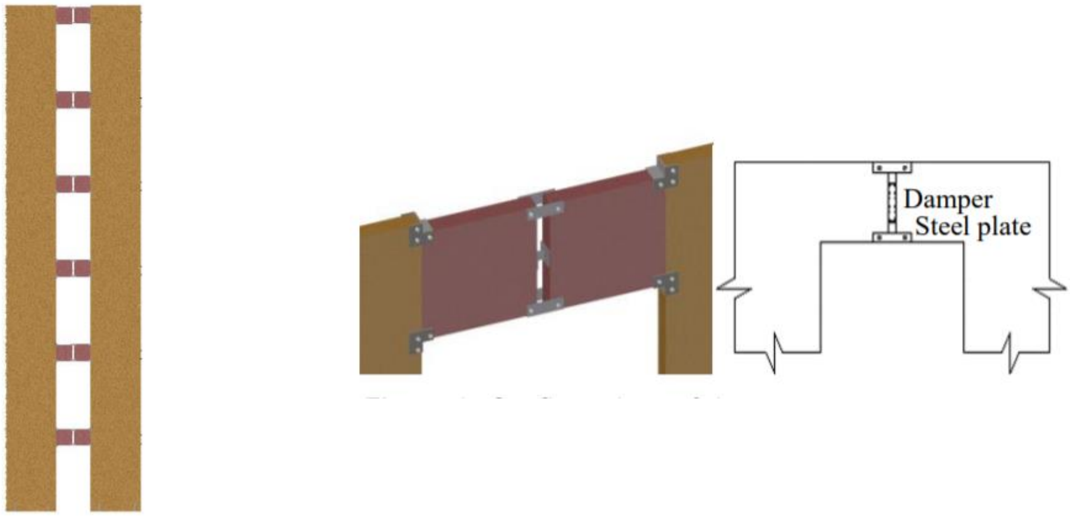


Figure II.1-9– Shear walls with coupling beams (left) and detail of dampers and steel plates (right). [39]

It is worth nothing that the detailing between beams and walls strongly influence the structural performance of the system. Improper design of coupling beams may fail to meet the lateral strength and stiffness requirements of the CLT shear walls.

II.1.1.1.4 High ductility and dissipative capacity systems

In order to overcome the limitations provided by the adoption of the classical connection generally used in CLT structures (hold-down and angle brackets), several authors proposed dissipative capacity systems able to guarantee high ductility of CLT buildings. In the following are reported some solutions developed in recent years and available in literature.

Latour and Rizzano [41] introduced a new type of dissipative connector, called “Xl-stub” to be applied in substitution of the classical hold-down in order to increase the dissipative capacity and therefore the behaviour factor of CLT buildings. Figure II.1-10 reports Xl-Stub connector in undeformed and failure configuration after a cyclic test.

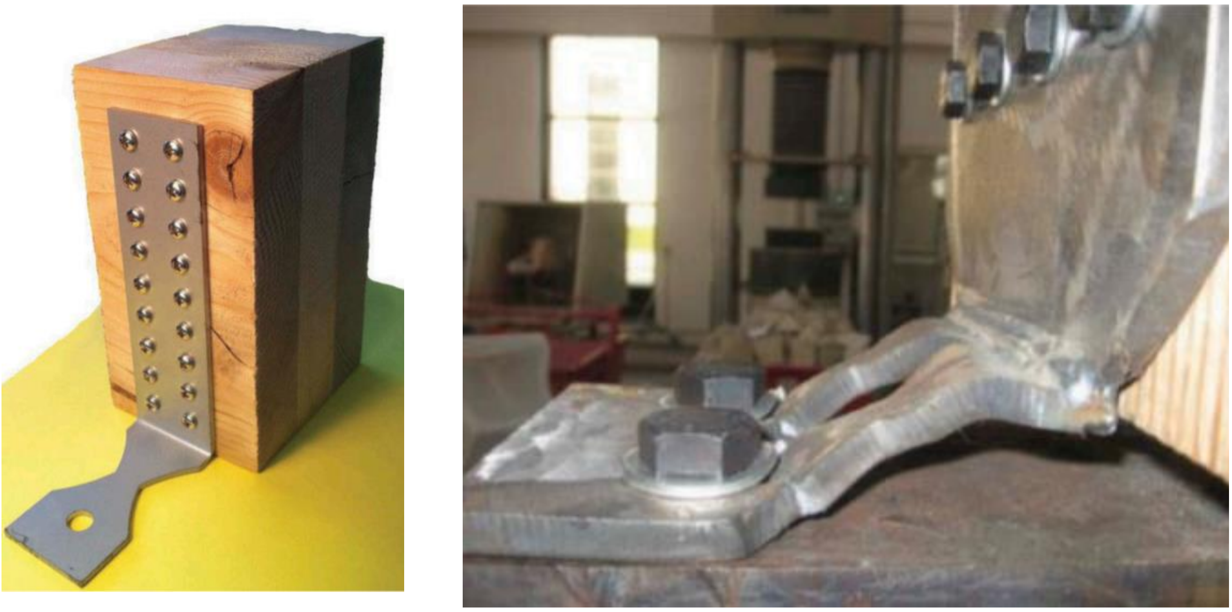


Figure II.1-10– Xl-stub brackets in undeformed (left) and failure (right) configurations [41]

It is possible to observe how the proposed angle was designed to dissipate the energy in the flange plate that was studied in order to maximize the energy dissipation capacity. Results obtained by authors from experimental tests demonstrates the possibility of improving the seismic behavior of CLT buildings by enhancing the cyclic behavior of the structural fuses by adopting the proposed XL-stubs.

A similar idea was proposed by Scotta et al. [45] with the development of the dissipative connector for CLT buildings called “X-bracket”. The connection has the peculiarity that operates properly in both directions and resisting to axial and shear forces. X-bracket can be used to realize both panel-to-foundation and panel-to-panel joints of shear walls, as shown in Figure II.1-11.

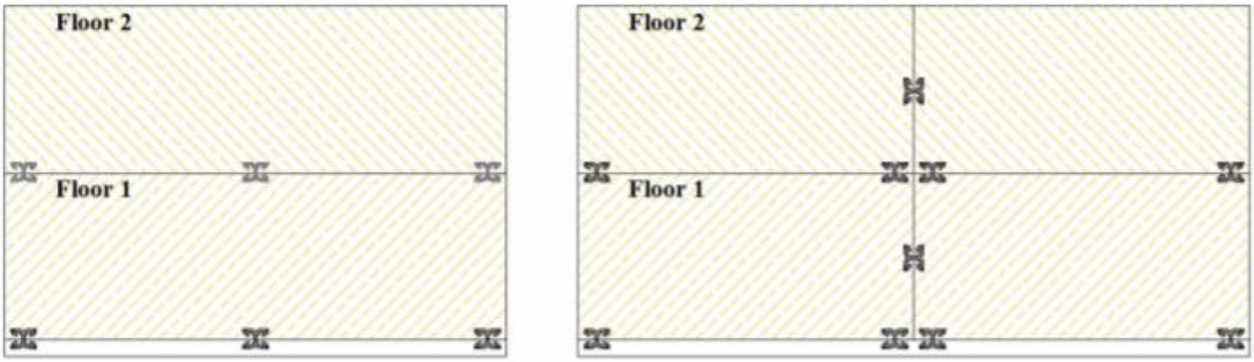


Figure II.1-11– Positioning of X-bracket connectors in CLT shear walls [45]

Panel to foundation and panel to panel joints are reported in FigureII.1-12.

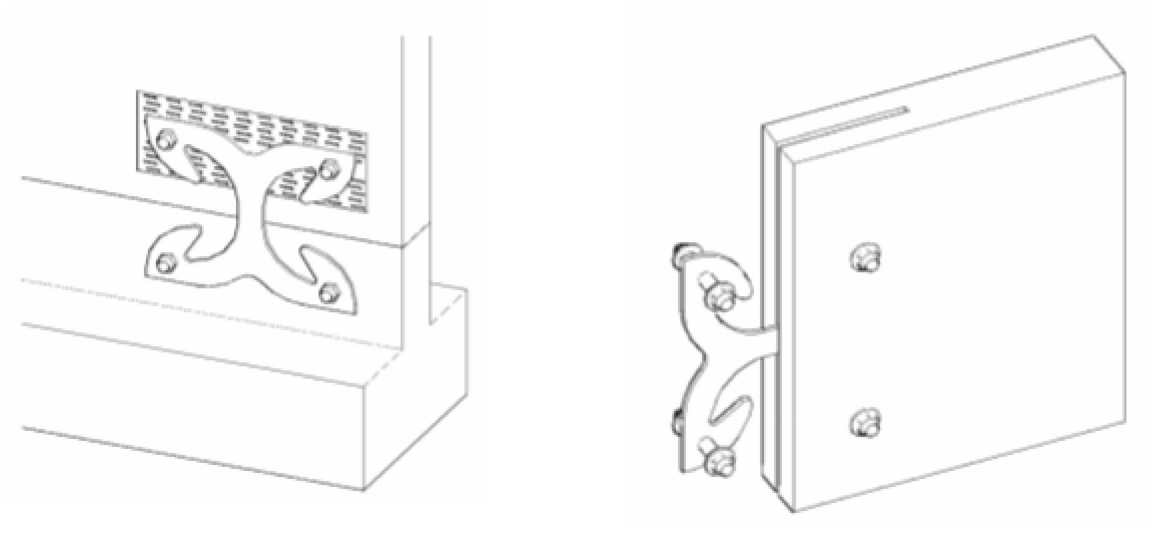


Figure II.1-12– Panel to foundation and panel to panel joints [45]

Geometry of the X-bracket assures high ductility before failure and demonstrates negligible pinching behavior, allowing one to emphasize the dissipative capacity under cyclic loading.

The last example of dissipative connection analyzed is represented by the slip-friction connectors developed by Loo et al. [43]. It is worth nothing that slip-friction connectors were originally developed for use in steel construction. Authors adapted the idea in order to use slip-friction connectors as hold-downs that resist to tensile forces, as reported in Figure II.1-13.

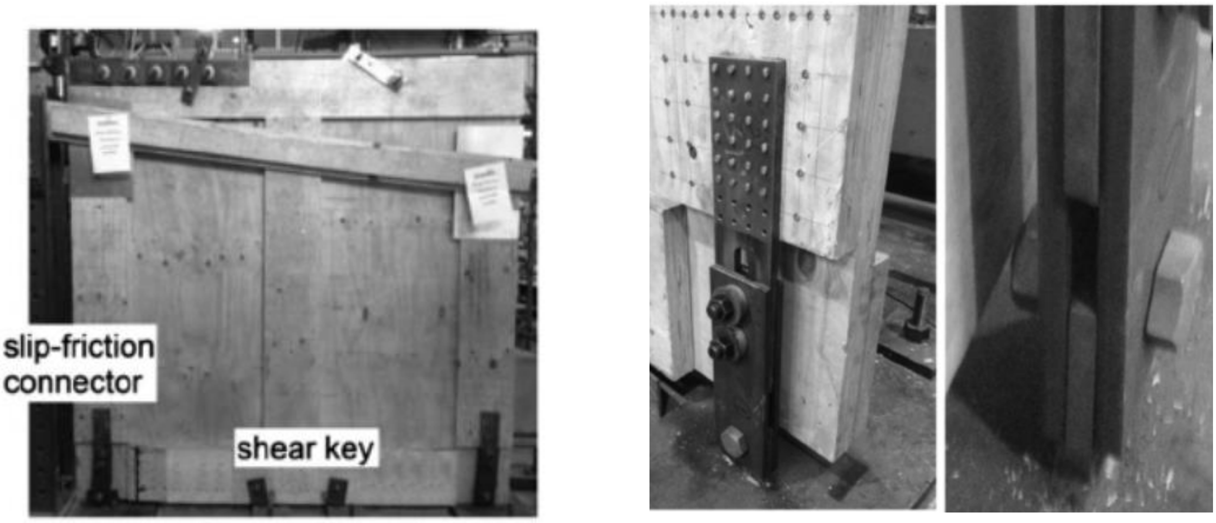


Figure II.1-13– Position of slip-friction connectors in shear wall (left) and detail of connectors (right) [43]

Results obtained from experimental tests demonstrate that slip-friction connectors can impart ductile and elasto-plastic characteristics to CLT shear walls. Self-centering potential was also found to be excellent

It is important to highlight how all the strategies reported in Section II.1.1 require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations. For this reason, in this work an original earthquake-resistant system based on the use of vertical steel ties (without the use of prestress, special connections or hybrid structures) is proposed and will be discussed in Section II.2.

II.2 Earthquake-resistant CLT buildings stiffened with vertical steel ties

Abstract

The interest in multi-storey CLT buildings in seismic areas is leading to the development of new strategies to increase the lateral stiffness of shear walls and to resist high tensile forces due to rocking. Both these purposes can be achieved with vertical steel ties placed at each shear-wall end, to transfer directly tensile forces from each storey to foundation. Three technologies are proposed for transferring forces from the CLT panels to the ties: the use of nailed plates, of the screwed connection X-RAD, or directly by contact with a thick plate at the top of each storey wall. The dynamic behaviour of CLT shear walls, representing the bracing system of a building, anchored with the aforementioned technologies has been investigated by means of dynamic analyses and a comparison with the use of common nailed plates or screwed connections without ties is presented. Results, varying the number of storeys and the seismic mass, show that the proposed technology is an effective strategy to increase the feasibility of multi-storey CLT buildings. Moreover, the substantial increase of the lateral strength makes possible to decrease the ductility demand, reducing lateral displacements and damages. Complementary non-linear static analyses have been performed to evaluate the actual displacement capacity and ductility of the systems.

II.2.1 Description of technology

In low- and mid-rise CLT structures, tensile and shear forces, which are generated at each storey level from lateral loads, are generally resisted respectively by hold-downs and angle brackets. Alternative connections can be used, suitably designed to optimize one or more targets: strength, ductility, dissipative capacity. Independently from the type of connection, they are attributed both to resist the abovementioned forces and to limit the lateral displacements of the structure. As the number of storeys of the building increases, there is an increase in tensile forces at connections especially for slender shear walls. As already mentioned, the rocking contribution to lateral displacements takes on increasingly importance as the height of the building grows, with a consequent increase in the deformability of the structure.
To limit the lateral displacements and to withstand the high tensile forces in multi-storey CLT buildings, connections with high stiffness and strength are needed. Traditional hold-downs, composed of a steel plate and many nails or screws, may not be enough. For this reason, steel ties, which may consist of steel plates, bars or profiles have been analysed, to work in addition to or in place of traditional hold-downs or other connections with the same purpose. It is important to underline that suitable connections for absorbing shear forces, that are not scope of this study, must be designed in order to assure the cinematic compatibility of such shear connections with the rocking of CLT panels deriving from ties elongation.
It is worth nothing that the proposed technology based on the use of strong and stiff steel ties may reduce the dissipative capacity of the structure, compared to the use of nailed plates or the dissipative connections described in Section II.1.1. For this reason, a non-dissipative seismic design of the wall systems has been followed in the calculations presented in the following Sections, adopting a behaviour factor q equal to 1.5 according to [18]. This choice does not penalize particularly the design of the structure, as current regulations limit significantly the behaviour factor associated to CLT buildings. For example, current European Seismic Code gives a maximum value equal to 2 [18], and current Italian Code for Constructions equal to 2.5 [118]. The design with a behaviour factor equal to 1.5, i.e., in accordance with low-dissipative structural behaviour concept [18], leads to a significant increase of the lateral stiffness of the structure: damages after an earthquake are limited and the structure is less flexible also for wind action, with a clear benefit also in terms of reduced discomfort for wind-induced vibrations in high-rise buildings [134].
In detail, three original connection systems are proposed to resist the tensile forces due to rocking:

a) steel ties coupled with nailed plates;
b) steel ties coupled with the screwed connection X-RAD [135];
c) steel ties only.

The following subsections present the concept of these three technologies.

II.2.1.1 Steel ties coupled with nailed plates

The system is composed by steel ties welded to steel plates, which are nailed to the panels at each storey level, Figure II.2-1. In this way, the steel ties connect the nailed plates together and transmit the tensile forces derived from them directly to the concrete foundation. According to this strategy, the nailed plates work as hold-downs at each storey but have to withstand only the tensile force deriving from rocking at that storey, whereas the tensile forces derived from the storeys above are directly transferred to the foundation through the ties. This results in reducing the tensile forces at the nailed connections, permitting of overcoming the typical limitations in the realization of multi-storey buildings.
To avoid eccentricities, the nailed plates and the steel ties should be placed at both sides of the wall. Since the steel tie is continuous from the foundation to the top storey, it is necessary to realize a slot through the timber floors. To avoid any problems related to compression perpendicular to the grain in floors CLT panels, it is

advisable to interrupt floors in order to guarantee the continuity of CLT wall. Furthermore, if required, it is possible to realize the tie with various steel elements and then to restore its continuity by welding or bolting.

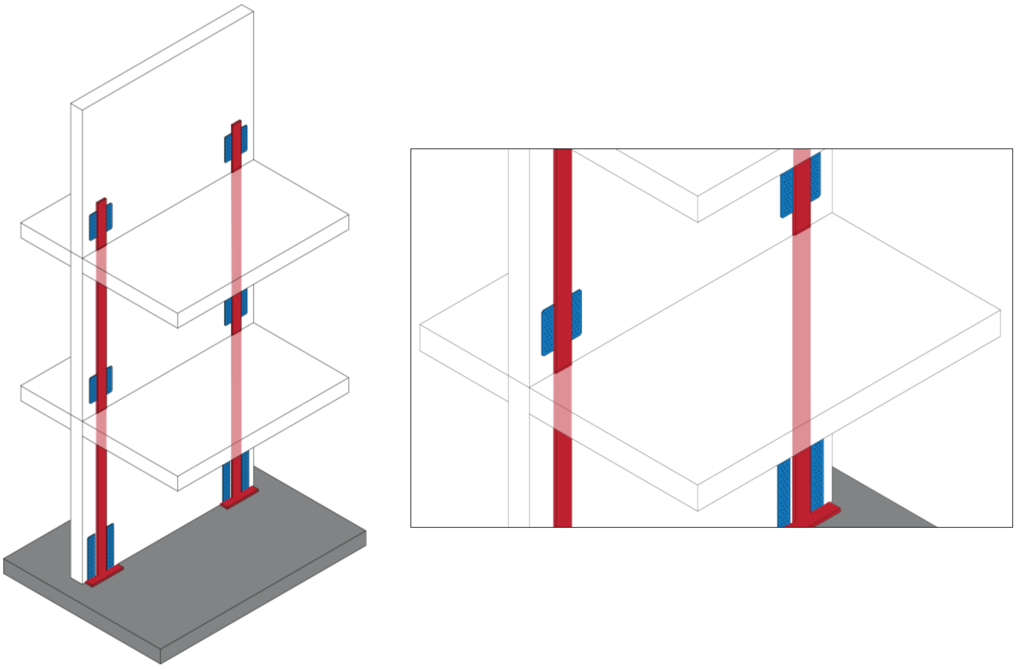


Figure II.2-1 - Conceptual representation of the steel ties coupled with nailed plates. 3D view and detail (only the front face is shown)

II.2.1.2 Steel ties coupled with the screwed connection X-RAD

The system is based on the combination of the screwed connection X-RAD [135] and the steel ties, which work together, Figure II.2-2. In detail, the steel ties are bolted to the X-RAD at each floor level. As the strategy described in previous Section, the ties are continuous along the height of the shear wall and can be composed by various bolted or welded elements. At the corners of the building, the steel ties can be opportunely realized with steel L profiles, whereas elsewhere steel plates can be placed at the narrow sides (thickness) of the wall, Figure II.2-2. According to this arrangement of the ties, eccentricities are avoided. As for the ties combined with nailed plates, the X-RAD for each storey must withstand only the tensile force deriving from rocking at that storey, whereas the tensile forces derived from the storeys above are directly transferred to the foundation through the tie.

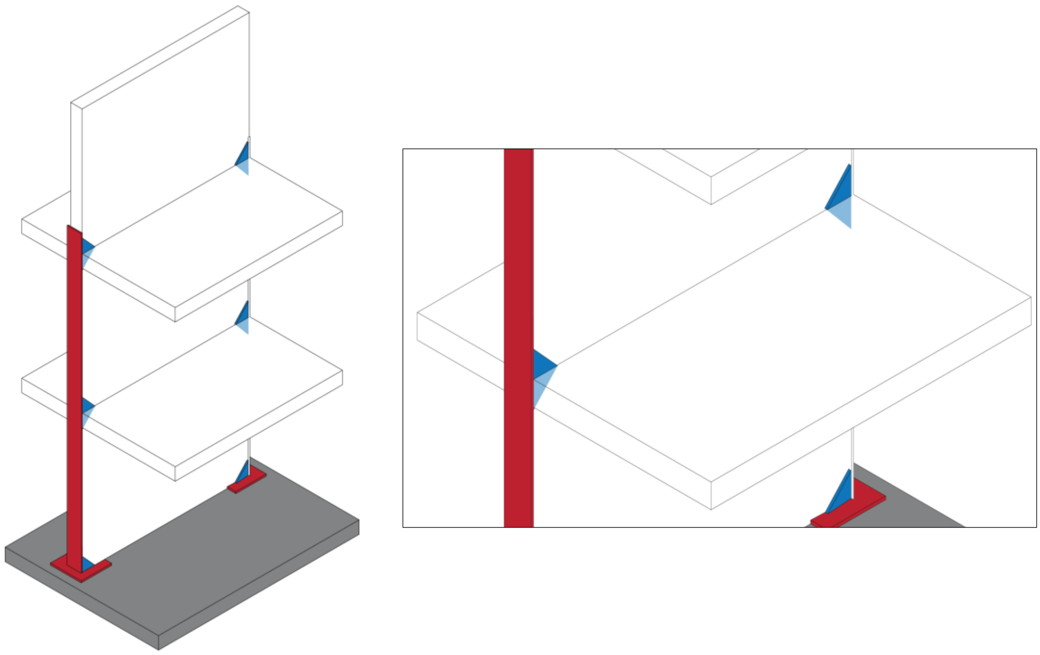


Figure II.2-2 - Conceptual representation of the steel ties coupled with X-RAD. 3D view and detail (only the front face is shown)

II.2.1.3 Steel ties only

This strategy, unlike the previous ones, does not involve the coupling of ties with other connections. The rocking behaviour at each storey is directly prevented by thick steel plates, in direct contact with the top narrow side of each wall panel. In this case, the ties can be realized with two steel bars placed at both sides of the wall; the bars are secured to the plates at each floor level, Figure II.2-3.

Compared to the other two systems (i.e., nailed plates or X-RAD combined with steel ties), this is definitely the stiffest, since the deformation contribution related to nails or screws is avoided. Moreover, this strategy should be also the most cost-effective, since dowel-type fasteners and additional brackets are avoided.

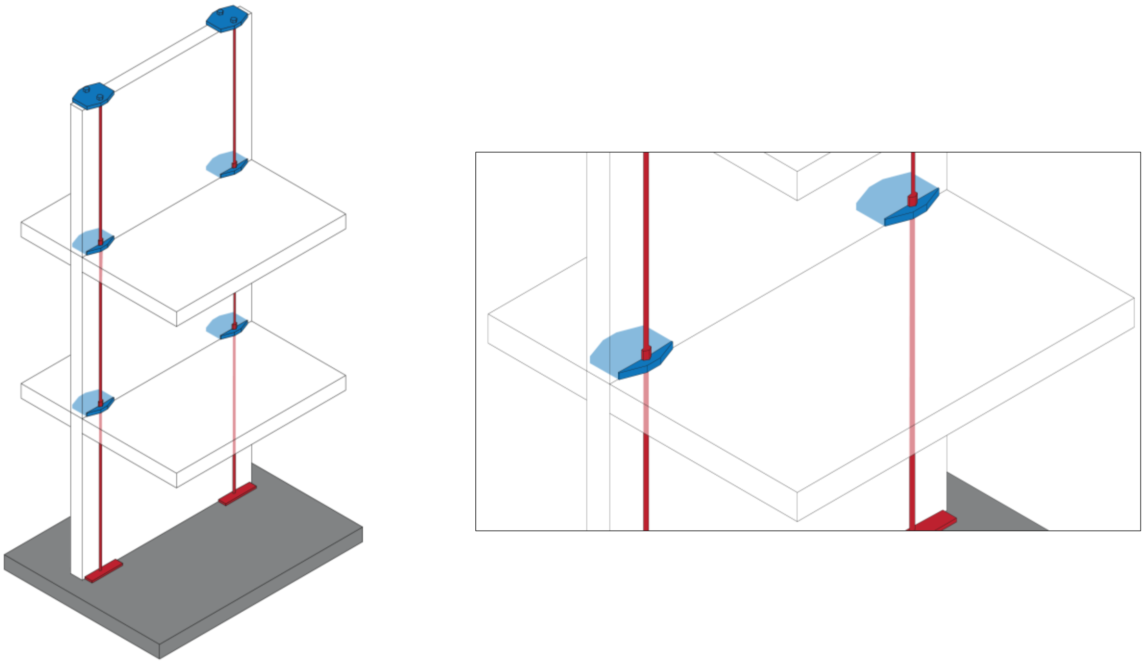


Figure II.2-3 - Conceptual representation of the steel ties. 3D view and detail (only the front face is shown)

II.2.2 Numerical model and analyses

In this Section, the effectiveness of the technology is evaluated, presenting parametric Linear Dynamic Analyses (LDA) of multi-storey CLT shear walls, which represent the bracing system of a building, supposed to be realized in an area with high earthquake hazard. Results are reported in Section II.2.3. Then, in Section II.2.4 results of non-linear static analyses are discussed to assess the actual capacity of the system and to compare the response of the new strategies with traditional systems.

II.2.2.1 Description of the buildings

Two CLT buildings have been considered, having rectangular plan with dimensions of 17.1 × 15.5 m (Building A) and 23.3 × 15.5 m (Building B), and a variable number of storeys (3, 5 and 7 storeys), with an inter-storey height of 3 m. The structure is braced with six shear walls per direction for a total of 12 shear walls in the building, symmetrically arranged in plan. The base length of the walls is equal to 6 m. They are intended to act only as lateral-load resisting systems, whereas all the vertical gravitational loads are resisted by a frame, which is supposed to be perfectly pendular, in order to neglect its effects in the lateral stiffness of the building. The perfect regularity of the buildings and the rigid floor diaphragms allowed to study the behaviour of the whole structure modelling only one shear wall, with the same number of storeys of the building and one sixth of the mass.

The mass was computed according to the seismic combination of EN 1990 [117], assuming floor and roof dead loads equal to 2.5 kN/m^2, floor live loads equal to 2 kN/m^2 and roof live loads equal to 0.5 kN/m^2. The seismic storey masses are listed in Table II.2-1.

Table II.2-1 –Seismic mass per storey relative to the modelled shear wall

	Building A	Building B
Dimensions	17.1 × 15.5 m	23.3 × 15.5 m
Floor mass	14.0 t	19.0 t
Roof mass	11.3 t	15.3 t
Wall mass	1.5 t	1.5 t

II.2.2.2 Earthquake-resistant connections

Two technologies have been preliminarily considered to connect the panels of the shear walls to the foundation and in-between storeys and to withstand the seismic forces: traditional hold-downs and angle brackets [136] or the innovative strong connection X-RAD [135]. Since for the considered high earthquake intensity, the seismic demand for the five-storey shear walls (in terms of force and/or stiffness) exceeded the capacity of these two connection systems, steel ties have been used to strengthen and stiffen the structure, according to the conceptual representations in Section II.2.1. The use of steel ties allowed also to extend the analyses to the seven-storey shear walls.

Table II.2-2 summarizes all the analysed configurations, with the following meaning of the labels:

- The first letter (A or B) identifies the dimension of the building, that is, the different storey mass assumed for the single shear wall;
- The number (3, 5 or 7) identifies the number of storeys of the shear wall;
- The last letters identify the connection system used or the combination of them: "H" is for hold-down, "X" is for X-RAD (i.e., screwed connection), "T" is for steel ties (Figure II.2-3), "NP+T" and "X+T" are for a combination of nailed plates or X-RAD with the steel ties (hereafter called also combined configurations or strengthened configurations, Figure II.2-1 and Figure II.2-2).

Table II.2-2 – Analysed configurations of the shear walls and labels

			Labels					
Config.	Connection system		Building A			Building B		
	Rocking	Sliding	3 storeys	5 storeys	7 storeys	3 storeys	5 storeys	7 storeys
H	Hold-downs	Angle brackets	**A3H**	**A5H**	-	**B3H**	**B5H**	-
NP+T	Nailed plates and steel ties	Angle brackets	-	**A5NP+T**	**A7NP+T**	-	**B5NP+T**	**B7NP+T**
X	X-RAD	X-RAD	**A3X**	**A5X**	-	**B3X**	**B5X**	-
X+T	X-RAD and steel ties	X-RAD	-	**A5X+T**	**A7X+T**	-	**B5X+T**	**B7X+T**
T	Steel ties	Angle brackets	-	**A5T**	**A7T**	-	**B5T**	**B7T**

II.2.2.3 Numerical models

The numerical models of the buildings were developed and analyses were performed using the SAP2000 Finite Element code [119]. The following finite elements were used: two-dimensional elastic shell elements to model the CLT panels; uniaxial linear elements to simulate the actual stiffness of the connections; one-dimensional truss elements to simulate the steel ties. Figure II.2-4 shows the models of the shear walls, performed with the characteristics hereafter summarized.

In the configurations H and NP+T (Figure II.2-4 a,c), the connections have been modelled with uniaxial elements according to the component approach presented in [26]. It can be noted that, according to the technology presented in Section II.2.1, in the model NP+T (Figure II.2-4 c), the tensile forces due to rocking at each storey level are firstly resisted by the nailed plates and then introduced in the steel tie and directly transferred to foundation. The values to be assigned to these models are: the axial stiffness of the hold-down $k_{H,i}$ or of the nailed plate $k_{NP,i}$; the shear stiffness of the element representing all the angle-brackets in a storey $k_{A,i}$; the Modulus of Elasticity E_T and the cross-section area A_T of the truss elements representing the steel ties. In the configurations X and X+T (Figure II.2-4 b,d), the X-RAD system that connects two panels at the same corner is represented by an assembly of four uniaxial elements: the two vertical elements simulate the stiffness at 90° to the horizontal ($k_{X,v}$); the other two elements the stiffness at 0° to the horizontal ($k_{X,h}$). The steel tie is represented also in this case with E_T and A_T. Finally, in the configurations T (Figure II.2-4 e), the steel ties are directly fixed to the top of each panel, for all the storeys, in order to transfer directly the force from the wall to the tie, according to the technology described in Section II.2.1.3.

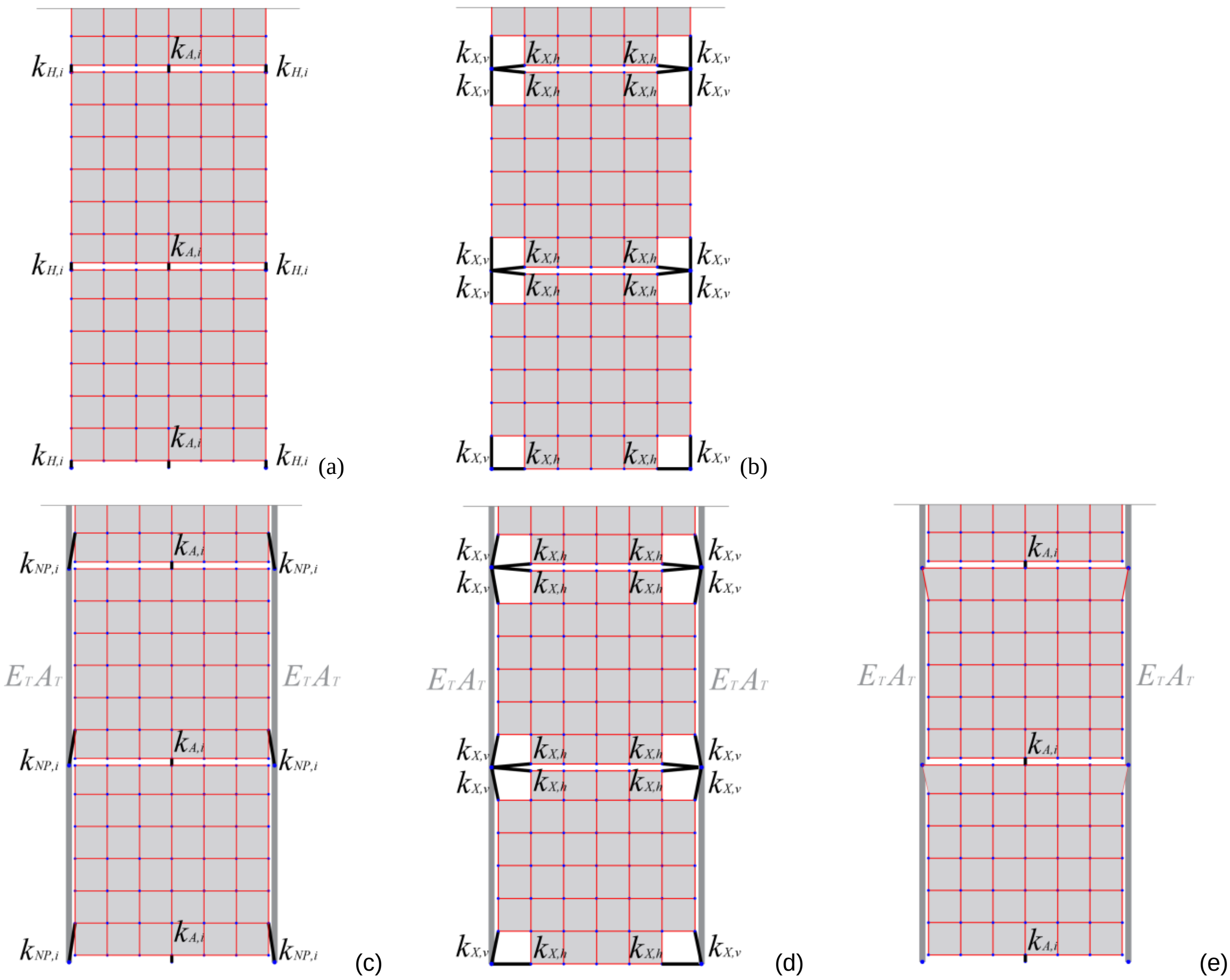


Figure II.2-4 - Numerical models of the shear walls. Configurations with: (a) hold-downs “H”; (b) X-RAD “X”; (c) nailed plates and steel ties “NP+T”; (d) X-RAD and steel ties “X+T”; (e) steel ties “T”

II.2.2.4 Design of the shear walls and stiffness of the connection systems

The shear walls were designed performing iterative LDA [18], assuring, for each iteration, the consistency between strength and stiffness of connections in the model, up to fulfilment of verifications for Ultimate Limit State (ULS) and Damage Limitation State (DLS) [16]. As anticipated in Section II.2.2, H and X configurations for the five-storey buildings did not fully comply with these verifications and the strengthening with the ties was required. Nevertheless, the results are anyway shown also for these configurations for comparative purposes.

The site location of Perugia in the region of Umbria has been chosen because it is representative of a high seismic hazard zone in Italy. It is characterised by the seismic spectra reported in Figure II.2-5, according to Italian Regulation for Constructions [118], assuming building foundations resting on ground type C and behaviour factor q equal to 1.5 or 2.0. A q equal to 2.0 was assumed for all the non-strengthened configurations (H and X), according to a dissipative structural behaviour in medium ductility class [18]. For the configurations with vertical steel ties (NP+T, X+T and T), it was assumed equal to 1.5. As mentioned above, the choice of designing these configurations according to a low-dissipative structural behaviour [18], was made since a reduced dissipative capacity is expected by using steel ties, which represent the leading earthquake-resistant system in the building.

The following connections have been considered in the models:

- Hold-downs WHT340 with 20 nails or WHT620 with 52 nails [137];
- Angle brackets TTF200 for floor-wall joints or TCF200 for wall-to-foundation joint, with 30 nails [138];
- 4-mm thick nailed plates with 12 nails (NP12) or 26 nails (NP26);
- X-RAD [139].

Table II.2-3 lists strength and stiffness values assumed for connections, according to values given in [26] for hold-downs and angle brackets and in [135] for X-RAD. Strength and stiffness of nailed plates were obtained multiplying the values available in [79] relative to a steel-timber connection with one 4×60-mm ring-shank nail, by twelve or twenty-six nails for NP12 and NP26 respectively. The steel ties were assumed to be realized with rectangular steel plates (or alternative shapes such as L or UPN profiles), realized in class S355, Modulus of Elasticity E_T equal to 210000 MPa [140].

The iterative design procedure resulted in the arrangement of connections listed in Table II.2-4 for the configurations H, NP+T and T. With reference to the configurations X and X+T, one X-RAD per panel corner was assumed for all configurations and storeys, with the exception of the 7-storey shear walls (A7X+T and B7X+T), for which a couple of X-RAD was added for all the storeys at the middle of the panel base and panel top, to increase the shear strength and comply with the seismic demand. For all the configurations with steel ties, an overall cross-section area of the ties A_T equal to 1500 mm^2 was assumed for the 5-storey shear wall, and equal to 2250 mm^2 for the 7-storey shear wall.

It is worth noting that the design of connections and ties depends strongly on the chosen aspect ratio of the shear wall: slender panels may require very strong ties; on the contrary, large panels may require high lateral forces to activate rocking, even if vertical loads are supported by the frame. A linear design is anyway sufficient to have a correct balance among strength and stiffness of all the connections used [16].

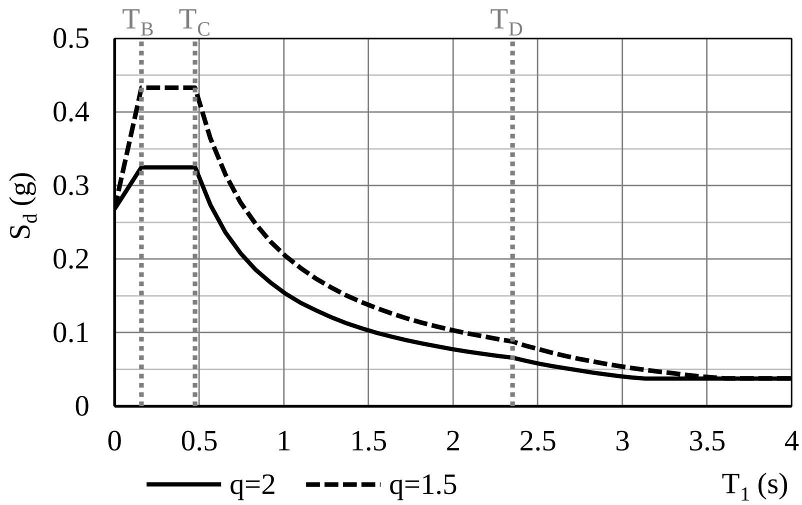


Figure II.2-5 - Design seismic spectra of Perugia in Italy, ground type C, behaviour factors q equal to 1.5 or 2

Table II.2-3 – Strength and stiffness of the earthquake-resistant connections

	Rocking				Sliding		Rocking and sliding
Metal connector	WHT340 hold-down [26]	WHT620 hold-down [26]	NP12 nailed plate [70]	NP26 nailed plate [70]	TTF200 angle bracket [26]	TCF200 angle bracket [26]	X-RAD [111]
Elastic stiffness (kN/mm)	5.70	13.25	5.80	12.58	8.21	8.48	15.2 (0°/90°) 17.3 (180°/270°)
Strength (kN)	46.20	93.70	41.58	90.09	39.10	39.10	*Capacity domain*

Table II.2-4 – Arrangement of the earthquake-resistant connections per panel of the multi-storey shear wall. Configurations H, NP+T and T

Storey	1	2	3	4	5	6	7
A3H	4 WHT620 3 TCF200	4 WHT340 3 TTF200	2 WHT340 2 TTF200	–	–	–	–
A5H	4 WHT620 3 TCF200	4 WHT620 3 TTF200	4 WHT340 2 TTF200	4 WHT340 2 TTF200	2 WHT340 2 TTF200	–	–
A5NP+T	4 NP26 5 TCF200	2 NP26 5 TTF200	2 NP26 4 TTF200	2 NP12 3 TTF200	2 NP12 2 TTF200	–	–
A5T	5 TCF200	5 TTF200	4 TTF200	3 TTF200	2 TTF200	–	–
A7NP+T	4 NP26 7 TCF200	4 NP26 6 TTF200	4 NP26 6 TTF200	4 NP26 5 TTF200	2 NP26 4 TTF200	2 NP26 3 TTF200	2 NP12 2 TTF200
A7T	7 TCF200	6 TTF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200	2 TTF200
B3H	4 WHT620 3 TCF200	4 WHT340 3 TTF200	2 WHT340 2 TTF200	–	–	–	–
B5H	4 WHT620 3 TCF200	4 WHT620 3 TTF200	4 WHT340 2 TTF200	4 WHT340 2 TTF200	2 WHT340 2 TTF200	–	–
B5NP+T	4 NP26 7 TCF200	2 NP26 6 TTF200	2 NP26 5 TTF200	2 NP12 4 TTF200	2 NP12 3 TTF200	–	–
B5T	7 TCF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200	–	–
B7NP+T	4 NP26 8 TCF200	4 NP26 7 TTF200	4 NP26 7 TTF200	4 NP26 6 TTF200	2 NP26 5 TTF200	2 NP26 4 TTF200	2 NP12 3 TTF200
B7T	8 TCF200	7 TTF200	7 TTF200	6 TTF200	5 TTF200	4 TTF200	3 TTF200

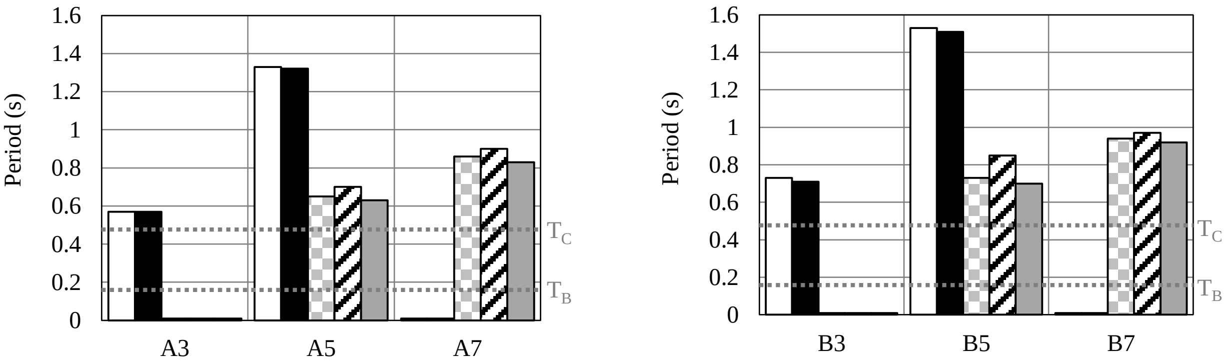
To compare the configurations in terms of resulting number of connections per panel (Table II.2-4), it is necessary to remember that the strengthening with the steel ties leads to an increase of the seismic demand due to an increase of stiffness and consequent decrease of periods of vibration and due to the lower behaviour factor used, Figure II.2-5. The consequent increase of strength demand clearly results in an increase of the number of angle-brackets per storey to resist higher shear forces, as can be noted by comparing results among the five-storey configurations. However, despite this apparent disadvantage, the addition of steel ties actually leads to a significant advantage in terms of rocking behaviour. Contrary to the non-strengthened five-storey buildings, both ULS and DLS verifications are fulfilled for all the five- and seven-storey NP+T and X+T configurations. Finally, as expected, the T configuration resulted the most cost-effective strategy in terms of optimization in the number of connections.

II.2.3 Results of linear dynamic analyses

In this Section, results from LDA are summarized and discussed in terms of fundamental periods of vibration, tensile forces at connections, inter-storey drifts and displacements.

II.2.3.1 Fundamental period of vibration

Figure II.2-6 shows the results of the analyses in terms of fundamental period of vibration of the structure. The periods T_B and T_C of the seismic spectra defined by Eurocode 8 [18] (see Figure II.2-5) are also represented to identify the limit of the constant spectral acceleration branch (plateau).
The configurations without steel ties (H and X) for both the buildings show similar fundamental periods of vibrations. The use of steel ties combined or not with the nailed plates and X-RAD (NP+T, X+T, T) increases strongly the global stiffness of the shear walls, resulting in a substantially lower fundamental period and a higher spectral acceleration. This is evident comparing results for the five-storey configurations in Figure II.2-7 (dashed black lines), where the fundamental periods of vibration have been superimposed to the design spectra.



	H	X	NP+T	X+T	T
A3	0.57	0.57	-	-	-
A5	1.33	1.32	0.65	0.70	0.63
A7	-	-	0.86	0.90	0.83

	H	X	NP+T	X+T	T
B3	0.73	0.71	-	-	-
B5	1.53	1.51	0.73	0.85	0.70
B7	-	-	0.94	0.97	0.92

Figure II.2-6 - Fundamental periods of vibration. Units: s

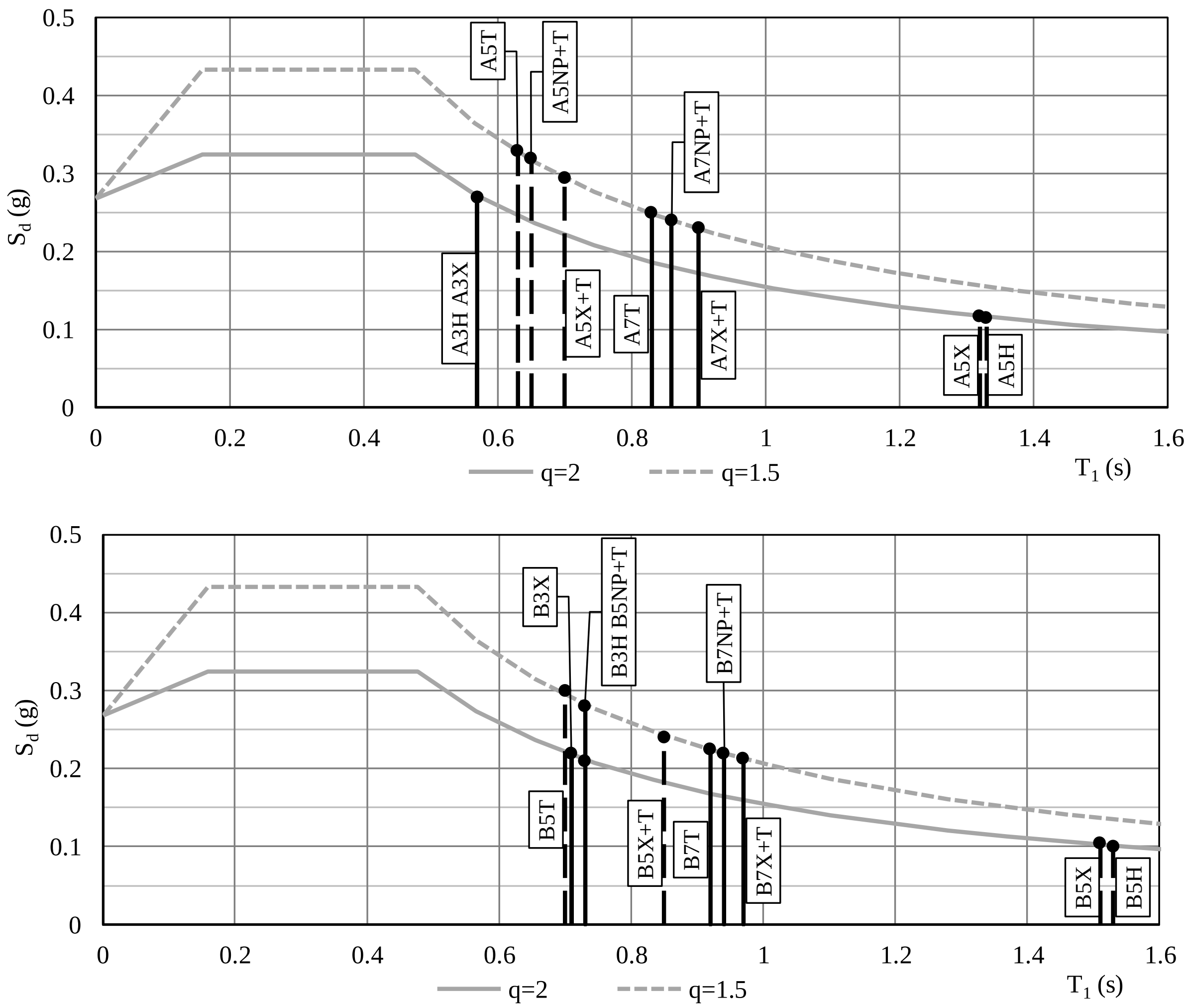


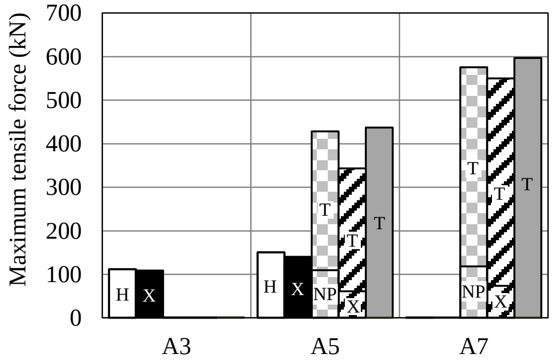
Figure II.2-7 - Fundamental periods of vibration and design spectra. (a) Building A; (b) Building B

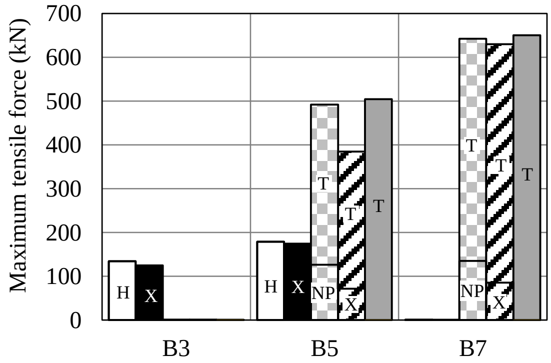
II.2.3.2 Tensile forces

Figure II.2-8 shows the results of the analyses in terms of maximum tensile forces due to rocking. The maximum values were recorded always in the connections at the base of the shear walls. The tensile force is showed for the rate resisted by the hold-downs (H), by the X-RAD (X), by the nailed plates (NP) and by the steel tie (T).

The comparison between the configurations without steel ties (H and X) for both the buildings shows similar forces. It is worth noting that for the 5-storey shear walls, very high tensile forces in the hold-downs and X-RAD have been recorded, reaching about 180 kN in the hold-downs of the B5H configuration, which is very close to the limit of about 187 kN of a couple of WHT 620 in the wall end, see Table II.2-3. Despite the higher spectral acceleration and consequent higher inertial forces due to the lower period of vibration and to the lower behaviour factor assumed, the strengthening with the steel ties allows to reduce the forces in the connections, and to have the possibility of increasing the number of storeys.

The T configurations behave similarly to the NP+T ones. The maximum tensile force at the base is almost the same, but in the case of the T configurations, the force must be resisted entirely by the tie. Conversely, in the NP+T and X+T configurations, tension in the ties is lower, due to the contribution of nailed plates and X-RAD at the base of the building, which are directly anchored to the foundation. The T configuration is confirmed to be more effective than the NP+T and X+T configurations, since only the ties are used, avoiding additional nailed or screwed connections to withstand tensile forces. Nevertheless, the combined configurations may become further interesting, when used in combination with dissipative device (e.g., [40-46]). In this way, the dissipators permit to reduce at each storey the inertial force transmitted from the panels to the tie. It is worth recalling also the possibility of pre-assembling the connections to the panels, making the X+T configuration particularly advantageous in terms of prefabrication and fast installation of the ties to the structure, see Section II.2.1.2.





	H □	X ■	NP+T	X+T	T
A3	112.1	109.0	-	-	-
A5	150.7	140.3	428.1 *NP 109.2* *T 318.9*	343.3 *X 61.5* *T 281.8*	436.9
A7	-	-	575.5 *NP 118.3* *T 457.2*	549.8 *X 73.4* *T 476.4*	597.3

	H □	X ■	NP+T	X+T	T
B3	134.2	125.0	-	-	-
B5	179.2	174.7	492.4 *NP 126.2* *T 366.2*	384.7 *X 71.5* *T 313.2*	504.3
B7	-	-	642.8 *NP 135.1* *T 507.7*	630.3 *X 85.3* *T 545.0*	650.4

Figure II.2-8 - Maximum tensile forces (acting at Level 1) at hold-downs, nailed plates, X-RAD connections and steel ties. Units: kN

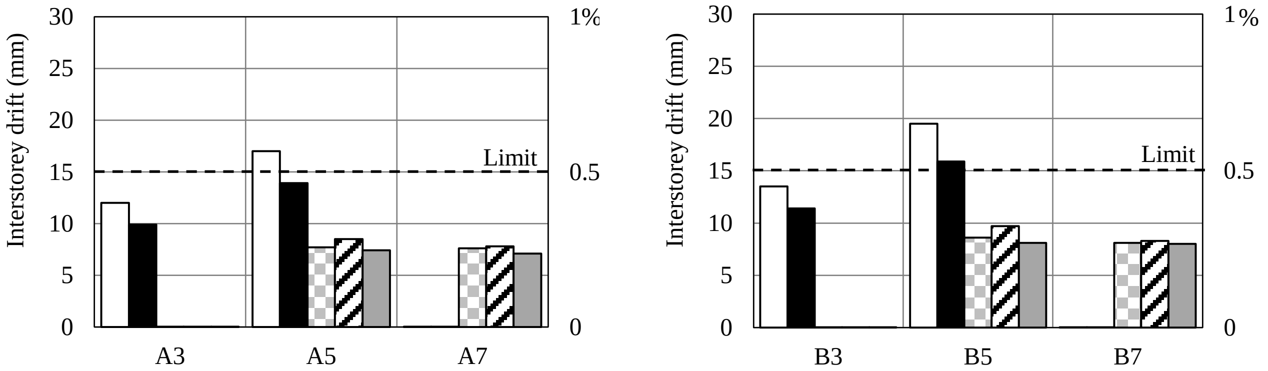
II.2.3.3 Displacements and inter-storey drifts

Figure II.2-9 and Figure II.2-10 show the results of the analyses in terms of maximum inter-storey lateral drift and maximum top displacement of the shear walls at the Damage Limitation State (DLS).

The configurations without steel ties (H and X) for both the buildings show similar top displacements, with an inter-storey drift slightly higher for the H configurations, mainly due to the greater rocking deformation. In these configurations, the global lateral displacements are mainly due to rocking for about an 80% rate and the contribution of shear deformations is limited to about 20%.

It can be noted that the use of the steel ties and the design of the structure according to a low-dissipative structural behaviour, result in limiting the lateral displacements of the building, and therefore in limiting the damage to structural and non-structural components.

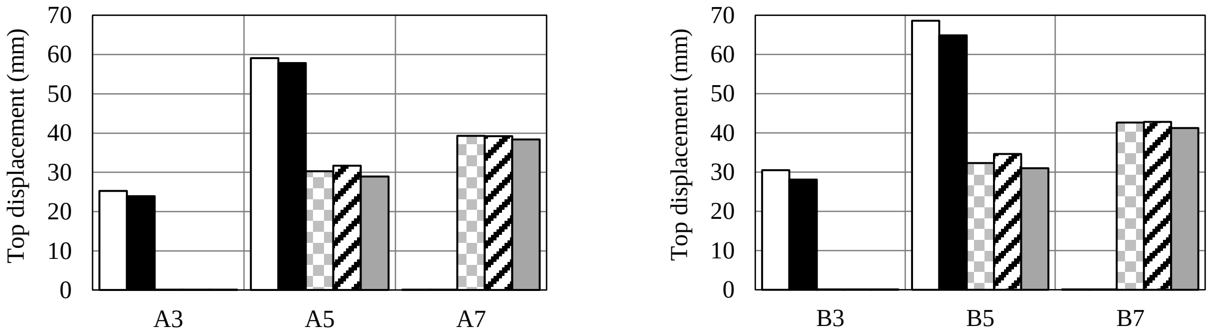
The 5-storey shear walls analysed in this work demonstrate that the addition of the steel ties almost halves the top displacements and inter-storey drifts, and, also for the 7-storey shear walls, displacements and drifts are smaller than the 5-storey configurations without ties and comply fully with the limit of 0.5% in Eurocode 8 [18]. In these strengthened configurations, shear deformations take on greater importance and contribute for about 50% to the global lateral displacements.



	H □	X ■	NP+T ⊠	X+T ▨	T ■
A3	12.0	9.9	-	-	-
A5	17.0	13.9	7.7	8.5	7.4
A7	-	-	7.6	7.8	7.1

	H □	X ■	NP+T ⊠	X+T ▨	T ■
B3	13.5	11.4	-	-	-
B5	19.5	15.9	8.6	9.7	8.1
B7	-	-	8.1	8.3	8.0

Figure II.2-9 - Maximum inter-storey drifts and DLS limit (at ground floor). Units: mm



	H □	X ■	NP+T ⊠	X+T ▨	T ■
A3	25.3	23.9	-	-	-
A5	59.1	57.8	30.3	31.7	28.9
A7	-	-	39.3	39.2	38.4

	H □	X ■	NP+T ⊠	X+T ▨	T ■
B3	30.5	28.1	-	-	-
B5	68.6	64.9	32.3	34.6	31.0
B7	-	-	42.6	42.8	41.2

Figure II.2-10 - Maximum top displacements. Units: mm

II.2.4 Complementary analyses and discussions

To complete and validate the analyses discussed in previous Section, complementary analyses have been performed. Section II.2.4.1 presents the results of linear analyses to evaluate the effects of variation of stiffness of ties per storey; Section II.2.4.2 presents a validation of analyses presented in previous Section with non-linear static analyses, to evaluate the actual capacity of the analysed techniques.

II.2.4.1 Effects of optimizing the capacity of steel ties per storey

In order to optimize the steel ties, the cross-section can be gradually reduced from the foundation to the top of the building, keeping almost constant among the storeys the ratio between the tensile strength demand to the capacity of the ties; this ratio is hereafter called "working rate".

To evaluate the influence of such optimization on the fundamental period of vibration, tensile forces and displacements, additional LDA were carried out for the strengthened configurations of the 5- and 7-storey

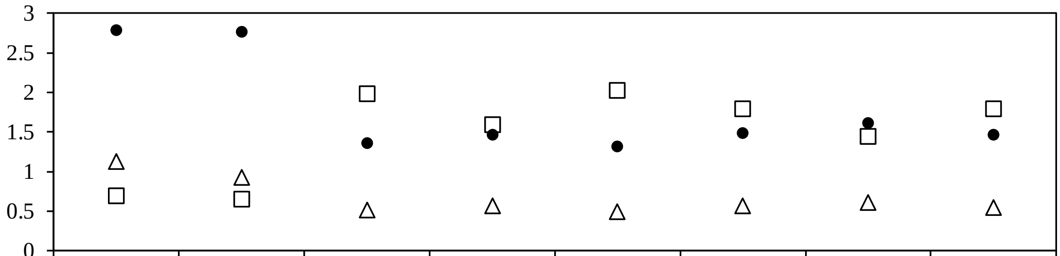
Building A. The optimized configurations are marked with a star; the additional configurations are therefore: A5NP+T*; A5X+T*; A5T*; A7NP+T*; A7X+T*; A7T*. The cross-section areas of the ties resulting from the design and the relative working rates are listed in Table II.2-5 and compared with the non-optimized counterparts. It is worth noting that the optimized design resulted in an important reduction of the cross-section of the ties for the upper storeys. Conversely, the number of the other connection elements (nailed plates, angle brackets and X-RAD), remained unchanged with respect to those of Table II.2-4 and negligible variations occurred in terms of periods (below 12%) and therefore in forces acting in connections (below 12%), as can be seen in Table II.2-6. Finally, the decrease in the overall stiffness of the optimized structures resulted in a negligible increase of inter-storey drifts (below 15%) and top displacements (below 12%).
A final comparison among all the analysed configurations for the 5- and 7-storey Building A is given in Figure II.2-11. Three dimensionless parameters have been introduced, namely: the period index τ equal to T_1/T_C that shows how far the fundamental period of vibration is from the plateau; the tensile force index ϕ equal to $F_{max}/(M \cdot PGA)$; and the drift index δ equal to the actual drift divided by 0.5%. In these parameters, T1 is the fundamental period of vibration, Tc is equal to 0.476 s, F_{max} is the maximum tensile force, *M* is the total seismic mass relative to the shear wall according to Table 1, PGA is the peak ground acceleration equal to 2.61 m/s^2, 0.5% is the drift limit for DLS.

Table II.2-5 – Cross-section areas AT of the steel ties, units mm2. In brackets the working rate of the steel ties

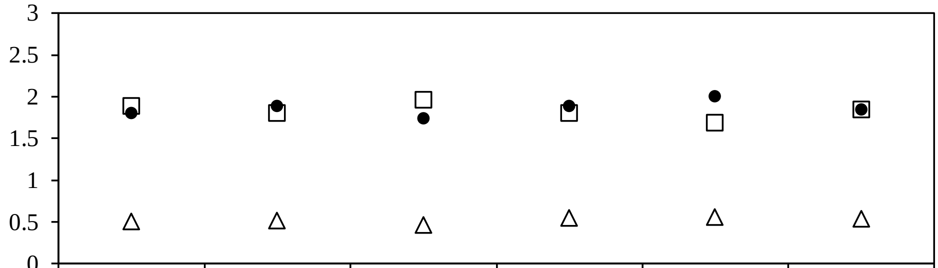
Storey	1	2	3	4	5	6	7
A5NP+T	1500 (60%)	1500 (60%)	1500 (38%)	1500 (21%)	1500 (8%)	-	-
A5NP+T*	1500 (54%)	1500 (54%)	950 (53%)	550 (53%)	200 (48%)	-	-
A5X+T	1500 (53%)	1500 (53%)	1500 (35%)	1500 (21%)	1500 (10%)	-	-
A5X+T*	1500 (48%)	1500 (48%)	1000 (47%)	600 (49%)	300 (46%)	-	-
A5T	1500 (82%)	1500 (58%)	1500 (38%)	1500 (21%)	1500 (7%)	-	-
A5T*	1500 (74%)	1100 (68%)	700 (67%)	400 (68%)	150 (65%)	-	-
A7NP+T	2250 (58%)	2250 (58%)	2250 (44%)	2250 (33%)	2250 (22%)	2250 (12%)	2250 (6%)
A7NP+T*	2250 (55%)	2250 (55%)	1750 (54%)	1300 (54%)	850 (56%)	500 (50%)	200 (48%)
A7X+T	2250 (60%)	2250 (60%)	2250 (47%)	2250 (35%)	2250 (24%)	2250 (14%)	2250 (7%)
A7X+T*	2250 (56%)	2250 (56%)	1750 (56%)	1300 (55%)	900 (55%)	600 (48%)	250 (49%)
A7T	2250 (75%)	2250 (65%)	2250 (50%)	2250 (36%)	2250 (23%)	2250 (13%)	2250 (5%)
A7T*	2250 (70%)	2000 (69%)	1500 (70%)	1100 (68%)	700 (70%)	400 (67%)	150 (66%)

Table II.2-6 – Results for the optimized configurations and percentage change from the non-optimized counterparts

	Period (s)		Maximum Tensile Force (kN)		Top displacement (mm)		Inter-storey drift (mm)	
A5NP+T*	0.71	*9.23%*	385.6	*-9.93%*	32.8	*8.25%*	8.6	*11.69%*
A5X+T*	0.77	*10.00%*	312.2	*-9.06%*	34.0	*7.26%*	9.2	*8.24%*
A5T*	0.70	*11.11%*	387.2	*-11.38%*	31.4	*8.65%*	8.3	*12.16%*
A7NP+T*	0.90	*4.65%*	549.7	*-4.48%*	43.1	*9.67%*	8.3	*9.21%*
A7X+T*	0.96	*6.67%*	514.2	*-6.48%*	43.6	*11.22%*	8.5	*8.97%*
A7T*	0.88	*6.02%*	560.9	*-6.09%*	42.8	*11.46%*	8.1	*14.08%*



	A5H	A5X	A5NP+T	A5X+T	A5T	A5NP+T*	A5X+T*	A5T*
● Period index τ	2.79	2.77	1.36	1.47	1.32	1.49	1.62	1.47
□ Force index ϕ	0.70	0.65	1.99	1.60	2.03	1.80	1.45	1.80
△ Drift index δ	1.13	0.93	0.51	0.57	0.49	0.57	0.61	0.55



	A7NP+T	A7X+T	A7T	A7NP+T*	A7X+T*	A7T*
● Period index τ	1.81	1.89	1.74	1.89	2.02	1.85
□ Force index ϕ	1.90	1.81	1.97	1.81	1.69	1.85
△ Drift index δ	0.51	0.52	0.47	0.55	0.57	0.54

Figure II.2-11 - Period, force and drift index for the 5- and 7-storey configurations of Building A. (a) 5 storeys; (b) 7 storeys

The comparison in Figure II.2-11 for the 5-storey building confirms the substantial decrease of fundamental periods of vibration and the increase of tensile forces for the strengthened shear walls. The improvement is evident in terms of DLS verification, confirming that it is fulfilled for all the strengthened configurations of the 5- and 7-storey shear walls. On the contrary, δ is higher than 1 for the A5H configuration, confirming that the DLS verification is not fulfilled and stiffening of the shear wall is needed. Finally, the results for the optimized configurations are confirmed to be similar to the non-optimized ones, with slight decrease of lateral stiffness and consequent increase of inter-storey drift.

II.2.4.2 Evaluation of the actual capacity with non-linear static analyses

The actual mechanical behaviour of the proposed innovative systems and their ultimate capacities have been assessed and compared with those of traditional systems performing non-linear static analyses. In detail, the 5-storey configurations of building A have been studied (A5H, A5X, A5NP+T, A5X+T, A5T) applying two imposed deformed shapes, i.e., two horizontal force patterns per floor: one providing the same forces for each floor (constant distribution) and the other proportional to the height of each floor (perfect triangular distribution). It is expected that the actual dynamic response can be within the two capacity curves (base shear force vs. horizontal top displacement) obtained with the two force patterns. The models described in Figure II.2-4 were adapted for the non-linear analyses, modifying the linear elements representing connections (hold-downs, nailed plates, angle brackets and X-RAD) with non-linear elements, according to an elastic-plastic behaviour reproducing the experimental behaviour available in [26,79,135] and adding compression-only elements to simulate the contact between panels at each floor level and between panel and foundation. Friction effects were not considered in the model. The non-linear static analyses returned the base shear force vs. top displacement curves shown in Figure II.2-12 and Figure II.2-13. The curves show a limited ductile behaviour regardless of the force pattern. In detail, the wall system equipped with the traditional connections behaves almost elastically up to failure of the hold-down connections, which represent the weakest elements. On the contrary, the wall systems equipped with ties show a clear transition between elastic and post-elastic phase, allowing the exploitation of the ductile phase of the shear resistant connection elements. The results in terms of global elastic (K_{el}) and post-elastic (K_{pl}) stiffness, ductility, maximum inter-storey drift at failure and actual failure mode are listed in Table II.2-7 and Table II.2-8.

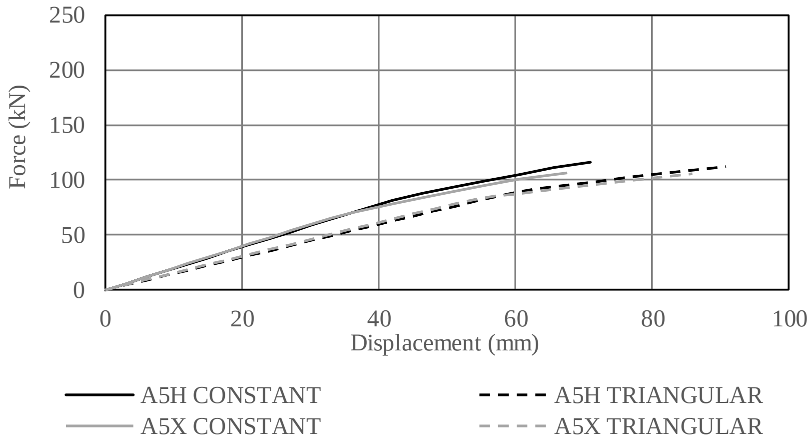


Figure II.2-12 - Base shear force vs. horizontal top displacement curves for traditional systems

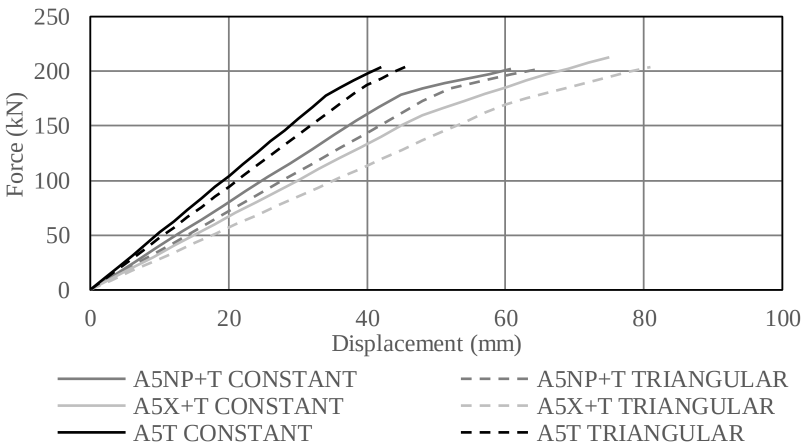


Figure II.2-13 - Base shear force vs. horizontal top displacement curves for the configurations with steel ties

Table II.2-7 – Results of non-linear static analyses for triangular deformed shape

	K_{el} (kN/mm)	K_{pl} (kN/mm)	Ductility	Maximum inter-storey drift at failure (‰)	Failure mode
A5H	1.47	0.8	1.6	7.1	Hold-downs
A5X	1.5	0.77	1.65	6.2	X-RAD
A5NP+T	3.54	1.5	1.25	5.4	Angle brackets
A5X+T	2.9	1.6	1.35	5.3	X-RAD
A5T	4.6	2.9	1.15	5.1	Angle brackets

Table II.2-8 – Results of non-linear static analyses for constant deformed shape

	K_{el} (kN/mm)	K_{pl} (kN/mm)	Ductility	Maximum inter-storey drift at failure (‰)	Failure mode
A5H	1.93	1.17	1.7	7.5	Hold-downs
A5X	1.95	1.14	1.8	6.6	X-RAD
A5NP+T	4.04	1.5	1.34	5.7	Angle brackets
A5X+T	3.35	1.6	1.48	5.5	X-RAD
A5T	5.1	3.1	1.15	5.4	Angle brackets

The obtained results highlight that all the configurations with steel ties show stiffness much greater than configurations without steel ties (about two-times greater). The displacement capacity showed by all the configurations is between 60 and 80 mm, except for the case with steel ties only, which reaches about 40 mm at failure. The ductility values obtained for all the investigated configurations are anyway limited, also for the case of CLT systems assembled with traditional high-capacity connectors, according to similar observations in the literature [21]. In order to account for the limited values of building ductility the analyses were performed using the lowest q-factor values suggested by codes and recommended in related literature. Moreover, according to Pozza and Trutalli [141] the q-factor values account for both ductility and overstrength confirming the adequacy of the adopted q-values. Finally, Figure II.2-14 illustrates conceptually the overall deformation distribution along the height of the wall. The weak link, where the first failure occurs, is highlighted with a circle, according to the position of connections in Figure II.2-14. The force pattern proportional to the height of each floor (perfect triangular distribution) is shown since deformation is similar to the other one. It can be noted that the configurations without steel ties are characterized by a prevalent rocking behaviour. On the contrary, in the configurations strengthened with steel ties, the sliding contribution for each storey is evident.

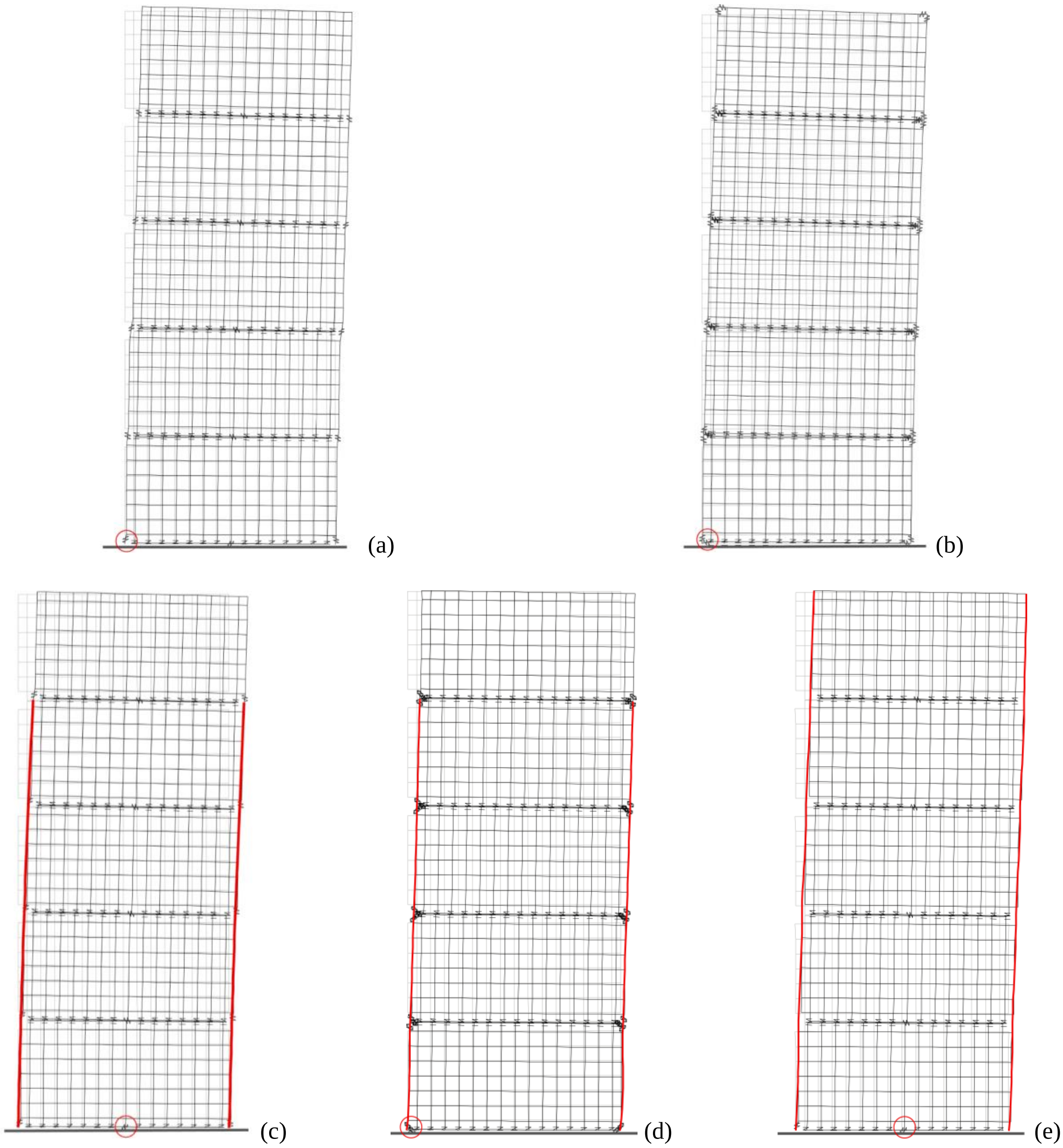


Figure II.2-14 - Deformation shape of the shear walls for the force pattern proportional to the height of each floor (perfect triangular distribution). (a) A5H; (b) A5X; (c) A5NP+T; (d) A5X+T; (e) A5T

II.2.5 Conclusions

Three earthquake-resistant systems for timber shear-wall structures have been presented, based on the use of vertical steel ties as alternative or in addition to traditional nailed plates or screwed connections. The proposed technologies are intended for multi-storey CLT buildings and have the aim of withstanding high tensile forces due to rocking of the timber panels, transmitting them directly from each storey level to the concrete foundation. At the same time, the ties stiffen the structure, reducing lateral displacements and inter-storey drifts. The fastening of the ties to the panels at each storey level can be realized with traditional nailed plates or screwed connections. As an alternative to the use of dowel-type fasteners, the ties can be directly secured to thick steel plates, placed in direct contact to the top narrow side of each wall panel.

Previous studies available in the literature proved that the main issues of multi-storey CLT buildings subjected to seismic action are the limited lateral stiffness and the high tensile forces concentrated in hold-down connections, which become the critical components of the building. The parametric dynamic analyses presented in this Section II.2 have confirmed this conclusion and have demonstrated that the proposed use of steel ties to realize high-strength shear walls can address both these issues, improving the feasibility of multi-storey CLT buildings in high-seismicity areas. This strategy makes also possible a reduction of the number of earthquake-resistant walls and connections in the building.

The parametric analyses have shown that the spectral accelerations increase by adding the ties to the structure, due to an increase in the global stiffness of the shear walls and consequent decrease of periods of vibration. Nevertheless, the tensile forces in the connections significantly decrease, being partially or totally resisted by the steel ties, depending on the fastening strategy adopted. Results in terms of top displacements and inter-storey drifts have demonstrated that the use of steel ties improves significantly the stiffness and the elastic response of the building, with clear advantages in complying with requirements for damage limitation state.

Finally, the reduction of the cross-section area of the ties from the foundation to the top of the building according to the seismic demand leads to the optimization of the structural material without affecting the response of the building.

The earthquake-resistant systems presented in this work have proven a good seismic performance, making them worthy of further developments and research. The non-linear static analyses have demonstrated the actual response of the analysed techniques, which ensure a significant increase of global stiffness without relevant reduction of ductility and displacement capacity when the ties are nailed or screwed to the CLT panels. Additional analyses can be performed to investigate the possible coupling of the proposed systems with ductile devices, able to dissipate energy, in order to reduce the inertial forces introduced in the steel ties at each floor level.

II.3 Technological details for the connection system based on the use of vertical steel ties

Abstract

The development of new strategies able to increase the lateral stiffness of multi-storey CLT buildings requires the study of technological details able to satisfy structural, durability and energy efficiency issues. Technological solutions implementing the use of vertical steel ties placed at each shear wall end are proposed paying particular attention to the connection node between the CLT panel and the reinforced concrete foundation that represents a critical detail for all timber buildings. A new solution for this node is presented: it consists of a linear concrete beam reinforced with steel elements that allows to keep the base of the panel raised with respect to the level of the foundations, to guarantee the continuity of the insulation placed on the external side of the panel and to resist to the high tensile and shear forces typical of tall CLT buildings. Moreover, additional technological details to connect CLT panel to tie down and to transfer shear forces form upper to lower floors are proposed.

II.3.1 Overview on technological details developed to connect CLT panel to foundation in multi-storey CLT buildings

The technological detail of the CLT wall to reinforced concrete foundation node represents one of the most critical issues in timber construction. In this Section some important issues related to this node and several solutions currently available in literature are described.

The above-mentioned construction detail, if not properly designed and expertly built on site, can compromise the durability, structural proprieties and energy efficiency of timber constructions. As regard durability, an inaccurate waterproofing of the CLT wall base could lead to a degradation and, in a short time, to the loss of the bearing capacity of the walls, compromising the structural safety of the entire building. Several calculations and tests were performed to determinate the interaction between reinforced concrete foundations and timber structure [141,142,143] that confirms the need of a moisture barrier at the interface between wood panels and concrete foundations in order to limit the moisture content and to preserve mechanical proprieties of timber walls. Guidelines, such as ÖNORM 2320 [144], provide proper constructive details to ensure the wood preservation. As for the mechanical properties, once the protection of the panels against humidity has been guaranteed, it is essential to design connection elements so as to transmit the shear and tensile forces to the foundation. In low- and mid-rise CLT structures, tensile and shear forces are generally resisted respectively by traditional connectors, such as hold-downs and angle brackets. As the number of storeys of the building increases, it has been demonstrated that the increase in tensile forces at connections requires the use of special connection elements, such as the solution based on vertical steel ties placed at each shear-wall end described in Section II.2. Technological details that implement steel ties will be proposed in Section II.3.2. Finally, as regard energy efficiency it is important to guarantee the continuity of the insulation layer on the external side of the panel avoiding thermal bridges.

Different techniques for supporting timber CLT walls at foundation have been developed in recent years; some of the most important solutions available in literature are:

1. Direct support to the reinforced concrete foundation;
2. Support on a reinforced concrete curb placed between CLT wall and foundation slab;
3. Support on innovative aluminium beam placed between CLT wall and foundation slab.

A description of such structural system is reported in the following.

II.3.1.1.1 <u>Direct support to the reinforced concrete foundation;</u>

In this solution, the base of the wooden walls is positioned at foundation level below the water flow surface, as reported in Figure II.3-1.

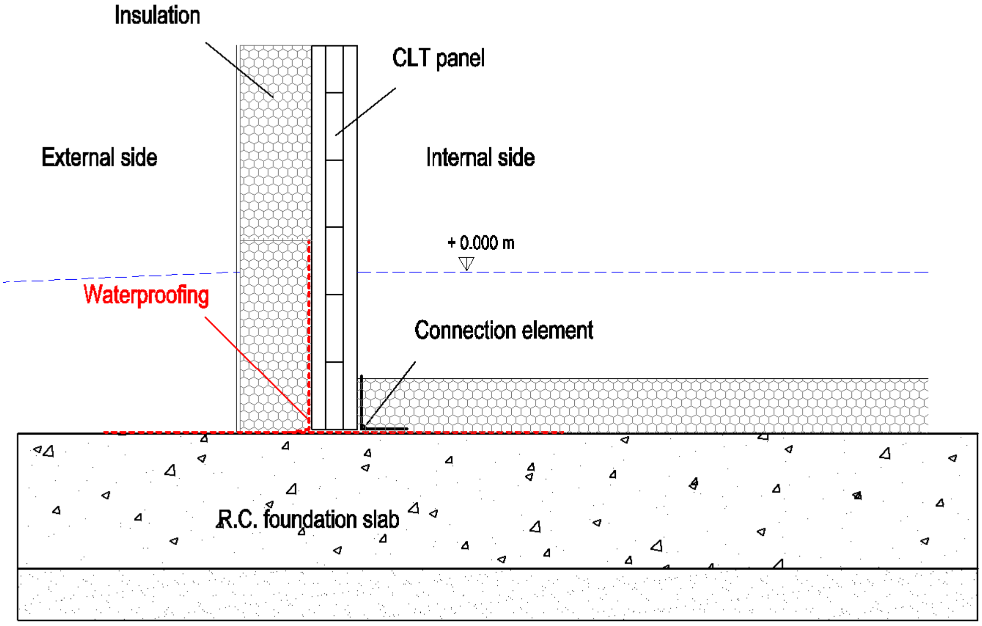


Figure II.3-1 - Direct support to the reinforced concrete foundation level

In this case, the durability of the wall is entirely assigned to the good execution and quality of the waterproofing system. Due to the inherent difficulties in the realization of a tight waterproofing, this solution is nowadays highly discouraged since several applications have shown sealing and stagnation problems that have induced degradation of the base of the wooden wall, see Figure II.3-2.

It should be noted that waterproofing must be applied to permit a correct drying of timber, when a moisture source is present.

Figure II.3-2 – Examples of damaged internal timber wall due to water attack [146]

II.3.1.1.2 Support on a reinforced concrete curb placed between CLT wall and foundation slab

In this solution, two different reinforced concrete curbs can be realized: squat curb (Figure II.3-3) and slender curb (Figure II.3-4).

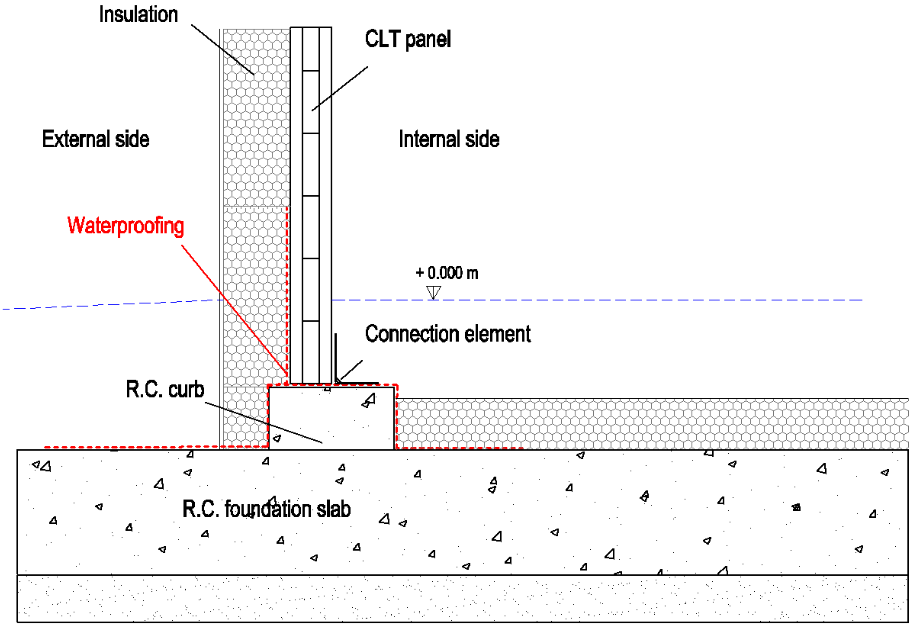


Figure II.3-3 - Support on squat concrete curb

Squat curb solution is not sufficient to prevent durability problems because the base of CLT panel remains generally below the water flow surface. Despite this, the use of a raised curb allows a better result than the direct attack on the foundation because the contact with any water infiltration at foundation level is avoided. In some cases the renifoced concrete curb is sostituited by the use of wooden beam (generally larch beam), but the compression perpendicular to grain limits its load-bearing capacity for mid- and high-rise buildings.

In order to avoid any problem of durability a slender concrete curb, where the base of CLT panel is raised respect the water flow surface, should be realized, as reported in Figure II.3-4.

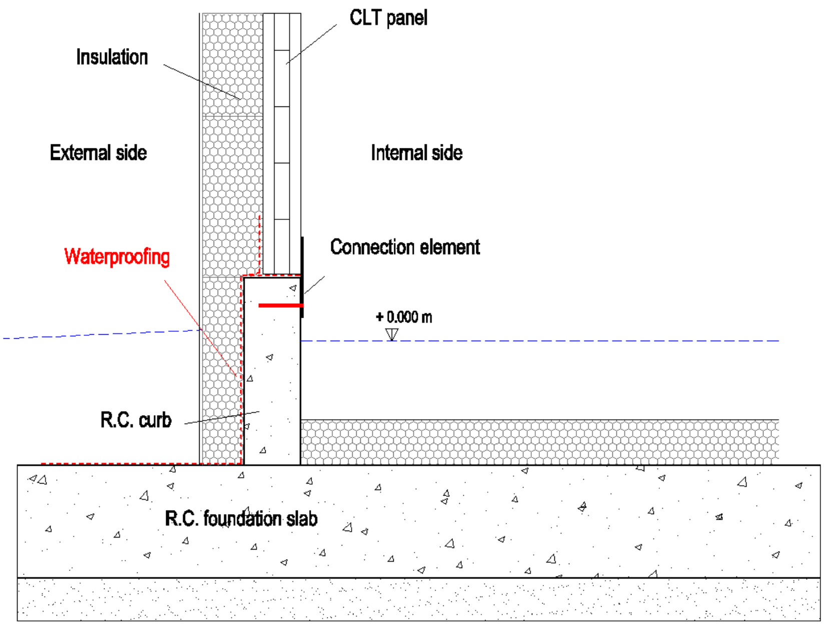


Figure II.3-4 - Support on slender concrete curb

This solution represents the better solution from durability point of view, but it penalize the mechanical prorieties of node. In fact it is impossible to reach high resistances using traditional connectors, such as threaded rods or mechanical anchors, inserted in the limited width of the curb. It is worth noting that the slender RC curb must be realized with geometric accuracies in order to avoid discrepancies between the concrete surfaces and the timber panels. An interesting solution able to exceed this limit has been devolped by STP srl [145] and is reported in Figure II.3-5, where tensile and shear connections are integrated in reinforced concrete curb, see Figure II.3-6, Figure II.3-7 and Figure II.3-8.

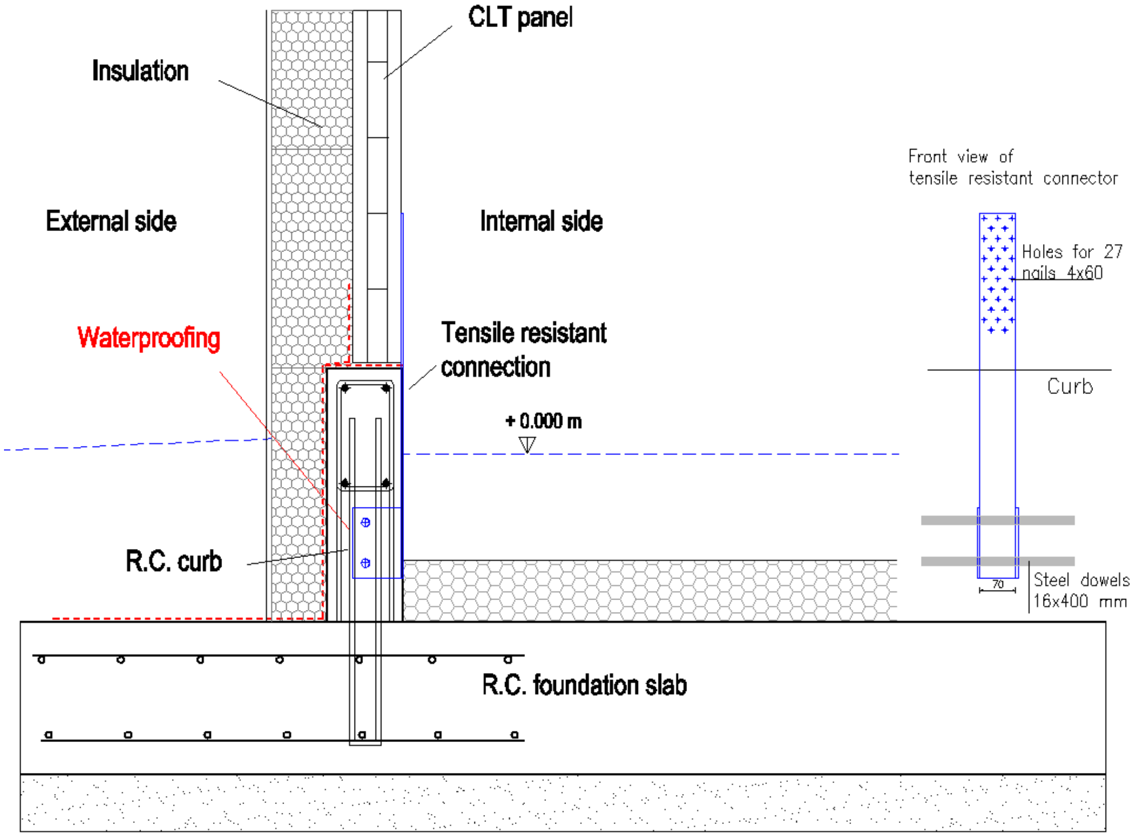


Figure II.3-5 - Support on slender concrete curb with tensile resistant connector integrated

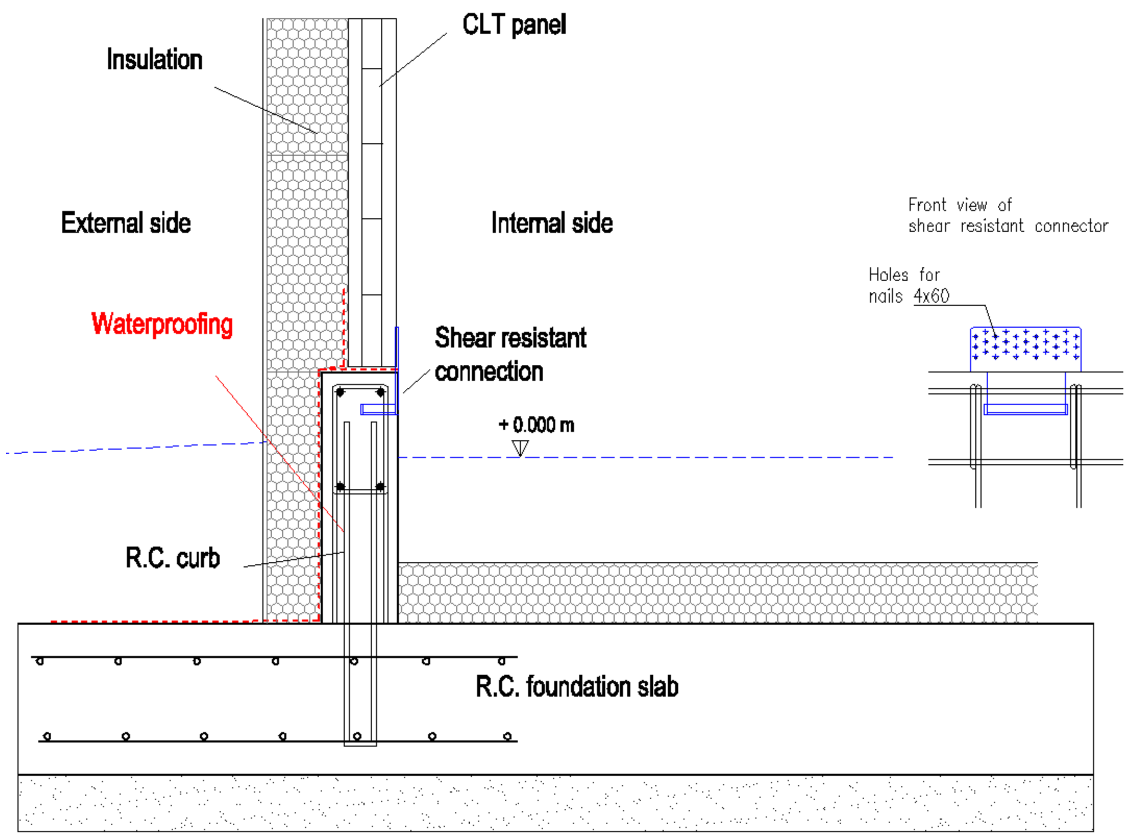


Figure II.3-6 - Support on slender concrete curb with shear resistant connector integrated

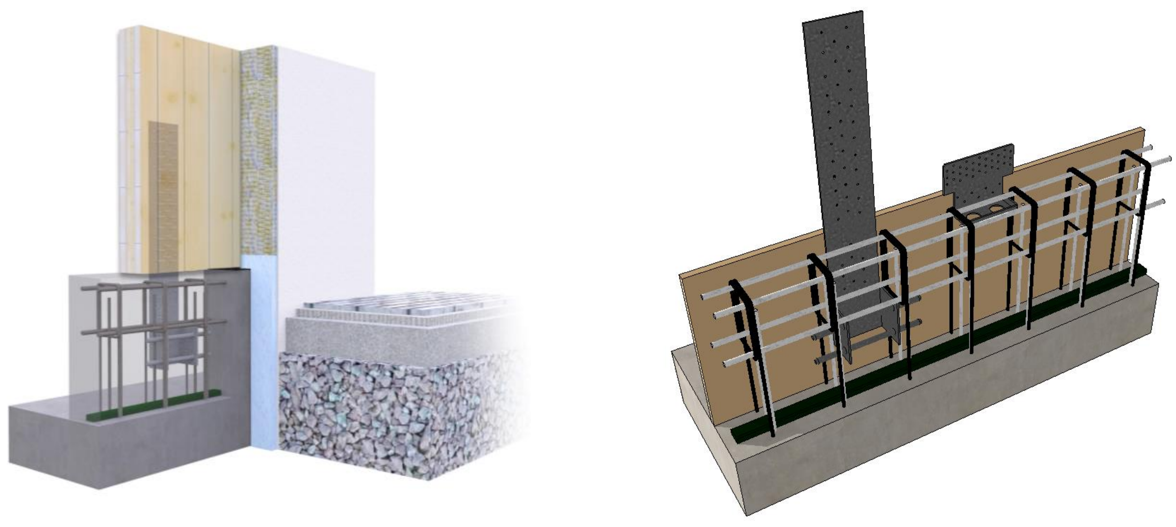


Figure II.3-7 – Tensile and shear resistant connectors integrated in RC curb [145]

As can be noted in figures above reported, slender curb solution with connectors integrated is able to:

- avoid durability problems being the base of CLT panel raised respect the water flow surface;
- guarantee good mechanical proprieties thanks to the use of connectors directly integrated in concrete curb avoiding the use of threaded rods with chemical anchor or screwed/expansion anchor for concrete that represents the weak point of the connection;
- guarantee energy efficancy thanks to the continuity of the insulation placed on the external side of the panel.

An example of application of this solution is reported in Figure II.3-8.

Figure II.3-8 – Example of tensile and shear resistant connectors integrated in RC curb [145]

II.3.1.1.3 <u>Support on innovative aluminium beam placed between CLT wall and foundation slab</u>

Foundation systems based on the use of innovative aluminium elements have been recently developed. The system studied by Scotta et al. [146,147] consists of a bottom-rail beam, realized using an extruded aluminium profile and conceived to avoid the rising dampness from the foundation to timber walls. Different types of the aluminium profile are reported in Figure II.3-9.

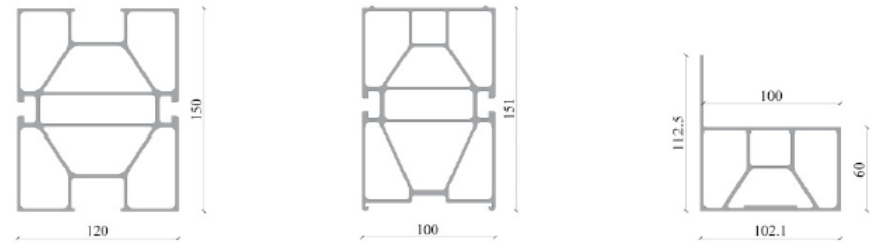


Figure II.3-9 – Versions of the aluminium profile developed by Scotta et al. [146]

In this solution, brackets are rigidly fixed to the beam by self-drilling bolts that block both the bracket and the linear joint to the vertical web of the channel, as reported in Figure II.3-10.

Figure II.3-10 – Brackets disposition [148]

A symmetric disposition of the brackets with the wall centred with respect to the beam axis represents the most efficient solution. Another advantage of this system is represented by the possibility to align aluminium beam by means of adjustable bolts attached to special provisional inserts, fastened to the lateral grooves. These use of this type of system allow an accurate and rapid positioning, reducing costs and time for installation. It is worth nothing that cross section of this system was designed by authors to withstand to vertical loads typical of two- to three- storey timber buildings.
On the basis of the solution just described, similar solutions have been recently developed by other companies. Two examples are reported in Figure II.3-11.

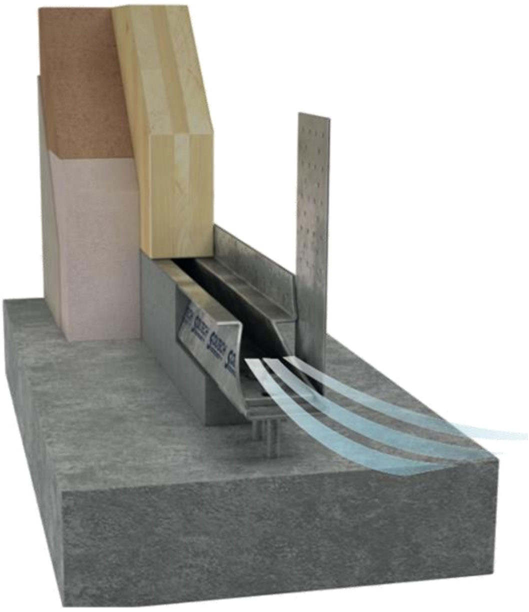


Figure II.3-11 – Innovative beam proposed by Soltech [149] and Rothoblaas [150]

II.3.2 Technological details implementing vertical steel ties for high CLT buildings

Technological details of the CLT wall to reinforced concrete foundation (proposed in previous Section II.3.1) and details of wall-to-floor and wall-to-roof connection (reported in Section I.1.1.4) can be used in low and mid-raise buildings. In Section II.2 it has been proved that that the main issues of multi-storey CLT buildings subjected to seismic action are the limited lateral stiffness and the high tensile forces concentrated in hold-down connections, which become the critical components of the building. In order to overcome these limits, an innovative earthquake-resistant system for timber shear-wall structures based on the use of vertical steel ties has been presented. According to Section II.2, three technological detailing that implements the use of vertical steel ties have been studied and are described in the following:

a) steel ties coupled with nailed plates;

b) steel ties coupled with the screwed connection X-RAD;

c) steel ties only.

All the solutions present a reinforced-concrete curb placed between CLT wall and foundation slab that plays a key role in the preservation of timber panels, raising the latter respect the water flow surface and avoiding decay due to moisture. Tensile forces are transfer directly from steel ties to foundation slabs, while shear forces are transferred, first, form CLT panel to concrete curb, and then, from curb to foundation slab.

The following subsections present details of three technologies above mentioned.

II.3.2.1.1 Steel ties coupled with nailed plates

Proposed system is composed by steel ties welded to steel plates, which are nailed to the panels at each storey level, see Figure II.3-12. Dimension and number of nails of each plate varies according to the acting forces. Role of steel ties is to connect the nailed plates together and to transmit the tensile forces directly to the concrete foundation, while the nailed plates work as tensile rocking resistant connectors at each storey. According to this scheme, the storey rocking tensile forces are directly transferred to the foundation through the ties. For symmetry reason, it is preferable to place the nailed plates and the steel ties at both sides of the wall in order to avoid eccentricities. Full-threaded screws can be used to reinforce CLT panel where local high compression stresses occur. Vertical steel ties are bolted to a steel base boxes directly fixed to the foundation slab. In order to ensure the free rotation to the tie in the wall plane, slotted holes are used at the end of the ties preventing uncontrolled deflection phenomena. Steel plates are also be used to avoid out-of-plane displacements between the panels of the upper and lower floors. To transmit shear forces, a possible solution is to use wood to wood (for upper floors) and wood to concrete (for base floor) shear key.

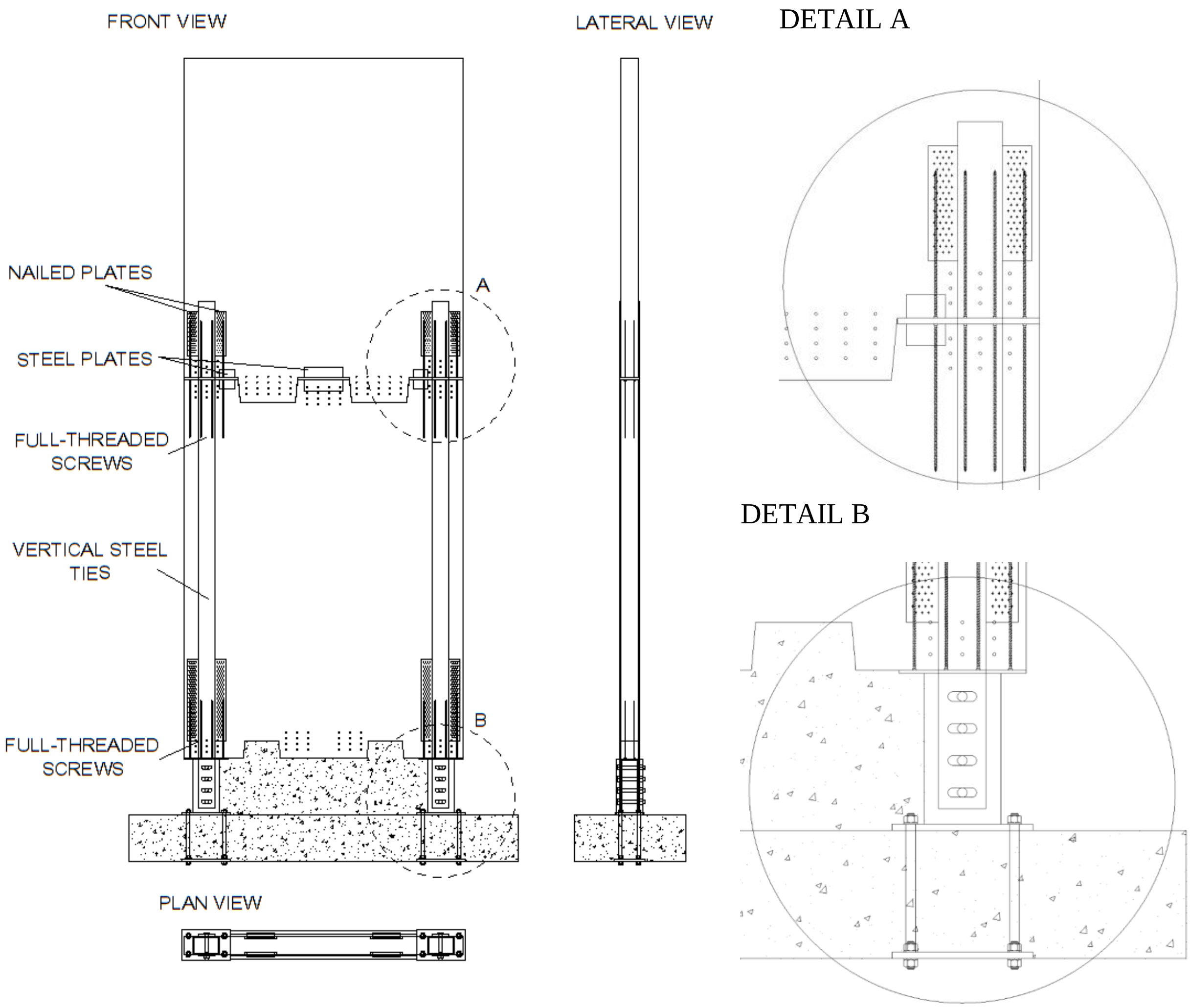


Figure II.3-12 – Solution with steel ties coupled with nailed plates

II.3.2.1.2 Steel ties coupled with the screwed connection X-RAD

This system is based on the combination of X-RAD connection and steel ties, which work together in order to resist to tensile forces: furthermore X-RAD can also be used to resist to shear forces placing an additional X-RAD in the middle of the CLT wall as depicted in Figure II.3-13. The steel ties, disposed in a symmetric configuration in order to avoid eccentricities, are fixed to the X-RAD at each floor level thanks to the use of bolts. As for the ties combined with nailed plates, at each storey the X-RAD withstand only the tensile force due to local rocking since the tensile forces induced by the global inter-storey rocking phenomenon are directly transferred to the foundation through the tie. Vertical steel ties are fixed to foundation slab using a steel element, according to the strategy explained in previous Section.

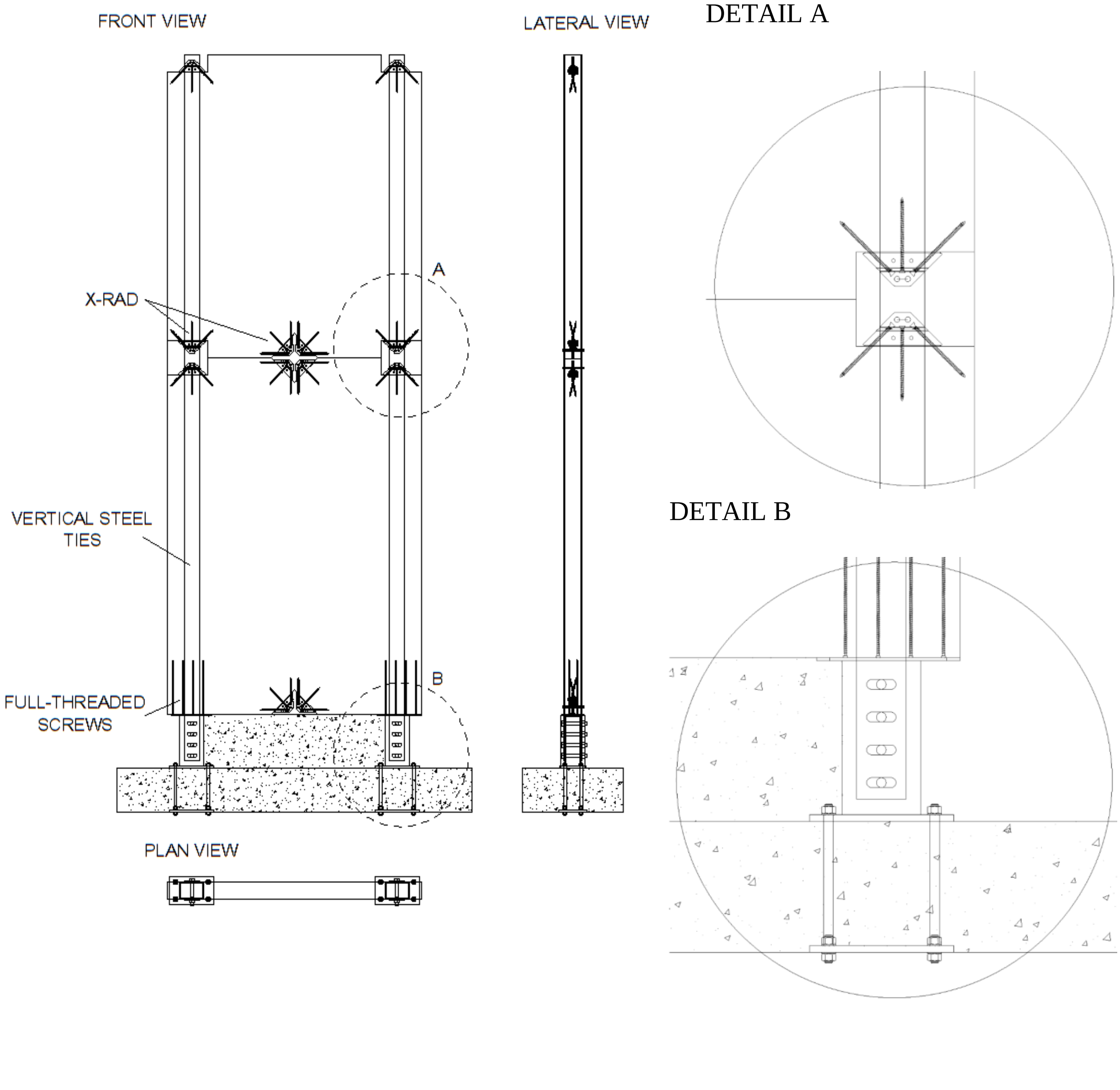


Figure II.3-13 – Solution with steel ties coupled with X-RAD

II.3.2.1.3 Steel ties only

Differently from previous solutions described in Section II.3.2.1.1 and in Section II.3.2.1.2, the steel ties only one does not involve the coupling of ties with other connections. In this configuration, the rocking behaviour at each storey is prevented by thick steel plates, in direct contact with the top narrow side of each wall panel. Full-threaded screws can be used to locally reinforce the CLT panel in the areas subjected to high compressive stress. Ties can be realized with two steel bars placed at each sides of the wall; the bars are secured to the plates at each floor level thanks to the use of bolts and to the foundations slab thanks to the use of a steel element. To transmit shear forces, a possible solution is to use wood to wood (for upper floors) and wood to concrete (for base floor) shear key.

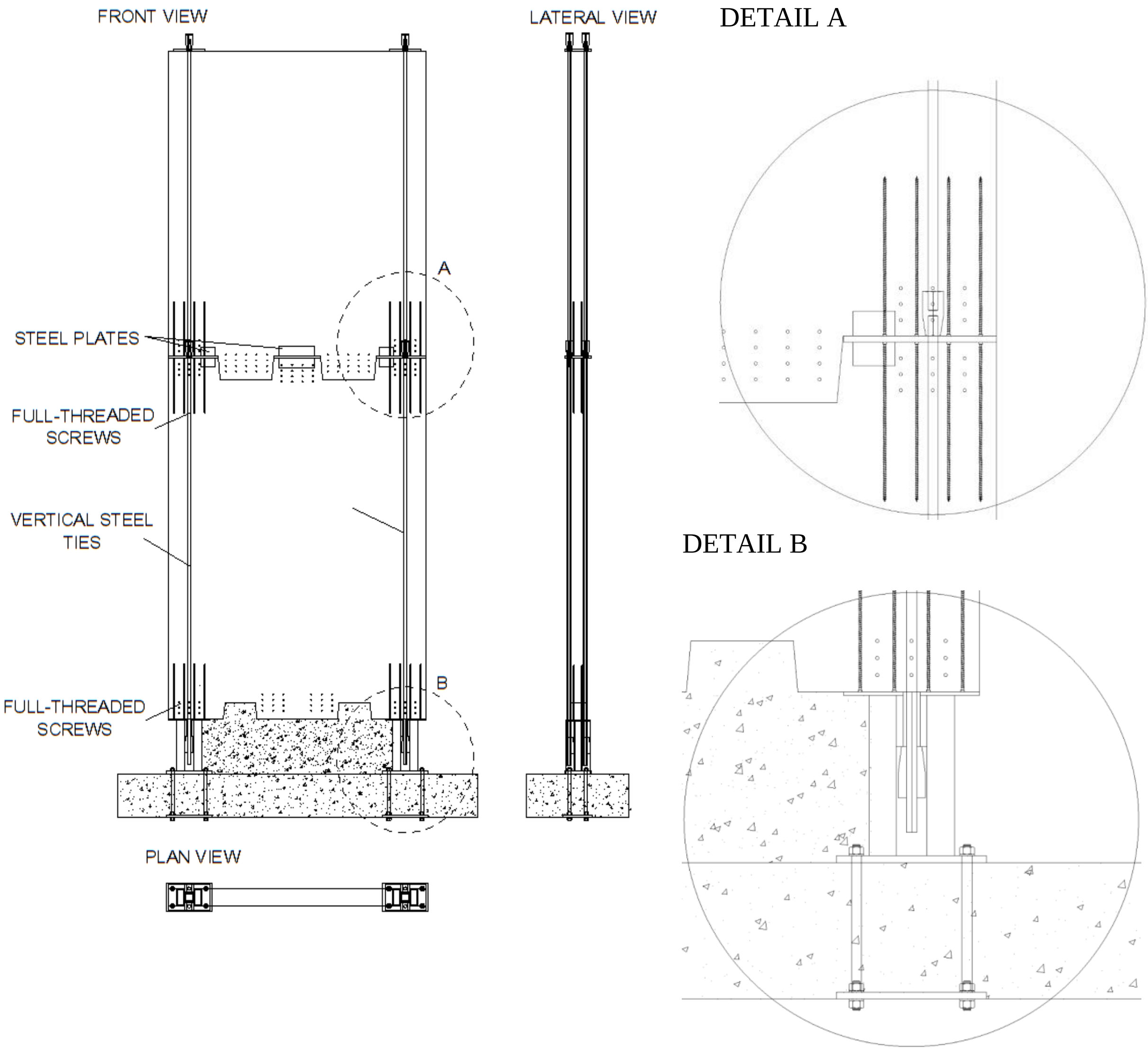


Figure II.3-14 – Solution with only steel ties

II.3.3 Conclusions

Technological details for the original connection system based on the use of vertical steel ties and able to satisfy structural, durability and energy efficiency issues have been proposed. The developed details are intended for multi-storey CLT buildings, where high tensile forces occur on tensile-resistant connector elements. Particular attention has been paid for the connection node between the CLT panel and the reinforced concrete foundation that represents one of the most critical issues in construction with a timber structure. Solutions available in literature, that can be used in low and medium-raise buildings, proved that several problems are related to this node. In order to overcome this limit, a new technological detail for the connection of CLT shear walls to the reinforced concrete foundation, that implements the use of vertical steel ties, has been developed. The proposed technology consists of a linear concrete beam reinforced with steel elements able to keep the base of the panel raised with respect to the plane of the foundations, able to guarantee the continuity of the insulation placed on the external side of the panel and able to resist to the high tensile forces typical of tall CLT buildings. Three different technological solutions for the connection between the CLT panel and tie down have been proposed: the first is based on the use of nailed plates, the second on the use of screwed connection X-RAD and the third on the use of only steel ties. An experimental campaign and additional analyses will be performed on the innovative earthquake-resistant system based on the use of vertical steel ties with the aim of verify the effectiveness of the proposed technological details.

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Conclusions and future developments

In this Thesis innovative numerical modelling strategies and original earthquake-resistant systems for CLT structures have been investigated and developed. In detail, three topics have been analysed: (i) numerical strategies for the characterization of seismic behaviour of CLT buildings (Section I.2), (ii) calculation method for the estimation of vibration periods of CLT structures (Section I.3), innovative earthquake-resistant systems for CLT shear-wall structures based on the use of vertical steel ties Section (II.2). In the following , a summary of the main findings from each research topics is reported.

As regard modelling strategies for CLT multi-storey buildings, two numerical approaches have been analyzed in order to define a numerical strategy able to reproduce the seismic response of CLT structures easy to implement in commercial software and characterized by a low computation afford that will be used by practitioners in the design phase. The first approach, named “component-level approach” is the most complex to implement and is generally used in research field. It requires the implementation of all the single components of the structural system (connections and CLT panels) and the calibration of their mechanical proprieties. This calibration is usually based on the results of experimental tests or on analytical procedures. Completed the calibration and implementation of all the single components of the structural system in numerical software, CM approach allows to simulate faithfully the structural behaviour of any buildings under seismic actions.
The second approach, named “phenomenological approach” could be used by engineers in the design phase since its simplicity and its direct and straightforward implementation in commercial software that leads to a time saving, making the project timing compatible with those of practitioners. Phenomenological model approach available in literature aims to reproduce the global response of the structure by means of elastic stiffness assigned to the CLT wall and disregards the contribution to the structural response given by connection systems but it has been demonstrated that this assumption leads to evaluate the behaviour of the structures in an inaccurate way because connections play a key role in the characterization of CLT structures influencing the main parameters such as the fundamental period T1 and displacements. To overcome this problem, a reduced equivalent elastic stiffness to be assigned to CLT panel and able to consider both the connections and panel deformability has been introduced and calibrated. The calibration phase of equivalent stiffness demonstrates that it depends from different parameters such as number of storeys, connection pattern, geometry and wall slenderness. A correlation between the equivalent elastic stiffness and the geometric proprieties of the structure was found thanks to least-squares method that minimizes the error on the fundamental parameters. Results have been included in an abacus and in a formulation that can be used by practitioners in the design phase.
Another crucial issue that regards numerical modelling of CLT structure was investigated in the first part of this Thesis where the relationship between the rocking behaviour of CLT shear walls and the natural period of CLT structures was evaluated. It has been observed that when the CLT system is subjected to significant vertical loads (capable of avoiding the overturning of the walls) the principal vibration period is not affected by the traction-resistant connection stiffness. On the contrary, when negligible vertical loads are applied to the CLT wall the principal vibration period is controlled by the hold-down connection stiffness. Modal tests on a full-scale timber mock-up and non-linear elastic incremental dynamic analyses were carried out to determine the natural period as function of the lateral response amplitude. The significant shift of natural period of CLT buildings, from the condition when rocking does not occur to the condition which the shearwalls rocking is activated, was demonstrated both from experimental campaign and from numerical analyses. Results obtained from the experimental campaign were adopted to validate a FE numerical models. Therefore, the model was used to perform extended parametric analyses with the aim of evaluating the effects of vertical load, stiffness, yield displacement of mechanical anchors and geometrical dimensions of CLT shearwalls on the determination

of the natural period of the CLT structures. Results of parametric analyses demonstrates a significant dependency of the natural period on the magnitude of vertical load as well as the stiffness of the hold-down, while changing the length of walls and number of storey not much affect values of the natural period for the same value of the lateral drift. An analytical expression to predict the maximum range of natural period which a CLT shearwall may exhibit under different levels of the dynamic lateral response is reported as function of the vertical load and the equivalent rocking slenderness. This formula can be adopted to validate the natural period adopted in the design of CLT building using linear-elastic Response Spectrum Modal analyses.

In the second part of the Thesis innovative earthquake-resistant systems based on the use of vertical steel ties able to allow the realization of high CLT buildings in high-seismicity areas have been proposed. Studies available in literature demonstrated that the main issues of multi-storey CLT buildings subjected to seismic action are the limited lateral stiffness and the high tensile forces concentrated in hold-down connections, which become the critical components of the building. The stiffness and load-bearing capacity of such connections may not be sufficient to realize tall CLT buildings taller than six or seven storeys in high-seismicity areas where rocking contributions plays a key role in the global lateral displacement of the structure. The various strategies proposed in literature for tall and slender CLT buildings in high-seismicity areas require the adoption of special technologies, innovative connections or the coupling of different materials, and the consequent development of new design methodologies, normally not implemented by regulations. To overcome this limit, three original connection system based on the use of vertical steel ties as alternative or in addition to traditional earthquake-resistant systems have been studied. The strategy is to realize high-strength vertical CLT cantilevers to brace the structure, optimizing the number of connections and shear walls in the building, allowing internal free spaces that are required in modern architecture. The proposed system is therefore conceived to withstand high seismic tensile forces that arise in tall multi-storey CLT buildings braced with a limited number of shear walls, limiting rocking, increasing the lateral strength and stiffness of the structure and therefore reducing inter-storey drifts and damages of structural and non-structural components. The optimization of the number of connections and of shear walls in the building allows internal free spaces, that are required in modern architecture, and facilitates the control and possible replacement of connections after a seismic event.
Finally, technological details for the original connection system based on the use of vertical steel ties have been studied with the aim of satisfy structural, durability and energy efficiency issues. Particular attention has been paid for the connection node between the CLT panel and the reinforced concrete foundation that represents one of the most critical issues in construction with a timber structure. The main problems related to this node were described and the several solutions currently available in literature were analyzed. All the solutions described can be used in low and medium-raise buildings, where low forces load the connectors. In order to overcome this limit, a new technological detail for the connection of CLT shear walls to the reinforced concrete that implements the use of vertical steel ties foundation has been developed. The proposed technology consists of a linear concrete beam reinforced with steel elements able to keep the base of the panel raised with respect to the plane of the foundations, able to guarantee the continuity of the insulation placed on the external side of the panel and able to resist to the high tensile forces typical of tall CLT buildings. Finally, three technological solutions for the connection between the CLT panel and tie down have been proposed.

As far as regards future developments, additional analyses will be implemented both in component-level and phenomenological modelling approaches in order to extend the research to cases where a vertical load is applied on the walls. The stabilizing contribution of the vertical load may be enough to prevent the rotation of the walls. In these cases, the deformation contribution due to the tensile resistant connectors must be neglected, thus leading to more rigid systems and higher equivalent stiffnesses to be assigned to CLT walls.

As regard the investigation of the rocking behaviour of shearwalls on the fundamental period of CLT structures, further analyses will be implemented in the next future in order to extend the results to other typologies of CLT shearwalls (e.g. walls with openings, multi-panel walls) and to 3D CLT structures.

Finally, an experimental campaign and additional analyses will be performed on the innovative earthquake-resistant system based on the use of vertical steel ties with the aim of verify their effectiveness and then investigate the possible coupling of the proposed systems with ductile devices, able to dissipate energy, in order to reduce the inertial forces introduced in the steel ties at each floor level.

List of figures

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List of publications and conferences

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