Università degli Studi di Cagliari

DOTTORATO DI RICERCA

IN INGEGNERIA STRUTTURALE

Ciclo XXVI

Seismic design of timber structures - A proposal for the revision of Chapter 8 of Eurocode 8

SSD: ICAR 09: Tecnica delle Costruzioni

Presentata da:	Maurizio Follesa
Coordinatore Dottorato	Prof. Ing. Francesco Ragnedda
Tutor/Relatore	Prof. Ing. Massimo Fragiacomo

Marzo 2015

Esame finale anno accademico 2013 – 2014

Abstract

Sustainability and energy efficiency, together with reductions in construction times and costs are becoming fundamental issues in the housing policies of most of the European and other International Countries. From all these point of views, timber building systems represent a winning choice, due to: (i) the capacity of storing carbon dioxide by displacing it from the atmosphere and reducing energy consumption during the production, transportation and erection compared to other construction materials, (ii) the good thermal properties of wood which, together with insulation materials, easily lead to reach excellent rates in the energy performance of the overall building, and (iii) the great speed of construction due to a completely dry construction process. These advantages, together with the excellent seismic performance witnessed by recent research results based on extensive numerical simulations and full-scale tests on multi-storey buildings, explains the reason why timber systems are becoming more and more popular in the construction of medium to high rise buildings in different seismic areas of Europe, replacing day by day other materials like reinforced concrete, masonry or steel buildings.

However this progress and the growing diffusion have not been accompanied by a corresponding update of the provisions to be used in the seismic design, which for the case of timber buildings are included within Section 8 of Eurocode 8. This chapter was published in 2004 and is very short and incomplete in many parts, especially when considering design provisions for modern construction systems nowadays widely used, thus causing real difficulties to structural engineers who have to apply these rules in the seismic design of timber buildings. Moreover, the differences from other building materials chapters of Eurocode 8 are significant, in some case substantial, and not always conservative.

The aim of this Ph.D. Thesis is to propose a comprehensive review of the current version of Section 8 of Eurocode 8 with the modification of most of the existing provisions and the implementation of some new parts, always from the point of view of a practicing engineer, with the intent to provide clear and simple rules for the seismic design of timber buildings in seismic regions. In this respect, the research work was aimed first at analyzing and investigating the evolution of the subsequent versions of this chapter and the technical-scientific background that led to the drafting of the current version. Subsequently, a detailed analysis of the different problems which arises in the application of the existing provisions is provided in this Ph.D. Thesis.

The research was then directed to the analysis of specific problems related to the seismic design of multi-storey timber buildings through linear and non-linear modelling and design applications performed with a widespread software package among structural engineers like SAP 2000. More specifically, buildings made with the two most used structural systems for the construction of medium to high-rise timber buildings, i.e.

the CLT and the Light-Frame system, were investigated by comparing the analysis results in terms of dynamic characterization and maximum displacements with those of experimental results conducted on full-scale buildings.

Additionally a topic of growing interest in the seismic design of multi-storey timber buildings, i.e. the seismic analysis of hybrid buildings with different lateral load resisting systems, was investigated for a specific application, i.e. the seismic design of multi-storey buildings with mixed CLT/Light Frame shear walls. This study was carried out through non-linear dynamic analysis conducted on a case-study four-storey building with different combinations of CLT and Light-Frame shear walls at each level conducted in order to establish the value of the seismic behaviour factor to be used in the seismic design. The analysis were conducted with DRAIN-3DX, a program for inelastic analysis of structures developed at the University of Berkeley, California, implemented with a numerical model developed at the University of Florence in order to study the non-linear behaviour of mechanical joints and shear walls in timber structures. The existing model has been further implemented with new features like strength and stiffness degradation in order to have a more precise reproduction of the actual non-linear behaviour under seismic excitations.

Finally the last part of the Thesis focuses on a proposal of Background Document for the drafting of a new version of Chapter 8 of Eurocode 8 with, according to the format of other material chapters of Eurocode 8, new provisions for the seismic design of existing and new structural systems not covered by the current version. Furthermore, capacity based design rules are implemented and new values of the over-strength factors and behavior factors to be used in the seismic design according to the different ductility classes currently proposed by Eurocode 8 are provided. Some changes related to the ductility rules for dissipative zones and partial safety factors for material properties to be adopted in the design according to the dissipative and non-dissipative behaviour are also given. New provisions regarding the design of buildings with different lateral load resisting systems, the different analysis methods to be adopted in the seismic design and inter-storey drift-limits are proposed. A code proposal for the evaluation of the behaviour factor q of mixed CLT/Light-Frame buildings is finally proposed.

The research work conducted and the changes of the code provisions proposed within this Ph.D. Thesis could be a sound basis for the next revision of the chapter for timber buildings of Eurocode 8. Further research is nevertheless needed in order to provide more detailed provisions about (i) non-linear static and dynamic analysis methods in order to foster their use in seismic design, (ii) the use of displacement-based design as an alternative to the force-based procedure, (iii) the seismic design of innovative low-damage structural systems and passive base isolation systems for timber buildings, and (iv) design provisions for the retrofitting of existing timber buildings for the next generation of Eurocodes.

Keywords: Eurocode 8, seismic design, Multi-storey timber buildings, Cross Laminated Timber system, Light-Frame system, Linear and non-linear analysis, Behaviour factor, Hybrid buildings

Sommario

La sostenibilità e l'efficienza energetica, insieme alla ottimizzazione dei tempi e costi di costruzione stanno diventando aspetti fondamentali nelle politiche abitative della maggior parte dei Paesi Europei ed Extraeuropei. Sotto questi aspetti gli edifici a struttura di legno rappresentano una scelta vincente grazie (i) alla capacità di stoccare l'anidride carbonica sottraendola all'atmosfera e ridurre i consumi di energia durante le fasi di produzione, trasporto e messa in opera rispetto ad altri materiali da costruzione, (ii) alle buone proprietà di isolamento termico del legno come materiale che, accoppiate all'utilizzo di isolanti termici, consentono di raggiungere livelli eccellenti di efficienza energetica per l'intero edificio e (iii) la grande velocità di costruzione attraverso un processo costruttivo realizzato interamente a secco. Questi vantaggi, accoppiati all'eccellente comportamento sismico testimoniato dai risultati di ricerche svolte recentemente e basate su numerose simulazioni numeriche e prove sperimentali su edifici multipiano in dimensioni reali, spiegano il perché i sistemi costruttivi a struttura di legno stiano diventando sempre più diffusi nella realizzazione di edifici pluripiano in diverse zone sismiche dell'Europa, sostituendo sempre di più soluzioni alternative in cemento armato, muratura o acciaio.

Tuttavia questi progressi e questa crescente diffusione non sono stati accompagnati da un corrispondente aggiornamento delle norme da utilizzare nella progettazione nei confronti delle azioni sismiche, che per il caso degli edifici a struttura di legno, sono contenute nel Capitolo 8 dell'Eurocodice 8. Questo capitolo, pubblicato nel 2004 è molto breve e incompleto in molte parti, specialmente se si fa riferimento alle regole per la progettazione di moderni sistemi costruttivi molto utilizzati attualmente, causando in tal modo delle vere e proprie difficoltà ai progettisti strutturali che si trovano a dover applicare tali regole nella progettazione di edifici a struttura di legno in zona sismica. Inoltre, le differenze rispetto ai corrispondenti capitoli per gli altri materiali da costruzione contenuti all'interno dell'Eurocodice 8 sono sostanziali e non sempre a vantaggio della sicurezza.

Lo scopo del lavoro contenuto in questa tesi è una sostanziale proposta di revisione dell'attuale versione del Capitolo 8 dell'Eurocodice 8 con la modifica della maggior parte delle indicazioni normative esistenti e l'implementazione di alcune nuove parti, tenendo sempre in considerazione il punto di vista del progettista strutturale, con l'obiettivo di fornire regole semplici e chiare per la progettazione di edifici a struttura di legno in zona sismica. A questo scopo il lavoro di ricerca è stato dapprima incentrato sull'analisi e lo studio dell'evoluzione delle diverse versioni di questo capitolo e delle motivazioni tecnico-scientifiche che hanno portato alla stesura della versione attuale e quindi ad un'attenta analisi dei diversi problemi che insorgono nell'applicazione delle regole attuali.

La ricerca è stata quindi indirizzata all'analisi di problemi specifici relativi alla progettazione di edifici multipiano in zona sismica attraverso casi di progettazione e modellazione lineare e non-lineare di edifici realizzati con i due sistemi più utilizzati per la realizzazione di edifici multipiano in legno, ovvero il sistema CLT e il sistema a telaio leggero, condotte con un programma per l'analisi strutturale molto diffuso tra i progettisti strutturali quale è SAP 2000, confrontando i risultati delle analisi effettuate in termini di caratterizzazione dinamica e spostamenti massimi con i risultati di ricerche sperimentali condotte su edifici in dimensioni reali.

In aggiunta è stato affrontato un tema di grande interesse nella progettazione di edifici multipiano a struttura di legno in zona sismica, ovvero l'analisi di strutture miste realizzate con combinazioni di diversi sistemi costruttivi resistenti alle azioni orizzontali, attraverso lo studio di una specifica applicazione, ovvero il caso di edifici multipiano misti CLT/telaio leggero. Questo studio è stato condotto attraverso analisi dinamiche non lineari condotte su un edificio campione di quattro piani con diverse combinazioni di pareti CLT e a telaio leggero, allo scopo di stabilire il valore del fattore di struttura da utilizzare nella progettazione di questa tipologia di edifici in zona sismica. Le analisi sono state condotte con DRAIN-3DX, un programma per l'analisi non-lineare delle strutture sviluppato all'Università di Berkeley, California, implementato con un modello numerico sviluppato all'Università di Firenze per studiare il comportamento non lineare di giunti meccanici e pareti di taglio nelle strutture di legno. Il modello esistente è stato ulteriormente completato con nuove caratteristiche come il degrado di resistenza e rigidezza allo scopo di ottenere una riproduzione più fedele del reale comportanto non-lineare sotto l'effetto delle azioni sismiche. La ricerca è stata quindi finalizzata con una proposta di formula da implementare in normative per la valutazione del valore del fattore di struttura q per sistemi misti CLT/telaio leggero.

Infine l'ultima parte della ricerca è incentrata su una proposta di Background Document per la stesura di una nuova versione del Capitolo 8 dell'Eurocodice 8, che, coerentemente al corrispondente formato dei capitoli relativi agli altri materiali contenuti nell'Eurocodice 8, contiene nuove regole per la progettazione in zona sismica di sistemi costruttivi esistenti e nuovi, non contemplati dalla versione attuale, insieme all'implementazione di nuove regole sui criteri di gerarchia delle resistenze, i valori del fattore di sovraresistenza e i valori del fattore di struttura da utilizzare nella progettazione per le diverse classi di duttilità contemplate dall'Eurocodice 8. Inoltre vengono proposte alcune modifiche alle regole di duttilità esistenti per le zone dissipative e per coefficienti parziali di sicurezza sui materiali da adottare nella progettazione secondo il comportamento strutturale dissipativo e non-dissipativo, insieme alla proposta di nuove regole relative alla progettazione di edifici con diverso sistema resistente alle azioni orizzontali, ai diversi metodi di analisi da utilizzare nella progettazione e ai limiti di spostamento interpiano da utilizzare nella progettazione in zona sismica.

Il lavoro di ricerca condotto e le modifiche normative proposte all'interno del lavoro di tesi possono costituire una solida base su cui impostare la prossima revisione del capitolo relativo agli edifici a struttura di legno dell'Eurocodice 8. Ciononostante ulteriori ricerche sono necessarie al fine di fornire indicazioni più precise nella prossima generazione degli Eurocodici su (i) i metodi di analisi lineare e non-lineare al fine di promuovere il loro utilizzo nella progettazione in zona sismica, (ii) l'utilizzo del metodo di progettazione basato sugli spostamenti al posto dell'attuale metodo basato sulle forze, (iii) la progettazione sismica di sistemi strutturali innovativi a basso livello di danno e sistemi di isolamento passivo per gli edifici in legno e (iv) indicazioni progettuali per il consolidamento strutturale di edifici in legno esistenti.

Parole chiave: Eurocodice 8, Progettazione nei confronti delle azioni sismiche, Edifici multipiano in legno, Sistema a pannelli portanti a strati incrociati, Sistema a telaio leggero, Analisi lineare e non lineare, Fattore di struttura, Edifici misti.

Acknowledgments

I would like to thank first my advisor, Prof. Massimo Fragiacomo, for his support and guidance, for his kind availability, for the interesting and challenging scientific discussions we had, and for his sincere belief in my work which greatly contributed to the final outcome. My gratitude is extended also to my thesis coordinators, Prof. Barbara De Nicolo and Prof. Francesco Ragnedda, for their encouragement during the period of my doctoral studies.

A very special thanks to my colleague Davide Vassallo for the encouragement and support and for the interesting discussion we had in our everyday work, part of which provided me hints and ideas which gave a great contribution to this work. Thanks also to my colleague Ioannis Christovasilis; his research experience, scientific knowledge and numerical skills were of great help.

I gratefully thank Dr. Igor Gavric for providing me the test results of his researches conducted at CNR-IVALSA on CLT connections and walls without which this work would not have been possible and Dr. Giovanni Rinaldin for his advices with the numerical analysis.

Furthermore I would like to express my deep gratitude to Dr. Chun Ni, Dr. Marjan Popovski and Dr. Erol Karacabeyli from FPInnovation for their kindness and for the test results provided on Light-Frame walls and to Prof. Motoi Yasumura, Dr. Chikahiro Minowa, Dr. Minoru Okabe and Prof. Naohito Kawai for their kind availability and for exchanging with me their ideas and invaluable experience during the research conducted within the Sofie Project in Japan, which were of great help for my professional work and academic studies.

Part of the work included in this thesis is due also to the teachings I had over the years from my Master Degree Professor and Director in the period of my research cooperation with CNR-IVALSA within the Sofie Project, Prof. Ario Ceccotti, and to the experience gained by working side by side with my former colleague Marco Pio Lauriola, to which I want to express my sincere thanks.

Last but not least I want to express my deep gratitude to my family, Yayoi and Momo, and apologize with them for all the time I've stolen during these last years and to my parents Benito and Ada who always encouraged and supported me throughout life and my brother and sisters Stefano, Cristina and Alessandra for their kind encouragement.

Table of contents

List of Figures

List of Tables

1 – Introduction

1.1. Research motivation

In the last twenty years the building technology in the field of timber structures has made great strides due to the improvements gained in the automation process and performance of CNC machinery, in the developments reached in the gluing process of wood-based products and with the introduction of new types of mechanical fasteners, especially screws, which have greatly enhanced the possibility of prefabrication of structural components and made the construction process easier and faster.

These progresses have made possible the development of new structural systems such as the CLT system, and the optimization of the construction process of existing ones such as the Light Frame system, the Log House system or the Post and Beam system.

At the same time, several important research projects have been recently (2005-2014) completed with the aim of investigating above all the seismic performance of medium rise timber buildings built with different structural systems, via both full scale tests on entire multi-storey buildings and numerical investigations. Noteworthy are: the SOFIE Project [2005-2012] conducted by CNR-IVALSA Italy, NIED, BRI and University of Shizuoka Japan on the seismic performance of multi-storey CLT buildings (Ceccotti et al. 2013); the NEESWood Project [2004-2010] conducted by Colorado State University, State University of New York at Buffalo, Rensselaer Polytechnic Institute, University of Delaware and Texas A&M University, USA on the seismic performance of multi-storey wood-frame buildings (van de Lindt et al. 2010); the SERIES Project [2010-2013] conducted by University of Trento, Italy, Graz University of Technology, Austria and University of Minho, Portugal, on the seismic performance of multi-storey CLT, Log House and Light Frame buildings (Tomasi and Piazza 2013); the NEES-Soft project [2011-2014] conducted by Colorado State University, Western Michigan University, Clemson University, Rensselaer Polytechnic Institute and California State Polytechnic University, USA on the seismic retrofit of soft-storey woodframe buildings (van de Lindt et al. 2014). All these projects have investigated the seismic performance of multi-storey timber building by means of shaking table tests conducted on full-scale buildings with a number of storeys ranging from 2 to 7, built with the most common structural systems used in the current timber construction practice.

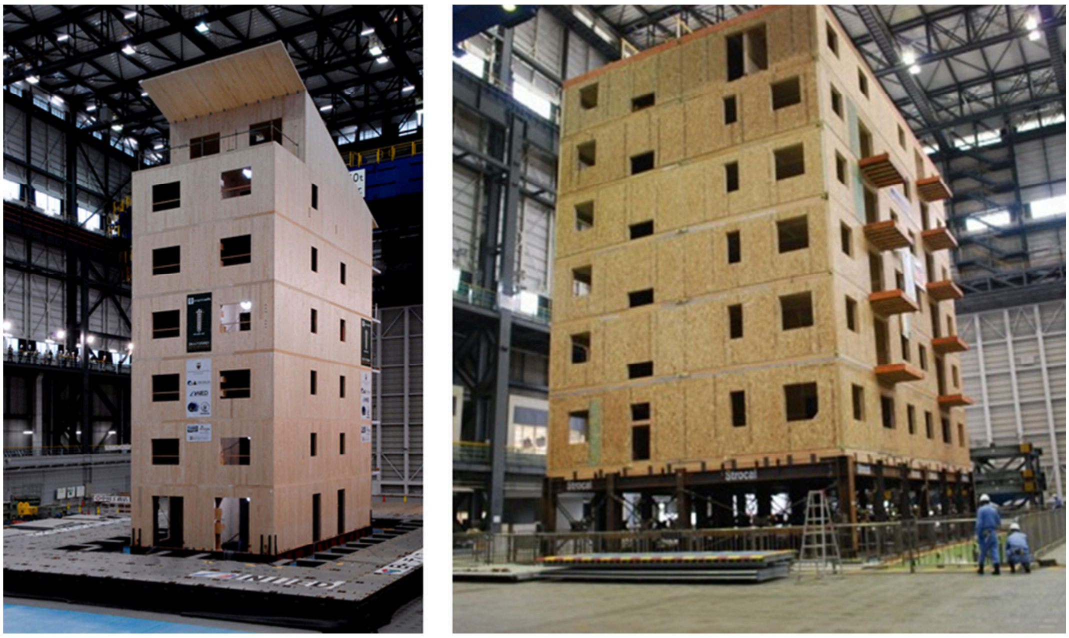


Figure 1-1: Left: 7-storey CLT building tested at the NIED Shaking Table Facility (E-Defence) in Miki, near Kobe, Japan, 2007, within the SOFIE Project. Right: 6-storey Light-Frame building tested at the same laboratory in 2009 within the NEESWood Project.

This background, coupled with the great advantages of timber building systems in terms of sustainability, energy efficiency, good fire performance and above all speed of construction, explains the reasons why in the last years a large number of medium to high-rise buildings have been constructed in areas with different seismic hazard levels in Italy and other European Countries (see Figure 1-2).

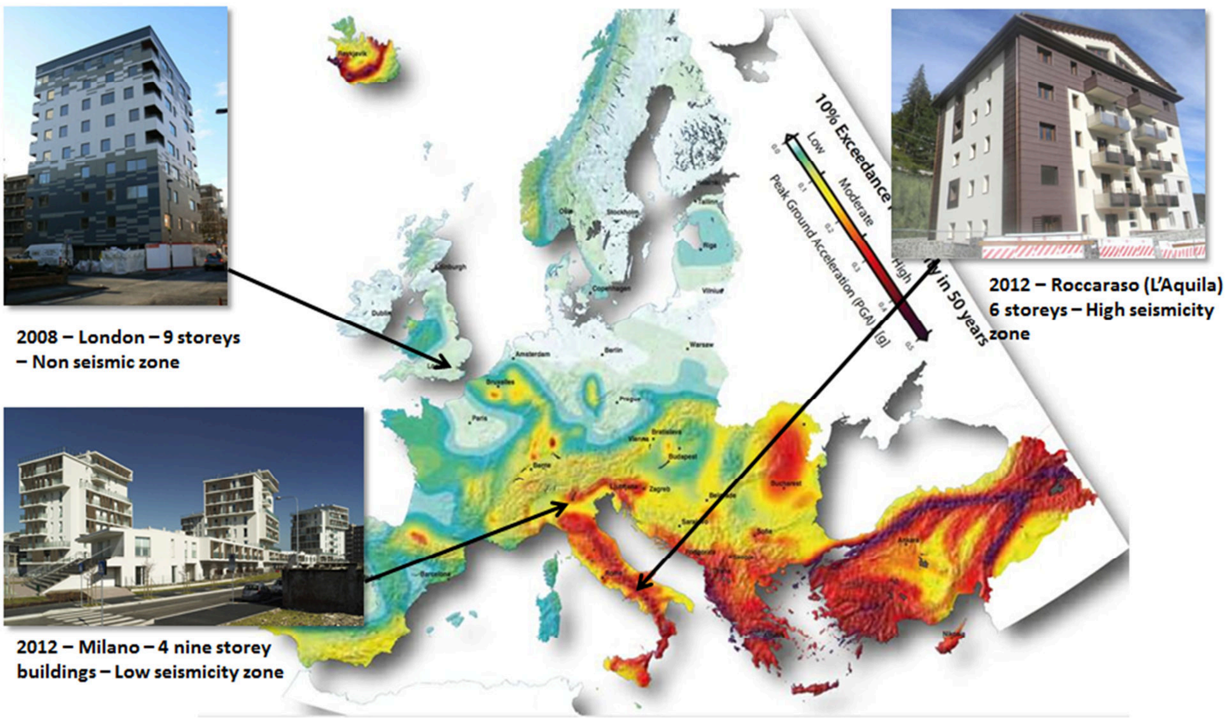


Figure 1-2: Medium to high-rise buildings built in recent years in European areas with different levels of seismic hazard (European Seismic Hazard map from the SHARE web site http://www.share-eu.org).

However these advances of the building technology in the field of timber structures were not followed by a corresponding update of the structural design codes. Particularly when talking about the seismic design, the chapter of the European Standard for the seismic design of structures (Eurocode 8: Design of structures for earthquake resistance, 2004) related to the rules for the seismic design of timber buildings which a European structural designer should apply contains very few indications and is not updated to the actual state of the art, having been written for the most part more than two decades ago, causing therefore real difficulties for its application.

1.2. Seismic design of timber structures according to Eurocode 8

The capacity of a structure to develop plastic deformations in some of its components and dissipate energy without reaching the collapse is an essential part of its ability to withstand earthquake actions. This is true for all types of structures, but particularly for timber structures.

Timber in fact has a linear elastic behaviour until failure, and, especially when talking of structural elements and when referring to the characteristic strength, i.e. the lower values of resistance, the behaviour is prevalently brittle, because collapse will occur in the weakest elements due to brittle failure mechanisms caused by natural defects such as, e.g., notches or grain deviation. The same brittle behaviour will occur in case of cyclic loads, and no energy dissipation will take place, except for perpendicular to the grain stresses or friction.

Glued joints also show an elastic behaviour and do not contribute either to the plastic behaviour or to the energy dissipation. However a very pronounced ductile behaviour and a high energy dissipation capacity for the whole structure can be achieved if the timber elements are connected through properly designed joints made with mechanical fasteners (e.g. screws, nails, dowels or bolts). This is essentially due to a dual phenomenon: embedding of timber due to the crushing of wood fibres on the sides of the fastener hole and plasticization of the metallic fastener with the formation of one or better two plastic hinges.

The combination of these two mechanisms, especially in the case of application of cyclic loading (like in the case of earthquake actions), give the possibility to reach the desired ductile and dissipative behaviour which is essential for the resistance to seismic actions. The load-displacement curves present a typical pinching shape whose central body becomes thinner as the fasteners goes toward higher values of the load. The thinning of cycles is due to the fact that mechanical fasteners have made place in the wood and as the load increase they continue to do with it more and more, with only a small part of the wood crushing deformation recovered elastically, therefore as the load change direction large deformations will occur for small load increases (see Figure 1-3). This behaviour allows the development of relevant deformations before the collapse is reached either due to wood splitting or to fastener failure, and consequently the dissipation of a considerable amount of energy due to hysteresis.

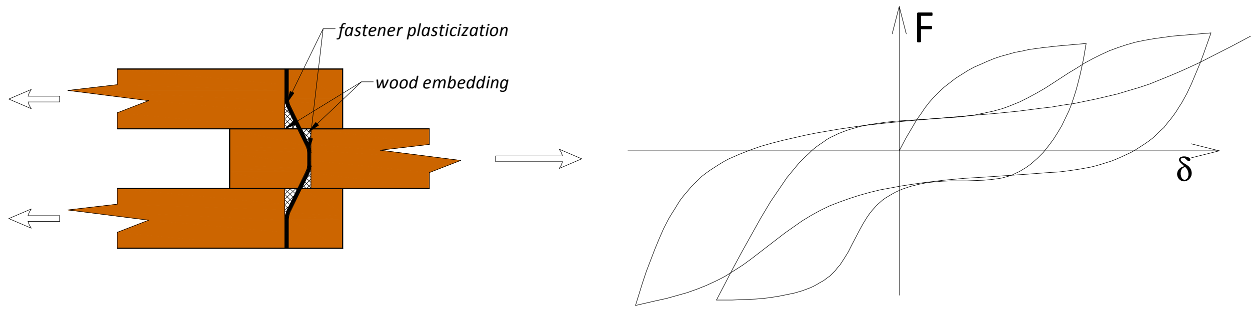


Figure 1-3: Typical cyclic behaviour of a mechanical joint in a timber structure.

This behaviour, coupled with the excellent strength-to-weight ratio of timber, especially if compared to other structural materials such as reinforced concrete or masonry, explains the excellent behaviour of timber structures observed during past earthquakes. When collapse happened, they occurred for reasons mostly independent from the inherent characteristics of wood itself such as (i) lacks or mistakes in the construction process (missing or mistaken installation of fasteners or connectors), (ii) failures due to the substructure or foundation, (iii) marked asymmetries in the structural configuration, (iv) presence of large openings on the ground floor and (v) insufficient resistance of brick chimneys often without any reinforcement (Reiner and Karacabeyli, 2000).

According to the general requirements of Eurocode 8 (CEN, 2004), all structures shall be designed to withstand the foreseen earthquake for that area (i.e. with a typical probability of exceedance of 10% in 50 years for the “no collapse requirement” corresponding to the Ultimate Limit State and of 10% in 10 years for the “damage limitation requirement” corresponding to the Serviceability Limit State) with an appropriate mixture of resistance and energy dissipation, following the Capacity Based Design philosophy.

As it is well explained in the definitions of Eurocode 8 (CEN, 2004), the Capacity Based Design is a design method in which some elements of the structure are chosen and suitably designed for energy dissipation while others are provided with sufficient strength so to keep the chosen means of energy dissipation. For the case of timber structures, the material itself has a brittle behaviour and the energy dissipation capacity of the whole structure could be achieved due to the ductile behaviour of properly designed connections with mechanical fasteners, for the above explained reasons.

In practical terms the energy dissipation capacity of the structure is only implicitly taken into account by dividing the seismic forces obtained from a linear static or modal analysis by the behaviour factor q corresponding to the associated ductility class, which accounts for the non-linear response of the structure associated with the material, the structural system and the design procedures.

Therefore, as it is well explained in §4.4.2.3 “Global and local ductility condition”, of Eurocode 8 (CEN, 2004), 1(P) “*It shall be verified that both the structural elements and the structure as a whole possess adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behaviour factor*.” and 2(P) “*Specific material related requirements, as defined in Sections 5 to 9, shall be satisfied, including, when indicated, capacity design provisions in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.”*

Therefore, in order to achieve the desired means of ductility and energy dissipation capacity, (i) the structural system should be clearly identified, (ii) capacity design provisions should be given so to address without any possible misinterpretation the components of the structural system devoted to the ductile behaviour and on the other end the parts of the structure which should be over-designed to avoid any possible anticipated brittle failures and (iii) ductility rules for the dissipative zones should be given.

This could be achieved by designing the brittle elements by the over-strength factors of the ductile elements in the following way:

over-strength factor X characteristic resistance of the ductile element < characteristic resistance of the brittle element

Summarizing, according to the Capacity Based Design Philosophy, the design procedure of a structure should be based on the following method:

1. Clearly identify the structural type and the associated behaviour factor q;

2. Follow the Capacity Based Design rules and the detailing provisions for dissipative zones defined for the corresponding structural type and

3. Adopt the over-strength factors defined for the design of the brittle elements.

The current version of timber structures in Eurocode 8 (CEN, 2004) is partially or totally missing the above mentioned conditions for most of the structural types currently used in the construction practice; therefore in most cases it is almost inapplicable for the seismic design of most of the structural systems nowadays used for the construction of timber buildings, forcing the structural designer to make assumptions in the seismic design which not necessarily could be on the safe side.

1.3. Background research on the seismic behaviour of timber structures

As reported in §1.1, especially in the last ten years, a relevant number of important research projects have been conducted with the aim of investigating the seismic performance of timber buildings built with different structural systems under earthquake actions. These projects have been undertaken in order to study the structural and seismic behaviour, both through experimental research and numerical investigations, of new structural systems recently developed such as the Cross Laminated Timber system, internationally acknowledged with the acronym CLT (see Chapter 3 for further details), of existing ones like the Light-Frame and Log House system, and also with the purpose of exploring the possible seismic retrofit techniques of existing buildings. In the following a brief overview of the most important ones is provided. The research projects carried out so far and referenced above brought also a huge amount of experimental data and useful information which has been used to develop the additional aspects related to the seismic design of timber buildings. At the same time, due also to the development of powerful software packages for structural analysis, new models for the linear and non-linear analysis of timber structures have been developed and used for research purposes especially in the evaluation of the seismic performance of medium to high rise timber buildings. In the following a brief overview of the most important experimental researches and numerical investigations conducted over the last ten years is provided.

1.3.1. Experimental research

With the construction and development of increasingly performing shaking table facilities in different Countries (Japan, Canada, USA, Portugal, Italy) the experimental research on the seismic behaviour of structures has made significant progress in the last years. Particularly for the case of timber structures, and unlike for e.g. concrete or masonry, due to the lightweight of the material it has been possible to test full-scale structures instead of scaled models reaching almost the highest levels of performance allowed by the seismic testing facilities. This explains the reason why in the last 10-15 years a large number of shaking table tests on full-scale multi-story building has been conducted.

Following some casualties and substantial damage observed in wooden structures after destructive earthquakes occurred in different part of the world (especially the January 17, 1994 Northridge, California Earthquake and the January 17, 1995 Kobe Earthquake), full-scale seismic tests on wooden buildings started since the late 90’s. In 1998 and 2000 within a joint research project between Forintek Co. (now FP Innovations), Canada and NIED, Japan shaking table tests on a 2-story symmetric Light-Frame building were performed at the 1D NIED shaking table facility in Tsukuba, Japan. The building was built with Light-Frame walls made with the same 9.5 mm Canadian softwood plywood (CSP) sheathing and with 2x4 inches framing members, with a 5000 kg mass at each floor and was subjected to different scaled reproductions of the 1940 El Centro N-S ground motion until reaching the "near collapse" state (Ceccotti and Karacabeyli, 2002). In 2000 the same 1998 building was tested with the same ground motion record and with 40% nail spacing at the first level with respect to the 1998 test (6.5 cm instead of 15 cm), while in the second level the nail spacing was left unchanged (15 cm in both tests), thus leading to the attainment of "near collapse" at the second level, though the story shear was lower, of course at higher level of earthquake intensity with respect to the 1998 test (0.60 g instead of 0.45g for the 1998 test). As reported in Ceccotti and Karacabeyli, 2002, the results of these two series of shaking table tests demonstrated that (i) a uniform nail spacing in multi-storey Light-Frame buildings will first induce failure in the lowest storey with the highest storey shear, potentially leading to a "pancacking" weak-storey building collapse, (ii) an increase in the overall lateral load resistance of the building is achieved by decreasing the nail spacing in the lower storey and (iii) as the lower storey resistance is increased, failure is forced into the weaker higher storey with a higher PGA of the seismic input.

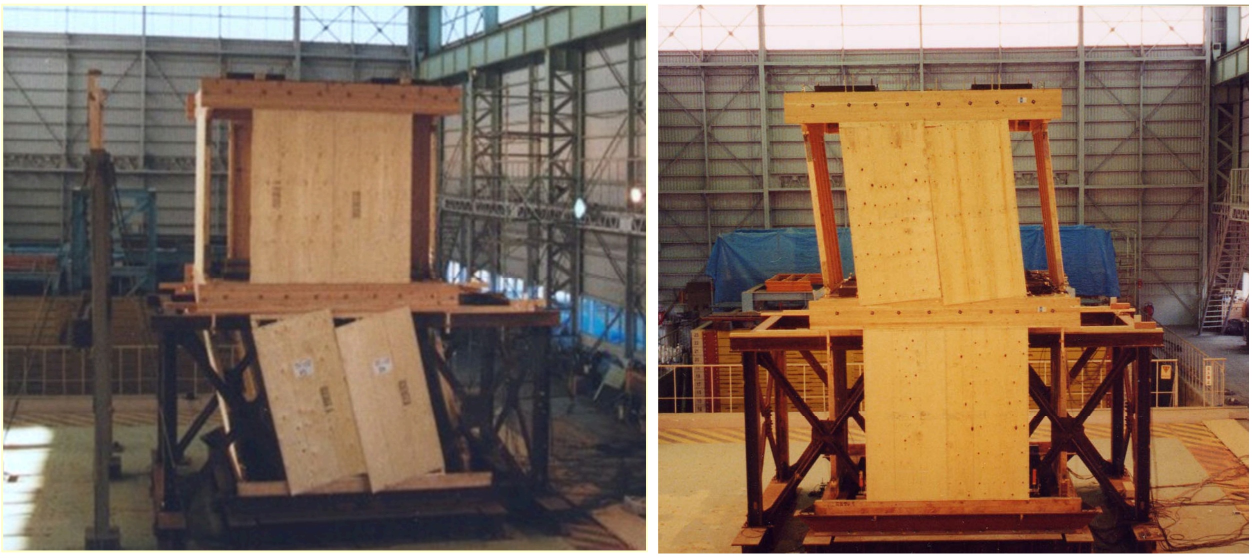


Figure 1-4: Shaking table tests of two-storey Light-Frame specimens: 1998 specimen collapse after El Centro N-S ground motion with a PGA=0.45g (left); 2000 specimen collapse after El Centro N-S ground motion with a PGA=0.60g (right).

In 2001, within the CUREE-Caltech Woodframe Project (where CUREE stands for Consortium of Universities for Earthquake Engineering and Caltech for California Institute of Technology), a joint research project aimed to improve the seismic performance of wood frame buildings which involved 14 different organizations, a 2-story single-family wood-frame house was tested at the 1D shaking table facility at the University of California, San Diego. The building had a simple rectangular plan with dimensions of approximately 5x6m with openings on all the four sides and with a large garage door opening at the first level in order to introduce a torsional eccentricity in the lateral load resisting system. The building was subjected to 5 shakes in total, 4 of which were scaled reproduction of the 1994 Northridge Earthquake recorded at Canoga Park with PGA ranging from 0.05g to 0.5g and the fifth one the unscaled reproduction of the 1994 Northridge Earthquake recorded at Rinaldi with a maximum PGA of 0.89g. The test results showed that, especially after the 0.5g test with the Canoga Park record and the 0.89g test with the Rinaldi record the building showed clearly as expected a torsional behaviour due to the asymmetry in the building layout resulting from the large garage door opening in the east wall. Also measurable in-plane deformations of the 2nd floor and roof diaphragms were observed (Fisher et al., 2001). In order to assess the state-of-the-art of 3D numerical models to predict the inelastic behaviour of Light-Frame structures at the time, the test outcomes were also used to invite the international engineering community to blind predict the dynamic characteristics and inelastic seismic response of the tested building. Thirteen international teams participated to the benchmark exercise and finally an international benchmark workshop was held in 2001 at the University of California, San Diego, to compare the results of the benchmark exercise provided by the different teams (Folz et al., 2001).

Other full-scale tests were performed in 2004 in Japan where a two-story Japanese conventional post-and-beam house was tested to investigate the collapse mechanism and predict the collapse margin for these types of buildings (Koshihara et al., 2004) and again in 2005 (Shimizu et al., 2008). However the most important full-scale tests on wooden building were conducted within some international research project detailed in the following, where buildings up to 7 stories built with different structural types were tested.

The **SOFIE Project** [2005-2008], where SOFIE stands for Sistema Costruttivo Fiemme (in English Fiemme Building System where Fiemme is a wooden region in the North East of Italy, near Trento), is probably the most comprehensive research conducted on the structural, seismic, energetic and fire performance of multi-storey timber buildings built with the CLT system. The project was entirely funded by the Province of Trento, Italy, and was led by CNR-IVALSA, Italy together with NIED, BRI and Shizuoka University. Especially the seismic structural behaviour was thoughtfully investigated through a testing program conducted in Italy and Japan which included:

- monotonic and reversed cyclic tests on wall panels with different configurations of joint, openings, panel size and rate of vertical loads applied conducted at CNR-IVALSA, Italy (Ceccotti et al., 2006);
- a pseudo-dynamic test on a one story CLT building without vertical loads and with three different opening layouts (two symmetric and the third one asymmetric) of the door openings parallel to the shaking direction conducted at the University of Trento, Italy (Lauriola and Sandhaas, 2006). The building was tested applying two earthquake records, El Centro and Kobe JMA, with two different intensities, 0.15g and 0.50g;

- a shaking table test on a 3-storey CLT building, 49 m^2 (7x7m) in plan and 10 m of total height, tested in three different layouts of the door openings in the 1^{st} floor parallel to the shaking direction (again two symmetric and the third one asymmetric as in the pseudo-dynamic test) and with 3 different ground motion records (1995 JMA Kobe, 1940 El Centro and 1997 Nocera Umbra) scaled to different values of peak ground acceleration, with a total of 26 shakings, 15 of which with a peak ground acceleration of 0.5 g or more up to 0.82g and 0.90g for Kobe earthquake and 1.20g for Nocera Umbra earthquake. Tests were conducted in 2006 at the NIED 1D shaking table facility in Tsukuba, Japan (Ceccotti and Follesa, 2006);

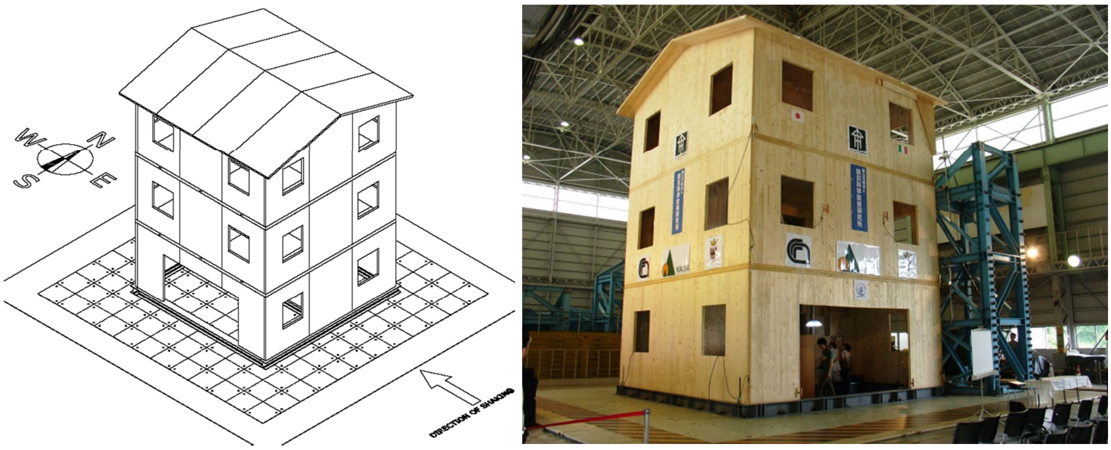


Figure 1-5: Shaking table tests of a three storey CLT building conducted in 2006 in Tsukuba, Japan within the Sofie Project. Specimen layout in Configuration C (left) and picture of the specimen during the shaking table tests (right).

- a shaking table test on a 7 story CLT building, 13,5x7,5 m in plan and 23,5 m of total height, performed in 2007 at the NIED 3D shaking table facility (E-Defence) in Miki, Hyogo (near Kobe), Japan (Ceccotti et al. 2013). The building was tested with 3 different ground motion records (1997 Nocera Umbra, 1995 JMA Kobe, 2007 Kashiwazaki R1) scaled to different values of peak ground accelerations in the 3 directions and subjected to 10 shakings in total, 6 of which 1D and the remaining 4 3D. The last two tests were performed with 100% intensity (0.60g in X, 0.82g in Y and 0.34g in Z direction) of JMA Kobe and 100% (0.31g in X, 0.68g in Y and 0.41g in Z direction) of Kasiwazaki R1 earthquake.

Figure 1-6: Shaking table tests of a seven storey CLT building conducted in 2007 in Miki (Kobe), Japan within the Sofie Project. Specimen layout with wall thickness at each storey (floors were all made with 142 mm CLT panels) and picture of the specimen over the shaking table (right).

Moreover preliminary numerical investigations have been conducted with the aim of evaluating the value of the behaviour factor q to be used in the seismic design of multi-storey CLT buildings (Ceccotti et al., 2007) and in 2007 a fire test on a full-scale 3-story CLT building was conducted at the BRI fire testing facility in Tsukuba Japan (Bochicchio et al, 2008).

Tests performed on wall panels (Ceccotti et al., 2006) showed that the overall behaviour is strongly dependent on the design of joints and wood panels behave almost completely rigid. The overall dissipation of energy is therefore provided only by connections with an average equivalent viscous damping of 14%, even greater than the value observed on Light-Frame walls, proving the suitability of the system for seismic purposes.

Pseudo-dynamic tests (Lauriola and Sandhaas, 2006) showed that the lateral stiffness in the asymmetric configuration with large openings on one side was very similar to the one measured for symmetric configuration. Moreover, like in the results of the wall test, proved to be very stiff if compared to other structural timber systems, but nevertheless still ductile, with all the energy dissipation due to mechanical joints.

Shaking table tests performed on the 3-storey building (Ceccotti and Follesa, 2006) confirmed the overall good ductile behaviour and seismic performance observed in the preceding tests. The building was designed according to the equivalent lateral force method in Eurocode 8 for a type B soil and applying a peak ground acceleration PGA=0.35 g, corresponding to the most hazardous seismic region in Italy and considering a value of the behaviour factor q=1. The total seismic weight was 465 kN to account for self-weight and additional dead and live loads and the calculated base shear was 509 kN. The test results showed again, like for the pseudo-dynamic tests, that the asymmetric configuration of the building in the third layout with a large opening on one side didn't lead to almost any pronounced torsional behaviour. According to the Author's opinion this may be explained by the fact that as the floors were rigid enough, the torsion of the building was counteracted by the contribution of the two perpendicular walls. The building did not suffer great damage for the 0.5g series of test, whereas a larger damage was observed for the most severe levels of earthquake intensity. However the observed damage was anyway concentrated only in some connections while the wooden panels remained undamaged, and the building could be easily repaired and brought almost again to the undamaged condition, as was demonstrated during the tests by measuring natural frequency before and after the repairs. By defining a near-collapse condition and by dividing the PGA values recorded in the seismic tests by the design value, a preliminary evaluation of the behaviour factor q was made (Ceccotti et al., 2006), finding that a value of q=3 could be considered as a good indicator for the seismic design of multi-storey CLT buildings. These results were confirmed by numerical simulations performed with non-linear dynamic analysis made with 8 different historical earthquakes (Ceccotti et al., 2007).

The results obtained from the 3-story building shaking table tests were used in the design of the 7-storey tests. The building was designed again with the equivalent lateral force procedure of Eurocode 8, but using two different design PGA in the two principal horizontal directions (0.82g and 0.60g in the long and short direction, respectively, according to the 100% intensities of JMA Kobe earthquake), but using a value of the behavior factor q=3 according to the preceding results and an importance factor γ_I=1.5 for 'buildings whose integrity is of vital importance for civil protection' according to Eurocode 8 (CEN, 2004). The results of the 7 storey shaking table test (Ceccotti et al., 2013) showed that after a series of 10 major earthquakes the building didn't show any residual damage, with maximum values of inter-storey drift, uplift and slip of joints which were not critical. Observed failures on joints were ductile and again easily repairable. High accelerations at the upper storeys, up to a maximum of 3.8g, were registered during the 3D JMA Kobe 100% test. Although these accelerations are still acceptable for human health, they are nevertheless uncomfortable, suggesting that maybe further investigation on the mitigation of such accelerations e.g. with the use of tuned mass dampers at the upper stories would be appropriate. Furthermore the Authors conclude by confirming the validity of the Eurocode 8 force-based approach for the seismic design of CLT buildings.

The **NEESWood Project** [2005-2009], where NEESWood stands for Network for Earthquake Engineering Simulation Wood, is a five university project (Colorado State University, State University of New York at Buffalo, Rensselaer Polytechnic Institute, University of Delaware and Texas A&M University) led by Colorado

State University whose aim is to investigate the possibility to increase the height of Light-Frame wood buildings to six stories in regions of moderate to high seismicity by means of full-scale testing and numerical analysis (van de Lindt et al. 2010). The project was funded by the U.S. National Science Foundation, with additional funding provided by industry and government in the U.S., Canada and Japan with the aim of developing a new Performance Based Seismic Design philosophy that could provide the necessary tools to designers in order to safely increase the height of woodframe structures in active seismic zones of the U.S. as well as mitigating damage to low-rise woodframe structures. Within this project the following activities have been conducted:

- shaking table tests on a 2-story Light-Frame building with an integrated two-car garage, with a total of 160 m^2 of living space, conducted at the State University of New York at 3D Buffalo's SESL NEES twin shaking table facility in 2006 (Filiatrault et al., 2010);

- shaking table tests on a 7-story building composed a first-story steel special moment frame (SMF) and six stories of light-frame wood for stories two through seven tested at the NIED 3D shaking table facility (E-Defence) in Miki, near Kobe, Japan in 2009. The building has a plan dimension of 220 square meters (approximately 18x12m) and a total height of 20m and was designed to host 23 living units (van de Lindt et al., 2011).

The 2-story Light-Frame building tested at Buffalo was designed according to the 80's design procedure and construction practice in California. Tests performed showed that the building performed relatively well surviving two destructive earthquakes but showing nevertheless significant and costly damage (Filiatrault et al., 2010). Moreover the test results confirmed previous outcomes of tests performed during the CUREE-Caltech Woodframe project (Filiatrault et al., 2002), i.e. that non-structural elements such as gypsum wall board (GWB) and exterior plaster significantly increase the overall strength and stiffness of wood frame buildings, therefore contributing to improve the seismic performance. Furthermore it was confirmed that such structures are prone to torsional behaviour and soft story mechanisms. This benchmark test served in order to obtain data for the calibration and verification of non-linear dynamic models (sapwood software package, Pei and van de Lindt, 2007) and to provide information which was used to develop the Direct Displacement Design procedure used in the design of the test on the 7-story building.

The shaking table tests performed on the 7-story building (called the Capstone Building) were divided into two phases (van de Lindt et al., 2011). In the first phase the 7 storey building composed of a first storey made with a steel special moment frame (SMF), to account for large open commercial spaces, and stories 2 to 7 with a Light-Frame construction was tested to evaluate the seismic behaviour of the mixed steel-frame and Light-Frame wood construction. In the second phase the steel moment frame was locked down by means of heavy bracings, thus becoming an effective extension of the shaking table, so that the six story woodframe building could be tested. The design of the 6-story woodframe building was not made in accordance with any existing building code, but was performed using the Direct Displacement Design (DDD) procedure (Pang et al., 2010) developed based on the results of the shaking table tests performed on the 2-story building previously referenced, with a specific performance target: the peak inter-story drift in the building should not exceed 4% when subjected to a maximum credible earthquake (MCE) 80% of the time, i.e. a 20% probability of exceeding 4% inter-story drift. The testing schedule program for the mixed-use building consisted of two ground motion records, both tri-axial scaling of the 1994 Northridge Earthquake recorded at Canoga Park, for two different levels of intensity, respectively 0.19g in X, 0.22g in Y, 0.26 in Z direction and 0.50g in X, 0.58g in Y and 0.69g in Z direction, corresponding to a probability of exceedance of 50% in 50 years and 7% in 50 years. The intensities were estimated to be the highest possible without causing a significant damage to the above wood portion which should be tested later. For the second phase the six storey woodframe building was subjected to three shakes of the same ground motion record used in phase 1, corresponding respectively to a probability of exceedance of 50%, 10% and 2% in 50 years. The lowest probability of exceedence (Maximum Credible Earthquake) corresponded to input accelerations of 0.64g in X, 0.76g in Y, 0.88 in Z direction respectively.

The steel frame was composed of a new type of partial strength moment-resisting beam-to-column connection properly designed in order to provide high rotational ductility in a manner that creates a plastic hinge outside of the beam itself, and played also an important role for moving and lifting the building over the shaking table, and, after the building was lifted and installed, for an easier connection to the shaking table. As for the wooden structure, shear walls were distributed evenly along the two main directions of the building, and were composed of 38 mm × 140 mm studs spaced at 406 mm on centre, sheathed with OSB on one side and GWB on the other side. The internal walls were made with the same shear walls, except for two high-capacity midply walls (Ni et al., 2008), i.e. Light-Frame shear wall assemblies with double studs (instead of single studs of regular shear walls) and three layers of wood-based sheathing (instead of two layers of sheathing as in regular shear walls) with nails working in double shear (instead of single shear as in regular shear walls) which provide higher strength and stiffness performance, located in the long direction of the building next to the internal staircases. Continuous steel rods with mechanical shrinkage compensating devices were used at each end of shear wall to prevent overturning (van de Lindt et al., 2009).

Figure 1-7: Shaking table tests of a six storey Light-Frame building conducted in 2009 in Miki (Kobe), Japan within the NeesWOOD Project.

The results of Phase 1 showed a limited non-structural damage to the mixed building, mostly in the corners near openings for doors and windows with small cracks in the GWB panels, with only few cases of slightly larger cracks where larger drift were experienced. The peak inter-storey drift of 1.31% was measured for the second shake in the longest direction of the building at the 5th storey and the peak roof displacement was 1% of the building height in the same direction. The maximum uplift force was 542 kN recorded in one end of one of the two internal midply walls.

During phase 2 the building performed extremely well and survived the strongest quake without any significant structural damage. The maximum average recorded displacement obtained at the roof level relative to the shaking table in the long direction was 60, 140 and 211 mm for the three earthquake intensities, respectively (van de Lindt et al., 2009) . As reported, during the MCE test (highest level of earthquake intensity) the building clearly showed a torsional response. Due to the presence of torsion the maximum inter-storey drift, 1.88%, was recorded again, for the MCE test, at the 5th storey in the longest direction, while the maximum recorded uplift was 768 kN, again in the same position as in phase 1.

The **SERIES Project** [2010-2013], where SERIES stands for Seismic Engineering Research Infrastructures for European Synergies, is a three university project (University of Trento, Italy, University of Minho, Portugal and Technical University of Graz, Austria) led by the University of Trento and funded within the European Union Seventh Framework Programme [FP7/2007-2013] with the goal of investigating the seismic performance of different type of structural systems, i.e. the Log House system, the Light-Frame system with two different types of sheathing and sheathing-to-framing connections and the CLT system through full-scale shaking table tests performed on the same building, a 3-storey dwelling house (even if only for the case of Log House system, it was reduced to 2 storeys) subjected to the same ground motion records (Tomasi and Piazza, 2013). It was the first time ever that different structural systems were compared with regard to their seismic behaviour by means of full-scale tests. All the tests were performed at the LNEC 3D shaking table facility (where LNEC stands for Laboratório Nacional de Engenharia Civil) located in Lisbon, Portugal. More in detail the research activities conducted within the project were:

- preliminary monotonic and reversed cyclic tests on different type of connections and on full scale walls for the different structural systems performed at the University of Trento for the Light-Frame system tests, at the University of Minho for the Log House test and at the Technical University of Graz for the CLT test.

- shaking table tests performed on a 2-storey Log House building, with a plan dimension of 5x7 m and 5.28 m of total height. Walls were composed by 16cm (for the external ones) and 8cm (for the internal ones) double or triple glue-laminated logs. Floor and roof construction was composed by solid wood timber beams and rafters sheathed with 15 mm OSB panels connected to the framing with 2.8x60 annular ringed nails. The base log was connected to the steel frame by means of M16 bolts. The building was designed according to Eurocode 5 (CEN, 2009) and Eurocode 8 (CEN, 2004) with a linear dynamic analysis performed with SAP2000 (CSI, 2000) and using a value of the behaviour factor q=2 to account for the dissipative contribution of friction (Campos Costa et al., 2013a). No reference was found about the seismic action considered in the design. The testing program included three different values of peak ground accelerations (respectively 0.07g, 0.28g and 0.50g) representative of three different levels of intensity (respectively low, moderate and high). Before and after each test a dynamic characterization in order to measure the natural frequency of the building. The ground motion record used was a 2D reproduction of one historical ground motion record (the 1979 Ulcinj - Hotel Albatros record of the Montenegro earthquake) scaled at different values of the peak ground acceleration as explained above. The building was subjected to a total of 12 shakings.

Figure 1-8: Shaking table tests of a two storey Log House building conducted in 2011 in Lisbon, Portugal within the SERIES Project. 3D model of the specimen and picture of the specimen over the shaking table (after Campos Costa et al., 2013a).

- shaking table tests on a 3 storey Light-Frame building, with the same plan dimension of the Log House building and 7.66 m of total height. Walls were made of 60x100/160 mm studs spaced at 600 mm average, sheathed with 15 mm OSB on one side and GWB on the other side. Floors were made of prefabricated box elements made with 31x78 mm timber joists and upper and lower 31 mm timber boards on top of which 15 mm OSB panels are nailed and roof with timber rafters, 20mm timber boards and reinforcing steel metal strips nailed to the timber boarding. The building was designed according to Eurocode 5 (CEN, 2009) and Eurocode 8 (CEN, 2004) with a linear static analysis considering the Italian area with the highest seismic hazard (Ferla (SR), with a PGA of 0.277g for a 10% probability of exceedance in 50 years and 0.402g for a 5% probability of exceedance in 50 years), for a type A soil and using a value of the behaviour factor q=5 (Campos Costa et al., 2013b). The testing program included three different values of peak ground accelerations (respectively 0.07g, 0.28g and 0.50g) representative of three different levels of intensity (respectively low, moderate and high). Before and after each test a dynamic characterization was carried out in order to measure the natural frequency of the building. The chosen ground motion records were a 2D reproduction of two different historical ground motion records, the above mentioned record of Montenegro earthquake and the 2011 Tohoku earthquake scaled at different values of the peak ground accelerations ranging from 0.07g to 0.50g for the Montenegro earthquake according to the above referenced procedure and of 0.27g and 0.54g for the Tohoku earthquake respectively. The building was subjected to a total of 22 shakings.

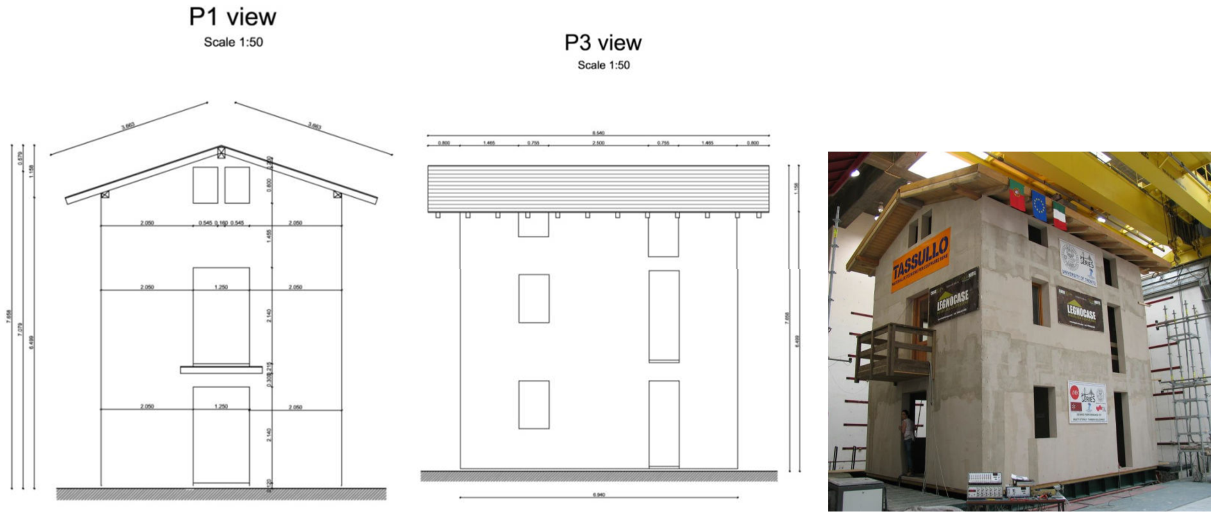


Figure 1-9: Shaking table tests of a three storey Light-Frame building conducted in 2011 in Lisbon, Portugal within the SERIES Project. Elevations of the specimen and picture of the specimen over the shaking table (after Campos Costa et al., 2013b).

- shaking table tests on a 3 storey Light-Frame building, with the same plan dimension and total height of the previous one. Walls were made of 60x100/160 mm studs spaced at 600 mm average, sheathed with 15 mm on and 12.5 mm gypsum fibre panels on both sides connected with metal staples with a 1.59x1.35 mm double round section. Floors were made of glulam timber beams sheathed with sheathed with 22 mm OSB upper panels and 12 mm OSB lower panels and roof with timber rafters, 20mm timber boards and reinforcing steel metal strips nailed to the timber boarding. The building was designed according to Eurocode 5 (CEN, 2009) and Eurocode 8 (CEN, 2004) with a linear static analysis considering the Italian area with the highest seismic hazard (see above for more details), for a type A soil and using a value of the behaviour factor q=2.5 (Campos Costa et al., 2013c). The testing program included four different values of peak ground accelerations, respectively 0.07g, 0.15g, 0.28g and 0.50g. Before and after each test a dynamic characterization was carried out in order to measure the natural frequency of the building. The chosen ground motion record was a 2D reproduction of the above mentioned record of Montenegro earthquake scaled at different values of the peak ground accelerations ranging from 0.07g to 0.50g. The building was subjected to a total of 32 shakings.

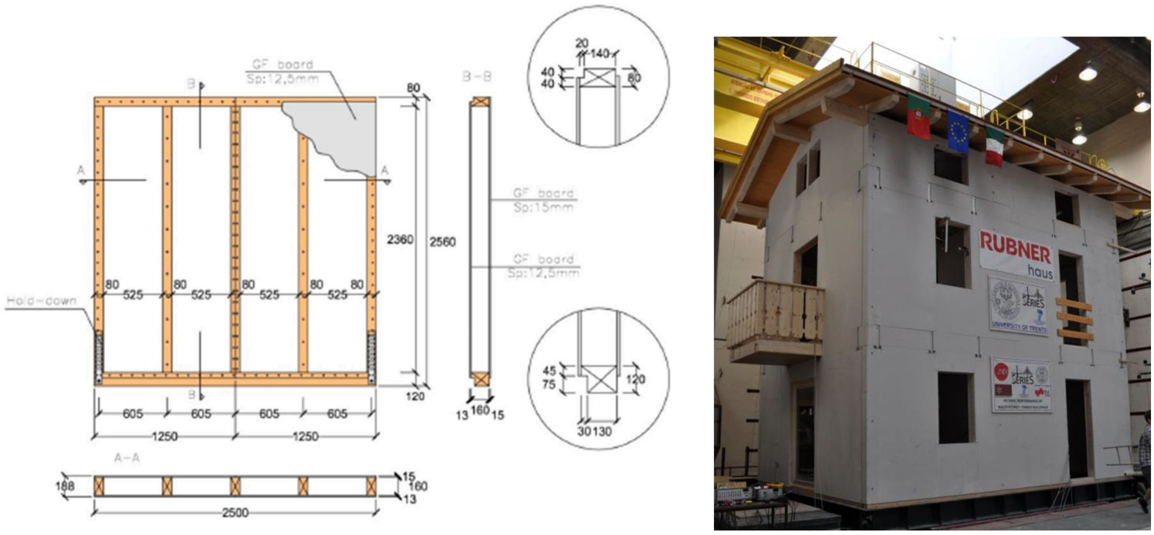


Figure 1-10: Shaking table tests of a three storey Light-Frame building sheathed with gypsum fibre panels conducted in 2011 in Lisbon, Portugal within the SERIES Project. Geometry of a standard shearwall and picture of the specimen over the shaking table (after Campos Costa et al., 2013c).

- shaking table tests on a 3 storey CLT building, with the same plan dimension and total height of the previous ones. Walls were made of 100 mm, 3 layers CLT panes. Floors were made of 150 mm, 5 layers CLT panels and roof with 99 mm, 3 layers CLT panels (Campos Costa et al., 2013d). No information about the design could be found. Before and after each test a dynamic characterization was carried out in order to measure the natural frequency of the building. The chosen ground motion records were a 2D reproduction of the above mentioned record of Montenegro and Tohoku earthquake scaled at different values of the peak ground accelerations, up to a maximum value of 0.50g. The building was subjected to a total of 32 shakings.

Figure 1-11: Shaking table tests of a three storey CLT conducted in 2011 in Lisbon, Portugal within the SERIES Project. Accelerometers placed inside a monitored room and picture of the specimen over the shaking table (after Campos Costa et al., 2013d).

In the Log House building test the maximum values of displacements were recorded obviously for the highest intensity quake with a measured maximum uplift of around 9 mm, maximum wall slip of 6.5 mm, maximum wall shear deformation of around 143 mm, maximum inter-storey drift of around 1%. The measured fundamental frequency of the buildings was 5.39 Hz, reduced to 5.11 Hz after all tests. The observed damage was limited to some cracks observed in the logs due to out of plane rotation of the walls, at corner joints due to shear failures in end sections of the logs, some sliding deformation between logs and some shear cracks of the top and bottom profiles of the log cross-section, probably due again to out of plane wall deformations. Anyway the overall damage was very limited, suggesting that this building system proved to have good seismic performance even in high seismicity zones.

Also the Light-Frame OSB sheathed test showed a low level of damage with the maximum values of displacements recorded obviously for the highest intensity quake with a measured maximum uplift of around 3 mm, maximum wall slip of 36 mm, maximum inter-storey drift of around 2% and maximum uplift forces of around 21 kN. The measured fundamental frequency of the buildings was 3.62 Hz, reduced to 3.35 Hz after all tests, confirming that the structural damage was very limited. The observed damage was very limited, and visible only in some plaster crack in the outside corners of the building.

Also the Light-Frame gypsum fibre board sheathed test showed a low level of damage with the maximum values of displacements recorded obviously for the highest intensity quake with a measured maximum uplift of around 10 mm, maximum wall slip of 5 mm, maximum inter-storey drift of around 0.8% and maximum uplift forces of around 17 kN. The measured fundamental frequency of the buildings was 3.28 Hz, reduced to 3.06 Hz after all tests, confirming that the structural damage was very limited. The observed damage was very limited, and visible only in some minor cracks in the external cladding in correspondence of the building corners and of the conjunction between floors and walls.

Finally the CLT building test showed a low level of damage with the maximum values of displacements, which were nevertheless very low, recorded obviously for the highest intensity quake. The measured fundamental frequency of the buildings was 3.98 Hz, reduced to 3.75 Hz after all tests, confirming that the structural damage was almost negligible. The observed damage was even more limited than for other structural types, showing that this structural type show a very limited damage for moderate to high seismic forces.

The **NEES-Soft Project** [2010-2013], where NEES-Soft stands for Seismic Risk Reduction for Soft-Story Wood-frame Buildings, is a five university project (Colorado State University, State University of New York at Buffalo, University of Alabama, California State Polytechnic University and University of California) led by Colorado State University on the seismic retrofit of existing woodframe buildings (van de Lindt et al. 2014). The project was funded by the U.S. National Science Foundation, with additional funding provided by industry contribution with the aim of developing a new Performance Based Seismic Design methodology for the seismic retrofit of existing commercial and multi-storey residential woodframe buildings, which, due to the presence of large openings at the ground level are potentially prone to soft and weak-story collapse due to earthquake actions. The objectives of the project, presented in van de Lindt et al. 2014, were (i) to enable performance-based seismic retrofit (PBSR) for soft-story wood-frame buildings prone to earthquake failure, (ii) validate the U.S. Federal Emergency Management Agency (FEMA) P-807 retrofit guidelines by means of experimental testing and (iii) validate an analytical/numerical collapse model via full-scale building collapse testing. Within this project the following activities have been conducted:

- Tests on a 6.1 m long hybrid damper Light-Frame wall with and without toggle-braced dampers performed at the University of Alabama Structural Engineering Laboratory;

- Reversed cycling testing on a Light-Frame assembly seismic retrofitted with distributed knee-bracings (DKB) performed at the California State Polytechnic University San Luis Obispo Structures Laboratory;

- Shaking table tests on a Light-Frame assembly seismic retrofitted with distributed knee-bracings (DKB) to collapse, identical to the one tested with cycling testing, performed at the Colorado State University Structural Engineering Laboratory;

- Slow hybrid testing on a three-storey Light-Frame building (one-story numerical substructure and a two-story physical substructure) in order to test the contribution of different type of seismic retrofit (cross laminated timber rocking walls, steel cantilevered columns, fluid viscous dampers (FVDs) and a distributed knee-brace (DKB) system) numerically computed in the first soft storey, performed at the NEES laboratory at the University of Buffalo;

- Shaking table tests performed on a full-scale four-story Light-Frame building without and with different type of seismic retrofit (cross laminated timber rocking walls, steel special moment frame and combination of steel special moment frames and supplemental damper assemblies together with wood shear walls retrofits for the upper stories) performed at the NEES laboratory at University of California, San Diego. The building was tested with scaled values of the ground motions records of Loma Prieta, Capo Mendocino earthquakes up to a maximum peak ground acceleration of 0.9g.

The hybrid testing performed at the NEES laboratory of Buffalo, showed that the overall seismic performance of the tested building with all the seismic retrofitting techniques was satisfactory, meeting the 4% drift limit at an even higher intensity than designed. The measured inter-story drift displacements demonstrated the effectiveness of the seismic retrofit of the only first story in strengthening and damping the seismic response of the whole building and achieved the intended objective of limiting the force transferring to the upper stories and eliminating any torsional behaviour.

The tests performed at the NEES shaking table facility of San Diego confirmed the good results of the hybrid testing performed in Buffalo for all the retrofitting techniques. In test phases 1 and 2 where CLT rocking walls and steel special moment frames were used as retrofit system and were designed according to the FEMA P807 guidelines, higher inter-story drift limits were recorded at the first story with minimal demand transferred to the upper three stories. In phase 3 and 4 the building was retrofitted like for the test performed in Buffalo with a combination of steel special moment frames and supplemental damper assemblies together with wood shear walls retrofits for the upper stories according to the Performance Based Seismic Retrofit methodology developed within the NEEs-Soft project. The seismic response in these two latter phases engaged all the four stories, with inter-story drifts more equally distributed along the height of the building, allowing the building to perform well with more intense earthquakes than in a single story retrofit design. The peak inter-story drift was recorded as 4.3% during phase 3 for the strongest earthquake.

Figure 1-12: Specimen retrofitted with CLT rocking walls according to FEMA P807 guidelines before the shaking table tests.

Finally full-scale collapse tests were conducted on the unreinforced building in order to meet three major objectives, as reported in van de Lindt et al., 2014: (i) a better understanding of the behaviour of light wood-frame buildings near and at collapse; (ii) the quantification of the collapse displacement; and (iii) the investigation of the collapse mechanism of soft-story buildings. The building was subjected to 8 subsequent earthquakes, which were scaled inputs of Loma Prieta, Capo Mendocino and Superstition Hills earthquakes. The collapse occurred with the Superstition Hills earthquake which had larger spectra displacements but lower spectral accelerations of the Loma Prieta and Capo Mendocino: this confirms previous results of tests collapse tests conducted in Japan, i.e. that while ground acceleration induces large inertial forces in wood-frame buildings, it is generally large reversing ground deformations that initiate collapse. The outcomes of these tests were also useful in order to develop a 3D modelling package in order to evaluate the collapse risk of soft-story wood-frame buildings.

Other experimental investigations were conducted in Slovenia in 2006 where a two-story CLT building was tested at the shaking table facility of Institute of Earthquake Engineering and Engineering Seismology in Skopje, Macedonia (Dujic et al. 2006) and in 2014 in Canada, where a two-story CLT building was tested under quasi-static monotonic and cyclic loading in two directions, one direction at a time (Popovski et al., 2014).

Of course, besides the experimental research via full-scale testing of timber buildings, a vast literature about tests performed on single connections or structural components (e.g. Light-Frame or CLT shear walls) both regarding the monotonic and cyclic non-linear behaviour could be found, which of course cannot be all referenced within the limited length of this brief dissertation. However two of them will be referenced as they had been used for the numerical non-linear analysis performed in Chapters 3 and 4 for the calibration of the numerical models.

In 2000 an extensive testing program has been performed at Forintek Canada Co., Canada (now FP Innovations) for the evaluation of the non-linear properties of full-size Light-Frame shear wall specimens under monotonic and cyclic displacement schedules with different type of sheathing (plywood, OSB and gypsum wall-boards), different sizes of specimens (2.44x.2.44m and 4.88x2.44m) and with different amount of vertical load applied (Ceccotti and Karacabeyli, 2000). The testing program was used for the evaluation of the Canadian seismic force modification factor (R) and the European behaviour factor (q) for lateral load resisting systems comprised of plywood nailed walls through two-dimensional non-linear dynamic analysis performed on a 4-storey case-study building with 28 twenty-eight earthquake accelerograms specially selected to be compatible with the seismic characteristics of the Vancouver area with an hysteretic model (Ceccotti and Vignoli, 1989) calibrated to the shear wall cyclic test data.

In 2011 and 2012, following the experimental research performed within the SOFIE project mentioned above, an extensive experimental programme was conducted at CNR-IVALSA, Italy, for the evaluation of the non-linear properties of all types of connections used in CLT construction (Gavric et al., 2011, Gavric et al., 2012, Gavric 2013). Tests included quasi-static monotonic and cyclic tests on hold-down and angle bracket connections, screwed step joints for the connection of CLT panels in shear walls and in CLT floors, screwed joints for the wall-to-wall and floor-to-wall connections, tested in both principal directions (horizontal and vertical), and CLT shear walls in different configurations of connections and amount of vertical load applied.

1.3.2. Numerical investigations

Besides the aim of understanding better the seismic performance of full-scale buildings built with different structural systems under real earthquake excitations in order to obtain information which could be used in the revision of existing building codes and in the seismic design practice for structural engineers, the main objective of all the experimental researches referenced in §1.3.1 is the development of hysteresis models capable to predict, with the highest possible reliability, the actual non-linear behaviour of timber buildings in different configurations of size, geometry and connection layout, under different ground motion inputs. As one can easily imagine, full-scale seismic tests on timber building are very expensive as they imply (i) the design of the test, with the definition of the objectives and the tasks to be investigated, (ii) an extensive experimental background research about the non-linear static and dynamic behaviour of connections and structural components typical of the reference structural type, (iii) a detailed design of the specimen to be tested by means of preliminary numerical analysis aimed also to predict the building performance at the foreseen different levels of seismic input (which is very important in order to avoid unexpected circumstances which may undermine the test objective or even produce damage to the seismic testing facility or to the instrumentation), (iv) the delivery of the construction materials (timber elements and connections) to the earthquake simulation facility, (v) the construction of the specimen on site, (vi) the instrumentation of the specimen to be tested, (vii) the execution of the testing programme, (viii) the dismounting of instrumentation and of the specimen and (ix) the analysis of the test results and the production of the test reports. However all this great effort may seem useless as the information which the test may provide, even if useful in order to understand the real seismic behaviour of timber buildings typical of the reference structural type, are referred only to that type of building with the relevant geometry, connection layout and number of stories, type and level of seismic input but they cannot be transferred automatically to other buildings of the same structural type with different combination of the above mentioned factors. On the other hand the information provided by full-scale tests are of the outmost importance for the calibration procedure and proof of reliability of existing numerical models, by means of which it will be possible to perform a great number of analysis with different combinations of the above mentioned factors, in an undoubtedly more economical way.

For these reasons several numerical models have been developed in the last 40 years using both commercial or developed finite element software for the evaluation of the non-linear behaviour of timber structures under static and dynamic loading, based on available results of experimental researches on (i) single fasteners (nails, screws, bolts or dowels mainly), (ii) mechanical connections (different types of angle brackets, hold-downs and metal plate connections fastened with different type of fasteners), (iii) timber sub-assemblies (moment resisting-frames, shear walls in Light-Frame and CLT structures) and (iv) full-scale buildings. The computing performance and reliability of these models have been greatly enhanced in recent years also due to the development of powerful software packages for structural analysis, which significantly reduced computation times and improved the possibility of developing increasingly complex models.

Numerical investigations on the dynamic performance for the evaluation of the behaviour factor of moment-resisting glulam frames with dowelled joints tested at the University of Florence were performed by Ceccotti and Vignoli (1987) using a simple bi-linear law integrated within the non-linear dynamic analysis software DRAIN-2D. The same model was then implemented in a tri-linear pinching hysteresis piece-wise relationship in Ceccotti and Vignoli (1989) and subsequently implemented in DRAIN-3DX in order to model 3D symmetric timber structures composed of macro-elements which incorporate the hysteresis model through rotational or translational springs simulating the actual behaviour of shear walls and diaphragms in Light-Frame structures based on available experimental cyclic results (Ceccotti et al. 2000).

Foschi (2000) proposed a non-linear model for the evaluation of the seismic performance of connections and sub-assemblies like Light-Frame shear walls, based on the modelling of each single fastener in three dimensions. This model was then implemented by He et al. (2001) in a computer program capable of modeling 3D light-frame wood structures (LightFrame 3D) at the nail level and performing load-controlled or displacement-controlled monotonic and cyclic analysis. However, even if this model is in principle very powerful, the cyclic characterization of the model is calculated at each step of the dynamic analysis for each single fastener, and this procedure require large elaboration times and therefore is computationally inefficient.

Other numerical models, were developed in most cases for the evaluations of the non-linear performance of Light-Frame shear walls and entire Light Frame buildings with different models implemented in purposely developed 2D or 3D Finite Element Software or available commercial programs. A comprehensive chronological and detailed review on the evolution of different numerical models can be found in van de Lindt (2004) and Christovasilis (2010).

There are much less examples of non-linear hysteretic models for the evaluation of the seismic performance of CLT structures. Ceccotti et al. (2006) proposed a 3D numerical model with DRAIN-3DX with CLT panels simulated as rigid frames connected by symmetric and asymmetric non-linear translational springs simulating angle brackets and hold-downs incorporating the hysteresis model reference above. Although the model could not fit precisely the hysteretic behaviour of each spring, the predicted global structural response was accurate enough. However, the strength and stiffness degradation which are typical features of CLT shear walls and connections subjected to cyclic loading could not be considered.

Pozza et al. (2012) modelled mechanical connections in CLT buildings with macro-springs made with a group of springs and dampers implemented in Straus 7 software package. Also this model however cannot take into account the strength and stiffness degradation.

The most powerful and reliable non-linear model for the seismic analysis of CLT structures have been developed by Rinaldin et al (2013). The model schematizes CLT connections as non-linear hysteretic multi-spring elements with different hysteretic behaviour for each degree of freedom, implemented as external subroutines in Abaqus software package. The hysteretic model consider all the features of the non-linear behaviour of CLT connections such as (i) strength and stiffness degradation, (ii) pinching behaviour, (iii) post-peak behaviour, (iv) interaction between axial and shear resistance of connectors, and (v) influence of friction. CLT panels are schematized as elastic shell elements. The model has the great advantage given by the possibility to predict the cyclic behaviour of entire panels, subassemblies and buildings once each spring simulating each single CLT connection has been calibrated. The reliability of this approach, even if computationally costing, has been prooved by comparing the numerical simulations with the results of the 3-story and 7-story seismic tests performed within the SOFIE Project and referenced above (Rinaldin et al., 2014).

1.4. Objectives and methods

The aim and motivation of this PhD Thesis is to propose a new Background Document for the next revision of the timber Chapter of Eurocode 8, which includes a comprehensive review of the current version and the implementation of some new parts, like some specification about the analysis methods and the design of mixed CLT/Light Frame buildings not actually covered by the current standard.

The objectives will be pursued through:

- a comprehensive and detailed discussion of the current version of the Chapter for timber buildings actually included in Eurocode 8 analysing each part or sentence which could be improved, changed or reviewed.

- the proposal of possible methods for the linear and non-linear analysis of timber structures made with a widely used software package as Sap 2000 which could be used by consulting engineers in the seismic analysis of CLT and Light Frame multi-storey buildings

- non-linear dynamic analysis performed with a numerical model for the analysis of the non-linear behaviour of timber structures implemented within DRAIN-3DX and improved with new features such the strength and stiffness degradation during cyclic loading of mechanical joints, not included in the previous version of the numerical model, incorporated in order to obtain a more detailed prediction of the actual non-linear behaviour and finally,

- a proposal of a new Background Document for the release of a new chapter for the seismic design of timber building which will include all the aspects analysed and discussed within the research work.

1.5. Thesis structure

The thesis is organized into 6 chapters, each one including a final Reference section. In each Chapter a first introductive section explains the topics which will be treated and the research background previously made. More specifically the organization of the different Chapters is the following:

- Chapter 1 introduces the research motivation and the research background on the seismic behaviour of timber structures. After a brief dicussion about the seismic design of structures according to EC8, and a detailed overview of the most important experimental researches conducted and numerical models developed, this Chapter explains the research objectives and methods that will be followed.

- Chapter 2 retraces the historical evolution of the current version of the timber Chapter of EC8 since its first release in 1988 and analyses in detail its content underlining all the critical points which could be improved or changed.

- Chapter 3 addresses the different analysis methods for the seismic design of the two most common structural types used for the construction of multi-storey timber buildings, i.e. CLT and Light Frame system, through linear and non-linear analysis compared and calibrated with experimental results, made with the most common structural analysis program used by consulting engineers, i.e. SAP 2000.

- Chapter 4 is related to a code proposal for the seismic design of mixed CLT/Light Frame structures. The seismic behaviour of mixed buildings is investigated by means of non-linear dynamic analysis, performed with a numerical model for the analysis of the non-linear behaviour of timber structures developed within DRAIN-3DX software package and purposely implemented with new features for a better reproduction of the actual seismic behaviour of these structures.

- Chapter 5 contains the Background Document for a new version of the timber chapter of Eurocode 8, with proposal of changes of some sentences currently included and with new parts which should be added related especially to the Capacity Based Design provisions for the different structural types and over-strength factors to be adopted in the seismic design.

- Chapter 6 finally addresses the possible future evolution of the chapter for the seismic design of timber structures in Eurocode 8 in a third generation of Eurocodes and possible development in the research of timber structures which could be followed in the future.

1.6. References – Chapter 1

Bochicchio G., Ceccotti A., Frangi A., Lauriola M.P. [2008] “Natural full-scale fire test on a 3 storey XLam timber building”, Proceedings of 10th World Conference on Timber Engineering (WCTE), Miyazaki, Japan, 2008.

Campos Costa A., Candeias P. X., Lourenco P. B., Branco J. M., Aranha C. [2013a] “Seismic performance of multi-storey timber buildings - Rusticasa building -- Final Report” – SERIES. Work Package [WP9-TA5 LNEC].

Campos Costa A., Candeias P. X., Bartolucci C., Piazza M., Tomasi R., Grossi P. [2013b] “Seismic performance of multi-storey timber buildings - Legnocase building -- Final Report” – SERIES. Work Package [WP9-TA5 LNEC].

Campos Costa A., Candeias P. X., Piazza M., Tomasi R., Grossi P. [2013c] “Seismic performance of multi-storey timber buildings - RubnerHaus building -- Final Report” – SERIES. Work Package [WP9-TA5 LNEC].

Campos Costa A., Candeias P. X., Schikhofer G., Flatscher G. [2013d] “Seismic performance of multi-storey timber buildings – TU Graz building -- Final Report” – SERIES. Work Package [WP9-TA5 LNEC].

Ceccotti, A., Follesa, M., Karacabeyli E. [2000] “3D seismic analysis of multi-storey wood frame construction” Proceedings of 6th World Conference on Timber Engineering (WCTE), Whistler Resort, Canada, 2000.

Ceccotti, A., Follesa, M. [2006] “Seismic Behaviour of Multi-Storey X-Lam Buildings.” Proceedings of 426 COST E29 International Workshop on Earthquake Engineering on Timber Structures, pages 81-95, Coimbra, Portugal.

Ceccotti A., Follesa M., Lauriola M.P. [2007] "Quale fattore di struttura per gli edifici multipiano a struttura di legno con pannelli a strati incrociati?" XII Convegno ANIDIS L’ingegneria sismica in Italia, Pisa 2007.

Ceccotti A., Follesa M., Lauriola M.P., Sandhaas C. [2006] “Sofie Project - Test results on the lateral resistance of cross-laminated wooden panels”, Proceedings of the First European Conference on Earthquake Engineering and Seismology Geneva, Switzerland, 3-8 September 2006, Paper Number: 1912.

Ceccotti A., Follesa M., Lauriola M.P., Sandhaas C., Minowa C., Yasumura M. [2006] “Which seismic behaviour factor for multi-storey buildings made of cross-laminated wooden panels?” Proceedings of the 39th CIB-W18-Meeting, paper 39-15-2. Florence, Italy, 2006.

Ceccotti A., Sandhaas C., Okabe M., Yasumura M., Minowa C., Kawai N., [2013] “SOFIE project – 3D shaking table test on a seven-storey full-scale Cross-Laminated building”, Earthquake Engineering & Structural Dynamics, DOI: 10.1002/eqe.2309.

Ceccotti, A., Karacabeyli, E. [2000] “Dynamic analysis of nailed wood-frame shear walls”. 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 720, 1-8.

Ceccotti, A., Karacabeyli, E. [2002] “Validation of seismic design parameters for wood-frame shearwalls systems”. Canadian Journal of Civil Engineering 29.3 (2002): 484-498.

Ceccotti, A., Vignoli, A. [1987]. "Behaviour factor for timber structures in seismic zones" Proceedings of the 20th CIB-W18-Meeting, paper 20-15-1. Dublin, Ireland, 1987.

Ceccotti, A., Vignoli, A. [1989]. "A Pinching Hysteretic Model for Semirigid Joints" European Earthquake Engineering Journal, Bologna, Italy, V3, 3-9.

Computers & Structures Inc. [2009]. “SAP2000—Integrated finite element analysis and design of structures. Ver. 14” Computers & Structures Inc.: Berkeley, CA.

Christovasilis I. [2010] “Numerical and experimental investigations of the seismic response of light-frame wood structures” Phd Thesis, University at Buffalo, State University of New York, USA.

Dujic, B., Hristovski, V., Stojmanovska, M., Zarnic, R. [2006] “Experimental Investigation of Massive Wooden Wall Panel Systems Subjected to Seismic Excitation.” Proceedings of the First European Conference on Earthquake Engineering and Seismology Geneva, Switzerland, 3-8 September 2006.

European Committee for Standardization (CEN), [2009] “Eurocode 5 – Design of timber structures – Part 1-1: General rules and rules for buildings”, Brussels, Belgium.

European Committee for Standardization (CEN), [2004] Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, Brussels, Belgium.

FEMA [2013], “Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories”, FEMA P-807, prepared by the Applied Technology Council (ATC 71-1 Project) for the Federal Emergency Management Agency, Washington, D.C.

Filiatrault A., Christovasilis I., Wanitkorkul A., van de Lindt J.W. [2010] "Experimental seismic response of a full-scale light-frame wood building". ASCE J Struct Eng 2010;136(3):246–54.

Filiatrault A., Fisher D., Folz B., Dang C. [2002] "Seismic testing of two-story woodframe house: influence of wall finish materials." J Struct Eng 2002;128(10):1337–45.

Fischer, D., Filiatrault, A., Folz, B., Uang, C-M., and Seible, F. [2001] "Shake Table Tests of a Two-Story House," CUREE Publication No. W-06, Richmond, CA.

Follesa M., Fragiacomo M., Lauriola M.P., [2011] "A proposal for revision of the current timber part (Section 8) of Eurocode 8 Part 1" Proceedings of 44th CIB W18 Meeting, Alghero 2011, paper n.44-15-1.

Folz B., Filiatrault A., Ming Uang C., Seible F. [2001] "Blind predictions of the seismic response of a two-story woodframe house: an International Benchmark". Report No. SSRP – 2001/15. Division of Structural Engineering, University of California, San Diego, La Jolla, California 92093-0085.

Foschi RO. [2000] "Modeling the hysteretic response of mechanical connections for wood structures." Proceedings of 6th World Conference on Timber Engineering (WCTE), Whistler Resort, Canada, 2000.

Gavric I., Ceccotti A., Fragiacomo M. [2011] "Experimental cyclic tests on cross-laminated timber panels and typical connections." Proceedings of the 14th ANIDIS Conference, Bari (Italy), September 2011.

Gavric I., Fragiacomo M, Ceccotti A. [2012] Strength and deformation characteristics of typical X-Lam connections. Proceedings of the 12th World conference on timber engineering, Auckland (New Zealand), July 2012.

Gavric I. [2013] "Seismic behaviour of cross-laminated timber buildings." Phd Thesis, Università degli Studi di Trieste, Italy.

He, M., Lam, F. and Foschi, R. O. [2001] "Modeling Three-Dimensional Timber Light-Frame Buildings," Journal of Structural Engineering, ASCE, 127(8):901-913.

Koshihara, M., Isoda, H., Minowa, C., Sakamoto, I. [2004] "An Experimental Study On The Collapsing Process Of Wood Conventional Houses – Shaking Table Tests of Real-Size Models", Proceeding of 13Th WCEE, Vancouver, B.C., Canada.

Lauriola, M.P., Sandhaas, C. [2006] "Quasi-Static and Pseudo-Dynamic Tests on XLAM Walls and Buildings." Proceedings of COST E29 International Workshop on Earthquake Engineering on Timber Structures, pages 119-133, Coimbra, Portugal.

Ni C., Follesa M., Popovski M., Karacabeyli E. [2008] “Assessment of Seismic Design Parameters for Midply Wood Shear Wall System”, Proceedings of 10th World Conference on Timber Engineering (WCTE), Miyazaki, Japan, 2008.

Pang W, Rosowsky DV, Pei S, van de Lindt JW. [2010] “Simplified direct displacement design of a six-story woodframe building and pre-test performance assessment”. J Struct Eng 2010;136(7):813–25.

Pei S., van de Lindt JW. [2007] “User’s manual for SAPWood for Windows, Version 1.0” SAPWood user’s manual available at http://www.engr.colostate.edu/NEESWood.

Popovski M., Gavric I., Schneider J. [2014] “Performance of two-storey CLT house subjected to lateral loads”. Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Reiner H.J., Karacabeyli E. [2000] “Wood-frame constructions in past earthquakes”, Proceedings of the 6th World Conference on Timber Engineering, Whistler Resort, BC, Canada.

Rinaldin G., Amadio C., Fragiacomo M. [2013a] “A component approach for the hysteretic behaviour of connections in cross-laminated wooden structures.” Earthquake Engineering and Structural Dynamics, Vol. 42 No. 13, pp. 1885–2042, doi: 10.1002/eqe.2310.

Rinaldin G., Poh’sie G.H., Amadio C., Fragiacomo M. [2013b] “Modelling of seismic behaviour of light-frame timber structures.” Ingegneria Sismica, Special Issue on Timber Structures, No. 4/2013, pp. 82-98.

Rinaldin G., Poh’sie G.H., Fragiacomo M. Amadio C., Pontarin F. [2014] “Non-linear modelling of the three and seven storey x-lam buildings tested within the Sofie project.” Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Shimizu, H. et al [2008], "Shaking Table Test on full Scale Wooden Houses, Special project for Earthquake Disaster Mitigation in Urban Areas in 2005 – Improvement of Seismic Performance of Structures by E - Defense", Technical Note of the National Research Institute for Earth Science and Disaster Prevention, No.320.

Tomasi R., Piazza M., [2013] "Investigation of seismic performance of multi-storey timber buildings within the framework of the SERIES Project" in International Conference on Structure and Architecture, Guimaraes, Portugal : [ICSA2013], 2013.

van de Lindt, J. W. [2004] “Evolution of Wood Shear Wall Testing, Modeling, and Reliability Analysis: Bibliography,” Practice Periodical on Structural Design and Construction, ASCE, 9(1):44-53.

van del Lindt J. W., Pei S, Pyor S. E., [2009], “Shake Table Testing of a Six-Story 14,000 ft^2 Light-Frame Wood Building”. Wood Design Focus Vol. 19 n.4, 2009.

van de Lindt J. W., Pei S., Pryor S. E., Shimizu H., Isoda H. [2010]. "Experimental Seismic Response of a Full-Scale Six-Story Light-Frame Wood Building." Journal of Structural Engineering, Vol. 136 No. 10, pp. 1262-1272.

van de Lindt J. W., Bahmani P., Mochizuki G., Pryor S. E., Gershfeld M., Tian J., Michael D. Symans M. D., Rammer D. [2014] "Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Wood-Frame Building with Retrofits. II: Shake Table Test Results." Journal of Structural Engineering, E4014004, doi: 10.1061/(ASCE)ST.1943-541X.0001206.

2 – The Chapter for Timber Buildings of Eurocode 8. Evolution and critical review

2.1 Foreword. The first issue of the European codes

The first issue of the Eurocodes, i.e. the European Codes for structural design, was dated 1988 with the intent of establishing a set of common rules for the structural design for all the Member States of the European Community, which, at first, could be applied as an alternative to the corresponding national rules of the same technical matters. The intent was to reach a common agreement between all the State Members so that common performance criteria and general principles concerning the safety, serviceability and durability of the different types of construction and materials could be gradually adopted by all member countries, replacing in the end the different Member State rules.

At the time of the first issue the proposed Eurocodes were 8, covering the following topics: common unified rules for different materials and construction (Eurocode 1), concrete structures (Eurocode 2), steel structures (Eurocode 3), composite steel-concrete structures (Eurocode 4), timber structures (Eurocode 5), masonry structures (Eurocode 6), foundations (Eurocode 7) and structures in seismic regions (Eurocode 8) plus a not numbered Eurocode for actions on structures.

The compliance of the common rules with the corresponding national safety level, was left to the specification of appropriate values for safety coefficients for actions and materials which were purposely "boxed", i.e. marked with a surrounding rectangle. Those values were left to the decision of the competent National Authorities which, for each Eurocode, should be published in a National Application Document (at present named National Annex).

Eurocode 8, since its first draft (more or less the same content remained until the current version), was divided into 5 parts as in the following:

Part 1: General and Buildings

1.1 - Seismic actions and general requirements and rules for design.

1.2 - Buildings in seismic regions. General rules for design.

1.3 - Buildings in seismic regions. Specific rules for different materials and elements (concrete, steel, composite, timber, masonry, mixed structures, elements).

1.4 - Buildings in seismic regions. Strengthening and repair

Part 2: Bridges

Part 3: Towers, masts

Part 4: Silos, tanks

Part 5: Foundations, retaining structures and geotechnical aspects

The 1988 version contained only Part 1.1, Part 1.2 and Part 1.3. Part 1.3 was the part related to the specific rules for different materials and building systems and was divided into 8 chapters (1 - Introduction, 2 - Specific rules for Concrete Structures, 3 - Specific rules for Steel Structures, 4 - Specific rules for Composite Structures (as defined by Eurocode 4), 5 - Specific rules for Timber Structures, 6 - Specific rules for Masonry Structures, 7 - Specific rules for Mixed Structures, 8 - Specific rules for non-structural elements).

The format of this first draft was more similar to a textbook than to that of a draft code with paragraphs of comments next to the text intended to provide explanations about principles and rules. Since this first draft the fundamental principles on which the current version of Eurocode 8 is based were already established, i.e.:

- The "no collapse requirement" associated to the Ultimate Limit State, according to which structures shall be designed to withstand the design seismic action with a typical probability of exceedance of 10% in 50 years without local or general collapse, thus retaining its integrity and a residual capacity after this seismic action has ceased[1].

- The "no damage requirement" associated to the Serviceability Limit State, according to which structures shall be designed to withstand a design seismic action with a larger probability of exceedance (usually 50% in 50 years) without occurrence of damages and limitations of use[1].

- As it is explained in the code, "*For design purposes and in order to avoid the need for a nonlinear analysis, the concept of behaviour factor q is introduced. This parameter, which takes into account the energy dissipation capacity through ductile behaviour, is used to correct the results obtainable from a linear analysis in order to get an estimate of the nonlinear response*".

Therefore the seismic design of structures should be performed according to the Capacity Based Design philosophy and the values of the behaviour factor q can be found in the relevant chapters for the different materials, according to the ductility rules for the dissipative zones given for each structural type and the associated Ductility Class.

[1] This definition of the two different seismic design actions according to the relevant Limit State was indeed given in the next editions of Eurocode 8. In the 1988 edition only one design value was defined for the seismic action, associated to the "no-collapse requirement", and different deformation limits were given for the two conditions.

However, as it is explained in the introduction, "*it must be pointed out that differences exist between the various chapters of Part 1.3 with respect to the quantity of the contents, the extent of the details, the degree of justification of the proposed values and/or expressions, the spectrum of structural configurations for which solutions are proposed etc. The basic reason behind these differences is to be found in the varying degree of seismic experience and accumulated knowledge intrinsic to the various materials both from a theoretical point of view (research, laboratory testing, etc.) and from actual seismic events.*"

This statement explains why some chapters, like the ones for RC and the steel structures, were well detailed and already to an acceptable level of completeness, while others, like for the case of timber structures, were very short and concise and contained very few indications.

At the time of this first draft in fact, the knowledge about the seismic performance of timber structures was rather limited, and studies conducted about the seismic performance of timber buildings were confined to the experiences conducted mainly in New Zealand, California and Japan (Dean et al. 1986, Patton-Mallory et al. 1984, Yasumura et al. 1988) about the racking performance of Light-Frame walls and buildings subjected to lateral loading, and in Europe (Ceccotti and Vignoli 1985, 1987) about the dynamic performance of glulam arches and moment-resisting frames.

These experiences, together with the existing rules for the seismic design of timber structures in some international building codes and the observation of the behavior of timber buildings under real earthquakes occurred in Alaska, Chile and California, were collected together for the release of the first version of the chapter for the seismic design of timber structures in 1988 edition of Eurocode 8.

In the following paragraphs the evolution of the timber chapter of Eurocode 8 is presented, starting from the 1989 Background Document from Ario Ceccotti and Hans Jorgen Larsen written for the first 1988 release of the chapter up to the current version, released in 2004. The parts of text taken directly from the standard will be reported in italic and "boxed", i.e. market by a surrounding border, and the sentences included in the different versions will be discussed within the paragraph text. Moreover comments will be provided with remarks and observations based on the outcomes of recent research projects.

2.2 The 1989 Background Document and the first version of the Chapter for Timber Buildings in Eurocode 8

The Background Document for specific rules on timber structures in Eurocode 8 (Ceccotti and Larsen, 1989) was drafted around the end of 1987 and, as it was stated in the introduction, presented the state of the art of the researches conducted since then and their outcomes related to the design of timber buildings. The document, as previously mentioned, provided the basis for the first release of the timber chapter of Eurocode 8 which, in the 1988 edition, was composed only by 4 pages and very few sentences together with some paragraphs of comments provided with the same numbering and preceded by a “C” (e.g. C.5.1 in order to provide comments to the paragraph 5.1), as for the other parts of this release of Eurocode 8, explaining further details and some background about the rules provided.

In the following, the text of the first standard is provided together with comments reported from the Background Document.

5.1 General criteria

Timber elements and glued joints shall be designed and verified to behave linearly without demand for hysteretic energy dissipation under seismic actions.
In structures having joints with mechanical fasteners, their plastic behaviour and capacity to dissipate energy may be taken into consideration, provided that the behaviour of the joints under cyclic loading is determined by tests either on single joints or on whole structures or parts thereof.

5.2 Materials

Eurocode No 5, chapter 3, applies.
Only mechanical fasteners and connections with appropriate low-cycle fatigue properties are allowed to be used, except for type A structures.

This first two paragraphs in the *General Criteria* section explain the basis of the seismic design of timber structures, as wood elements in a timber structure shows a linear elastic behaviour until the collapse is reached, especially in bending or tension, while some dissipation may occur for compression perpendicular to the grain stresses or for friction between members in carpentry joints. The same applies for glued joints, while the energy dissipation capacity through a ductile behaviour may be obtained for well-designed joints with mechanical fasteners.

The non-linear behaviour of joints with mechanical fasteners is well explained in the Background Document where a typical force-deformation behaviour for a semi-rigid joint in a moment resisting frame subjected to cyclic loading is illustrated as shown in Figure 2-1.

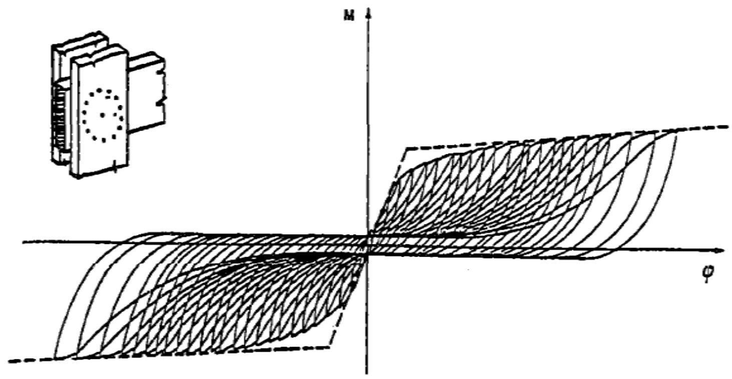


Figure 2-1 Moment-rotation relationship for a dowelled joint subjected to cyclic loading (after Ceccotti and Vignoli, 1988).

According to the explanation given in the Background Document, the features of the joints behaviour are listed the following:

a) *The joints exhibits for static loading a very pronounced ductility with static ductility factors D_S in the range of 8-13 (Soltis and Patton-Mallory 1986). D_S is defined as the ratio between the ultimate deformation or deformation at the maximum load and the yield deformation (defined by the intersection of the initial tangent and the asymptote of the parent curve, see Figure 2-1).*

b) *The peak values follow an increasing parent curve, which is practically the same as would be found in a normal ramp test to failure.*

c) *The degradation, i.e. the ratio between the loads in two successive cycles, is relatively small and comparable to that of materials normally associated with good seismic behaviour (except for punched metal plates and toothed plate connectors).*

Particularly statement a) above led to the following proposal which was not introduced in text code but in the annexed comments:

C.5.2. Materials

For type B and C structures (see Cl. 5.3) a deemed-to satisfy rule is as follows. Connections between elements must be able to deform plastically (according to their intended deformational mechanism) for at least three fully reversed cycles at a ductility ratio of 4 without impairment of their strength larger than 20%. This rule needs not to be satisfied for type A structures.

As it is further explained in the Background Document, *to ensure sufficient ductility it is proposed to introduce in the final Eurocode 8* (ed. this requirement was indeed never introduced in the final version) *the requirement that the static ductility should be greater than the assumed behaviour factor q, by a factor k:*

$$D_s \geq k \cdot q \tag{1}$$

with k in the range 3-4.

The reason for the above statement is explained later in the Background Document where it is written, discussing the results of two-dimensional non-linear dynamic analysis conducted in order to evaluate the behaviour factor q for moment-resisting frames, "*In many cases the equal displacement concept has been advocated. Stated simply this concept requires that if a structure is designed to yield at a force which is a fraction 1/r of the elastic response force, the structure must be capable of accommodating a displacement of r times the yield displacement. In other words: the behaviour factor q is assumed equal to the static ductility ratio (k equal to unity in formula 1). This is, however, only true for some earthquakes and for structures having ideal elastic-plastic behaviour. For practical structures the q-values are lower than the static ductility. Based on theoretical and empirical considerations Pozzati in (Pozzati, 1983) proposes a value of k=4 in formula 1 for concrete structures, and Macchi (Macchi, 1983) proposes a value of k=2.5 for masonry structures. A similar value was found for the frames described above. For these the minimum values for D_s was about 12.8 and q=3-3.5, corresponding to k=4-3.5 for the worst case and k=2-2.5 average.*"

It must be pointed out that, at the time when the Background Document was written, no international standard on the evaluation of the seismic properties of mechanical joints in timber structures existed. In 2001 a European Standard for the evaluation of the cyclic properties of joints made with mechanical fasteners was published (*EN 12512:2001: Timber structures- Test methods. Cyclic testing of joints made with mechanical fasteners*). Besides in 2003 an ISO Standard was published (*ISO 16670:2003:* "*Timber structures -- Joints made with mechanical fasteners -- Quasi-static reversed-cyclic test method*") and other standards on the same topic have been published in different countries (USA, Australia, Japan, etc.).

As it can be easily understood, the estimation of the static ductility depends on (i) the definition of the yield displacement, (ii) the definition of the ultimate displacement and (iii) the used loadings protocol for cyclic testing.

While there is an international agreement about the definition of the ultimate displacement (defined as the displacement corresponding to 80% of the maximum load in the descending portion of the envelope curve in a cyclic testing), different methods are proposed for the evaluation of the yield displacement of mechanical joints in timber structures and of the loading protocol for cyclic testing. Munoz et al. (Munoz et al., 2008) demonstrated by comparing six different methods for the evaluation of the yield point, that a difference up to 100% can be found in the estimation of the static ductility. Vogt et al. (Vogt at al., 2012) demonstrated that, depending on the loading protocol (especially the number of cycles with a high deformation and the rate of displacement), significant differences in the maximum forces could be found. Also significant differences of maximum force and displacement were found between monotonic and cyclic loading with the different protocols.

5.3 Structural types

Based on the ductility and the capacity to dissipate energy under seismic action, distinction shall be made between the following types:

5.3.1 Type A: Non dissipative structures, such as

- structures without or with only a few joints with mechanical fasteners

- arches with hinged joints

- cantilever structures with built-in columns

- structures with diaphragms solely with glued joints

5.3.2 Type B: Low dissipative structures, such as

- frames or beam-column structures with semirigid joints between all members and the foundations

- trussed frame structures with mechanical fasteners

5.3.3 Type C: Medium dissipative structures, such as

- structures with diaphragms resisting the horizontal forces, connected by nails. Panels may be either glued or nailed.

The structural types considered reflected the international state of the art for timber structures at the time when these provisions were written (1988). In the Background Document explanations about the assignment to Low and Medium dissipative classes for the considered structural types are given based on an extensive programme of two-dimensional non-linear analysis carried out on moment resisting frames (Ceccotti and Vignoli 1987, Ceccotti and Vignoli 1988) and on an extensive literature review about the seismic performance of Light-Frame buildings during past earthquakes and about Light-Frame shear wall behaviour under lateral loading.

5.4 Behaviour factors and damping ratio

The values for the q factors given in Table 5.4 shall be applied.

Table 5.4: Value of the behaviour factor q

Type of structure:	q
A Non dissipative	1
B Low dissipative	1
C Medium dissipative	1

For all structural types a viscous damping ratio of ξ = 10 % (see Cl. 4.3.1 of part 1.1) may be assumed.

As explained in the annexed comment C.5.4. provided in the code text, the knowledge about the seismic performance of timber structures at the time these provisions were written was limited, therefore conservative values of the behaviour factor q were proposed.

However in the Background Document a first proposal of behaviour factor greater than unity based on recent researches conducted and referenced in the document, was given for future reviews of the code:

Type of structure:	*q*	ξ
A Non dissipative	*1.0*	*5%*
B Low dissipative	*2.0*	*5%*
C Medium dissipative	*2.5*	*10%*

As the design spectrum was defined for a 5% elastic viscous damping, in 1988 edition of Eurocode 8 a correction factor for the design spectra ordinates was proposed in order to give the possibility for some structural systems to account for values of the viscous damping different from the reference one. Values of the viscous damping ξ were given in the different materials chapters.

5.5 Safety verifications, limitations, detailing

For safety verifications the relevant provisions in chapter 4 of part 1.2 and in Eurocode N° 5 apply with the following additions and modifications:

1. *In diaphragms the strength of the panels shall be so high that failure will take place in the joints, not in the panels.*
2. *If q-values higher than 1.0 are taken into account, the members shall be designed such that their load-bearing resistance is higher than that of the connections.*
3. *k_{mod} for instantaneous load apply.*

4. *For ultimate limit state verifications the partial safety coefficients for material properties γ_M from Eurocode 5, table 2.3.3.2 for fundamental load combinations apply.*
5. *Compression members and their connections which may fail due to deformations caused by load reversals shall be so designed as to retain their original position at all times.*
6. *Bolts shall only be used in secondary members, and they shall be tight and tightfitting in the holes.*
7. *Smooth nails shall not be used.*

This section provides together a first draft of Capacity Based Design rules, safety verification provisions and detailing rules for dissipative and non-dissipative parts of the structure to be followed in the seismic design. As it is further explained in the Background Document, "*The use of q-values greater than unity is only permitted if a number of detailing requirements are met. The most important are:*

- *It must be verified that the failure mechanism is the one assumed in the design, and especially that failure takes place in the joints after ductile behaviour and not in the wood or other non-ductile members. The strength of all timber members shall be higher than the strength of the relevant joints.*
- *The structures shall be structurally regular as described in Eurocode 8, Part 1.2, clause 2."*

Further provisions in the Background Document are also given for the detailing of some type of joints where perpendicular to the grain compression deformations and brittle failures due to tension perpendicular to the grain may occur.

The above quoted references provide, as mentioned above, a first proposal of Capacity Based Design rules to be followed. However these provisions, even if correct in principle, were not sufficient to provide guidance to the structural designers, since no indications about the over-strength factors to be used in the design to avoid anticipated brittle failures in wood members or other brittle elements were given.

5.6 Limitation of damage

The provisions given in part 1.2 apply

5.7 Control of design, construction and use

The provisions given in part 1.1 apply

These paragraphs were written in order to provide consistency with the other relevant chapters of Eurocode 8 for other materials, but no specific provisions in the code text or in the Background Document for timber structures were given.

2.3 The 1995 ENV version of Eurocode 8

A comprehensive revision of the chapter for timber buildings was provided with the release of the 1995 ENV version of Eurocode 8. After the publication of the first draft of the Eurocodes the European Commission gave mandates to CEN for the development of the Structural Eurocodes, initially published as European Prestandards (ENV) and later as European Standards (EN). With the publication of the Prestandard version, a new Eurocode was included, Eurocode 9: Design of aluminium structures, while the definition of actions on structures was included in Eurocode 1.

Like the first version of Eurocode 8, the new ENV version comprised the following parts:

- Part 1-1: General rules, seismic actions and general requirements for structures;
- Part 1-2: General rules - General rules for buildings
- Part 1-3: General rules - Specific rules for various materials and elements
- Part 1-4: General rules - Assessment and retrofitting of buildings;
- Part 2: Bridges;
- Part 3: Towers, masts and chimneys;
- Part 4: Silos, tanks and pipelines
- Part 5: Foundations, retaining structures and geotechnical aspects.

The chapter for timber buildings (Chapter 4 of Part 1.3) was entirely re-written by a group of international experts and presented in his first draft at the 1993 26th CIB-W18 Meeting in Athens, Georgia, USA (Becker et al., 1993). This paper provided explanations and comments about the provisions included in this new chapter and could be regarded, in some way, as a sort of “Background Document” or commentary of the Code text.

The chapter was completely changed with respect to the previous version. New paragraphs were introduced (e.g. a new paragraph including provisions on Structural Analysis) and the existing ones were largely improved and detailed (the “General criteria” paragraph was detailed with definitions and design concepts to be adopted in the design, the “Material” paragraph was detailed with new provisions about properties of wood-based panels and of dissipative connections, the “Structural types” section was largely improved with an increased number of ductility classes, new structural types described with graphic sketches and new values of the behaviour factor, and the paragraphs about “Detailing Rules”, “Safety verifications” and “Control of design and construction” were completely rewritten). It is by far the most complete version of this chapter ever provided. As it will be discussed later, the subsequent edition simplified the rules here included, not always providing a clearer understanding.

Like for the other part of this new version of Eurocode 8, and in the 1988 edition, the text was divided into Principles, i.e.:

- numbered clauses marked with a P which includes (i) general statements and definitions for which there is no alternative and (ii) requirements and analytical models for which no alternative is permitted, if not otherwise established;

- Application Rules, i.e. simply numbered clauses, which are generally recognized rules which follow the principles and meet the requirements.

The new chapter was composed of 8 pages and was included in Part 1.3. related to the general and specific rules for various materials and elements and, like for the relevant chapters related to other materials, the formulation of several parts depended on the regulations given in other parts of Eurocode 8 (e.g. the regularity classes and damping ratios) and on the provisions given in the corresponding Eurocode for the design of timber structures, i.e. Eurocode 5.

In the following Sections, the code text is provided in boxed paragraphs and comments are provided on the relevant parts, partly taken from the CIB-W18 paper, which explains the background behind some provisions, together with some discussion related to the current state of the art in the field of seismic research on timber structures.

4.1. General

4.1.1 Scope

P(1) For the design of timber buildings Eurocode 5 applies. The following rules are additional to those given in Eurocode 5 with respect to seismic design.

4.1.2 Definitions

P(1) The following definitions and symbols are used in this chapter with the following meanings:

Static ductility:	*Ratio between the ultimate deformation and the deformation at the end of the elastic behaviour evaluated in quasi-static cyclic tests (see 4.3 P(4)).*
Semi-rigid joints:	*Joints allowing significant loading deformation, the influence of which has to be considered in structural analysis according to Eurocode 5 (e.g. mechanical timber joints).*
Rigid joints:	*Joints with insignificant loading deformation, which is negligible according to Eurocode 5 (e.g. glued solid timber joints).*

Dowel-type joints:	*Joints with dowel-type mechanical fasteners (nails, staples, screws, dowels, bolts, etc.) loaded perpendicular to their axis (activating embedding resistances).*
Carpenter joints:	*Joints where loads are transferred by means of pressure areas and without mechanical fasteners (e.g. skew notch, tenon, half joint).*

This part was completely rewritten with regard to the first 1988 version, providing definitions and criteria which give a clear understanding of the different terms referenced in the code. However, some definitions remains too vague, such as the "significant" and "insignificant" loading deformation used for the classification of semi-rigid and rigid joints, since no limits are provided. Also the definition of the static ductility, as it is dependent on the method used for the evaluation of the yield and ultimate displacement, suffers from the missing reference to an agreed European Standard on cyclic testing of timber joints (at the time not yet available) where these definitions are clearly stated.

4.1.3 Design concepts

P(1) Earthquake-resistant timber structures shall be designed according to one of the following concepts:

a) Concept of dissipative structural behaviour.

b) Concept of non-dissipative structural behaviour.

(2) When using concept a) the capability of parts of the structure (dissipative zones) to resist earthquake actions by moving out of the elastic range is taken into account. When using the design response spectrum defined in Clause 4.2.4. of ENV1998-1-1 the behaviour factor is taken as q>1. The value of q depends on the structural type (see 4.3).

P(3) Dissipative zones shall be regarded as located in joints and connections with mechanical fasteners, whereas the timber members themselves shall be regarded as behaving elastically.

(4) The properties of dissipative zones shall be determined by tests either on single joints, on whole structures or on parts thereof according to EN XX[2]. Provisions for avoiding such tests are given in 4.3.(5).

(5) In concept b) the action effects regardless of the structural type are calculated on the basis of an elastic global analysis without taking into account non-linear material behaviour. When using the design spectrum defined in clause 4.2.4 of Part 1-1, the behaviour factor q is taken equal to 1,0.

The two design concepts of dissipative and non-dissipative structural behaviour are introduced. In case of design with the concept of dissipative behaviour, some rules regarding the properties of dissipative zones (which as it is stated in P(3) should be located in joints and connections with mechanical fasteners) should

[2] At the time of publication of this ENV no related EN exists. In the meantime reference should be made to agreed international recommendations (e.g. RILEM – TC 109 TSA).

be fulfilled. The application rule (4) was proposed in order to avoid limitations to the use of new developed types of connections and new structural types in seismic regions for timber structures. Therefore the meaning of this rule should be "unless the dissipative zones follow the provisions given in 4.3.(5) (i.e. detailing rules given for dowel type connections, shear walls and diaphragms in Light-Frame structures) the non-linear properties should be evaluated by tests according to the relevant European Standard". However, as it is explained in the footnote, at the time when these provisions were published, no European Standard for the evaluation of the non-linear properties of mechanical joints existed (the EN 12512 referenced above was published in 2001), and therefore the designer should make reference to agreed international recommendations.

4.2. Materials and properties of dissipative zones

P(1) Clauses 3, 6 and 7 of Part 1-1 of Eurocode 5 apply. With respect to the properties of steel elements, clause 3 of Part 1-1 of Eurocode 3 applies.

P(2) When using the concept of dissipative structural behaviour, the following provisions apply:

a) Only materials and mechanical fasteners providing appropriate low cycle fatigue behaviour may be used in joints regarded as dissipative zones.

b) Glued joints shall be considered as non-dissipative zones.

c) Carpenter joints may only be used when they can provide sufficient energy dissipation capacity. The decision on their use shall be based on appropriate test results.

(3) Paragraph (2) a) is deemed to be satisfied if clause 4.3 (4) is fulfilled.

When tested according to EN XX[1] joints shall be verified to have appropriate low cycle fatigue properties under large amplitudes to ensure a sufficient ductility in respect to their intended deformational mechanism and to justify the q value assumed in the analysis (see 4.3.(4)).

(4) For sheathing-material in diaphragms, paragraph (2) a) is deemed to be satisfied, if the following conditions are met:

a) Particleboard-panels have a density of at least 650 kg/m^3.

b) Plywood-sheathing is at least 9 mm thick.

c) Particleboard - and fibreboard sheathing are at least 13 mm thick.

P(5) Steel material for connections shall comply with the following conditions:

a) All connection elements made of cast steel shall fulfil the relevant requirements in Eurocode 3 and in section 3 of this Part.

b) The ductility properties of the connections between the sheathing material and the timber framing in type C and D structures (see 4.3) shall be tested for compliance with 4.3.(4) by cyclic tests on the relevant combination of sheathing material and fastener.

As it is explained in the above referenced CIB-W18 paper (Becker et al., 1993) the provisions given in P(2) are not intended to limit the use of glued or carpentry joints in seismic regions, as there is no reason to do so, but to avoid or limit their use as dissipative connections, as it is correctly stated at the beginning.

Rule (3) follows Rule (4) of 4.1.3. In other terms, the two types of connections which could be regarded as dissipative, if the detailing rules specified in 4.3.(5) are met, are:

- doweled, bolted and nailed timber-to-timber and steel-to-timber joints;
- nailed sheathing-to-framing connections in shear walls and diaphragms of Light-frame assemblies.

Other type of connections with mechanical fasteners, or the same type of connections mentioned above if the rule specified in 4.3(5) is not satisfied, could be regarded as dissipative if the provisions given in 4.3P(4) are met. Comments about the provision 4.3P(4) will be provided later.

Regarding rule (4) it should be underlined that the most common type of wood-based product currently used worldwide as sheathing material in Light-Frame assemblies, i.e. Oriented Strand Board (OSB), is not mentioned. Moreover, recent research conducted at the University of Trento, Italy (Sartori and Tomasi, 2013) and within the SERIES Project (Piazza et al., 2013) have proved the suitability of Gypsum Fibre Panels (PF-GF) connected to the timber framing with staples as a sheathing material for shear walls in Light-Frame construction.

4.3. Structural types and behaviour factors

P(1) According to their ductile behaviour and energy dissipation capacity under seismic actions timber buildings shall be assigned to one of the four types A - D given in table 4.1 with the corresponding behaviour factors.

Table 4.1 Structural types and behaviour factors

Type	*Description*	*Behaviour factor q*
A	*Non-dissipative structures*	*1,0*
B	*Structures having low capacity to dissipative energy*	*1,5*
C	*Structures having medium capacity to dissipative energy*	*2,0*
D	*Structures having good capacity to dissipative energy*	*3,0*

(2) Examples of structural systems belonging to these different types are given in fig. 4.1.

(3) If the building is non-regular in elevation (see clause 2.2.3 of Part 1-2) the q-values listed in table 4.1 should be reduced by 20% (but need not be taken less than q = 1,0).

P(4) In order to ensure that the given values of the behaviour factor can be used, the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for type B structures and at a static ductility ratio of 6 for type C and D structures, without more than a 20% reduction of their resistance.

(5) The provisions of paragraph (4) above and of 4.2.(2) a) and 4.2.(5) b) may be regarded as satisfied in the dissipative zones of all structural types if the following provisions are met:

a) In doweled and nailed timber-to-timber and steel-to-timber joints, the minimum thickness of the connected members is 8d and the dowel-diameter does not exceed 12 mm.

b) In shear walls and diaphragms, the connection of sheathing to the timber framing of diaphragms the sheathing material is wood-based and the minimum thickness of the sheathing material is 4d, where d does not exceed 3,0 mm.

(6) Behaviour factors different from those presented in table 4.1 may be used for specific structures, if their validity is demonstrated on the base of analytical simulations and tests under a representative number of earthquakes (see clause 4.3.2 of Part 1-1).

This paragraph was completely changed with respect to the 1988 edition. The number of structural types was increased from 3 to 4 and values of the behaviour factor q were provided for all the different structural types. Moreover the number of structural systems for each structural type was increased and graphical sketches were provided for a better understanding, even if some drawings were partly mistaken in the published version[3].

As already discussed above in this paragraph and mentioned in §2.3 the definition of static ductility ratio depends on the methods used for the evaluation of the ultimate and yield displacement. Vogt et al. (Vogt et al., 2012) and Boudaud at al. (Boudaud et al., 2010) demonstrated that by applying the EN12512 procedure for the evaluation of the static ductility, in some cases Light-Frame shear walls do not meet the requirements given in P(4) for the static ductility for medium and high-dissipative structures. Gavric et al. (Gavric et al., 2011) estimated the value of static ductility for hold-down connections and angle bracket

[3] Probably due to a misinterpretation, some graphic sketches provided in the referenced CIB-W18 paper (Becker et al., 1993) were mistaken in the final document. For instance, the graphic sketch provided in the CIB-W18 paper for the three-hinged glulam arch was the one on the left, which was completely different form the one on the right and shown in Fig. 4.1a of the code text, where the arch cross-section at supports is very thin in comparison with the apex, which is wrong and never realized in practical cases.

This may be the reason why the graphic sketches were deleted in the subsequent versions of the chapter for timber buildings in Eurocode 8.

connections in CLT buildings (a structural system which was not yet developed at the time these provisions were written) and found that they do not even meet the requirement for low-dissipative structures. Moreover as it is explained in Becker et al., 1993, "*In this clause the above mentioned related provisions concerning the testing of ductility properties are to be found. The described method to determinate the performance is just a simple proposal, but a more detailed evaluation would exceed the justifiable size of proofing provisions in the design code. For the time being the values of "4" and "6" are so-called "boxed values" to be stipulated by the National Authorities during the ENV phase of the code. But as the National Authorities should only influence values concerning the safety level, these (research related) values should be fixed*".

The provisions given in Clause (5) for the dissipative zones of all structural types are prescriptive, and they do not consider the type of failure mode calculated according to the Johansen theory given in Eurocode 5, even if, as explained again in Becker et al., 1993, they are intended to provide a failure mode characterized by the formation of two plastic hinges in the fastener, i.e. a ductile failure mode. Reasons for these provisions are explained in Ceccotti, 1995. However this limitation could be better addressed by requiring directly to the designer a failure mode characterized by the formation of one or two plastic hinges in the mechanical fastener, which can be easily checked using the Johansen equations prescribed by the Eurocode 5 Part 1-1.

Figure 4.1a Examples of structural systems (Part 1).

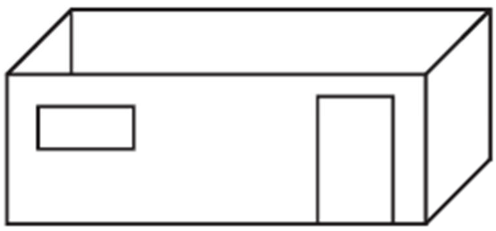
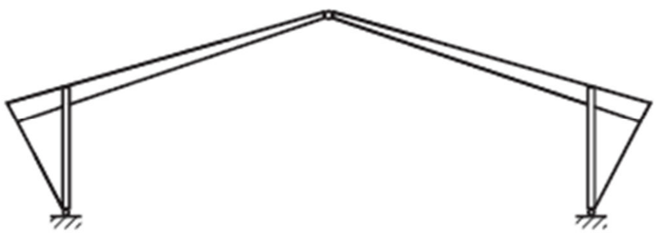
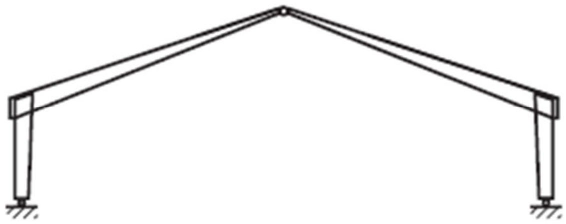
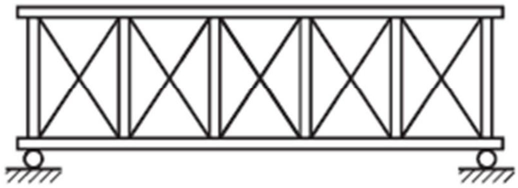
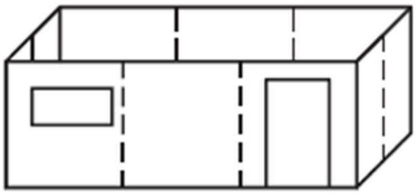
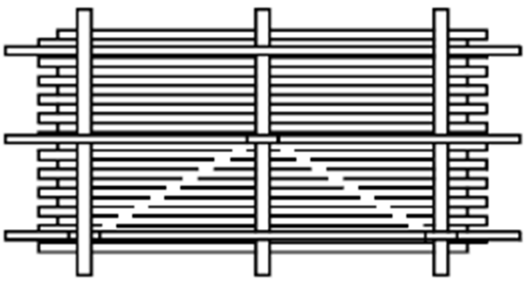
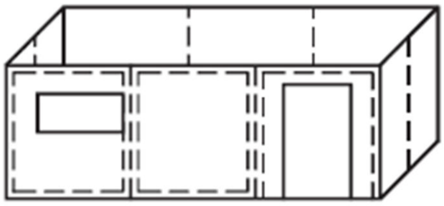
Type	*Description*	*q*	*Example*
A	***Non dissipative structures*** *Having none or only few joints with mechanical fasteners beyond the dissipative zones*	*1,0*	*Arches with hinged joints*   *Cantilever structures with rigid connections at the base*  *Buildings with vertical diaphragms resisting the horizontal forces without mechanical fasteners for both interconnection and between sheathing and timber framing* 
B	***Structures having low capacity of energy-dissipation***	*1,5*	*Structures with semi-rigid connections at the base* 
			Structures with few but effective dissipative zones 

Figure 4.1b   Examples of structural systems (Part 2).

Type	*Description*	*q*	*Example*
C	***Structures having medium capacity of energy-dissipation***	*2,0*	*Frames or beam-column structures with semi-rigid joints between all members, Connections with foundations may be semi-rigid as well as hinged (due to the load carrying system)*  *Trussed frame structures with mechanical fasteners in the joints of the frame and/or the connections of the bracing elements*  *Buildings with vertical diaphragms resisting the horizontal forces where sheathing is glued to the framing. Diaphragms are interconnected by mechanical fasteners (horizontal diaphragms may be glued or nailed)*  *Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infillment* 

D	***Structures having good capacity of energy-dissipation***	*3,0*	*Buildings with vertical diaphragms resisting the horizontal forces where sheathing is fixed to the framing by mechanical fasteners as well as the interconnection of the wall-systems (horizontal diaphragms may be glued or nailed)* 

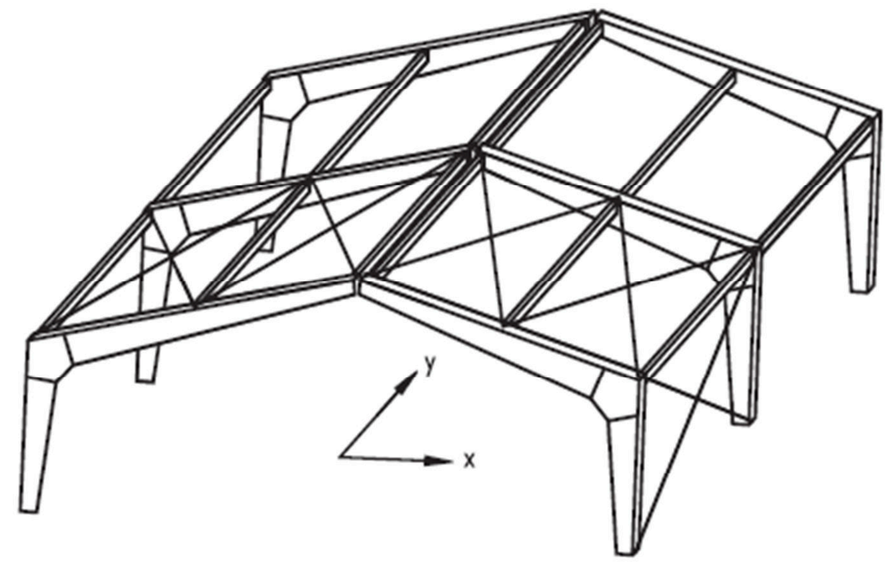
As mentioned before, the number of structural types was increased with respect to the 1988 version. Though it does not seem completely clear at a first reading, all the structural type referenced in Figure 4.1a and 4.1b refer to lateral load resisting systems of buildings (i.e. frames with hinged, rigid or semi-rigid connections, bracing systems and shear walls).

At the time when this table was written there was a discussion whether for type D structures a greater value of the behaviour factor q could be adopted (e.g. 3,5). Evidences about the good seismic performance of Light-Frame buildings were given from the earthquakes occurred in California and New Zealand and from the test results on timber sheathed shear walls, which showed that an higher value of the elastic damping ratio (greater than 5%) could be considered due to friction mechanisms for these type of structures. For this reason the following note was provided in Becker et al., 1993 *"To enable an effective q-value of 3,5 initially the following footnote was foreseen <<The q-value for type D-structures may be increased, if the existence of sufficient friction mechanism is provided. Under application of the rules given in chapter 5.6* (ed. 4.7 in the final version) *this is to be valid>>. Most of the timber structures show a damping ratio generally of more than 5% in the elastic range; the equivalent viscous damping is even higher in the inelastic range. To correct the use of a 5%-based response spectrum there are two possibilities: either to estimate the "real" q-factors (according to the real damping ratios) and use the correction factor from the May 1988-version or to include the diverging damping ratio in a modified q-value. The type D-structures are supposed to show the maximum of dissipation properties in timber buildings. A significant part of that is provided by friction mechanism. As there is a (still running) discussion to implement the correction factor in Eurocode 8, if the friction is exceeding the 5%-value, which is the base of the response-spectrum, the proposed values (mainly for type C and D-structures) are not fixed yet."*

There is no evidence whether it was this discussion about the possibility of considering higher q-values due to friction mechanisms leading to higher values of elastic damping ratios or the comparison with other international Standards (e.g. the Canadian Building Code) where the value of the action reduction factors for the different structural systems are higher, but as discussed later, in the subsequent edition of this chapter the q-values were largely incremented.

(7) For structures having different and independent properties in the x - and y - directions (see fig. 4.2), different q-factors may be used for the calculation of the seismic action effects in each main direction.

Figure 4.2 Example of a structure with different behaviour in the main directions (type A in x and type C in y direction)



As it is further specified in Becker et al., 1993, "*The demand for a use of spatial model (without a previous determination of the both "main directions") which is ruled in part 1.1 of Eurocode 8 according to the regularity classes, is independent from that, as the q-values are influencing the analysis on the actions side, so that there's no problem to consider the properties of the different directions by different q-values."*

4.4. Structural analysis

P(1) In the analysis the slip in the joints of the structure shall be taken into account.

P(2) An E_O-modulus-value for instantaneous loading (10% higher than the short term one) shall be used.

(3) Floor diaphragms may be considered rigid in the structural model without further verification, if

a) the detailing rules for horizontal diaphragms given in 4.5.3 are applied

and

b) their openings do not significantly affect the overall in-plane rigidity of the floors.

As also mentioned in Becker et al., 1993, since in Eurocode 5 no value of an E_0-modulus for instantaneous loading is given, it is questionable whether this provision should be left in the code text.

Regarding the application rule (3) it is further specified in Becker et al., 1993, *"Often the horizontal diaphragms are supposed to be rigid in plane without verification. But the knowledge of the realistic* *stiffness of the horizontal diaphragms is necessary for a control of deformation of the supporting elements. Initially the formulation was >…openings are limited in number and area…<. Limits and regarding application rules could be:*

- *openings in corners should be avoided,*
- *dimensions of an opening adjacent to a support should not exceed 20% of that support,*
- *edges and borders of openings as well as jump-in corners should be stiffened by appropriate means.*

But it seems to be impossible to give generally valid limitation-values. So the formulation aims at the responsibility of the designer to ensure the rigidity respectively the omission of a precise calculation of the in-plane stiffness."

4.5. Detailing rules

4.5.1 General

P(1) The detailing rules given in 4.5.2 and 4.5.3 apply for earthquake-resistant parts of structures designed according to the concept of dissipative structural behaviour.

P(2) Structures with dissipative zones shall be designed so that these zones are located mainly in those parts of the structure where yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

This section was completely re-written with respect to the 1988 edition. As mentioned above, the list of structural types given in Figures 4.1a and 4.1b is referred to buildings with lateral load resisting systems composed by different types of frames with hinged, rigid or semi-rigid connections, bracing systems connected with joints made with mechanical fasteners and glued or nailed shear walls. Therefore these detailing provisions are addressed only to joints with mechanical fasteners and horizontal diaphragms in Light-Frame buildings, in order to ensure in the seismic design of these systems the desired means of energy dissipation in the dissipative zones within the Capacity Based Design philosophy.

4.5.2 Detailing rules for connections

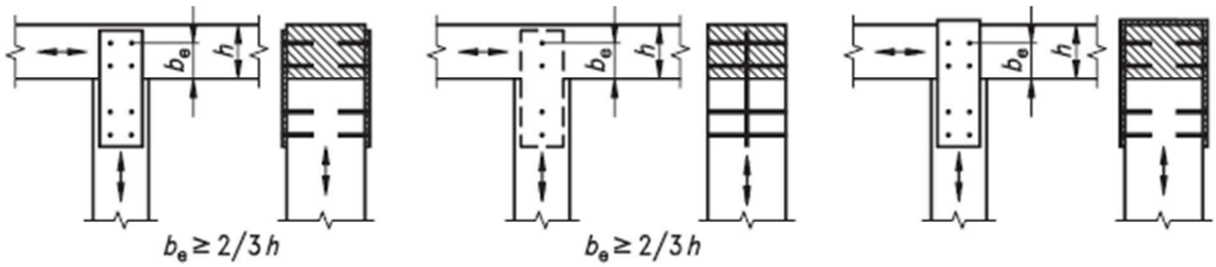
P(1) Compression members and their connections (e.g. carpenter joints), which may fail due to deformations caused by load reversals, shall be designed in such a way that they are prevented from separating and remain in their original position.

P(2) Bolts and dowels shall be tightened and tight fitted in the holes. Large bolts and dowels (d > 16 mm) shall not be used in timber-to-timber and steel-to-timber connections, except in combination with timber connectors.

(3) Smooth nails and staples should not generally be used without additional provision against withdrawal. They are however admissible in diaphragms for the connection of sheathings to the timber framing (see 4.3.(5) b)) and in secondary members.

(4) In the case of tension perpendicular to the grain, additional provisions should be met to avoid splitting, as for instance shown in fig. 4.3.

Figure 4.3 Examples of acceptable provisions the case of tension perpendicular to the grain.



a) By two external nailed steel plates b) By an inner dowelled steel plate c) by a U shaped cover steel plate

Again the provision P(2) give a prescriptive rule in order to avoid non-ductile failures in joints with mechanical fasteners, i.e. referring to the failure modes of Johansen theory, to those failure modes were only embedding in timber elements occurs, with no plasticization at all of mechanical fasteners. However there is no reason in principle to give a limitation to the fasteners diameter, but again a better formulation should address to the failure modes of the connection, specifying that non-ductile failure modes, where only embedment in timber elements occur, should be avoided.

Like for the additional provisions referenced in Fig. 4.3, they are now incorporated into Eurocode 5 together with the provisions for the design against other types of brittle connection failures such as block shear and plug shear failures at multiple dowel-type steel to timber connections.

4.5.3 Detailing rules for horizontal diaphragms

P(1) For horizontal diaphragms under seismic actions clause 5.4.2 of Part 1-1 of Eurocode 5 applies with the following modifications:

a) Paragraphs (2) and (6) shall not be applied.

b) Contrary to paragraph (5) the distribution of the shear forces in the diaphragms shall be evaluated by taking into account the in-plan position of the lateral load resisting vertical elements.

P(2) *All sheathing edges not meeting on framing members shall be supported on and connected to blocking (see fig. 4.4). Blocking shall also be provided in the horizontal diaphragms above the lateral load resisting vertical elements (e.g. walls).*

P(3) *The continuity of beams and especially of headers shall be ensured in areas of diaphragm-disturbances.*

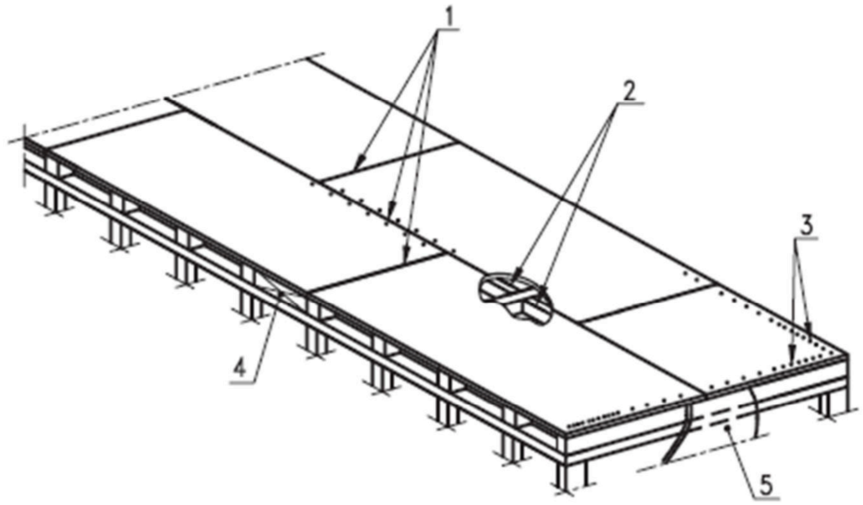
P(4) *The slenderness of beams shall be limited to b/h < 4.*

P(5) *In seismic zones with $a_g \geq [0,2]\cdot g$ the spacing of fasteners in areas of discontinuity shall be reduced by dividing by a factor of 1,3 but not less than the minimum spacing given in Eurocode 5 (see fig. 4.4).*

P(6) *When floors are considered as rigid in plan for structural analysis, there shall be no change of span-direction of the beams over supports, where horizontal forces are transferred to vertical elements (e.g. shear-walls).*

Figure 4.4 ***Supporting and nail-spacing at the edges of sheathing panels***

Legend
1 Splice
2 Supported splice (blocking)
3 Reduced spacing
4 Blocking
5 Wall sheathing



A further explanation to P(6) is given in Becker et al., 1993. *"The stiffness of diaphragms changes with the direction of the beams. If areas of different span direction of the beams are touching over a subsequent bearing and stiffening member (e.g. a wall element), the behaviour of the diaphragm cannot be estimated realistically."*

4.6. Safety verifications

P(1) The strength values of the timber material shall be determined taking into account the k_{mod}-values for instantaneous loading according to clause 3.1.7 of Part 1-1 of Eurocode 5.

P(2) For ultimate limit state verifications of structures designed according to the concept of non - dissipative structural behaviour the partial safety factors for material properties γ_M for accidental load combinations from table 2.3.3.2 of Eurocode 5 apply.

P(3) For ultimate limit state verifications of structures designed according to the concept of dissipative structural behaviour the partial safety factors for material properties γ_M for fundamental load combinations from table 2.3.3.2 of Eurocode 5 apply.

P(4) In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength. This overstrength requirement applies especially for

- anchor-ties and any connections to massive subelements,

- connections between horizontal diaphragms and lateral load resisting vertical elements.

The provisions P(2) and P(3) seem to be mistaken and they were corrected by inverting the proposed values for the partial safety factors for material properties γ_M for the respective verifications according to the non-dissipative and dissipative structural behaviour in the next edition of this chapter in the EN version. However, analysing further in detail, they were correctly formulated and the subsequent correction made in the current EN version was wrong. A further explanation is given in Becker et al., 1993 *"To use the partial safety factors for the material properties γ_M for fundamental load combinations was used with respect to all other materials chapter in Eurocode 8. Additionally the material has to fulfil the requirements of 5.4 P(4)* (ed. 4.3 P(4) in the published version) *so a general use of the partial safety factors for the material properties γ_M for accidental load combinations would be sufficient. For type A-structures the factor for γ_M for fundamental load combinations would simply be wrong according to the concept of non-dissipative behaviour".*

In other words the use of partial safety factors for the material properties γ_M for accidental load combinations for dissipative structures can be allowed only if the reference strength takes into account the strength degradation due to cyclic loading. However, for the case of timber structures, the dissipative elements are mechanical connections, whose strength is calculated from Eurocode 5 (EN 1995-1, CEN, 2009) according to Johansen theory. This strength however is calculated for monotonic loading and doesn't consider the degradation due to cyclic loading. Therefore, since the degraded strength due to cyclic loading is generally unknown (unless the structural designer makes reference to available experimental data from which the degraded strength due to cyclic loading could be derived), the partial safety factors for the material properties γ_M for fundamental load combinations should be used, assuming that the ratio between the partial safety factors for the material properties for fundamental load combinations (i.e. γ_M=1.3 according to Eurocode 5) and the partial safety factors for the material properties for accidental load combinations (i.e. γ_M=1.0 according to Eurocode 5) is equal to the ratio between the strength for monotonic loading and the degraded strength due to cyclic loading. For non-dissipative timber structures (e.g. structures with only glued joints), as there's no degradation due to cyclic loading considering the linear-elastic stress/strain relationship for timber structures until collapse is reached, the partial safety factors for the material properties for accidental load combinations (i.e. γ_M=1.0 according to Eurocode 5) could be used. This last consideration however is different from the case of other material properties of Eurocode 8.

A further explanation could be given by making reference to the corresponding section for RC structures, as the above referenced comment make reference to the relevant provisions of all other materials chapters in Eurocode 8, where the corresponding provisions are:

(1)P For ultimate limit state verifications the partial factors for material properties γ_C and γ_s shall take into account the possible strength degradation of the materials due to cyclic deformations.

(2) If more specific data are not available, the values of the partial factors γ_C and γ_s adopted for the persistent and transient design situations should be applied, assuming that due to the local ductility provisions the ratio between the residual strength after degradation and the initial one is roughly equal to the ratio between the γ_M values for accidental and fundamental load combinations.

(3) If the strength degradation is appropriately accounted for in the evaluation of the material properties, the γ_M values adopted for the accidental design situation may be used.

A similar provision is included also, for all structural types and materials, in the current version of the chapter for the seismic design of buildings (Chapter 7) of the Italian Building Code (Norme Tecniche per le Costruzioni, 2008). Therefore, for compliance with the safety verifications of other materials chapters in Eurocode 8, the use of the partial safety factor γ_M for fundamental load combinations was suggested for the verifications in case of dissipative behaviour.

As for provision P(4), the over-strength requirements provided are very few and they do not refer to a specific structural type. Moreover the over-strength factors to be used in the design are not given. Again Becker et al., 1993 provide a further explanation to this provision *"The main difference between the design of steel or concrete and timber structures is the fact, that the ductility is to be placed in the connections and* *not in the members. To ensure the development of the hysteretic mechanisms, which provide the justification of the load reducing by the q-values, the members have to be designed with an overstrength. The value of the overdimensioning (2%, 5%, 10% or 20%) depends on the special application and counts on the knowledge and responsibility of the designer."*

4.7. Control of design and construction

P(1) The provisions given in Part 1-1 and in Eurocode 5 apply.

P(2) In accordance to paragraph 2.2.4.3.(2) of Part 1-1 the following structural elements shall be identified on the design drawings and specifications for their special control during construction shall be provided:

- *anchor-ties and any connections to massive subelements,*
- *diagonal tension steel trusses used for bracing,*
- *connections between horizontal diaphragms and lateral load resisting vertical elements,*
- *connections between sheathing panels and timber framing in horizontal and vertical diaphragms.*

P(3) The special construction control shall refer to the material properties and the accuracy of execution.

This section is completely new with respect to the 1988 version were no specific provisions were given. Again they refer to the structural types given in Figures 4.1a and 4.1b. However, as discussed later on for the current version of the timber chapter in Eurocode 8, a review is needed in order to make reference to other structural systems not included in this chapter.

2.4 The current 2004 edition

The 1995 ENV edition of Eurocode 8 was completely redrafted between 1999 and 2003 and published in the current EN version in 2004. The document was completely re-organized with respect to the ENV version, Part 1-1 (General rules, seismic actions and general requirements for structures), Part 1-2 (General rules for buildings) and Part 1.3 (Specific rules for various materials and elements) were incorporated into a unique Part 1 divided itself into 10 chapters plus 3 annexes, Part 1-4 (Assessment and retrofitting of buildings) became Part 3 and the previous Part 3 (Towers, masts and chimneys) became Part 6. The other parts kept the same order and numbering as before. The new chapter for timber buildings became chapter 8 of Part 1.1.

The parameters defining the elastic response spectrum on which the calculation of the seismic action is based were changed with respect to the 1995 ENV Version, mainly due to a re-definition of the ground types which passed from the 3 of the ENV version to 5, and two different types of elastic response spectrum were introduced depending on the seismic hazard level of the site based on the foreseen surface wave Magnitude M_s.

The seismic design approach remained the same defined in the previous edition of Eurocode 8, i.e. a force-based approach within the Capacity Based Design philosophy. However, some substantial changes were introduced, and the most important one is the minimum value of the behaviour factor q, to be adopted for seismic verification in case of non-dissipative behaviour, which passed for all types of material from 1,0 to 1,5.

The reason for this change is explained in Section 2.2.2 *Ultimate Limit State*, where the application rule (2) reports "*The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterized by the values of the behaviour factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. <u>As a limiting case, for the design of structures classified as low-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may not be taken, in general, as being greater than the value of 1,5 considered to account for overstrengths.</u> For steel or composite steel concrete buildings, this limiting value of the q factor may be taken as being between 1,5 and 2 (see Note 1 of Table 6.1 or Note 1 of Table 7.1, respectively). For dissipative structures the behaviour factor is taken as being greater than these limiting values accounting for the hysteretic energy dissipation that mainly occurs in specifically designed zones, called dissipative zones or critical regions.*" Note that the same period underlined above, in the previous ENV version was the following: "*As a limiting case, for the design of structures classified as non dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor is equal to 1,0.*"

For all the different material chapters, three different Ductility Classes (replacing, also in the definition, the preceding Structural Types) were introduced; (i) Low capacity to dissipate energy – DCL, (ii) Medium Capacity to dissipate energy – DCM and (iii) high capacity to dissipate energy – DCH, and all the different structural types were included in one of the three classes.

The chapter for timber buildings, also due to the re-organization of the entire document, was subjected to substantial changes especially in the definition of the structural types and the corresponding values of the behaviour factor. However, differently from the previous editions, no scientific background was provided about the proposed changes and therefore no research paper or related document is available.

Chapter 2 - The Chapter for Timber Buildings of Eurocode 8. Evolution and critical review

In the following the code text is provided in boxed paragraphs and comments are provided on the relevant parts with reference to recent research results.

8.1 General

8.1.1 Scope

(1)P For the design of timber buildings EN 1995 applies. The following rules are additional to those given in EN 1995.

8.1.2 Definitions

(1)P The following terms are used in this section with the following meanings:

static ductility
ratio between the ultimate deformation and the deformation at the end of elastic behaviour evaluated in quasi-static cyclic tests (see ***8.3(3)****P);*
semi-rigid joints
joints with significant flexibility, the influence of which has to be taken into account in structural analysis in accordance with EN 1995 (e.g. dowel-type joints);
rigid joints
joints with negligible flexibility in accordance with EN 1995 (e.g. glued solid timber joints);
Dowel-type joints
joints with dowel-type mechanical fasteners (nails, staples, screws, dowels, bolts etc.) loaded perpendicular to their axis;
Carpenter joints
joints, where loads are transferred by means of pressure areas and without mechanical fasteners (e.g. skew notch, tenon, half joint).

This part remained identical to the ENV version.

8.1.3 Design concepts

(1)P Earthquake-resistant timber buildings shall be designed in accordance with one of the following concepts:
a) dissipative structural behaviour;
b) low-dissipative structural behaviour.

*(2) In concept a) the capability of parts of the structure (dissipative zones) to resist earthquake actions out of their elastic range is taken into account. When using the design spectrum defined in **3.2.2.5**, the behaviour factor q may be taken as being greater than 1,5. The value of q depends on the ductility class (see **8.3**).*

(3)P Structures designed in accordance with concept a) shall belong to structural ductility classes M or H. A structure belonging to a given ductility class shall meet specific requirements in one or more of the following aspects: structural type, type and rotational ductility capacity of connections.

(4)P Dissipative zones shall be located in joints and connections, whereas the timber members themselves shall be regarded as behaving elastically.

(5) The properties of dissipative zones should be determined by tests either on single joints, on whole structures or on parts thereof in accordance with prEN 12512.

*(6) In concept b) the action effects are calculated on the basis of an elastic global analysis without taking into account non-linear material behaviour. When using the design spectrum defined in **3.2.2.5**, the behaviour factor q should not be taken greater than 1,5. The resistance of the members and connections should be calculated in accordance with EN 1995-1:2004 without any additional requirements. This concept is termed ductility class L (low) and is appropriate only for certain structural types (see Table 8.1).*

Within this part, very similar to the ENV version, there are three major changes:

- A new Principle (3)P is included, according to the new Ductility Classification of Eurocode 8 e similarly to other material chapters, stating that for structures classified in Medium (M) and High (H) ductility class, specific requirements shall be met regarding structural type and ductility capacity of connections (it is not completely clear why “rotational”, maybe the author was referring to dowel-type connections in moment-resisting frames). This principle is correct in order to apply the Capacity Based Design for the different structural systems according to the relevant definition of the behaviour factor q, however these specific requirements are missing for most of the structural types defined later in Table 8.1.
- According to the new provisions provided in section 2.2.2 and discussed above a value of the behaviour factor q=1,5 should be applied in the design of structures designed according to the non-dissipative behavior, differently from the value 1,0 given in the ENV version for the same situation.
- In the application rule (5) ((4) in the ENV version) it is referenced the European Standard prEN 12512 for the cyclic testing of joints made with mechanical fasteners. The reference is made to the prEN version, even if at the time the final EN version of Eurocode 8 was published (2004), the EN version of the 12512 was already published (2001).

8.2 Materials and properties of dissipative zones

(1)P The relevant provisions of EN 1995 apply. With respect to the properties of steel elements, EN 1993 applies.

(2)P When using the concept of dissipative structural behaviour, the following provisions apply:
a) only materials and mechanical fasteners providing appropriate low cycle fatigue behaviour may be used in joints regarded as dissipative zones;
b) glued joints shall be considered as non-dissipative zones;
c) carpenter joints may only be used when they can provide sufficient energy dissipation capacity, without presenting risks of brittle failure in shear or tension perpendicular to the grain. The decision on their use shall be based on appropriate test results.

*(3) **(2)**P a) of this subclause is deemed to be satisfied if **8.3(3)**P is fulfilled.*

*(4) For sheathing-material in shear walls and diaphragms, **(2)**P a) is deemed to be satisfied, if the following conditions are met:*
a) particleboard-panels have a density of at least 650 kg/m^3;
b) plywood-sheathing is at least 9 mm thick;
c) particleboard - and fibreboard-sheathing are at least 13 mm thick.

(5)P Steel material for connections shall conform to the following conditions:
a) all connection elements made of steel shall fulfil the relevant requirements in EN 1993;
*b) The ductility properties of the connections in trusses and between the sheathing material and the timber framing in Ductility Class M or H structures (see (**8.3**)) shall be tested for compliance with **8.3(3)**P by cyclic tests on the relevant combination of the connected parts and fastener.*

This part also remained basically unchanged with respect to the ENV version. In application rule (3) the second sentence of the ENV version "*When tested according to EN XX*[1] *joints shall be verified to have appropriate low cycle fatigue properties under large amplitudes to ensure a sufficient ductility in respect to their intended deformational mechanism and to justify the q value assumed in the analysis (see 4.3.(4)).*" was correctly deleted since it was a repetition of the preceding period.

Moreover, since in the structural types of 8.3 timber trusses with nailed, doweled and bolted joints were included in the Medium and High ductility classes, also the connection of trusses were included in the ductility assessment provision of dissipative zones according to 8.3.(3)P (in the ENV version only connections between sheathing material and timber framing were referenced).

8.3 Ductility classes and behaviour factors

(1)P Depending on their ductile behaviour and energy dissipation capacity under seismic actions, timber buildings shall be assigned to one of the three ductility classes L, M or H as given in Table 8.1, where the corresponding upper limit values of the behaviour factors are also given.

NOTE Geographical limitations on the use of ductility classes M and H may be found in the relevant National Annex.

Table 8.1: Design concept, Structural types and upper limit values of the behaviour factors for the three ductility classes.

Design concept and ductility class	*q*	*Examples of structures*
High capacity to dissipate energy - DCH	*3,0*	*Nailed wall panels with glued diaphragms, connected with nails and bolts; Trusses with nailed joints.*
	4,0	*Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).*
	5,0	*Nailed wall panels with nailed diaphragms, connected with nails and bolts.*
Medium capacity to dissipate energy - DCM	*2,0*	*Glued wall panels with glued diaphragms, connected with nails and bolts; Trusses with doweled and bolted joints; Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill.)*
	2,5	*Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).*
Low capacity to dissipate energy - DCL	*1,5*	*Cantilevers; Beams; Arches with two or three pinned joints; Trusses joined with connectors.*

This is the part that was subjected to major changes with respect to the ENV version, and, being also the most important part of the entire chapter, such modifications had a determinant influence on the seismic design of timber structures. Moreover, since no research paper explaining the scientific background of such important modifications is available, some questions which arise remain unanswered. The following modifications were made:

- The Ductility Classes (referenced as Structural Types in the ENV version) were changed from 4 to 3. Anyway, unlike the ENV version, more than one value of the behaviour factor q was provided for each Ductility Class.

- The structural types were modified with respect to the ENV version and some structural assemblies for building roofs were introduced like trusses with nailed, doweled or bolted joints or with connectors. The reason for this modification is not completely clear since in the ENV version, essentially four types of structures were introduced for buildings: (i) large span glulam roofs with hinged arches, beams or frames, (ii) moment-resisting frames or generally frame structures with rigid or semi-rigid connections or bracings made with timber and/or steel members connected with mechanical fasteners, (iii) buildings with timber frames resisting to horizontal forces and masonry infill (a structural type known as Fackwerk in Germany, Colombage in France and Half-Timbered in UK, no more used for the construction of new buildings) and (iv) Light-Frame buildings with nailed or glued shear walls.

  All these structural systems are lateral load resisting systems used in buildings (even large span glulam roofs, where the timber elements are directly connected to the foundation and resist to vertical and horizontal loads). The trussed frames referenced in the corresponding table of the ENV version are vertical bracing systems for buildings and not roof trusses.

  Just for comparison, Table 2-1 lists the structural types and the seismic force modification factors according to the chapter for seismic of timber structures of the National Building Code of Canada (NBCC 2010), together with the corresponding behaviour factors q according to EC8 (note that the total seismic force modification factor R for each structural type, is given by the product of the ductility related force modification factor R_d and the over-strength related force modification factor $R_{0,}$).

Table 2-1 Definition of structural types and behaviour factors according to NBCC 2010 and corresponding q values according to EC8

Type of Seismic Force Resisting System	Ductility related force modification factor R_d	Overstrength related force modification factor R_0	Seismic force modification factor $R=R_d \cdot R_0$	q value according to EC8
Shear walls				
Nailed shear walls: wood-based panels	3,0	1,7	5,1	3,0/5,0
Shear walls: wood-based and gypsum panels in combination	2,0	1,7	3,4	N/A
Braced or moment-resisting frames in combination				
Moderately ductile	2,0	1,5	3,0	4,0*
Limited ductility	1,5	1,5	2,25	2,5*
Other wood- or gypsum-based SFRS(s) not listed above	1,0	1,0	1,5	1,5**

** Assuming that the structural types "Braced or moment-resisting frames in combination" and "Hyperstatic portal frames with doweled and bolted joints" are corresponding.*

*** In EC8 the number of structural types or Seismic Force Resisting Systems, as they're referenced in NBCC 2010, is much greater, with different q-values. However, assuming that minimum q-value is here reported, assuming that reference is made to structural types not referenced in the code text.*

Therefore, unless the referenced timber trusses are related to large glulam roofs (and in that case the distinction between nailed and dowelled and bolted trusses does not find enough justification as in large glulam roof nailed trusses are almost never used), the inclusion of different types of timber trusses, which are commonly used for building's roofs, as structural types does not seem appropriate.

Moreover the distinction for Light-Frame buildings in terms of energy dissipation capacity between glued and nailed diaphragms does not seem to find an appropriate justification in the research literature, also considering that, in practical design applications, horizontal diaphragms of Light-Frame buildings are almost always modelled as rigid in their plane.

Finally, the structural type "Hyperstatic portal frames with doweled and bolted joints" is mentioned twice, as a High Ductility Class with a q factor of 4.0 and as a Medium Ductility Class with a q factor of 2.5, depending on whether the specific requirements regarding the ductility capacity of connections given in 8.3.3(P) are or not satisfied, as specified in the subsequent Table 8.2. However, the structural system "Nailed wall panels with nailed diaphragms" is mentioned only once in Table 8.1 with the higher behaviour factor q, although the same ductility rule applies also for this system, thus generating possible confusion (Follesa et al., 2011).

- The values of the behaviour factor q for the different structural types corresponding to the relevant Ductility Classes were completely modified with respect to the ENV version and largely incremented for some structural types, even if in the EN version the reference is "upper limit values of behaviour factors" instead of "behaviour factors" given in the ENV version. If the lowest value of the behaviour factor q, changed from 1.0 to 1.5 for the lowest Ductility Class, find a justification in Section 2.2.2 *Ultimate Limit State* of Eurocode 8 as explained above in order to account for system over-strength, the increased value of the behaviour factor q for some structural types (e.g. for "Nailed wall panels with nailed diaphragms" changed from 3 to 5) or the high values given for new structural types (e.g. for "*Hyperstatic portal frames"* where the proposed value for the higher Ductility Class is 4) does not find a proper justification in literature. Such high values for portal frames could be justified only for moment resisting frames with high ductility joints (e.g. densified veneer wood reinforced joints with expanded tube fasteners) as reported in Lejiten, 1998; anyway, even considering this explanation, the description of the structural system should be better detailed. Again the absence of a published paper explaining the reasons for such important changes does not provide any help in understanding the reasons of such important modifications.

- The graphical sketches included in the previous ENV version were deleted, generating if possible more confusion in the understanding of the right structural type to be assumed in the design, thus leading to possible mistakes in the adoption of the correct value of the behaviour factor q. It should be also considered that in other materials chapters, graphical sketches are often included for a better understanding.

- Unlike other material chapters, where for each structural type designed according to a dissipative structural behaviour, a different value is given both for Medium and High Ductility Classes depending on whether the specific requirements regarding the ductility capacity of dissipative zones are fulfilled, in this case a single value is given for all structural types.

(2) If the building is non-regular in elevation (see ***4.2.3.3****) the q-values listed in Table 8.1 should be reduced by 20%, but need not be taken less than q = 1,5 (see* ***4.2.3.1(7)*** *and Table 4.1).*

(3)P In order to ensure that the given values of the behaviour factor may be used, the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class M structures and at a static ductility ratio of 6 for ductility class H structures, without more than a 20% reduction of their resistance.

(4) The provisions of ***(3)****P of this subclause and of* ***8.2(2)*** *a) and* ***8.2(5)*** *b) may be regarded as satisfied in the dissipative zones of all structural types if the following provisions are met:*

a) in doweled, bolted and nailed timber-to-timber and steel-to-timber joints, the minimum thickness of the connected members is 10-d and the fastener-diameter d does not exceed 12 mm;

b) In shear walls and diaphragms, the sheathing material is wood-based with a minimum thickness of 4d, where the nail diameter d does not exceed 3,1 mm.

If the above requirements are not met, but the minimum member thickness of 8d and 3d for case a) and case b), respectively, is assured, reduced upper limit values for the behaviour factor q, as given in Table 8.2, should be used.

Table 8.2: Structural types and reduced upper limits of behaviour factors

Structural types	*Behaviour factor q*
Hyperstatic portal frames with doweled and bolted joints	*2,5*
Nailed wall panels with nailed diaphragms	*4,0*

(5) For structures having different and independent properties in the two horizontal directions, the q factors to be used for the calculation of the seismic action effects in each main direction should correspond to the properties of the structural system in that direction and can be different.

Clauses (2), (3)P and (5) are the same as in the 1995 ENV version. Clause (4) was modified with the introduction of Table 8.2 with the reduced upper value of behaviour factors for the two structural types "Hyperstatic portal frames" and "Nailed wall panels with nailed diaphragms" in case the ductility requirements for dowel-type joints with mechanical fasteners and sheathing to framing connection in Light-Frame walls is not met. Again the scientific background behind these rules is unknown.

8.4 Structural analysis

(1)P In the analysis the slip in the joints of the structure shall be taken into account.

(2)P An E_0-modulus-value for instantaneous loading (10% higher than the short term one) shall be used.

(3) Floor diaphragms may be considered as rigid in the structural model without further verification, if both of the following conditions are met:
a) the detailing rules for horizontal diaphragms given in ***8.5.3*** *are applied;*
and
b) their openings do not significantly affect the overall in-plane rigidity of the floors.

This section remained unchanged with respect to the ENV version.

8.5 Detailing rules

8.5.1 General

(1)P The detailing rules given in ***8.5.2*** *and* ***8.5.3*** *apply for earthquake-resistant parts of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M and H).*

(2)P Structures with dissipative zones shall be designed so that these zones are located mainly in those parts of the structure where yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

8.5.2 Detailing rules for connections

(1)P Compression members and their connections (e.g. carpenter joints), which may fail due to deformations caused by load reversals, shall be designed in such a way that they are prevented from separating and remain in their original position.

(2)P Bolts and dowels shall be tightened and tight fitted in the holes. Large bolts and dowels (d > 16 mm) shall not be used in timber-to-timber and steel-to-timber connections, except in combination with timber connectors.

(3) Dowels, smooth nails and staples should not be used without additional provision against withdrawal.

(4) In the case of tension perpendicular to the grain, additional provisions should be met to avoid splitting (e.g. nailed metal or plywood plates).

In Clause (3) “dowels” were added with respect to the ENV version and the sentence “*They are however admissible in diaphragms for the connection of sheathings to the timber framing (see 4.3.(5) b)) and in secondary members.”* which was present in the ENV version was deleted. However there is no reason to prohibit the use of staples or smooth nails if used in sheathing-to-timber connections in wall and floor diaphragms as there are scientific evidences (Piazza et al. 2013) of the effectiveness of such connections in the ductile seismic performance of Light-Frame buildings. Moreover smooth nail are largely used in sheathing-to-timber connections of Light-Frame shear walls in Canada and USA and there is a vast literature about the good seismic performance of shear walls assemblies connected with smooth nails.

Clause (4) in the ENV version was followed by a Figure 2-showing the different possible solutions in order to avoid brittle failures caused by tension perpendicular to the grain in some type of connections between two perpendicular timber members. The important reference in that Figure 2-was the distance between the outer loaded fastener in the connection and the loaded edge of the horizontal element which should be greater than 2/3 of the height of the cross-section of the horizontal member and not the presence of steel plates (except for the U shaped metal plate). Therefore the reference reported in parenthesis "*e.g. nailed metal or plywood plates"* may lead to an incorrect interpretation of such provision. Anyway, as commented in the corresponding provision of the ENV version, such provisions are already incorporated into Eurocode 5 and maybe a more generic provision about avoiding any possible brittle failure caused by tension perpendicular to the grain or block or plug shear failures in steel-to-timber connections seems more appropriate.

8.5.3 Detailing rules for horizontal diaphragms

(1)P For horizontal diaphragms under seismic actions EN 1995-1-1:2004 applies with the following modifications:

a) the increasing factor 1,2 for resistance of fasteners at sheet edges shall not be used;

b) when the sheets are staggered, the increasing factor of 1,5 for the nail spacing along the discontinuous panel edges shall not be used;

c) the distribution of the shear forces in the diaphragms shall be evaluated by taking into account the in-plan position of the lateral load resisting vertical elements.

(2)P All sheathing edges not meeting on framing members shall be supported on and connected to transverse blocking placed between the wooden beams. Blocking shall also be provided in the horizontal diaphragms above the lateral load resisting vertical elements (e.g. walls).

(3)P The continuity of beams shall be ensured, including the trimmer joists in areas where the diaphragm is disturbed by holes.

(4)P Without intermediate transverse blocking over the full height of the beams, the height-to-width ratio (h/b) of the timber beams should be less than 4.

(5)P If $a_g.S \geqslant 0{,}2g$ *the spacing of fasteners in areas of discontinuity shall be reduced by 25%, but not to less than the minimum spacing given in EN 1995-1:2004.*

(6)P When floors are considered as rigid in plan for structural analysis, there shall be no change of span-direction of the beams over supports, where horizontal forces are transferred to vertical elements (e.g. shear-walls).

Some minor changes were introduced in this section with respect to the ENV version in order to improve the understanding of some provisions. The Figure 2-explaining supporting and nail-spacing at the edges of sheathing panels was deleted.

8.6 Safety verifications

(1)P The strength values of the timber material shall be determined taking into account the k_{mod}-values for instantaneous loading in accordance with EN 1995-1-1:2004.

(2)P For ultimate limit state verifications of structures designed in accordance with the concept of low-dissipative structural behaviour (Ductility class L), the partial factors for material properties γ_M for fundamental load combinations from EN 1995 apply.

(3)P For ultimate limit state verifications of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M or H), the partial factors for material properties γ_M for accidental load combinations from EN 1995 apply.

(4)P In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength.

This overstrength requirement applies especially to:

- *anchor-ties and any connections to massive sub-elements;*
- *connections between horizontal diaphragms and lateral load resisting vertical elements.*

(5) Carpenter joints do not present risks of brittle failure if the verification of the shear stress in accordance with EN 1995 is made with an additional partial factor of 1,3.

Also this part was subjected, like Section 8.3, to substantial changes with respect to the 1995 ENV version. In Clauses (2)P and (3)P the use of the partial safety factors γ_M for fundamental and accidental load combinations for the ultimate limit state verifications in case of dissipative and non-dissipative structural behaviour were inverted. However, as it was discussed in the corresponding part of the ENV version the previous version was proposed for compliance with the same section in other material chapters (e.g. RC structures) and based on the considerations discussed in §2.3, which should be nevertheless applied only to dissipative zones. Maybe a better formulation of (3)P could be "*For ultimate limit state verifications of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M or H), the partial factors for material properties γ_M for fundamental load combinations from EN 1995 apply. If the strength degradation due to cyclic loading is appropriately accounted for in the evaluation of the* *connection resistance, the partial factors for material properties γ_M for accidental load combinations from EN 1995 apply. This provision applies only to the dissipative zones of each structural type according to the capacity based design criteria given in 8.3.3. Other structural elements shall be designed according to 8.6(2)P".*

As for clause (2)P it seem more correct to make reference to the partial factors for material properties γ_M for accidental load combinations as it was stated in the ENV version, since there is no degradation due to cyclic loading considering the linear-elastic stress/strain relationship for timber structures until collapse is reached.

Clause (5) was added with respect to the 1995 ENV version.

8.7 Control of design and construction

(1)P The provisions given in EN 1995 apply.

(2)P The following structural elements shall be identified on the design drawings and specifications for their special control during construction shall be provided:

- *anchor-ties and any connections to foundation elements;*
- *diagonal tension steel trusses used for bracing;*
- *connections between horizontal diaphragms and lateral load resisting vertical elements;*
- *connections between sheathing panels and timber framing in horizontal and vertical diaphragms.*

(3)P The special construction control shall refer to the material properties and the accuracy of execution.

This section remained unchanged with respect to the ENV version.

2.5 A critical analysis of the current version

The current provisions for timber buildings included in Section 8 of EC8 follow the same order for headings, definitions and topics of the Sections related to other building materials. However, unlike RC structures for instance where the different structural types together with the capacity based design and seismic detailing rules are well described and detailed, the corresponding parts for timber structures are too short and, furthermore, they do not cover some new structural systems which are widely used in the construction practice in seismic prone areas.

The revision of the timber Section should cover different aspects in order to include the current state of the art while keeping the same approach of the current version and following the general principles on which the seismic design for other type of materials on EC8 is based. More specifically, there are some clauses which should be improved and some others which deserve further explanations in order to give a correct guidance to the structural designers, as listed in the following (Follesa et al. 2011).

1. The standard does not currently cover some construction systems recently developed and nonetheless already widespread throughout Europe for the construction of multi-storey buildings like the cross-laminated (CLT) technology, and other building systems such as the Log House, also widely used in seismic areas. A more detailed description of each structural type, also including possible graphical sketches, should be provided.
2. Capacity based design rules and detailing provisions for dissipative zones are totally missing for most of the structural types (the few ones only refer to the Light Frame system typically used in North America and not to the prefabricated system generally used in Central and Southern Europe) thus making difficult the choice of the correct value of the behaviour factor to be applied in the design. Like for other building materials, these provisions should be addressed for each structural type. Moreover, even for the Light-Frame system, the existing provisions doesn't help the structural designers to clearly identify the correct hierarchy of resistance of the various structural components in order to guarantee a consistent global ductility of the structure and any analytical expression is suggested for the application of the capacity design approach, therefore these provisions should be better detailed (Casagrande et al., 2014).
3. The values of the behaviour factor currently given for some structural types seem too high and some other too low and some sentences should be corrected in order to avoid confusion or misunderstanding. As an example in Table 8.1 the structural system "Hyperstatic portal frames with doweled and bolted joints" is mentioned twice, as a High Ductility Class with a q factor of 4.0 and as a Medium Ductility Class with a q factor of 2.5, depending on whether the specific requirements regarding the ductility capacity of connections given in 8.3.3(P) are or not satisfied, as specified in the subsequent Table 8.2. However, the structural system "Nailed wall panels with nailed diaphragms" is mentioned only once in Table 8.1 with the higher behaviour factor q, although the same ductility rule applies also for this system, thus generating possible confusion (Follesa et al., 2011).
4. The definition of static ductility should be better clarified or re-defined. According to the current definition, the requirements given for DCM and DCH are not always reached by some structural elements (e.g. nailed wall panels, see Boudaud et al., 2010 and Vogt et al., 2012) and connections for timber structures (e.g. hold-downs and angle brackets in CLT structures, see Gavric et al., 2011).
5. The ductility rules currently given for dissipative zones, i.e. mechanical joints in the case of timber structures, are "prescriptive", e.g. the fastener diameter is limited to 12 mm and the thickness of the timber members to 10 times the fastener diameter for DCH. It would be more appropriate to move to "performance based" design rules, e.g. requiring the attainment of a certain ductile failure mode

according to the Johansen theory as given in Eurocode 5, in accordance with the basic philosophy of Eurocodes.

6. Similarly to other materials such as reinforced concrete and steel structures, the values of the behaviour factor q should be provided, for some structural types such as Light Frame construction and moment resisting frames, for two ductility classes (e.g. Medium and High). Different detailing rules and capacity based design should be given for each ductility class so as to ensure the attainment of the corresponding behaviour factor (Casagrande et al. 2014).
7. New provisions for materials and properties of dissipative zones should be added, including new wood-based materials recently developed such as CLT panels, and the existing provisions regarding wood-based panels used as sheathing material should be updated to incorporate for example oriented strand board (OSB) or new sheathing panels such as Gypsum Fibre Panels (Piazza et al., 2013).
8. The values of the over-strength factors to be used in seismic design to oversize the non-dissipative parts of the structure so as to avoid anticipated brittle failure mechanisms are not provided for the different structural types. These values should be included (Casagrande et al., 2014, Gavric et al. 2014, 2015).
9. Inter-storey drift limits for the Damage Limit State verifications should be provided (Follesa et al., 2011).
10. The partial safety factors γ_M for fundamental and accidental load combinations for the ultimate limit state verifications in case of dissipative and non-dissipative structural behaviour should be changed and made consistent with the relevant provisions of all other materials chapters in Eurocode 8.

Other provisions which are related to additional aspects not covered at present and which should possibly be included in the next generation of the timber chapter of Eurocode 8 are listed in the following.

1. Additional provisions should be given in the Structural Analysis paragraph regarding: (i) the different analysis methods to be adopted in the design; (ii) the need to consider the flexibility and, for certain types of analyses, the nonlinear and hysteretic behaviour of mechanical joints in FEM models in order to schematize the correct horizontal stiffness of the whole structure; and (iii) specific provisions for non-linear static and dynamic analysis methods (Fragiacomo et al. 2011, Follesa et al. 2013).
2. Rules for the design of the floor diaphragm should be provided, both for heavy (CLT and glulam) and lightweight construction, with special emphasis on detailing and force transmission around holes. However these rules should be provided only after the corresponding provisions for the design of CLT and glulam floor diaphragms for fundamental load combinations will be included within Eurocode 5.
3. Some specific guidance should be given about the analysis methods to be used for the seismic design of mixed structures, some of which are already partly used in current design practice, such as buildings with mixed CLT/Light Frame bracing systems or timber buildings with a concrete stair/lift shaft. This part should be developed in co-operation with the Section of EC8 on reinforced concrete.

4. General provisions should be given for the assessment of the seismic performance of new structural types which could be developed in future and are not yet covered by the current standard.

All these aspects will be analysed in detail in the following chapters and a new proposal about all the above mentioned topics will be included and discussed in the new Background Document proposed in Chapter 5.

References – Chapter 2

Becker K., Ceccotti A., Charlier H., Katsaragakis E., Larsen H.J, Zeitter H. [1993] "Eurocode 8 – Part 1.3 – Chapter 5 – Specific rules for timber buildings in seismic regions", 26th CIB W18, Athens, Georgia, paper 26-15-2.

Boudaud C., Haumeury S., Faye C., Daudeville L. [2010] "European seismic design of shear walls: experimental and numerical tests and observations", Proceedings of the 11th World Conference on Timber Engineering, Riva del Garda (TN), Italy.

Buchanan A.H., Dean J.A. [1988] "Practical design of timber structures to resist earthquakes". Proceedings of the 1988 International Timber Engineering Conference, Seattle, WA, 813-822.

Casagrande D., Sartori T., Tomasi R. [2014] "Capacity design approach for multi-storey timber-frame buildings." Proc., The International Network on Timber Engineering Research INTER, Bath, United Kingdom, paper 47-15-3.

Ceccotti, A. [1995] "Timber connections under seismic actions". In: Timber engineering–STEP 1. 1st Edition. STEP/EUROFORTECH. The Netherlands, ISBN 90-5645-001-08. Pp. C17/1-C17/10.

Ceccotti A., Larsen H.J., [1988] "Background document for specific rules for timber structures in Eurocode 8". Report EUR 12266 EN for the Commission of the European Communities, Brussels, Belgium.

Ceccotti A., Vignoli A. [1985] "Full-scale structures in glue-laminated timber. Dynamic tests: theoretical and experimental studies". 18th CIB-W18 Meeting, Beit Oren, Israel.

Ceccotti A., Vignoli A. [1987] "Behaviour factors for timber structures in seismic zones". 20th CIB-W18 Meeting, Dublin, Ireland.

Ceccotti A., Vignoli A. [1988] "The effect of seismic events on the behaviour of semi-rigid joints in timber structures". Proceedings of the 1988 International Timber Engineering Conference, Seattle, WA.

Commission of the European Communities – Industrial Processes – Building and Civil Engineering [1988] Eurocode 8—Structures in Seismic Regions – Design - Part 1: General and buildings, May 1988 Edition – Report EUR 12266 EN, Directorate-General Telecommunications, Information Industries and Innovation L-2920 Luxembourg.

Dean J. A., Stewart W. G., Carr A. J. [1986] "The seismic behaviour of plywood sheathed shear walls", The New Zealand Journal of timber construction, Vol. 2, n. 3.

Decreto Ministeriale Ministero Infrastrutture e Trasporti del 14/01/08 "Norme tecniche per le costruzioni" [2008].

European Committee for Standardization (CEN), [2001] - EN 12512: Timber structures- Test methods. Cyclic testing of joints made with mechanical fasteners.

European Committee for Standardization (CEN), [1998] – European Prestandard: Eurocode 8 - Design provisions for earthquake resistance of structures - Part 1-3: General rules - Specific rules for various materials and elements, Brussels, Belgium.

European Committee for Standardization (CEN), [2000] – Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings - Draft No 1, Brussels, Belgium.

European Committee for Standardization (CEN), [2002] – Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings - Draft No 5 -Revised Final Project Team Draft (preStage 49), Brussels, Belgium.

European Committee for Standardization (CEN), [2004] Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, Brussels, Belgium.

Follesa M., Fragiacomo M., Lauriola M.P. [2011] "A proposal for revision of the current timber part (Section 8) of Eurocode 8 Part 1." Proc., the 44th CIB W18 Meeting, Alghero, Italy, paper n.44-15-1.

Follesa M., Christovasilis I., Vassallo D., Fragiacomo M., Ceccotti A. [2013] "Seismic design of multi-storey CLT buildings according to Eurocode 8." Ingegneria Sismica, Special Issue on Timber Structures, n. 04/2013, pp. 27-53.

Gavric I., Fragiacomo M., Ceccotti A. [2011] "Experimental cyclic tests on cross-laminated timber panels and typical connections", Proceedings of the 14th ANIDIS Conference, Bari (Italy), September 18th-22nd 2011.

Gavric I., Fragiacomo M., Ceccotti A. [2014] "Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures." RILEM Materials and Structures, published online, doi: 10.1617/s11527-014-0278-7.

Gavric I., Fragiacomo M., Ceccotti A. [2015] "Cyclic behaviour of typical screwed connections for cross-laminated (CLT) structures." European Journal of Wood and Wood Products, in print.

ISO 16670:2003: "Timber structures -- Joints made with mechanical fasteners -- Quasi-static reversed-cyclic test method".

Lejiten A.J.M. [1998] "Densified veneer wood reinforced timber joints with expanded tube fasteners: the development of a new joint." Ph.D. Thesis, Delft University Press, the Netherlands.

Macchi G. [1983] "Costruzioni in muratura" Ingegneria Sismica, Bologna.

Munoz W., Mohammad M., Salenikovich A., Quenneville P. [2008]: "Determination of yield point and ductility of timber assemblies: in search for a harmonised approach". Engineered Wood Products Association.

National Research Council Canada, [2010] "National Building Code of Canada 2010".

Patton-Mallory M., Gutkowski R. M., Soltis L. A. [1984] "Racking performance of Light-Frame walls sheathed on two sides", United States Department of Agriculture, Forest Services, Forest Products Laboratory, Research Paper FPL 448.

Piazza M., Tomasi R., Grossi P., Campos Costa A., Candeias P. X. [2013] "Seismic performance of multi-storey timber buildings - RubnerHaus building – Final Report" - Seismic Engineering Research Infrastructures For European Synergies (SERIES), Work package [WP9 – TA5 LNEC].

Pozzati P. [1983] "Introduzione" in Fondamenti di Ingegneria Sismica, Bologna, Italy.

Sartori T., Tomasi R. [2013] "Experimental investigation on sheathing-to-framing connections in wood shear walls" ENGINEERING STRUCTURES, v. 56, (2013), p. 2197-2205.

Soltis L.A., Patton-Mallory M. [1986] "Strength and ductility of sheathed walls". Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Portugal.

Tomasi R., Piazza M. [2013] "Investigation of seismic performance of multi-storey timber buildings within the framework of the SERIES Project." Proceedings of the International Conference on Structure and Architecture ICSA2013, Guimaraes, Portugal.

Vogt T, Hummel J., Seim W. [2012] "Timber framed wall elements under cyclic loading", Proceedings of the 12th World Conference on Timber Engineering, Auckland, New Zealand.

Yasumura, M., Nishiyama, I., Murota, T., Yamaguchi, N. [1988]. "Experiments on a three-storied wooden frame building subjected to horizontal load". Proceedings of International Conference on Timber Engineering (pp. 19-22), 1988.

3 – Numerical modelling of CLT and Light-Frame Buildings

3.1 General considerations and analysis methods of Eurocode 8

The two main structural systems widely used nowadays in Italy and Europe for the construction of multi-storey timber buildings are the Light-Frame system (known also as Platform-Frame system) which is well-known and widely used in North America, Asia, Australasia, and also in Europe, and the Cross-Laminated building system (CLT), which despite its relatively recent development, is already regarded as the main structural system used in Europe for the construction of mid-rise multi-story buildings, with a large number of significant buildings up to 9 stories recently built in different seismic and non-seismic regions of Europe (see §1.1).

However, also due to the lack of guidance and design rules in building codes and literature, the numerical modelling of a wooden building is one of the main concerns for structural engineers, especially when performing seismic design, as a not correct modelling approach may lead to non-conservative results with possible catastrophic consequences.

Different methods may be applied according to the type of analysis used in the design, the type of structural system and, of course, depending on the features of the software package used in the analysis. A correct approach should anyway consider the following aspects:

- construction features of the building system,
- capacity design rules to be applied in the seismic design,

and, especially for non-linear analysis,

- non-linear properties of the structural components devoted to the ductile behaviour and energy dissipation, i.e. mechanical connections.

Eurocode 8, Part 1 (CEN, 2004), propose four possible methods for the seismic analysis of timber buildings (Follesa et al., 2013):

a) the lateral force method of analysis, i.e. the linear static analysis;

b) the modal response spectrum analysis, i.e. the linear dynamic analysis;

c) the non-linear static analysis, i.e. the pushover analysis.

d) the non-linear dynamic analysis, i.e. the time-history analysis.

According to Eurocode 8 (CEN, 2004) the reference method to be used in the evaluation of the seismic effects is method b). However, if the building meet the regularity criteria in elevation given in §4.2.2.3 and the natural period is lower than the minimum value between four times the period defining the end of the horizontal segment defining the peak value of the elastic design spectrum and 2,0 sec, method a) could be applied. Methods a) and b) are both based on a linear elastic analysis, with the design action calculated according to the design spectrum defined in §3.2.2.5 of Eurocode 8, and the energy dissipation capacity of the building is implicitly taken into account by dividing the forces obtained from the linear elastic analysis by the behaviour factor q, which is associated with the relevant type of structure and ductility class, as defined in Table 8.1 of Eurocode 8 (Follesa et al., 2013).

However, since Table 8.1 is not covering all the structural types currently used in the current construction practice and since the capacity design rules and the definition of the overstrength factors, on which the correct application of the behaviour factor q is based, are often missing for most of the structural types, as discussed in Chapter 2, the application of these two methods often relies, for the case of timber buildings, on the decisions taken by the structural designer.

Method a) is usually followed for low-rise and simple buildings which meet the regularity criteria in elevation and according to the definition of Eurocode 8 "…*whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction*". The horizontal forces may be calculated with an inverted triangular distribution based on the mass and the height of each storey.

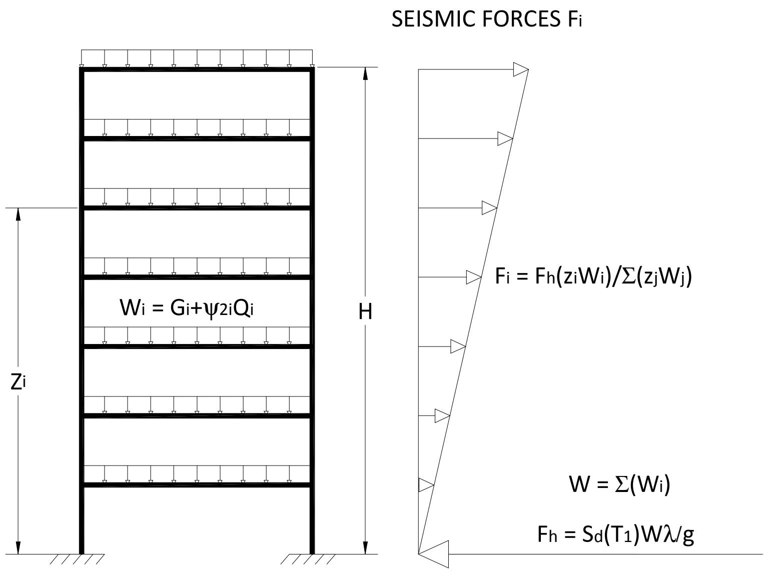


Figure 3-1 Calculation of the seismic design forces acting at each story according to the lateral force method of analysis of Eurocode 8 (CEN, 2004).

The natural vibration period may be calculated directly from the formula 4.6 of Eurocode 8:

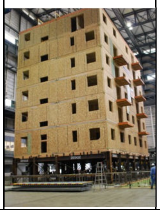
$$T_1 = C_T \cdot h^{3/4} \tag{1}$$

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

The same formula for the calculation of the natural vibration period for Light-Frame buildings and other structures is proposed by the National Building Code of Canada 2010 (NRC, 2010) with the limitation that, if the natural period is calculated by means of a structural model, should not be greater than two times the value calculated with formula (1).

Just for reference in Table 3-1 is presented a comparison between the natural vibration period calculated with the code formula (1) and the measured value obtained from some experimental tests referenced in §1.3.1.

Table 3-1 Comparison between code formula and experimental results for the natural vibration period of different structural systems.

	3-story CLT building SOFIE Project – Conf. A	3-story CLT building SOFIE Project – Conf. C	7-story CLT building SOFIE Project	6-story Light-Frame building NEESWood Project	2-story Log-House building – SERIES Project	3-story Light-Frame building – SERIES Project	3-story CLT building – SERIES Project
							
Code	0.28 s	0.28 s	0.53 s	0.42 s	0.17 s	0.23 s	0.23 s
Experimental	0.17 s	0.19 s	0.43 s	0.41 s	0.19 s	0.28 s	0.25 s
% Difference	-39%	-32%	-19%	-2%	12%	22%	9%

The experimental periods are of course referred to the shorter or anyway weaker horizontal direction of the building. From Table 3-1 no particular trend can be extrapolated, as the natural period is related to (i) the seismic weight, (ii) the lateral stiffness of the building which is reliant on the structural type and the mechanical properties of the connections and (iii) the friction, which are not accounted in the code formula where only referred to the building's height is made. However the comparison shows nevertheless an acceptable correlation between experimental results and code formula. The formula seems to provide better approximation for the more deformable systems (Light-Frame and Log-house) and slightly worse approximation for the most rigid system (CLT).

Most of the times however, in practical design cases, the building does not meet the criteria for regularity in elevation, and therefore the modal response spectrum analysis (Method b) should be used (Follesa et al., 2013).

In this case a proper evaluation of the natural vibration period of the building is crucial, as it may lead to non-conservative design if underestimated or overestimated, depending on the type and size of structure and on the shape of the design spectrum (Sustersic and Dujic, 2012). Moreover, the correct estimation of the lateral stiffness and therefore, for the case of timber structures, of the stiffness properties of the mechanical joints used in the design, is also important to estimate accurately the displacements of the building for the verification at the SLS and ULS. This is essential when the building is adjacent to other structures dynamically independent divided by a seismic separation (Follesa et al., 2013).

Non-linear methods, like method c) and d) require the knowledge of the non-linear monotonic (in case of static analysis) or cyclic (in case of time-history analysis) behaviour of the dissipative zones, which for the case of timber structures are mechanical joints, and the availability of a suitable finite element software, which includes a proper non-linear monotonic or hysteretic model capable of accurately simulating the actual non-linear response of the mechanical fasteners.

The design approach is therefore more complex than for methods a) and b), even because designers usually do not have access to experimental data that can be used to calibrate the non-linear behaviour of the mechanical fasteners. Current products certification (ETA, CE marking based on product standards) for connections and fasteners does not include any reference about the non-linear properties of such elements. Moreover, even when experimental data is available, it may refer to connections with different types, number and diameter of fasteners from those used in the actual design (Follesa et al., 2013).

However, in most cases, a reliable design of a timber building, either with linear or non-linear analysis could be done by formulating a proper finite element model of the structure with most of the commercially available software, provided that the linear or non-linear properties of the actual connections used in the design are correctly modelled. The most used software package by consulting engineers for the seismic design of structure is SAP2000 (CSI, 2000), which was proved to be suitable for the linear (Fragiacomo et al., 2011, Follesa et al., 2013) and non-linear (Fragiacomo and Rinaldin 2011, Rinaldin et al., 2013) analysis of multi-storey timber buildings built with different structural types.

In this Chapter information and details for the linear and non-linear analysis modelling methods of CLT and Light-Frame multi-storey with SAP2000 (CSI, 2000) will be provided.

3.2 Seismic design of CLT buildings

Cross Laminated Timber (CLT) buildings are structures in which walls and floors are composed of cross laminated timber panels, i.e. solid timber panels, connected together and to the basement by means of mechanical joints made with different types of steel plates and angle brackets, annular ringed nails and self-tapping screws. With this technique buildings with different type of use (schools, residential, office and commercial buildings), number of stories (currently buildings up to 10 stories have been built, and a 14 story-building is under construction) and architectural design can be built.

Figure 3-2 Internal view of the structure of a 3-story residential CLT building (left), completed 4-story CLT residential building (center) and completed CLT school (right).

3.2.1 CLT production

CLT panels are a plate-like product composed of several layers (from a minimum of 3 layers up to a maximum of 11) of timber boards assembled together and glued crosswise in order to form massive wooden panels which are used for walls and floors. The laminations may be composed of one or two layers of parallel timber boards.

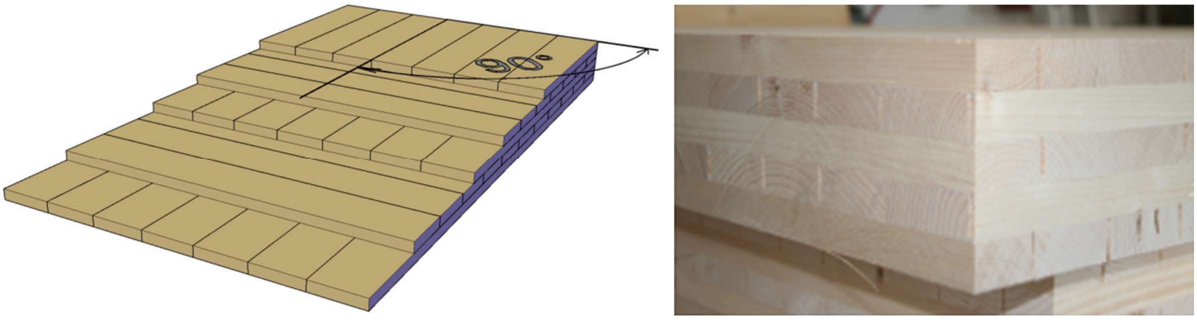


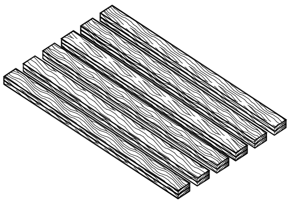
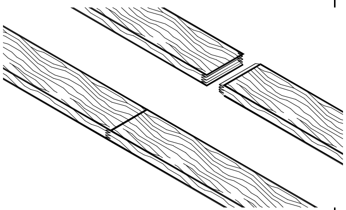
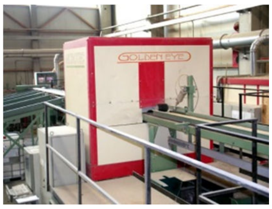
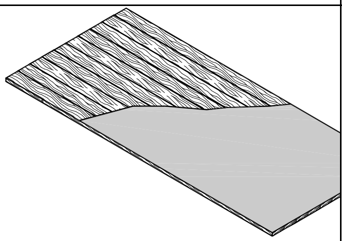
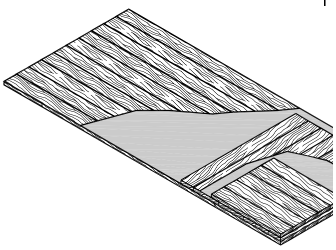
Figure 3-3 CLT panel layup.

The production process starts from the selection and grading of timber boards of different species (generally Spruce, Pine or Douglas Fir) of thicknesses varying from 17 to 40 mm and width from 150 to 250 mm which are machine dried up to 10-12 % of moisture content and planed.

The timber boards are then finger-jointed in length, assembled in layers and cross-glued with polyurethane glue or melamine-urea-formaldehyde glue.

The layers are then assembled in big size master panels up to a dimension of 4.80x20m and then cold pressed by applying a pressure of 0.6-1.0 MPa either with vacuum or mechanical systems. Finally, the panels are cut using CNC machinery to the required sizes and delivered to the building site.

In Figure 3-4 the entire production process is illustrated.

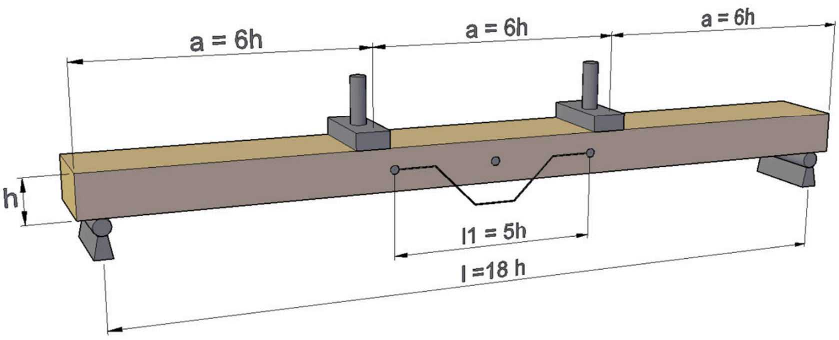
A - selection and drying of timber boards		
B – grading and finger jointing in length of timber boards		 
C – assembling and glue casting over the first layer		 
D – assembling and glue casting over the subsequent layers		 
E – vacuum pressure of the completed master panel		
E (alternatively) – mechanical pressure of the completed master panel		

F CNC machine cutting of completed panels	
G Loading and transportation of the cut panels to the building site	

Figure 3-4 Production process of CLT panels.

The qualification process of the structural panels is made by CE marking each panel according to European Technical Approvals (ETA) which are specific of each single producer and are generally based on the procedure described in CUAP 03.04/06. According to this procedure, in order to obtain the ETA a certain number of tests are necessary, both to verify the bond strength by means of bending tests on finger joints and delamination tests, and the mechanical properties of the entire panel composed of timber boards strength graded according to the strength classes of the European Standard for solid timber (EN 338).

Mechanical tests consist of bending and shear test both in- and out-of-plane.



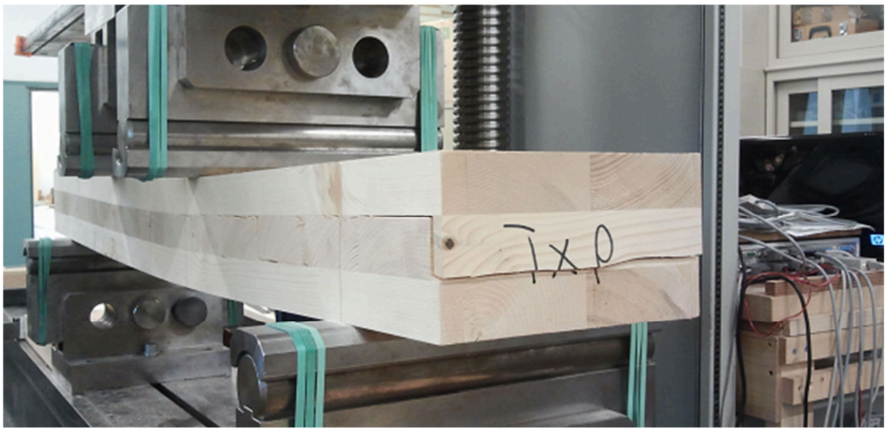


Figure 3-5 Out-of-plane bending test of the CLT panel. Test set-up and execution.

Once the tests are completed it is possible to obtain the mechanical properties of the panel both in- and out-of-plane which could then be used in the design of CLT panels.

At present a European Standard for CLT (EN 16351 Timber structures — Cross Laminated Timber — Requirements) is under formal vote and will be published soon. After the publication, the CE marking of CLT panels will be possible only according to the procedure established in this standard and all the single ETA's will expire.

3.2.2 Construction process of a CLT building

The construction process of a CLT building starts from a concrete basement or from concrete foundation beams over which the 1st storey walls are placed and connected.

The connection to the foundation is usually made with two types of connections: hold-down anchors placed at wall ends and at opening ends in order to restrain the walls against uplift, and steel brackets placed uniformly along the length of the wall as sliding restraint connections. Both hold-down anchors and steel brackets are connected by means of anchor bolts to the foundation and by means of annular ringed shanks nails to the CLT walls.

Figure 3-6 Connection of a CLT panel to the foundation with hold-down anchors and steel brackets.

The connections of the ground floor walls to the foundation must have a dual function as shown in Figure 3-7: to resist overturning and sliding of the walls caused by the horizontal actions (wind or earthquake) acting in the plane of the wall .

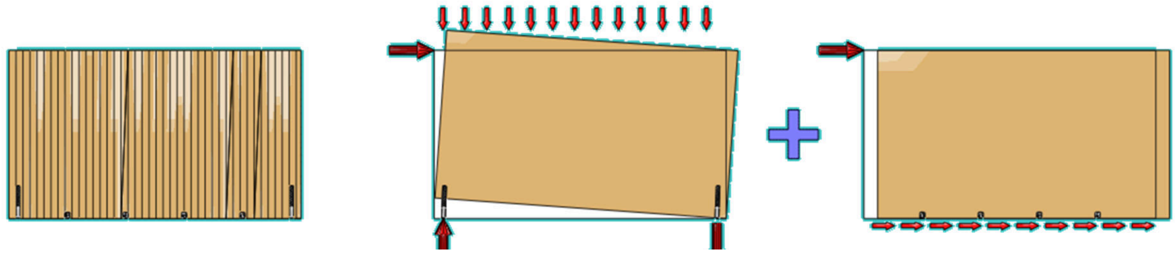


Figure 3-7 Effects of the seismic forces acting on a wall and different function of the connection elements (after Follesa et al., 2013).

At the inter-story levels the hold-down connection is usually made with (i) nailed steel plates connecting the upper and lower wall for external walls, and with (ii) a double hold-down connection, fixed to the upper and lower wall and with an anchor bolt passing through the floor panel to connect the two hold-downs for interior walls (Figure 3-8).

Figure 3-8 Inter-storey hold-down connection for inner walls (Left - in this case two hold-downs below connected to the corresponding two upper hold-downs not visible in this picture) and for external walls (Right) with two nailed steel plates at opening ends.

The erection process is a platform type of construction with walls of height equal to the inter-storey height which can be either made of a unique element up to the maximum transportable length (usually not more than 16 m) or may be composed of more than one panel. The last option is often done for transportation reasons: the panels have a width usually not greater than 2.5 m, and are connected together by means of step joints made with mechanical fasteners (screws or nails). Walls in perpendicular directions shall be connected by means of mechanical fasteners (usually screws).

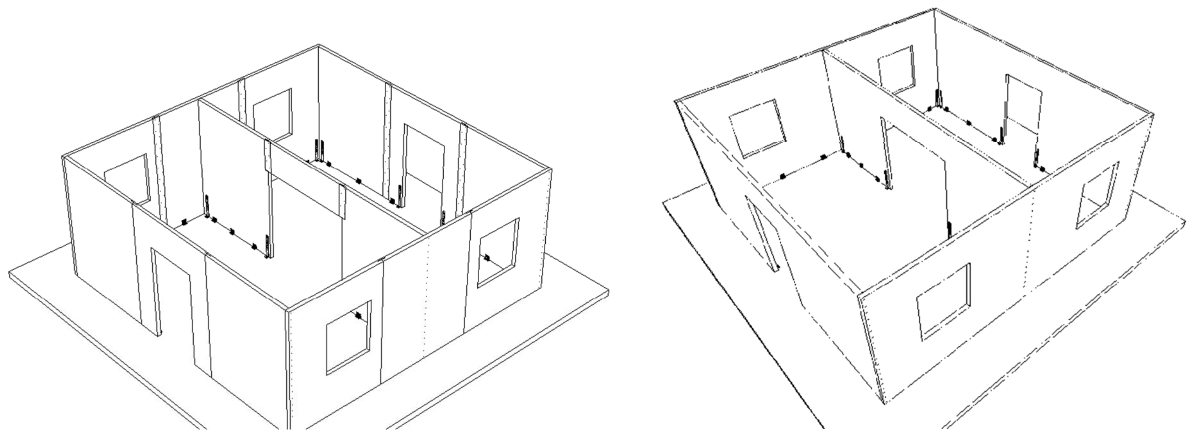


Figure 3-9 CLT walls composed of several panels connected by means of vertical joints (left) and CLT walls made of a single panel in which openings for doors and windows are pre-cut in the production factory (right).

The connections between wall panels are usually done with the interposition of a wood-based multi-layer panel (e.g. plywood or cross-laminated LVL) that can be inserted in grooves cut inside the wall or on one of its faces. Sometimes the same connection is made with a half-butt joint. The connection is always made with self-tapping screws with a diameter varying from 6 to 10 mm, or nails of 3 mm in diameter and distance depending on the seismic loads. Figure 3-10 illustrates some of the typical types of step joints used in CLT construction.

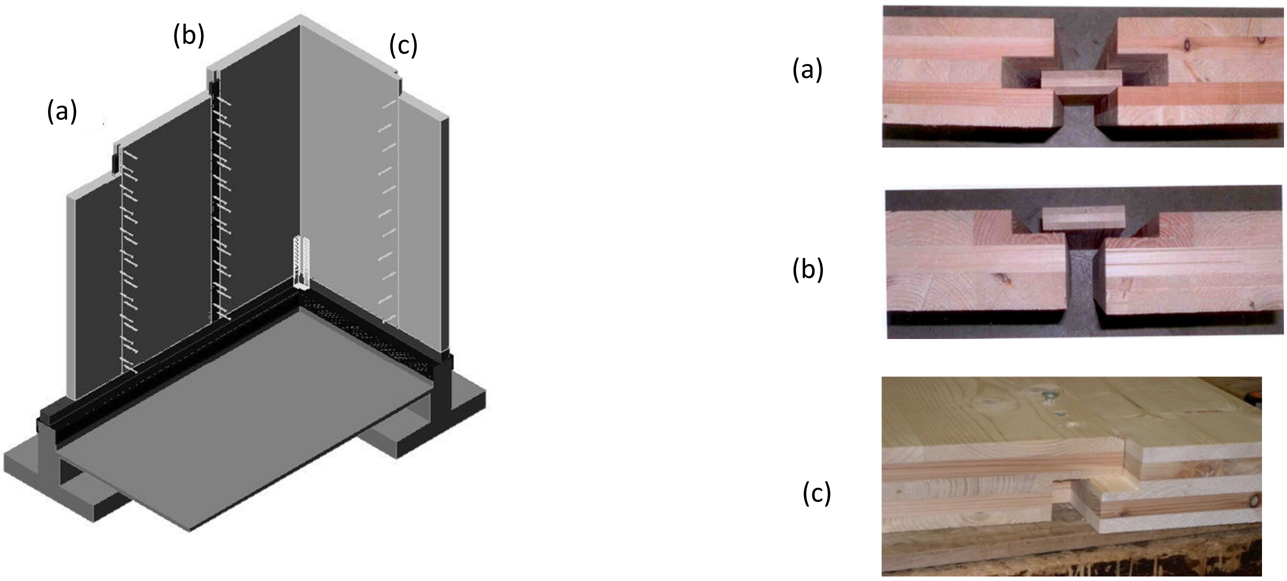


Figure 3-10 Three different step joints between vertical wall panels; (a) a cross-layer panel inserted into internal grooves, (b) a cross-layer panel inserted in grooves on the internal side of the wall, and (c) a half-lap joint (after Follesa et al. 2009).

Horizontal diaphragms are usually made of CLT timber panels connected together by means of horizontal joints made with mechanical fasteners (screws or nails) of the same type of the vertical step joints or with glulam beams and wood-based panels, i.e. thinner CLT panels or OSB or plywood panels. The floor panels are connected to the lower wall panels by means of mechanical fasteners (usually screws). Once the floor construction is completed, the construction continues with the same process, with the upper walls placed over the floor panels, and connected to the lower walls using mechanical joints similar to those used for the wall-foundation connection (Figure 3-11).

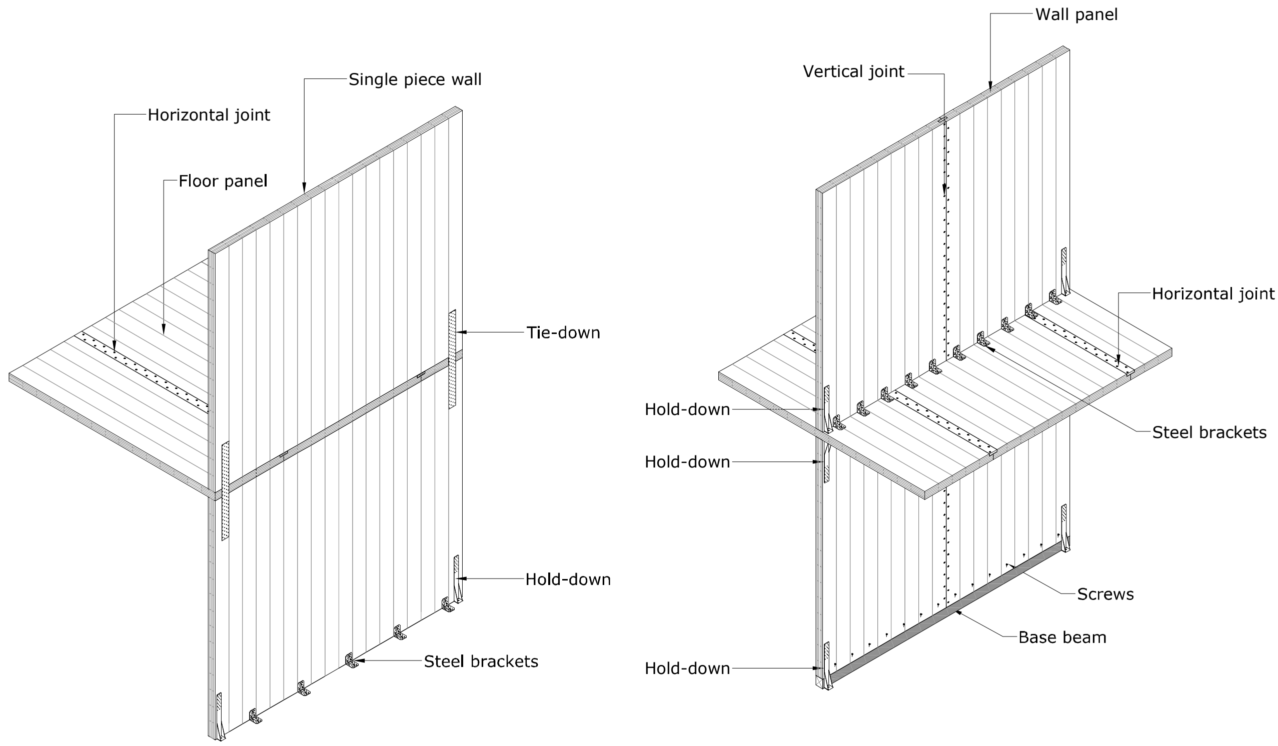


Figure 3-11 Walls and floors in platform type CLT buildings (after Follesa et al. 2011).

The roof construction is made usually in the same way using again CLT panels but of course it may be done following other more traditional construction practices like for example the case of roofs made with ridge beams, rafters and purlins.

Starting from the inside, the wall construction is completed usually with the following materials:

- a single or double layer of gypsum wall board;
- a gap usually of 40-50 mm for the electric installations usually filled with soft insulation;
- the CLT wall;
- an external single or double layer of insulation, typically wood fibre or rock wool;
- the exterior finishing which may be plaster, wood planks or covering panels.

The floor construction is completed, starting from below, with the following materials:

- a single layer of thick gypsum wall board (unless the CLT floor panel is left uncovered);
- a gap of 25 mm for the electric installation given by the steel supporting structure of the gypsum panels;
- the CLT floor panel;
- a layer of dry filling (sand or other materials) for acoustic insulation;
- a single or double light concrete slab of 50-100 mm of total thickness which contains all the fittings and eventually an underfloor heating;

- the flooring.

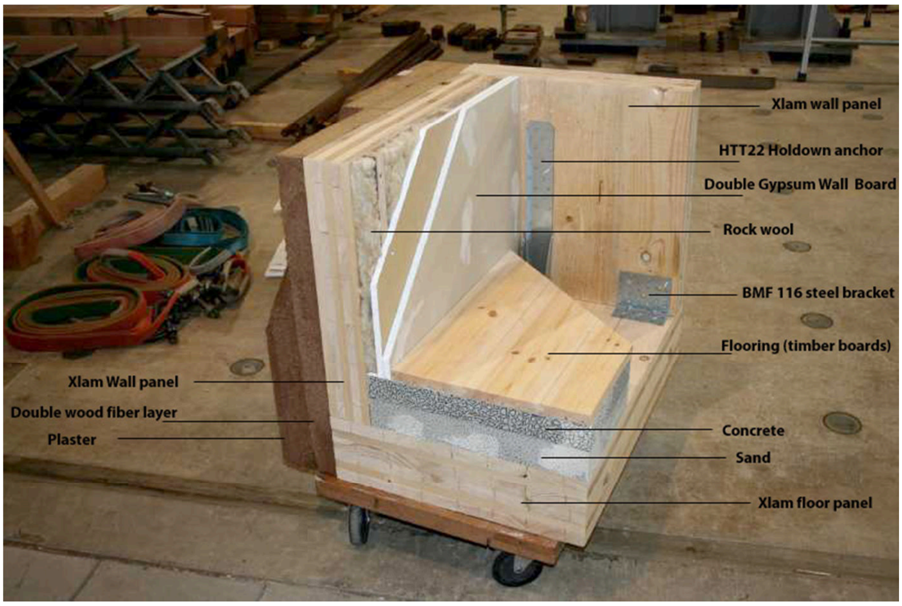


Figure 3-12 Typical wall and floor connection (detail).

3.2.3 Seismic design of CLT buildings

Unlike for other more “traditional” materials such as reinforced concrete and masonry, the structural design of CLT buildings is a highly detailed process, which should properly consider: (i) the design of each single structural panel, (ii) the specifications (such as different size of master panels and different thicknesses and layer configurations) of the CLT producer, and (iii) the specifications of the construction company (e.g. walls made by a single panel or by different panels connected together with vertical joints according to the customer specifications and/or site conditions and accessibility). For a proper design of a CLT building, it is required the knowledge of: (i) the panel geometrical properties with all the pre-cuts for openings, joints and beams or lintels support, (ii) the layout of connections, and (iii) a 3D-model of the whole building (Figure 3-13).

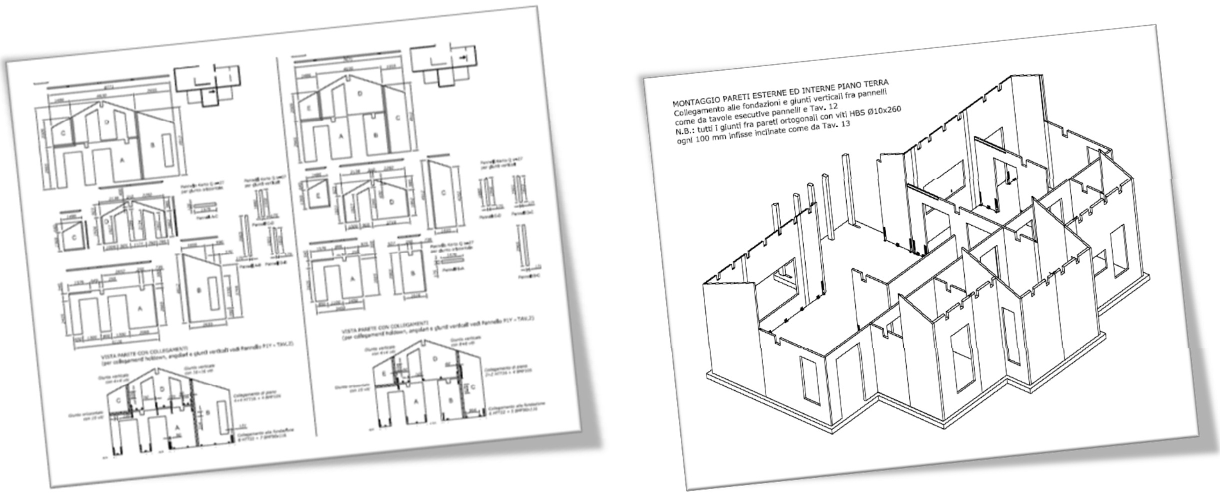


Figure 3-13 Example of detailed drawings of a CLT building.

It is very important that this first stage of the design process is decided by the structural engineer and not, e.g., by the construction company in relation to the speed or ease of assembly. The panel layout influences the layout of connections and, thus, the structural behaviour under horizontal loading and finally the numerical model of the whole building.

As an example Figure 3-14 shows four possible different configurations of panels and connections for the same CLT wall. In case A the wall is made with a single CLT panel inside which the two windows openings are cut. Two hold-downs are placed at the two ends of the wall and angle brackets are uniformly distributed along the wall length. The wall could be modelled as monolithic using shell elements (see §3.2.5 for further details), whose stiffness may be calculated by considering the equivalent stiffness of the CLT panel depending on the layer configuration and the timber grade, and possibly a reduction due to the presence of the openings. In case B the wall is composed of four CLT panels connected with step joints (which e.g. could be made in one of the three ways showed in Figure 3-10), the connection to the foundation is made in the same way as for case A and the two window openings are cut inside two wall panels. In this case the equivalent stiffness of the wall composed by multiple wall segments with mechanical joints will be lower than for case A and could be considered in the numerical model either modelling the wall as a single monolithic element whose stiffness should be calculated by taking into account also the stiffness of the vertical step joints, or could be modelled as four monolithic elements connected by means of equivalent spring or truss elements simulating the vertical step joints, and possibly reducing the stiffness of the panels with the window openings. In case C the CLT wall is divided into three wall segments by the two window openings. Windows are made with an upper CLT or glulam lintel which is supported by two notches cut in the panels and connected to the wall segments and a lower CLT parapet connected e.g. with self-drilling screws to the adjacent wall segments. Two out of three wall segments are composed of two CLT panels connected with vertical step joints. Hold-downs are placed at the ends of each wall segment and angle brackets are distributed along the length of each wall segment. In this case, the wall behaviour will be completely different from case A and B, as in the numerical model the wall will be composed by three monolithic elements schematizing the corresponding wall segments or by five monolithic elements schematizing the actual CLT panels and modelling the vertical step joints as described above. Lintels may be schematized as beam elements with hinged connections. Finally, case D corresponds to case C but in this case the three wall segments are made of a single CLT panels without vertical step joints. Hold-downs and angle brackets are placed in the same way as for case C. Again, the three wall segments could be modelled as three monolithic elements using shell elements and the opening lintels as simply supported hinged beams.

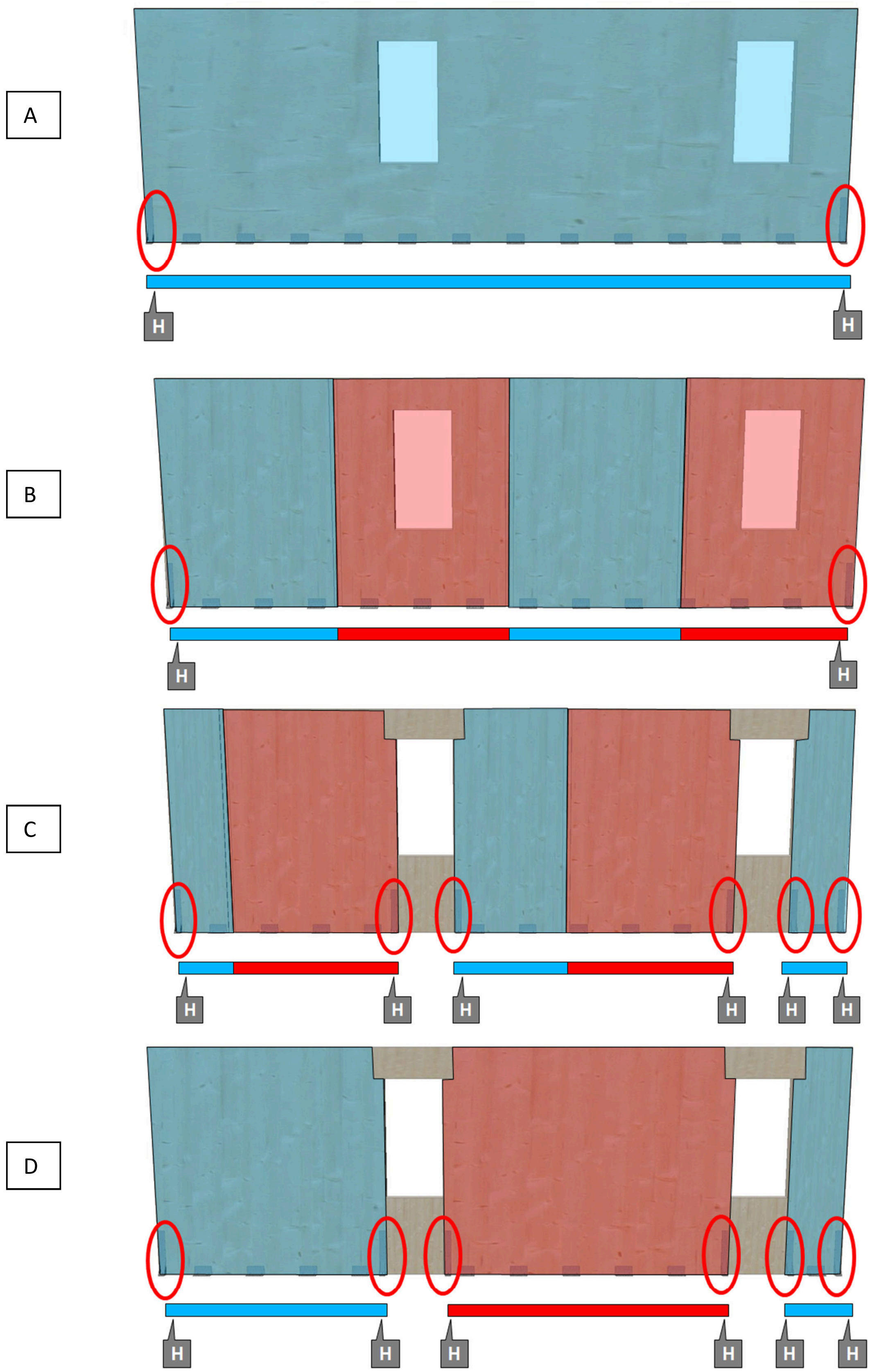


Figure 3-14 Four possible choices for panels and connections layout in a CLT wall with openings: A – Single CLT panel in which openings are cut inside, with hold-downs at wall ends and angle brackets distributed along the wall length. B –CLT wall composed by several panels connected with step joints and base connection like in A. Wall openings are cut inside two wall panels. C – Wall divided into three segments by the two window openings, with two wall segments composed by two panels connected with step joints and the third one by a single panel. Hold-downs are placed at the ends of each wall segment and

angle brackets are distributed along the length of each wall segment. D – Like case C but with wall segments composed by a single CLT panel.

Considering the seismic behaviour, a CLT building may be schematized as a box-type structure in which vertical and horizontal diaphragms are composed of cross-laminated panels with high in-plane stiffness and strength connected together by mechanical fasteners (Follesa et al., 2013).

At each level the story shear is distributed to the walls underneath proportionally to their stiffness considering, as prescribed by Eurocode 8 (CEN, 2004), an accidental eccentricity of +/- 5% of the largest dimension of the building plan measured perpendicularly to the direction of the seismic action in addition to any eccentricity between center of mass and center of stiffness in case of asymmetric distribution of stiffness.

Tests performed on CLT panels loaded by in-plane horizontal loads showed that a CLT wall deformation can be divided into four components: (i) rocking, (ii) sliding, (iii) bending deformation of the panel and (iv) shear deformation of the panel. However, being the CLT wall panels very rigid, studies conducted so far demonstrated that only the rocking and sliding component are significant, while the other two components are generally negligible, especially in the case of wall panels without openings (Figure 3-15).

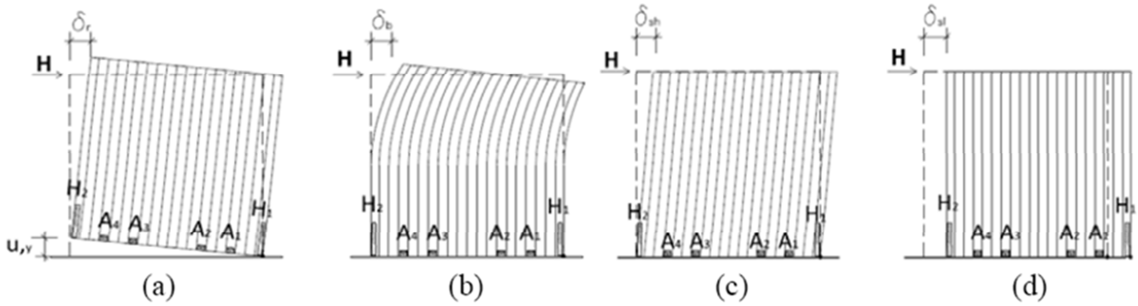


Figure 3-15 Deflection components of a CLT panel: (a) panel rocking, (b) bending, (c) shear deformation, (d) slip of base connections (after Gavric et al., 2011).

Thus, for practical purposes, CLT panels may be schematized with shell elements whose stiffness may be defined according to the corresponding layout and thickness of layers, as in the procedure described in §3.2.5 or even as infinitely rigid in their plane connected by deformable mechanical joints. The numerical model should nevertheless include a proper schematization of the mechanical joints by means of equivalent spring or truss elements as a model with only shell elements schematizing CLT panels and rigid connections may lead to significant mistakes in the prediction of the natural vibration period and of the building deformations (Fragiacomo et al., 2011, Follesa et al., 2013).

Regarding the value of the behaviour factor q to be adopted in a linear static or dynamic analysis for the seismic design of CLT buildings according to the force-based procedure of EC8 (CEN, 2004), the results of research conducted so far (Ceccotti et al., 2007) showed that, when the building is designed with walls composed of multiple CLT panels up to a panel width of 2.5m, designed in compliance with the capacity based design criteria explained in §3.2.4, a greater ductility and energy dissipation capacity for the whole building may be assumed and a value of q=3 can be used.

3.2.4 Capacity-based design criteria

The background research conducted on the static and seismic behaviour of CLT buildings and the design and the construction experience gained so far led to the conception of some design rules with regard to the capacity design criteria to be applied in the seismic design (Follesa et al., 2011a, Follesa et al. 2013).

As explained in §1.2 these rules are needed for each structural type and material in order to achieve the desired level of ductility and energy dissipation capacity for the whole building. This could be done by establishing a hierarchy of resistance among the different structural components so that the whole structure should have an adequate capacity to develop plastic deformations without substantial reduction in the overall resistance against horizontal and vertical loads. Besides, any possible global instability mechanism or soft-story mechanism should be avoided. Furthermore any possible anticipated brittle failure mechanism in the elements devoted to the energy dissipations, i.e. mechanical joints, should be avoided. This could be done, as explained in §1.2, by designing the brittle elements by the over-strength factors of the ductile elements in order to ensure that the ductile failure mechanisms will take place with sufficient reliability before the failure of the brittle structural components. Therefore Gavric et al. (Gavric et al., 2013), suggested the adoption of capacity design rules at three different levels: (i) connection level, (ii) wall level and (iii) building level. The reasons to suggest also capacity design rules at wall level are in order to achieve a rocking failure mechanism or a combined rocking-sliding failure mechanism in walls composed by multiple CLT panels with vertical step joints, which is more ductile than a pure sliding failure mechanism and in addition ensure recentering of the walls after seismic events by limiting permanent horizontal slip deformations. However this rocking mechanism can be observed in quasi-static cyclic tests like the ones described in §3.2.7.2 only near the collapse. In practical design cases, where hold-downs are designed to resist the seismic uplift forces and angle brackets are designed to resist sliding forces the rocking behaviour of the wall will be much less pronounced under the design seismic forces, due also to the uplift restraint contribution of the angle brackets. In principle also the uplift restraint of angle brackets should be taken into account in design, but this is possible only if the exact position of each angle bracket in the real building is known. This is a very difficult task to achieve in the construction of CLT buildings, where angle brackets are usually evenly distributed along the length of the wall and there uplift stiffness and strength are not considered in common design practice. Gavric at al. (2013) suggest, in order to achieve the rocking behaviour, to overdesign the angle brackets considering an overstrength factor of 1.3. Yet it should be considered that, if we take into account the design strength of commercially available angle brackets for CLT, this rule may result in too many connectors distributed at a close distance along the length of the wall, especially for medium to high-rise buildings, thus leading to an undesired lower ductile behaviour of the whole building. This is particularly true in Italy where the material partial coefficient is higher than elsewhere, and the Eurocode 8 clause of using a unitary material partial coefficient in seismic design is not used. Moreover it should be considered that, by applying the capacity design rules provided in the following which do not take into account the overdesign of angle brackets, a pronounced coupled rocking-sliding behaviour of walls was nevertheless observed during the full-scale tests performed within the Sofie project described in §1.3.1, with a clearly observable deformation of vertical step joints between wall panels, which led to a good overall ductile behaviour of the whole building (Figure 3-16).

Figure 3-16 Uplift of the middle panel in 2nd story wall (marked with the white circle) and corresponding slip of the vertical step joint observed during the 100% JMA Kobe (0.82g) test on a three story CLT building performed within the SOFIE Project at the NIED shaking table facility in Tsukuba, Japan, July 2006 (www.progettosofie.it).

Therefore, for the above explained reasons, only capacity design rules at building and connection level will be presented here.

3.2.4.1 Capacity based design at building level

As anticipated in §3.2.3, concerning its seismic behaviour a CLT building should be regarded as a box-type structure. In order to achieve this behaviour some structural components should be considered as dissipative zones and some other should be considered as non-dissipative and properly designed with sufficient overstrength so to avoid any possible brittle failure mechanism (Follesa et al., 2011a, Follesa et al. 2013).

The connections devoted to the dissipative behaviour in a CLT building are (Figure 3-17):

- vertical screwed or nailed step joints between parallel wall panels in case of walls composed of multiple CLT panels;

- connections against sliding between walls and floor underneath, and between walls and foundation (usually steel brackets or screwed connections);
- anchoring connections against uplift placed at wall ends and at wall openings (usually hold-down anchors).

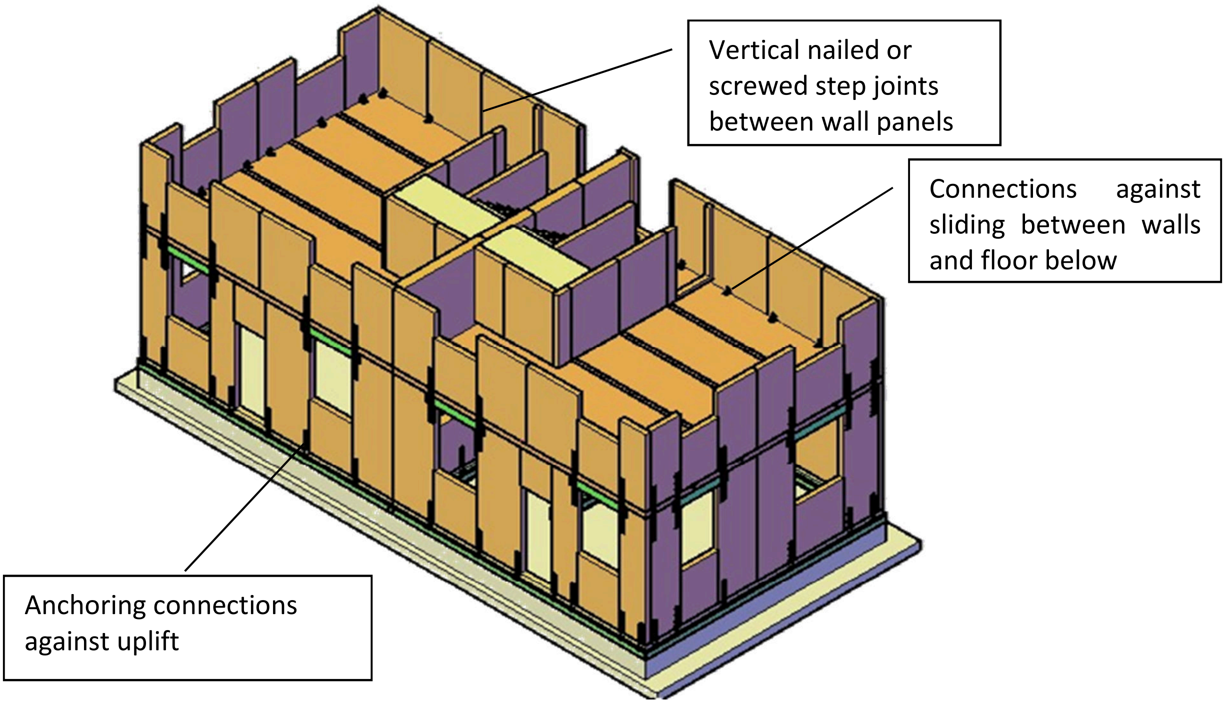


Figure 3-17 Connections devoted to the dissipative behaviour in a CLT building (note: for the sake of clarity hold-down anchors at the ground level are placed on the external side of the walls, whereas usually they are placed internally).

The structural elements which should be over-designed with sufficient overstrength so as to ensure the development of cyclic yielding in the dissipative zones are (Figure 3-18):

- all CLT wall and floor panels;
- connections between adjacent floor panels in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;
- connections between floors and walls underneath thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;
- connections between perpendicular walls, particularly at the building corners, so that the stability of the walls themselves and of the structural box is always assured.

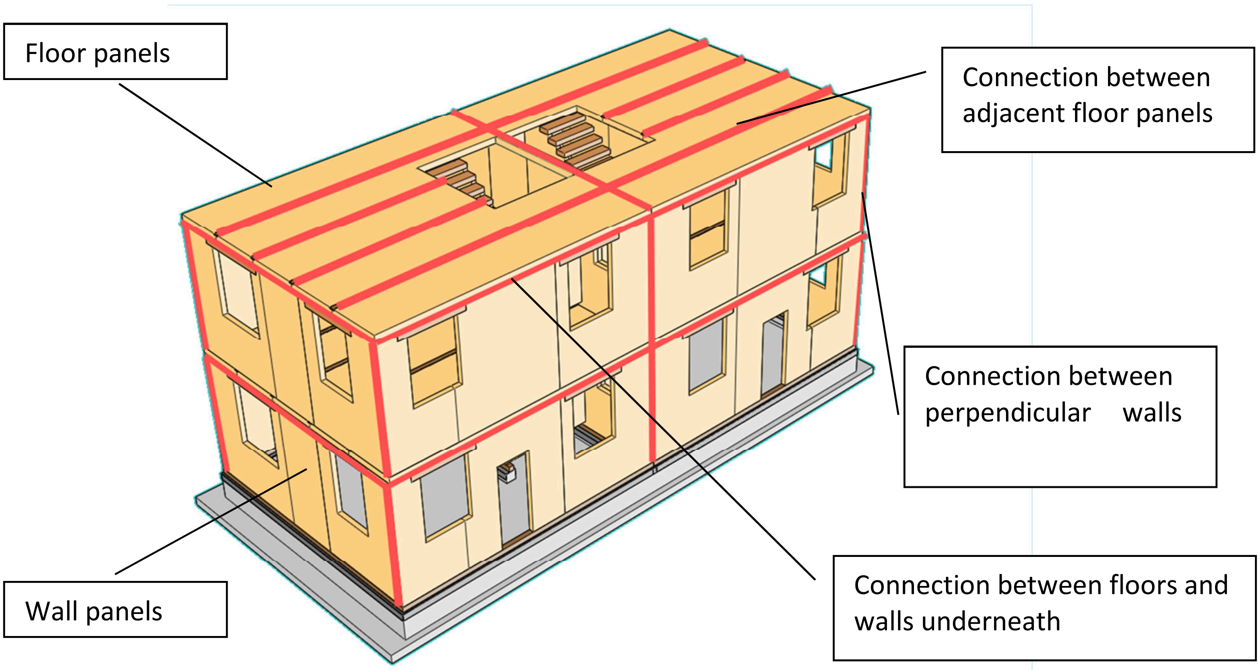


Figure 3-18 Connections (highlighted in red) and elements to be designed with overstrength in order to fulfil the capacity design criteria in Cross Laminated buildings (after Follesa et al., 2013).

The research experience conducted through full-scale seismic tests on CLT buildings (Ceccotti et al., 2007) designed with a constant ratio between story resistance and story shear at each level, confirmed that a simultaneous plasticization of the connections at each level is possible therefore maximizing the energy dissipation for the whole building.

Therefore, following these outcomes, the seismic resistance of shear walls should be higher at lower storeys and should decrease at higher storeys proportionally to the decrease of the storey seismic shear (Figure 3-19).

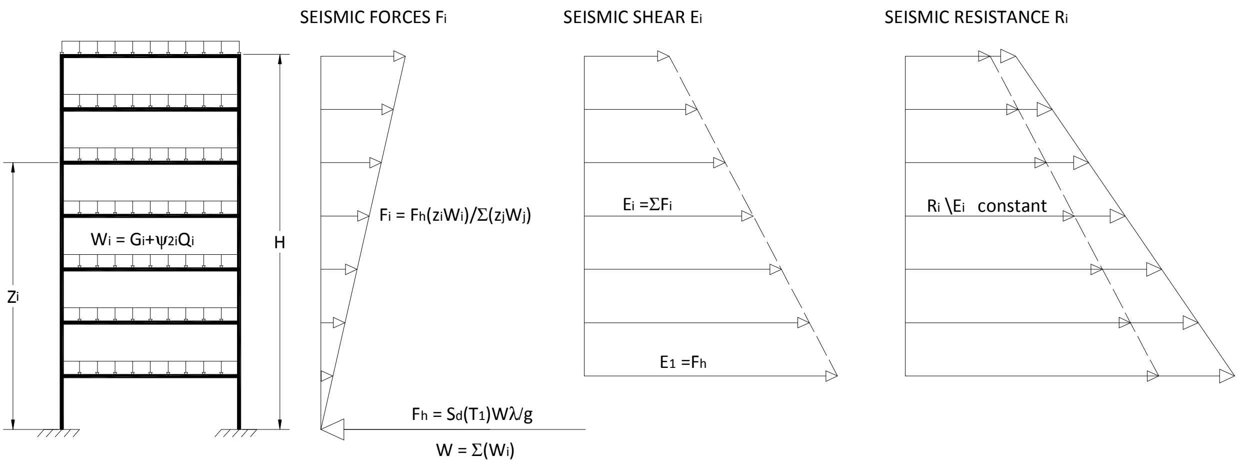


Figure 3-19 In order to maximize the energy dissipation of the whole building the ratio between seismic shear demand and seismic shear capacity should be kept almost constant at each storey.

3.2.4.2 Overstrength factors

As already discussed several times before, in order to achieve the desired ductile behaviour for the whole building and fulfil the capacity based provisions given for the specified structural system and material in order to maximize the energy dissipation, the brittle elements should be designed for the overstrength of the ductile parts. However as already mentioned in §1.2 and §2.4 currently there is no value suggested for timber buildings in Eurocode 8 (CEN, 2004). The overstrength factor is defined as the ratio between the 95th percentile of the connection strength distribution and the design strength F_d (Jorissen and Fragiacomo 2011, Fragiacomo et al., 2011). Studies conducted on the evaluation of the overstrength factor of hold-down, steel angle brackets and screwed connections for CLT buildings, based on statistical analysis conducted over experimental cyclic tests showed that a conservative value of 1.3 could be used for hold-down and steel brackets connections, and of 1.6 for screwed connections (Follesa et al., 2011a, Gavric et al., 2011, Gavric et al., 2012, Gavric 2013).

3.2.4.3 Capacity based design at connection level

In order to achieve the desired ductile behaviour, brittle failure mechanisms should be avoided through a proper design and detailing of each single connection (Follesa et al., 2013). As the ductile failure mechanism in CLT connection is characterized by yielding of fasteners (nails or screws) in steel-to-timber or timber-to-timber connections (Eurocode 5, 2009), special care should be used when designing the dissipative connections to ensure the attainment of a ductile failure mechanism characterized by the formation of one or two plastic hinges in the mechanical fastener (cases b, d, e for steel-to-timber connections, cases d, e, f for timber-to-timber connections in Figure 3-20).

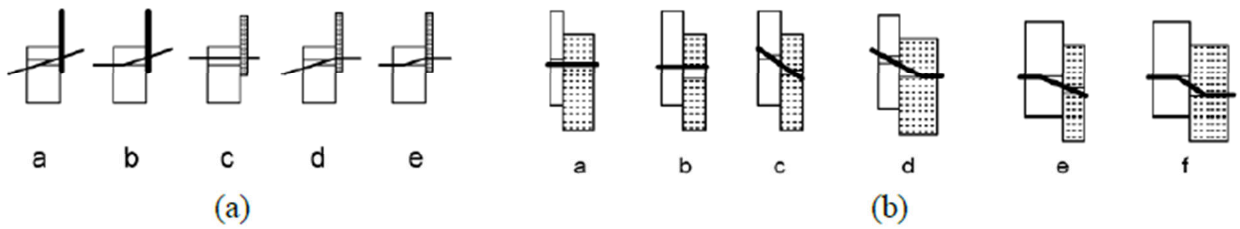
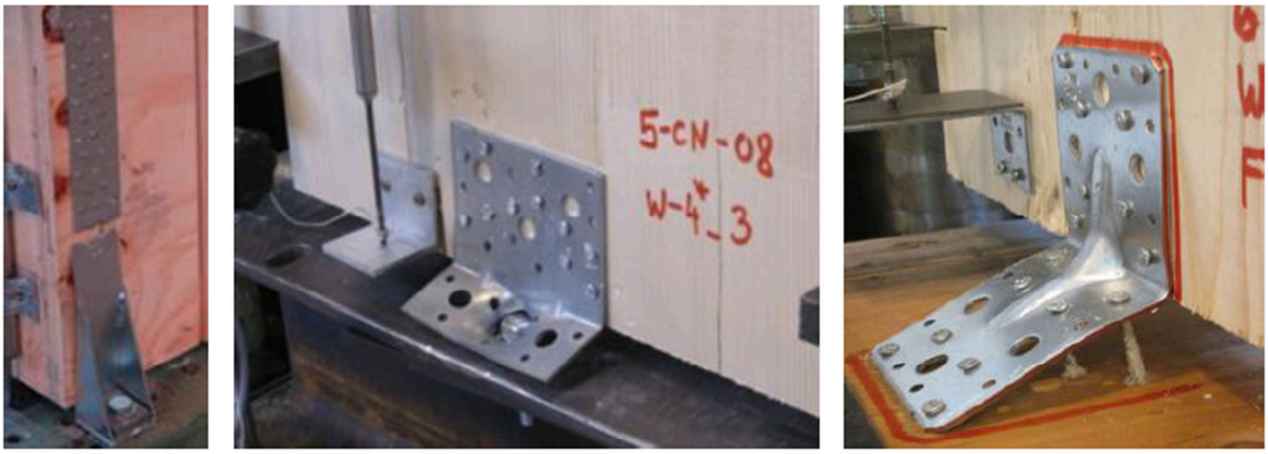


Figure 3-20 Failure modes according to Johansen theory for steel-to-timber connections (a) and timber-to-timber connections (b) (EC5, 2004).

Furthermore brittle failure mechanisms due to the steel plate failure in the weaker section of hold-down connections (like in case (a) of Figure 3-21), due to the pull-through of the head of the anchor bolt through the steel plate in steel bracket and hold-down connection to the foundation (like in case (b) of Figure 3-21), or to the sudden withdrawal of nails in the inter-story wall-to floor angle brackets connection (like in case (c) of Figure 3-21) should always be avoided. Other brittle failures such as splitting, shear plug, tear out and tensile fracture of wood in the connection regions should be always avoided.



(a) (b) (c)

Figure 3-21 Brittle failure mechanisms in angle brackets and hold-down connections due to the steel plate failure in the weaker section of hold-down connections (a), due to the pull-through of the head of the anchor bolt through the steel plate in steel bracket and hold-down connection to the foundation (b), due to the sudden withdrawal of nails in the inter-story wall-to floor angle brackets connection (c) (after Gavric et al., 2013).

Brittle failure mechanisms like in case (a) of Figure 3-21 could be avoided by calculating the maximum number of nails as to ensure that failure happens on the nail side and not on the hold-down side.

This could be achieved by applying the capacity based design rules to the single connector, i.e. by multiplying an overstrength factor γ_{Rd} for the calculation of the design strength of the brittle part as in the following equation:

$$\gamma_{Rd} \cdot F_{Rd,d} \leq F_{Rd,b} \tag{2}$$

where $F_{Rd,d}$ is the design strength of the ductile part (the nailed steel-to-timber connection) and $F_{Rd,b}$ is the design strength of the brittle parts (the steel plate and the bolted/nailed steel-to-foundation/floor connection) and γ_{Rd} could be taken as 1.3. The design strength capacity F_{Rd} could be calculated by dividing the characteristic strength F_k by the strength partial factor γ_M, assumed equal to one according to the Eurocode 8 (CEN, 2004) for dissipative structures. The correct application of this rule is strictly related to the exact estimation of the characteristic strength of the fasteners, as if the fastener's strength is over- or under-evaluated the theory will obviously fail.

However, since experimental results showed that the characteristic strength of dowel type fasteners in steel-to-timber and timber-to-timber connections in CLT is greater than the corresponding strength for solid timber and glulam due to reduced possibility of splitting given by the presence of the cross-layers, the Johansen theory proposed in Eurocode 5 (CEN, 2009) for the calculation of the characteristic strength of the connection cannot be applied. Therefore, since no adjustment of the current Johansen theory for CLT is found, reference should be made either to the available characteristic values data obtained from tests conducted on the same type of connection or to analytical formulation for the calculation of the characteristic values of dowel type connections in CLT panels. The most used and international recognized analytical method for the calculation of the characteristic strength of steel-to-CLT and CLT-to-CLT connections was proposed by Uibel and Blaß (Uibel and Blaß, 2006, Uibel and Blaß, 2007) by introducing a modified formulation for the calculation of the embedment strength in Johansen theory given in Eurocode 5 (CEN, 2009). However, as explained in Uibel and Blaß, 2007, the proposed equation is limited to panels with layers of 9 mm in maximum thickness, whereas the minimum layer thickness of European CLT producer is 19 mm.

As an example, Table 3-2, Equation (3) and Table 3-3 show the calculation of the maximum number of nails, according to the above referenced procedure, for a HTT22 hold-down connection produced by Simpson Strong Tie according to the Johansen theory current explained in Eurocode 5 (CEN, 2009), according to the modified Johansen theory proposed by Uibel and Blaß, 2007, and based on experimental results within the SOFIE project referenced in §1.3.

Table 3-2 Calculation of the characteristic strength on the steel side.

	Steel Plate	Anchor Bolt in tension	Ancoring to the concrete foundation
	R_k=0.9xA_{net}xf_{uk}=(0.9x(70-2x5)x3x360)/1000=**58.32 kN**	A_{res}xf_{uk}=157x400/1000=62.80 kN	N_{rk}=63.15 kN

Therefore the maximum number of nails should be calculated as:

$$N = \frac{R_{k,steel}}{\gamma_{Rd} \cdot R_{k,nail}} \tag{3}$$

Table 3-3 Calculation of the maximum number of nails according to Equation (3) for Eurocode 5 and Uibel and Blaß, 2007 and evaluation through experimental research within the SOFIE project.

Eurocode 5	Uibel and Blaß, 2007	SOFIE test
$F_{v,Rk}$=1.93 kN (mode d)	$F_{v,Rk}$=2.37 kN (mode d)	
$N = \frac{R_{k,steel}}{\gamma_{Rd} \cdot R_{k,nail}} = \frac{58.32}{1.3 \cdot 1.93} = 23$	$N = \frac{R_{k,steel}}{\gamma_{Rd} \cdot R_{k,nail}} = \frac{58.32}{1.3 \cdot 2.37} = 18$	12

Though it should be underlined that the result obtained from the SOFIE test refer to mean values of the connection strength, while the other two values are calculated characteristic values, the results displayed in Table 3-3 show that without a correct evaluation of the connection resistance, the design may lead to unsafe results as the capacity design theory fails.

Brittle failures like in case (b) of Figure 3-21 could be avoided by applying a steel washer to the anchor bolt in order to increase the steel resistance (as showed in Figure 3-22, case (b)), or by using more than one anchor bolt and then calculating the maximum number of nails according to the procedure described above.

Also brittle failures like in case (c) of Figure 3-21 could be avoided by using one or more screws with larger diameter for the steel bracket-to-floor connection (as showed in Figure 3-22, case (c)), in order to increase the withdrawal resistance of the angle bracket and again by calculating the maximum number of nails according to the procedure described above.

Figure 3-22 Possible solution to avoid the anticipated brittle failures showed in Figure 3-21.

Finally special care should be taken when placing the connectors in the CLT panel to avoid positions corresponding to wood defects such as evident knots or wood fractures, in order to avoid brittle failure mechanism such as the one showed in Figure 3-23.

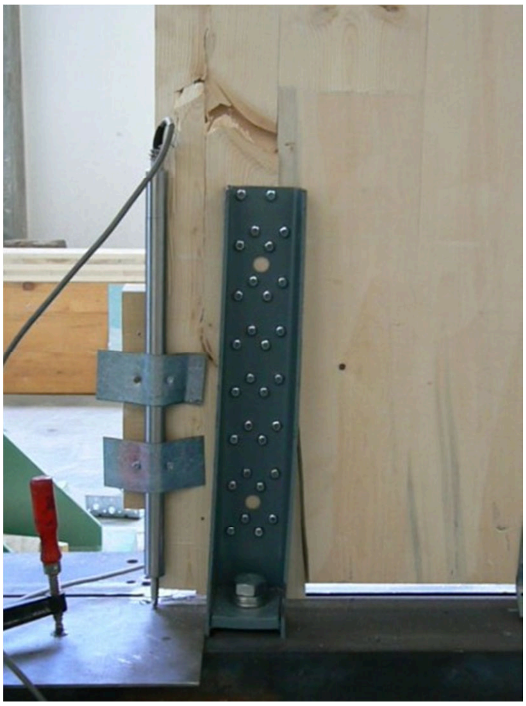


Figure 3-23 Brittle failure mechanism in a hold-down connection due to the presence of a knot in the connection area.

3.2.5 Numerical modelling of CLT buildings with Sap2000

For all the above explained reasons, it is crucial that the numerical modelling of a CLT building includes finite elements representing the connections used in the real construction, modelled with the correct stiffness. As explained in §3.2.3, numerical models which include only shell elements representing the CLT walls and rigid connections may lead to non-conservative results, both in the estimation of the natural period and of the deformation characteristics of the building.

In the following some suggestions on how to model CLT buildings will be given by making reference to a software package which is very well-known and used by consulting engineers, i.e. SAP2000 (CSI, 2000).

3.2.5.1 CLT walls modelling

CLT walls may be modelled using 2D elastic shell elements in one of the following ways:

a) Modelling each CLT wall as a "Layered Shell Element" where each layer is modelled as homogeneous and isotropic, and the adjacent layers are rotated by 90° as real layers in CLT.

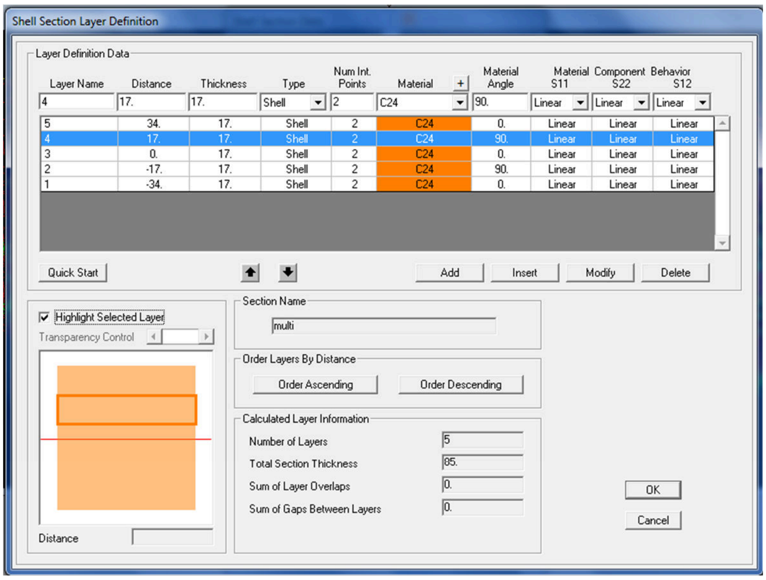


Figure 3-24 Definition of a layered shell element for a 85 mm CLT panel.

b) By assigning the element an equivalent homogenized section, with the same dimensions of the actual CLT panel and with different properties in different directions (orthotropic model) in the following way.

For the orthotropic material properties (Follesa et al. 2013), nine values have to be defined, i.e. the 3 moduli of elasticity in the 3 orthogonal directions E_x, E_y, E_z, the 3 Poisson's ratios ν_{xy}, ν_{yz}, ν_{zx}, and the 3 shear moduli G_{xy}, G_{yz}, G_{zx}. The x axis is assumed along the length of the wall (in the horizontal direction), the y axis in the direction perpendicular to the wall plane, and the z axis along the height of the wall (in the vertical direction), as shown in Figure 3-25. Out of these properties the most important ones are: (*i*)

E_z that affects the vertical axial stiffness and the bending stiffness; and (*ii*) G_{zx} that affects the in-plane shear stiffness of the wall.

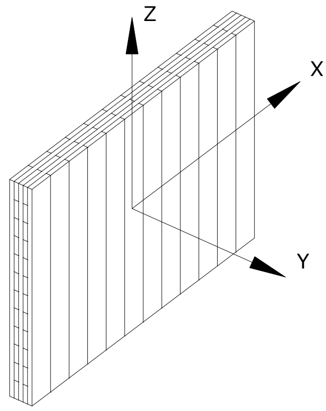


Figure 3-25 Principal axes of a vertical wall element (after Follesa et al., 2013).

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

$$E_{0,eq} = \left[1 - \left(1 - \frac{E_{90,T}}{E_{0,L}}\right) \cdot \frac{a_3 - a_1}{a_5}\right] \cdot E_{0,L} \tag{4}$$

where $E_{0,L}$ is the modulus of elasticity parallel to the grain of the longitudinal layers, $E_{90,T}$ is the modulus of elasticity perpendicular to the grain of the transversal layers and a_1, a_3, a_5 are shown in Figure 3-26.

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

$$E_{90,eq} = \left[\frac{E_{90,L}}{E_{0,T}} + \left(1 - \frac{E_{90,L}}{E_{0,T}}\right) \cdot \frac{a_3 - a_1}{a_5}\right] \cdot E_{0,L} \tag{5}$$

Figure 3-26 Definition of various thicknesses in a generic 5-layer CLT panel (after Follesa et al., 2013).

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

Furthermore, the equivalent modulus of elasticity to be used as E_y in the material properties of the numerical model (see Figure 3-25) is calculated as:

$$E_y = t_{tot} \cdot \left(\sum_{i=1,3,\ldots,n} t_i / E_{90,L,i} + \sum_{j=2,4,\ldots,n-1} t_j / E_{90,T,j} \right)^{-1} \tag{6}$$

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

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

$$G = t_{tot} \cdot \left(\sum_{i=1,3,\ldots,n} t_i / G_{L,i} + \sum_{j=2,4,\ldots,n-1} t_j / G_{T,j} \right)^{-1} \tag{7}$$

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

c) By using a pair of elastically deformable elements (links or springs) hinged on a frame of rigid hinged elements and calibrated on the bending stiffness and shear stiffness of the CLT panel in the following way.

$$K_{spring} = \frac{K_{lat}}{2\,cos^2(\alpha)} \tag{8}$$

where:

$$K_{lat} = \frac{F_{lat}}{\frac{F_{lat}h^3}{3E_zJy} + \frac{6F_{lat}h}{5GA}} = \frac{1}{\frac{h^3}{3E_zJy} + \frac{6h}{5GA}} \tag{9}$$

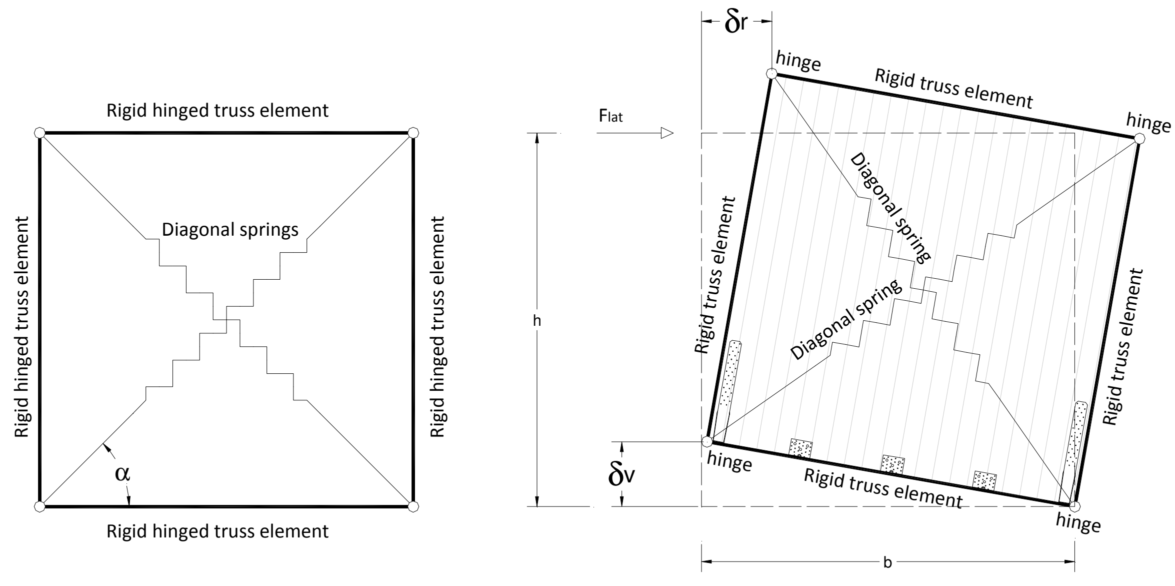


Figure 3-27 Schematization of a CLT wall using a pair of diagonal link or spring elements.

d) Even by modelling the CLT panels as rigid shell elements, especially in the case where no opening are present.

3.2.5.2 CLT floors modelling

CLT floors are usually modelled using a kinematic constraint of infinitely rigid in-plane diaphragm. In SAP 2000 this can be done by applying a diaphragm constraint to all the floor nodes. This is of course a simplification which is enough reliable for regular buildings with even distribution of shear walls spaced at regular distances like for residential buildings (usually no more than 6-7 m). For other type of buildings (e.g. office or commercial buildings) with few shear walls and high span-to-depth ratios, this assumption should be carefully considered, and provisions should be taken in order to assure that joints between floor panels are rigid enough, e.g. by realizing glued joints or using stiffer connection with screws running at 45°.

It is also possible and of course closer to the real construction to model the CLT floors using shell elements by assigning again an equivalent homogenized section with different properties in different directions (orthotropic model) as described above for wall elements, with the same length and width of the actual floor CLT panels and by schematizing the floor-to-floor connections as described in the following for the wall-to-wall connection.

In this way it is also possible to consider the out of plane stiffness of each floor which may have some contribution in the global behaviour of the building and in the distribution of the tension forces on hold-down connectors, especially in non-linear analysis. However, this schematization of course greatly increases the computational time, especially for multi-storey buildings.

3.2.5.3 CLT connections modelling

As stated above, the numerical modelling of connections is crucial in order to obtain reliable results both in the evaluation of the natural period and mode shapes of the building and to estimate the correct inter-storey drifts of the building.

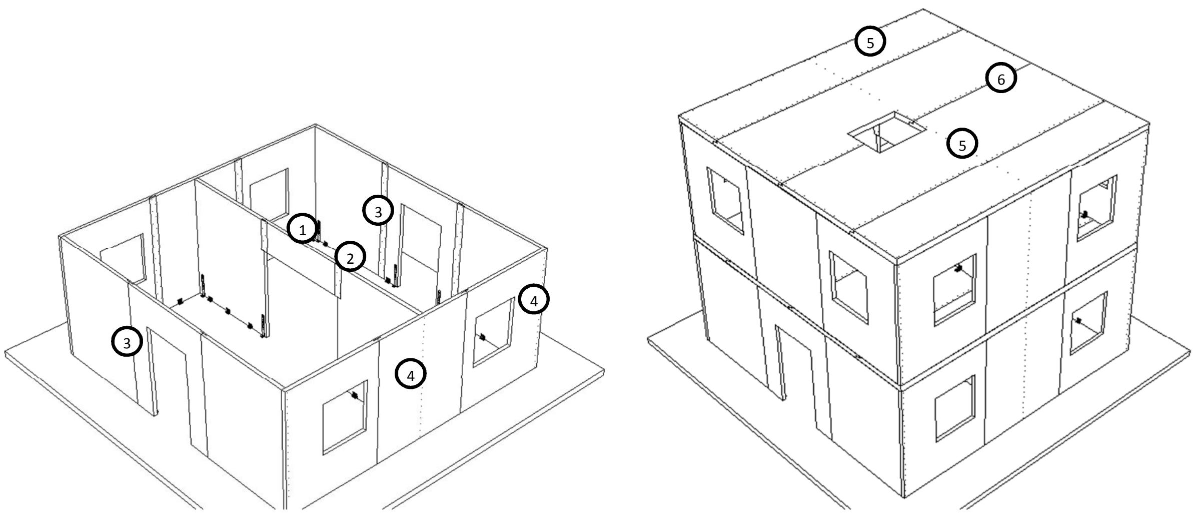


Figure 3-28 Connections in CLT buildings: 1 – Hold-downs, 2 - Angle brackets, 3 - Vertical screwed or nailed step joints between adjacent wall panels, 4 - Vertical screwed joint between perpendicular walls, 5 - Horizontal screwed connection between floor panels and walls underneath, 6 - Horizontal joint between floor panels.

By making reference to Figure 3-28 we can identify 6 types of connections in a CLT building, respectively:

1. Uplift restraint connection at the foundation and at floors made with hold-down anchors,
2. Slip (and also uplift) restraint connection made with angle brackets
3. Vertical joint (if present) between adjacent wall panels made with nails or screws
4. Vertical joint between perpendicular walls made usually with screws
5. Horizontal connection between floor panels and walls underneath usually made with screws
6. Horizontal connection between floor panels usually made with screws

Considering the capacity based design criteria described in §3.2.4, connections #4, #5 and #6 could be modelled as rigid.

All the above mentioned connections may be schematized with 2 degrees of freedom link or spring elements as they restrain 2 translational degrees of freedom, which may or may not be located in the exact position of the real connection.

Particularly for the case of **linear elastic static or dynamic analysis**, the main problem is the correct evaluation of the connection stiffness, as it has a great influence on the evaluation of the natural period of the building and of its overall stiffness.

Therefore there are two possibilities:

1. Making reference to available experimental data to evaluate the connection stiffness.

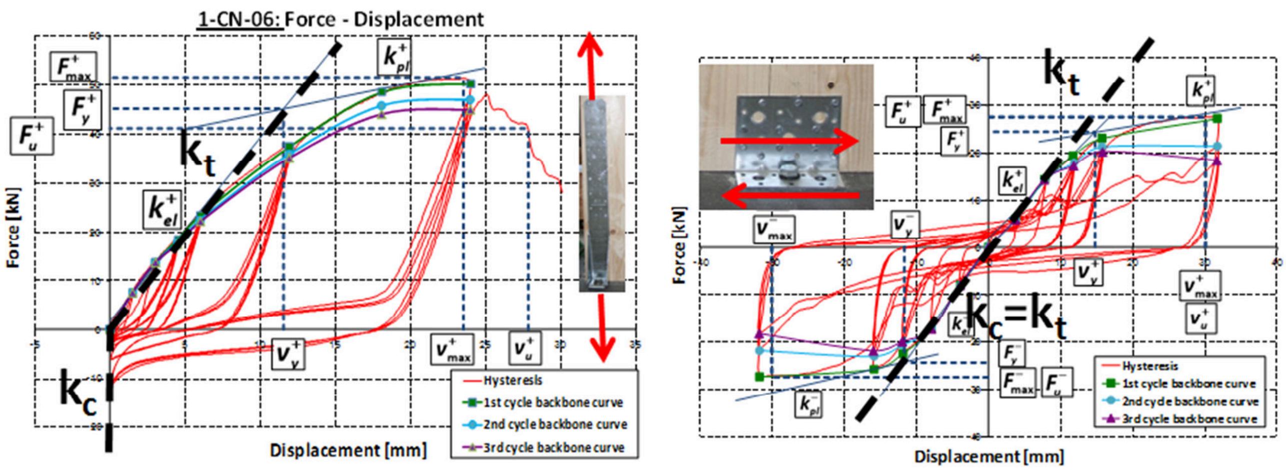


Figure 3-29 Experimental hysteresis curves of hold-down (left) and steel angle bracket (right) connection subjected to cyclic loading (after Fragiacomo, 2013).

From available experimental data, referred to the same type of connector used in the design with the same type and number of fasteners, it is possible to evaluate the elastic stiffness. This is a relatively simple procedure for connection like steel brackets or screwed vertical or horizontal joints loaded in shear which has a nearly symmetric behaviour. The stiffness k_t may be evaluated as a secant value at 40% of the peak strength (thick black dashed line in Figure 3-29).

However for hold-down or steel bracket connections axially loaded the behaviour is completely different and asymmetric between tension, where the value of the elastic stiffness K_t is that of the nailed connection between steel plate and CLT panel, and compression where the value of the elastic stiffness K_c is that of the CLT wall panel loaded in compression parallel to the grain for the foundation connection or that of the floor panel loaded in compression perpendicular to the grain for the inter-storey connection.

In this case one procedure could be the calibration of the elastic stiffness of the two linear springs K_e, simulating the hold-down connection, which will have an intermediate value between K_c and K_t, by equalizing the vertical displacement Δ due to the symmetric panel rotation represented in the model schematization with the vertical displacement Δ due to the panel rotation in the real asymmetric behaviour (Figure 3-30).

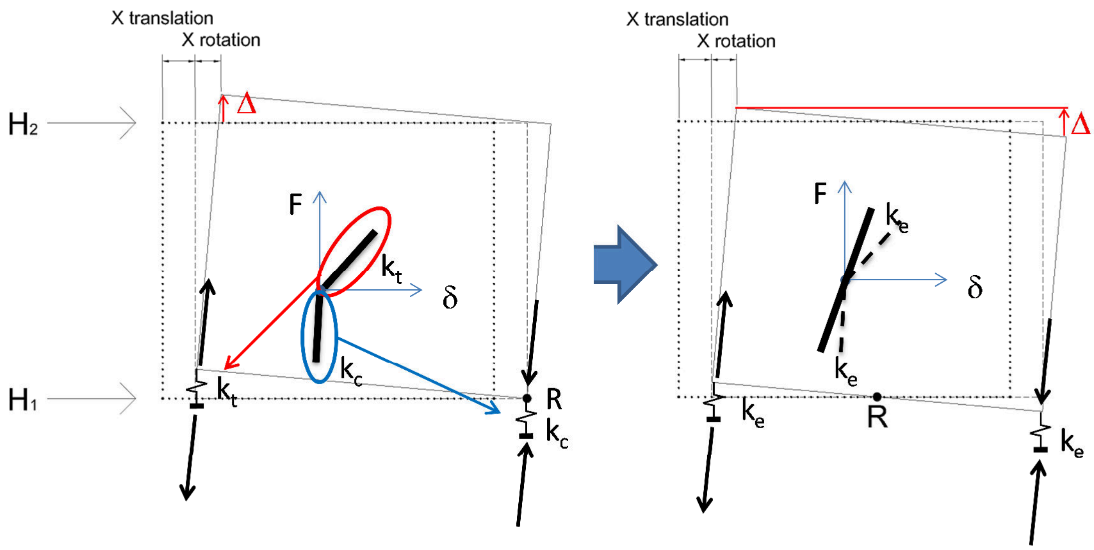


Figure 3-30 Possible procedure to evaluate the equivalent stiffness of two symmetric linear springs simulating the asymmetric hold-down behaviour by equalizing the vertical displacement Δ in the two cases (after Fragiacomo, 2013).

Anyway being this procedure rather complex, a possible solution could be not to explicitly model the hold-down connection as in the example described in §3.2.6.

2. The second possibility is to calculate the stiffness of each connection according to the formula proposed by EN1995-1-1 for the calculation of the slip modulus of timber to timber and steel to timber connections as in the procedure described below for an angle bracket connection, but applicable also to other cases (Follesa et al., 2013).

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

$$K_{con} = 2 \cdot \rho_m^{1.5} \cdot d / 23 \qquad (10)$$

In Equation (10), ρ_m and d are the mean density of timber in [kg/m^3] and the fastener diameter in [mm], respectively, with the values of K_{con} in [N/mm]. This equation is given in table 7.1 of Eurocode 5 (EC5, 2009) for the calculation of the slip modulus under service load for dowels, bolts, screws and nails with pre-drilling used in steel to timber connections. Since the nails are usually banged, a different formula for nail without pre-drilling should be used. However, it was observed from the results of tests conducted on CLT structures (Ceccotti et al., 2006) that Equation (10) provides a better match with the experimental values. A possible reason for that is the effect of the cross layers, which increases the stiffness of nails with respect to the calculated value for solid timber according to the formula proposed for nails without pre-drilling. Thus, for n nails, the horizontal stiffness of the wall-foundation connection is given by:

$$K_H = n \cdot K_{con} \tag{11}$$

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

$$K_H = \frac{1}{\frac{1}{K_{H1}} + \frac{1}{K_{H2}}} \tag{12}$$

In case of **non-linear static or dynamic analysis** it is possible to schematize each connection as a multi-linear plastic link element. Multi-linear plastic link elements are non-linear springs which follow the Pivot hysteresis rule. The Pivot hysteresis rule, so called from the presence of the pivot points in the force-displacement diagram, which define the slopes of unloading and reloading branches, among the different hysteresis laws included in SAP2000 library is the model which better approximate the pinching hysteresis behaviour of mechanical joints in timber structures.

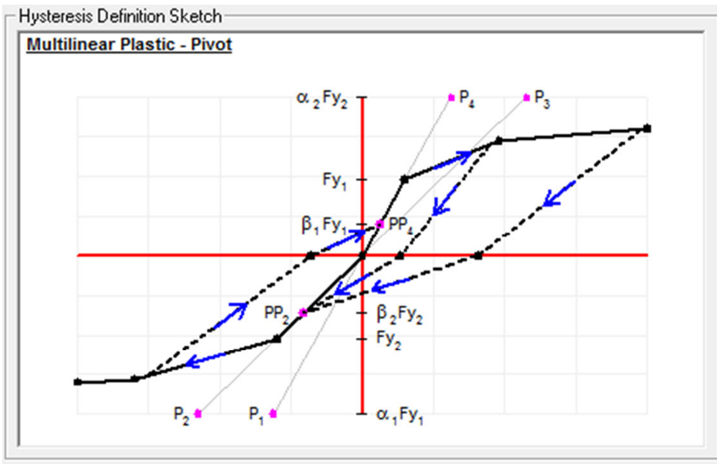


Figure 3-31 Multilinear Plastic – Pivot hysteretic rule in Sap2000 (CSI, 2000) which could be used to schematize the non-linear behaviour of hold-down, angle brackets and screwed connections in CLT structures.

In order to calibrate the model the following parameters shall be provided:

- α_1 and α_2 which define the target pivot points for the unloading from the backbone curve in the positive and negative quadrant, respectively,
- β_1 and β_2 which define the target pivot points for the reloading towards the positive and negative quadrant, respectively,
- η which define the elastic stiffness degradation.

Multi-linear plastic link elements are not able to simulate the strength degradation which occur between the first and subsequent cycles when a mechanical joint in a timber structure is subjected to more cycles at the same displacement. Therefore, an accurate reproduction of the experimental hysteresis cycles is not possible. However, a reliable enough reproduction of the actual non-linear behaviour of mechanical joints may be obtained by making an appropriate calibration of the Pivot model based on the dissipated energy as in the procedure described in §3.2.7.

3.2.6 Linear static and dynamic analysis of CLT buildings with SAP2000. Possible procedure and design example

Considering the numerical modelling procedure described in 3.2.5 (Follesa et al., 2013) for the schematization of the main structural components of a CLT building, in this paragraph a numerical model for the linear static and dynamic analysis of CLT buildings is proposed.

The proposed 3-dimensional numerical model of a CLT building has been implemented in the widespread software package for structural analysis SAP2000 (CSI, 2000), which is utilized to perform the modal response-spectrum or the static analysis and to solve the associated equilibrium equations. A pre- and post-processing software specifically developed by Tecnisoft (Modest, Ver. 8.6, 2015) was used to aid in the implementation.

Three types of elements are utilized, namely:

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

Figure 3-32 illustrates a typical schematization of a pair of CLT wall panels and the connections between the walls, at the base and with the upper floor walls.

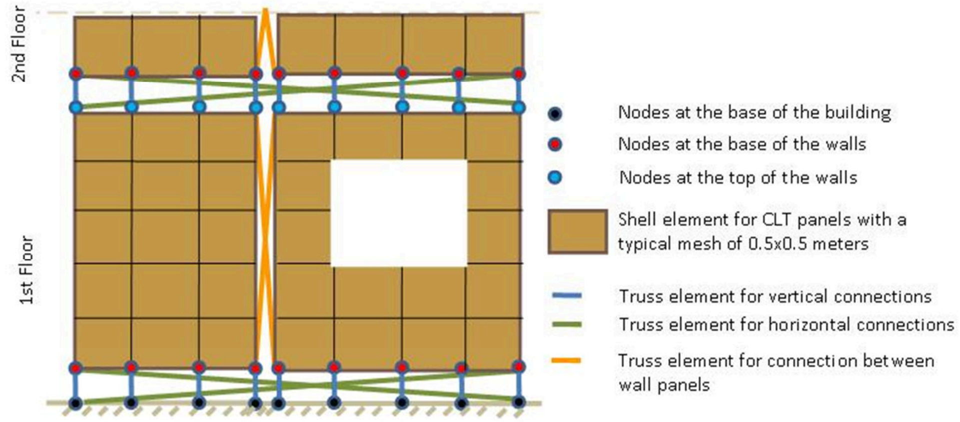


Figure 3-32 Typical wall schematization (after Follesa et al., 2013).

With this representation, the in-plane shear forces transmitted from the walls to the walls underneath can be directly obtained from the axial forces in the horizontal truss elements, while the uplifting vertical forces are obtained from the tensile forces in the vertical truss elements. A rigid diaphragm constraint is used to constrain all nodes at the same level in Figure 3-32. It should be noticed that a separate constraint is used to constraint the nodes at the bottom of the wall and at the top of the wall underneath.

The model described above is based on some simplified assumptions:

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

Shell elements are defined with the same length, height and thickness as the associated CLT wall panels considering an orthotropic schematization as described in §3.2.5.1 (b). The shear connection of the panels to the foundation or to the floor below is schematized for each wall with a pair of truss elements whose axial stiffness is calculated in the following way:

$$E_f A_H = \frac{K_H \cdot L_W}{2} \tag{13}$$

where L_w is the length of the wall section, which can be made of a single panel or more adjacent panels connected with vertical step joints, and K_H is the horizontal stiffness of the steel bracket connection calculated as in §3.2.5.3.

Similarly to the horizontal connection, the vertical step joints between wall panels are simulated with a pair of vertical cross truss elements which are used to connect each wall panel to the next one (see Figure 3-32). The section and material properties of the trusses are computed based on the stiffness of the connections used to transfer the shear force from adjacent wall panels. These connections are typically made of a cross-layered wood-based panel (usually cross banded LVL panels or plywood) inserted in an internal groove or in a groove cut on the internal side of the wall (case (a) and (b) of Figure 3-10 respectively) and the connection is usually made of self-tapping screws with a diameter varying from 6 to 10 mm or, sometimes, with 3 mm diameter nails and spacing depending on the seismic load.

The vertical stiffness K_V of the connection is computed again based on the slip modulus at serviceability limit state K_{con} of each fastener of the panel to timber connection given in table 7.1 of Eurocode 5 (EC5, 2009) and in Equation (10) multiplied by the number of fasteners. This value should then be divided by 2, considering the double panel to timber connection, and either multiplied by 2 for double shear connection like in the case of internal groove step joint, or by 1 for a groove on the internal side of the panel. Similarly to the horizontal connections, the axial stiffness $E_f A_V$ of each vertical truss element is then calculated using Eq. (14):

$$E_f A_V = \frac{K_V \cdot H_W}{2} \tag{14}$$

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

Vertical truss elements are also used to simulate the deformability of the floor diaphragms along their thickness, namely perpendicular to their plane, as the walls bear on the floor panels. Thus, the modulus of elasticity perpendicular to grain (E_{90} from Equation (5)) is selected for the isotropic material properties. Since the shell elements are meshed with a grid of 0.5 meters length, the cross sectional area of the vertical trusses is equal to 0.5 meters times the thickness of the wall above. At the foundation level, the modulus of elasticity of concrete is used for the isotropic material properties. Forces in the vertical truss elements of a wall are then utilized to calculate the tensile forces for the design of the hold-downs, typically installed at each end of the wall to resist overturning moments from horizontal seismic loads. It should be noted that although hold-down anchors play a major role in the actual lateral stiffness and strength of the wall, they are not explicitly simulated in the linear numerical model, as explained in §3.2.5.3.

3.2.6.1 Design example of a three storey case-study building

The case study building considered in the design example (Follesa et al., 2013) is a 3-storey residential CLT structure which was built and tested on the unidirectional 14,5m x 15m NIED Shaking Table facility in Tsukuba, Japan in July 2006 by a joint group of Italian (CNR-IVALSA) and Japanese (NIED) researchers within the SOFIE Project, which was a research project on the seismic and fire behaviour of multi-storey CLT buildings funded by the Province of Trento, Italy, and coordinated by CNR-IVALSA (Italian National Research Council – Trees and Timber Institute) (Ceccotti and Follesa, 2006).

3.2.6.1.1 Geometric and structural configuration

The building has a square plan with dimensions of 7.0x7.0 m and a total height of 10.0 m with a double-pitched roof. The floor plan is symmetric in both directions and at each floor there is a central wall parallel to the E-W direction with a central door opening of 2.25x2.40 m. The two outer walls at the first floor parallel to the E-W direction have two openings; one of 2.25x2.20 m and one of 4.00x2.20 m. The outer walls parallel to the N-S direction incorporate two window openings of 1.20x1.20 m. The net storey height is 2.95 m for the first two floors and varies from 2.75 m at eaves to 3.80 m at the ridge for the third floor. Plan and elevation drawings are displayed in Figure 3-33 while Figure 3-34 shows the construction sequence of the building.

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

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

Figure 3-33 Plans and elevations of the case study building (dimensions in m) (after Follesa et al., 2013).

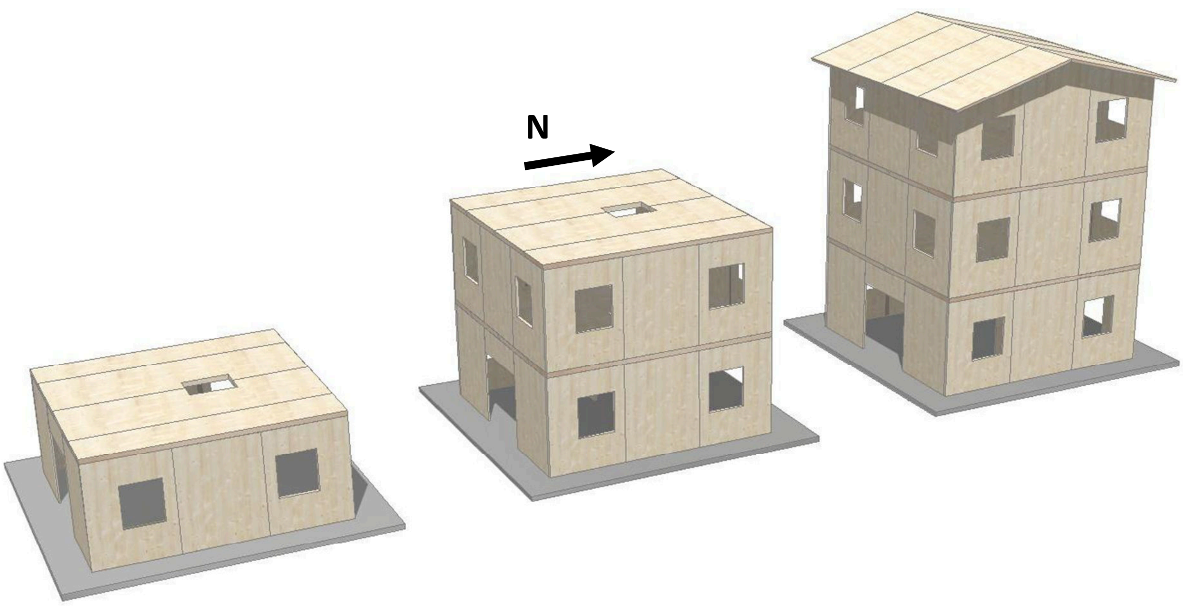


Figure 3-34 Erection sequence of the case study building and direction of floor panels (after Follesa et al., 2013).

3.2.6.1.2 Gravity loads and seismic weight

Gravity loads for the seismic combination (Follesa et al., 2013) were estimated based on the structural and non-structural elements. The dead loads G of external and internal walls are 0.94 kPa and 0.79 kPa, respectively, while those of the floors and roof are 3.43 kPa and 1.28 kPa, respectively, where the roof loads refer to the inclined area. The live loads Q for the floors are 2.00 kPa for residential use and for the roof diaphragm no accidental load is considered for the seismic combination. Based on these gravity loads, Table 3-4 lists the total dead and live loads as well as the seismic weight of each floor of the building. The total seismic weight is W = 692.45 kN.

Table 3-4 Total dead load, live loads and seismic weight for each level of the building (from Follesa et al., 2013).

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

3.2.6.1.3 Design spectrum

The design response spectrum, illustrated in Figure 3-35 (Follesa et al., 2013), is defined from Section 3 of Eurocode 8 (CEN, 2004), based on a Type 1 elastic response spectrum with the following parameters:

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

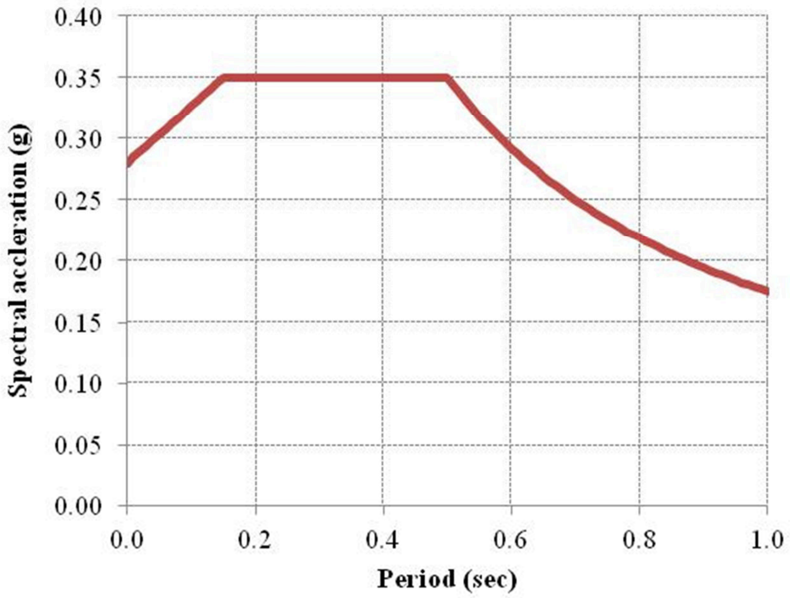


Figure 3-35 Design response spectrum considered in the design example (after Follesa et al., 2013).

3.2.6.1.4 Numerical model of the test building

Based on the description provided in the previous section, a 3-dimensional numerical model has been developed for the 3-storey case study building (Follesa et al., 2013). Floor and roof diaphragms are considered rigid in their plane and schematized by means of master-slave constraints applied to the reference nodes. Figure 3-36 shows the undeformed shape of the FE model and Figure 3-37 shows the Wall IDs of the building. The properties of the shell elements representing wall panels are computed according to Equations (4-7) as:

$$\begin{aligned} E_x &= 4622\ MPa \\ E_y &= 370\ MPa \\ E_z &= 6748\ MPa \\ G_{xz} = G_{xy} = G_{yz} &= 690\ MPa \end{aligned} \tag{15}$$

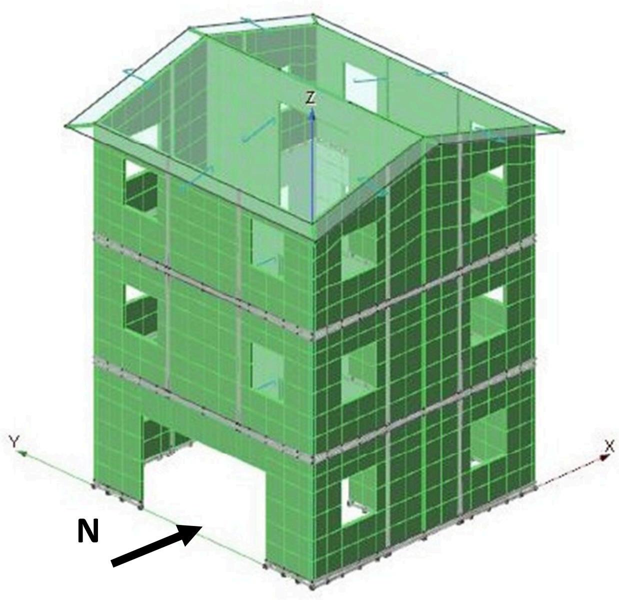


Figure 3-36 Undeformed shape of the building FE model (after Follesa et al., 2013).

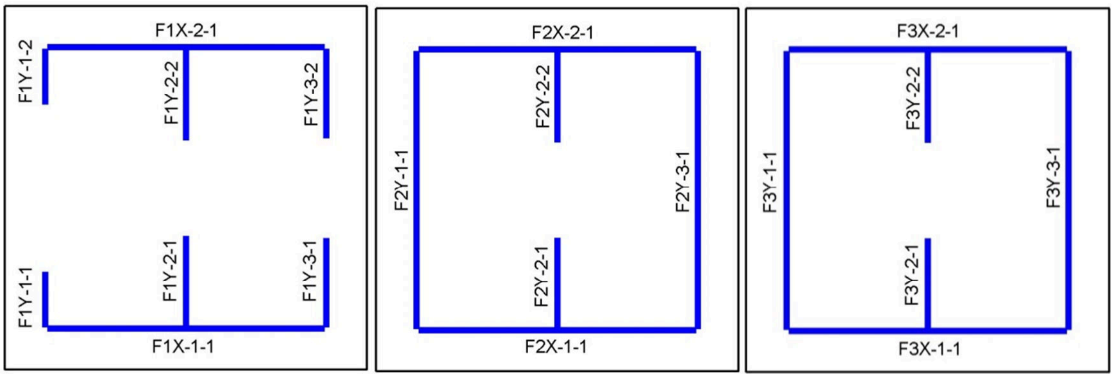


Figure 3-37 Wall IDs of structure for first (left), second (middle) and third (right) floor (after Follesa et al., 2013).

3.2.6.1.5 Preliminary analysis and design of the building

Before the final design of the building, a preliminary design is conducted using the lateral force method of analysis (Follesa et al., 2013). The natural vibration period of the structure is calculated from the code equation shown in Equation (1), which in this case gives:

$$T_1 = C_T \cdot h^{3/4} = 0.05 \cdot 10^{3/4} = 0.28\,\text{s} \tag{16}$$

The purpose of the preliminary design is to provide all the necessary information needed to define the appropriate material and section properties of the elements used in the final finite element model. The lateral stiffness of the connections of the wall panels to the floor diaphragm below is assumed to be the same per linear meter for all wall assemblies of the structure. A value of 20 MN/m/m is used, based on engineering judgement, to calculate the required cross section of the pair of horizontal springs of each wall assembly according to Equation (13). Similarly, vertical step joints between panels of the same wall assembly are considered to provide the same generic stiffness per linear meter of 5 MN/m/m and the required cross section of each pair of vertical springs is computed according to Equation (14).

The results of the preliminary analysis in terms of shear per unit length for each wall and shear per unit height for each vertical step joint are presented in Table 3-5. The storey shear forces are 206.2 kN, 165.0 kN and 78.8 kN for the first, second and third storey, respectively, with a correction factor λ = 0.85 applied according to Section 4.3.3.2.2 of EC8 (CEN, 2004).

Table 3-5 Preliminary analysis and design results (after Follesa et al., 2013).

Wall ID	Length (m)	Height (m)	Shear per unit length (kN/m)	Vert. joint shear per unit height (kN/m)	No of ABs	AB stiffness (kN/m)	Vert. joint screw spacing (cm)	Vert. joint screw stiffness (kN/m)
P1X-1-1	6.94	2.95	16.82	9.44	11	362260.8	10.0	16242.5
P1X-2-1	6.94	2.95	16.82	9.44	11	362260.8	10.0	16242.5
P1Y-1-1	1.47	2.95	20.50	N/A	3	98798.4	N/A	N/A
P1Y-1-2	1.47	2.95	20.53	N/A	3	98798.4	N/A	N/A
P1Y-2-1	2.34	2.95	16.92	N/A	4	131731.2	N/A	N/A
P1Y-2-2	2.34	2.95	16.92	N/A	4	131731.2	N/A	N/A
P1Y-3-1	2.29	2.95	17.55	N/A	4	131731.2	N/A	N/A
P1Y-3-2	2.29	2.95	17.55	N/A	4	131731.2	N/A	N/A
P2X-1-1	6.94	2.95	12.85	10.94	16	191609.6	7.5	53615.7
P2X-2-1	6.94	2.95	12.85	10.94	16	191609.6	7.5	53615.7
P2Y-1-1	6.94	2.95	9.97	8.76	12	143707.2	10.0	16242.5
P2Y-2-1	2.34	2.95	8.81	N/A	4	47902.4	N/A	N/A
P2Y-2-2	2.34	2.95	8.81	N/A	4	47902.4	N/A	N/A
P2Y-3-1	6.94	2.95	9.92	8.11	12	143707.2	10.0	16242.5
P3X-1-1	6.94	3.40	6.14	8.19	8	95804.8	10.0	18338.3
P3X-2-1	6.94	3.40	6.14	8.19	8	95804.8	10.0	18338.3
P3Y-1-1	6.94	2.75	4.78	9.47	8	95804.8	10.0	15194.6
P3Y-2-1	2.34	3.70	4.15	N/A	4	47902.4	N/A	N/A
P3Y-2-2	2.34	3.70	4.15	N/A	4	47902.4	N/A	N/A
P3Y-3-1	6.94	2.75	4.76	8.15	8	95804.8	10.0	15194.6

Two types of angle brackets (AB) are used in the design of the building, which are manufactured by Rotho Blaas (2012): the WVS 90110 that is used for the shear-transferring connections of the 1st storey, and the WB90 that is used for the connections of the upper storeys. Table 3-6 summarizes the main properties of the angle brackets. The vertical step joints between panels of the same wall assembly are constructed with a single plywood strip 27x150 mm inserted into a groove in the internal side of the wall panels, as shown in Figure 3-10b, and nails ϕ2.8x80 that provide a design strength of 1.0 kN and a stiffness of 1047.9 kN/m per connector. Table 3-5 lists the required number of angle brackets and the required screw spacing in the vertical joints for each wall assembly as well as the corresponding stiffness for each type of connection.

Table 3-6 Design strength and stiffness of angle brackets (after Follesa et al., 2013).

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

3.2.6.1.6 Final analysis and design of the building

Based on the results of the preliminary design, the numerical model is updated with the correct stiffness contribution of the angle brackets and the vertical step joints to the horizontal and vertical pair of springs, respectively (Follesa et al., 2013). The building is analysed with the modal response spectrum analysis and Table 3-7 shows the fundamental periods and the mass participation factors for each mode shape of the structure. The first mode shape, with a period of 0.27 s that is similar to the 0.28 s computed with the code equation, is related to translation along the N-S direction of the building. The second mode shape, with a period of 0.24 s, is related to translation along the E-W direction of the building.

Table 3-7 Modal analysis results (after Follesa et al., 2013).

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

The results of the response spectrum analysis in terms of shear per unit length, uplift force for each wall, and shear per unit height for each vertical step joint are presented in Table 3-8. The storey shear forces along the N-S direction are 205.6 kN, 165.0 kN and 80.4 kN for the first, second and third storey, respectively, while along the E-W direction are 207.2 kN, 163.6 kN and 79.2 kN. These values are very similar to the values obtained from the lateral force method of analysis.

Two types of hold-downs (HDs), the WHT 340 and the WHT 440, which are manufactured by Rotho Blaas (2012), are used to restrain the building uplift. In addition, a steel strap (SS) with dimensions of 100x1000x1.5 mm is used as a tie-down to restrain external wall assemblies of the second and third storey against uplift.

Table 3-9 summarizes the main properties of the hold-downs and tie-downs. Self-tapping screws HBS ϕ8x300 and ϕ8x200 are prescribed for the connection of the floor and the roof panels, respectively, to the supporting wall panels. The design strength per screw connector is 4.56 kN and 4.24 kN for the ϕ8x300 and ϕ8x200, respectively. These screwed connections were designed for a force equal to the shear force of the supporting wall multiplied by an overstrength factor of 1.6. Self-tapping screws HBS ϕ8x200, with a design strength of 4.24 kN, are also prescribed for the connection between perpendicular walls. This type of connection is designed for a force equal to the maximum shear force per unit length between the two walls multiplied by the height of the connection and further multiplied by an overstrength factor of 1.6. Table 3-8 shows the required number of ABs and the type of hold-downs/tie-downs for each wall assembly, as well as the required spacing for the screws in the vertical step joints, the wall-to-floor connections and the perpendicular wall connections.

Table 3-8 Final analysis and design results (after Follesa et al., 2013).

Wall ID	Shear per unit length (kN/m)	Uplift (kN)	Vert. joint shear per unit height (kN/m)	No of ABs	Vert. Joint screw spacing (cm)	Type of hold-down/tie down	Wall-to-floor connection spacing (cm)	Perpendicular wall connection spacing (cm)
P1X-1-1	17.02	42.55	5.75	11	15.0	WHT440	15.0	10.0
P1X-2-1	17.02	42.55	5.75	11	15.0	WHT440	15.0	10.0
P1Y-1-1	22.52	34.84	N/A	3	N/A	WHT340	10.0	10.0
P1Y-1-2	22.56	34.84	N/A	3	N/A	WHT340	10.0	10.0
P1Y-2-1	17.52	34.01	N/A	4	N/A	WHT340	10.0	10.0
P1Y-2-2	17.52	34.02	N/A	4	N/A	WHT340	10.0	10.0
P1Y-3-1	15.90	46.61	N/A	4	N/A	WHT440	10.0	10.0
P1Y-3-2	15.90	46.61	N/A	4	N/A	WHT440	10.0	10.0
P2X-1-1	13.29	8.09	9.78	16	10.0	SS	20.0	15.0
P2X-2-1	13.29	8.09	9.78	16	10.0	SS	20.0	15.0
P2Y-1-1	10.41	24.25	5.67	13	15.0	SS	20.0	15.0
P2Y-2-1	8.42	4.65	N/A	4	N/A	WHT340	20.0	20.0
P2Y-2-2	8.42	4.65	N/A	4	N/A	WHT340	20.0	20.0
P2Y-3-1	9.29	14.65	4.69	12	15.0	SS	20.0	15.0
P3X-1-1	6.38	0.00	6.57	8	15.0	SS	20.0	20.0
P3X-2-1	6.38	0.00	6.57	8	15.0	SS	20.0	20.0
P3Y-1-1	4.83	0.00	8.19	8	10.0	SS	20.0	20.0
P3Y-2-1	5.22	7.97	N/A	4	N/A	WHT340	20.0	20.0
P3Y-2-2	5.22	7.97	N/A	4	N/A	WHT340	20.0	20.0
P3Y-3-1	4.18	0.00	6.39	8	15.0	SS	20.0	20.0

Table 3-9 Design strength and details of hold-downs and tie-downs (after Follesa et al., 2013).

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

3.2.6.2 Validation of the Numerical Model

In order to validate the numerical modelling approach, the experimental results from three low intensity shaking Table 3-tests of a three-storey full scale CLT building (Ceccotti and Follesa, 2006) are compared with the numerical predictions from linear dynamic analyses of a representative numerical model (Follesa et al., 2013).

The test structure was tested with three different configurations, as shown in Figure 3-37, where the differences were the openings at the first floor along the W-E direction, which is the direction of shaking. The third configuration is identical to the structure considered in the case study building designed in the previous sections. The first configuration, considered in this case for the validation of the numerical model, incorporated a 1.2 m long door opening at the middle of the wall assemblies.

The ground motion records used for the shaking table tests are listed in Table 3-10.

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

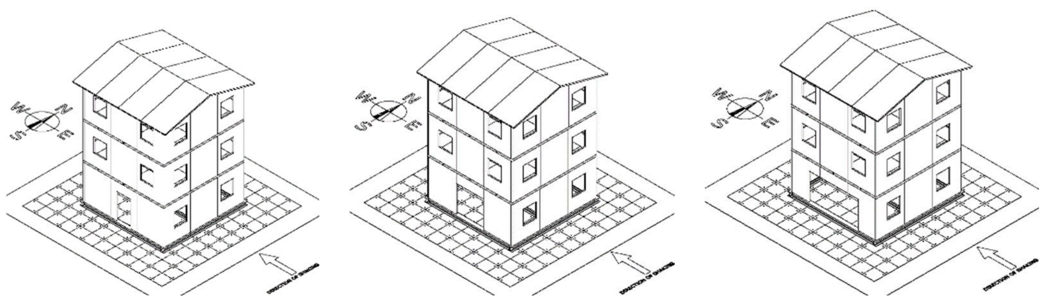


Figure 3-38 First (left), second (middle) and third (right) configuration of the test structure (after Ceccotti and Follesa, 2006).

Table 3-10 Details of the original ground motions used in the shaking table tests (after Ceccotti and Follesa, 2006).

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

3.2.6.2.1 Test structure and numerical model

The thickness and lay-up of wall and floor panels of the test structure were identical to the case study building described above and the orthotropic material properties considered in the model are shown in Equation (14) (Follesa et al., 2013). The details of the shear-transferring connections and the vertical step joints of the test building, which are those schematized in the numerical model, are listed in Table 3-11, while Table 3-12 lists the corresponding stiffness values of the connections according to the above described procedure. Steel blocks were anchored on each floor diaphragm to augment the gravity and seismic load and account for permanent and live loads in seismic combination as prescribed by Eurocode 8. The dead and additional loads of the test structure are listed in Table 3-13. Note that the dead loads are calculated for each storey by considering the floor panels, half of the lower walls and half of the upper walls. The undeformed shape of the FE model is shown in Figure 3-39.

Table 3-11 Details of the horizontal and vertical connections used in the test building (after Follesa et al., 2013).

Storey	Wall to floor horizontal connection for each floor	Vertical joint between wall panels
1	6 BMF 90x48x3,0x116 for each wall assembly, connected to the wall with 8 ϕ4x60 anker nails and to the foundation with one ϕ16 anchor bolt	Self tapping screws ϕ8x80 spaced at 450 mm c/c
2	4 BMF 105 for each wall assembly, connected to the wall and to the floor with 8 ϕ4x60 anker nails.	Self tapping screws ϕ8x80 spaced at 600 mm c/c
3	3 BMF 105 for each wall assembly, connected to the wall and to the floor with 5 ϕ4x60 anker nails.	Self tapping screws ϕ8x80 spaced at 900 mm c/c

Table 3-12 Total stiffness calculated for the horizontal and vertical springs (after Follesa et al., 2013).

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

Table 3-13 List of dead and additional weights used in the test building (after Follesa et al., 2013).

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

Figure 3-39 Undeformed shape of the FE model (after Follesa et al., 2013).

3.2.6.2.2 Comparison with the test results

A linear modal analysis was carried with a modal damping of 5% constant for all modes, to analyse the numerical model under the three recorded shake table motions that were scaled to a peak acceleration of 0.15g (Follesa et al., 2013). The comparison between the model and the test results is made in terms of natural period, maximum base shear and maximum displacements relative to the base measured at the centre of each storey. The experimental results and the numerical predictions are presented in Table 3-14.

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

Table 3-14 Comparison between model and experimental results (after Follesa et al., 2013).

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

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

$$F_f = \mu_k \cdot W_i \qquad (17)$$

The yield displacement is calculated by considering the yield displacement of the weakest fastener of the shear-transferring connection, i.e. the ϕ4 anker nails used in the connection of the angle brackets. This may be evaluated, in the absence of test results, by dividing the characteristic strength of the nail $R_{c,k}$, calculated according to EC5 with the embedment strength of wood taken from the equations provided by Uibel and Blaß [2006 and 2007], by the slip modulus under service load of Equation (10), i.e.:

$$u_y = \frac{R_{c,k}}{2 \cdot \rho_m^{1.5} \cdot d_f / 23} = \frac{2370}{2994} = 0.79\,mm \qquad (18)$$

Therefore, the stiffness due to friction is evaluated as:

$$K_f = \frac{F_f}{u_y} \tag{19}$$

Adding the contribution of friction to the stiffness provided by the angle brackets, the total horizontal stiffness of each wall assembly was recalculated and the results are shown in Table 3-15.

Table 3-15 Total stiffness calculated for the horizontal and vertical springs taking into account the friction contribution (after Follesa et al., 2013).

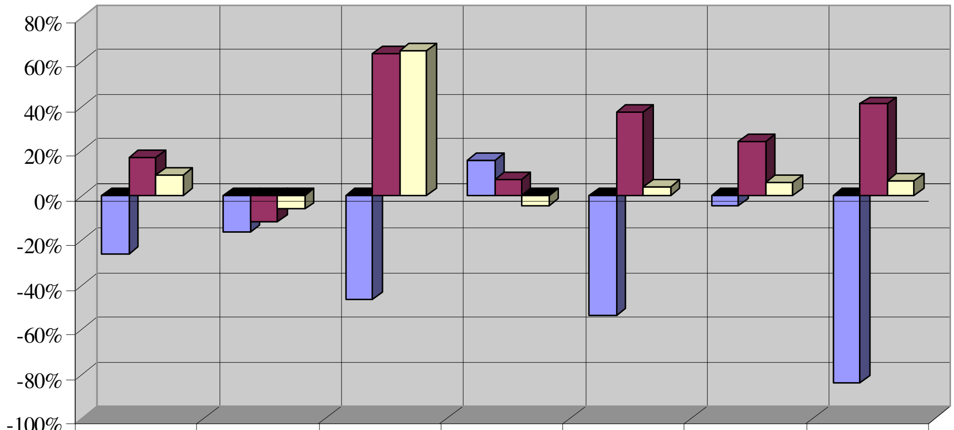
Storey	Total stiffness for angle bracket connection (kN/m)	Total stiffness for vertical step joint (kN/m)
1	K_1=197607 (wall length 6.935 m - 6 brackets); K_2=154189 (wall length 2.34 m - 3 brackets); K_3=124494 (wall length 2.868 m - 3 brackets)	K=10479 (wall height 2.95 m - 7 screws)
2	K_1=79252 (wall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

Table 3-16 shows the comparison with the updated numerical model with the friction contribution. The predicted natural period is still higher than the experimental one (0.182 s versus 0.166 s) but apart from the displacement predictions for Kobe earthquake, a very good correlation with the experimental results is obtained, thus leading to the conclusion that the friction contribution should be taken into account in the calculation of the horizontal stiffness.

The described modelling approach, where all the horizontal wall-to-floor and vertical panel-to-panel connections are schematized with equivalent truss elements, may require a long input procedure especially for large and tall buildings. For this reason, sometimes simplified numerical models where all the joints between CLT elements are considered as rigid are used by designers. In order to quantify the effect of modelling the flexibility of the mechanical connections, a third numerical model was generated without any connections among the shell elements. Figure 3-40 illustrates the comparison between the experimental results and the numerical predictions from all three numerical models. As it can be observed the differences from the experimental values of the third model are larger, especially in terms of natural periods and maximum displacements. This difference in the natural period can lead to significant differences in the calculation of the design seismic load, and confirms results of previous research (Fragiacomo et al. 2011).

Table 3-16 Comparison between model with friction contribution and experimental results (after Follesa et al., 2013).

	Period (s)	Base shear (kN)	3rd storey (roof) displacement (mm)	2nd storey displacement (mm)	1st storey displacement (mm)
Kobe 0.15g					
Model	0.182	82.3	2.54	1.75	0.76
Experimental	0.166	87.6	1.54	1.29	0.49
Difference (%)	9.64%	-6.03%	65.32%	35.38%	56.65%
El Centro 0.15g					
Model	0.182	123.3	4.05	2.74	1.15
Experimental	0.166	129.1	3.90	2.80	1.15
Difference (%)	9.64%	-4.48%	3.91%	-2.00%	-0.27%
Nocera Umbra 0.15g					
Model	0.182	165.90	5.41	3.69	1.55
Experimental	0.166	156.08	5.08	3.62	1.50
Difference (%)	9.64%	6.29%	6.54%	2.04%	3.00%



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

Figure 3-40 Percentage difference between numerical models and experimental results (after Follesa et al., 2013).

3.2.7 Non-linear dynamic analysis of CLT buildings with SAP2000

As explained in §3.2.5, SAP2000 (CSI, 2000) can be used to perform non-linear analysis of CLT buildings by modelling CLT walls as either layered shell elements or by using the orthotropic model, i.e. by assigning the element an equivalent homogenized section with different properties in different directions. Hold-down, angle brackets and screwed joints may be schematized by using the multi-linear plastic link elements, which although not capable to simulate correctly the strength degradation phenomenon in mechanical joints could be successfully used to simulate the non-linear behaviour of mechanical joints in timber structures as in the procedure described in this paragraph.

3.2.7.1 Calibration process for connections

The calibration process of the numerical model is made by making reference to the results of an experimental programme conducted by I. Gavric at CNR-Ivalsa laboratory in San Michele all'Adige (Gavric, 2013a) on the same connectors (hold-down, angle brackets and screws) used for a three-storey CLT building tested in 2006 on the NIED shaking table within the SOFIE project test (Ceccotti and Follesa, 2006).

For the calibration procedure results from cyclic tests on hold-down type HTT22 with 12 4x60 Anker nails (test 1-CN-08), and angle brackets BMF 90x48x3x116 with 11 4x60 Anker nails (test 5-CN-03 for uplift and test 7-CS-03 and 7-CS-06 for slip) were considered.

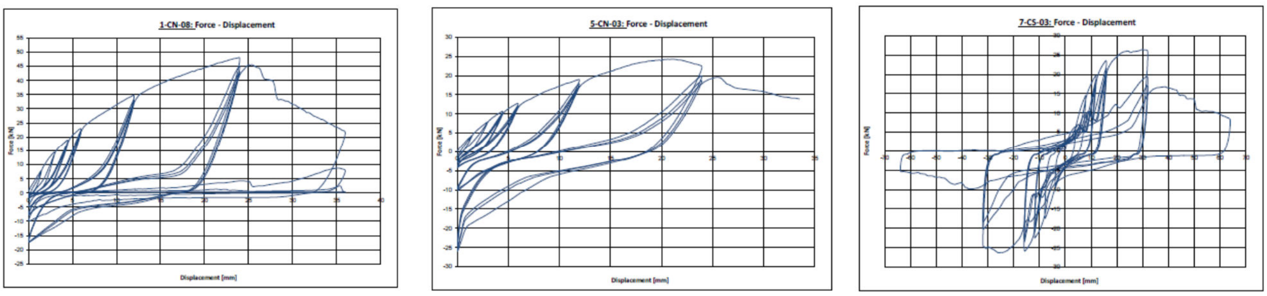


Figure 3-41 Force-uplift behaviour of hold-downs (left), angle brackets (center) and force-slip behaviour of angle brackets (right) taken into account for the calibration process (after Gavric, 2013a).

The calibration process was made by using at first the calibration program Sophie 4.52 developed by G. Rinaldin (Rinaldin, 2013) using both the scan and the genetic calibration process in terms of minimum dissipated energy and then "manually adjusting" the backbone curves and the hysteresis parameters in order to obtain both a better fitting to the test data and at the same time trying to keep at minimum the average difference of dissipated energy. Note that the calibration process was made in this way because the aim was to keep as close as possible the dissipated energy time-history curves rather than obtaining a closer value of final dissipated energy while trying to keep at the same time an acceptable fitting of the hysteresis curves to the test data.

3.2.7.1.1 Hold-down HTT22 with 12 4x60 Anker nails (test 1-CN-08)

Figure 3-42 and Figure 3-43 display the results of the calibration process with Sap2000 using the Pivot hysteresis rule for hold-downs based on the force-uplift relationship showed in Figure 3-41 (left). Hold-downs were schematized as two joints link elements.

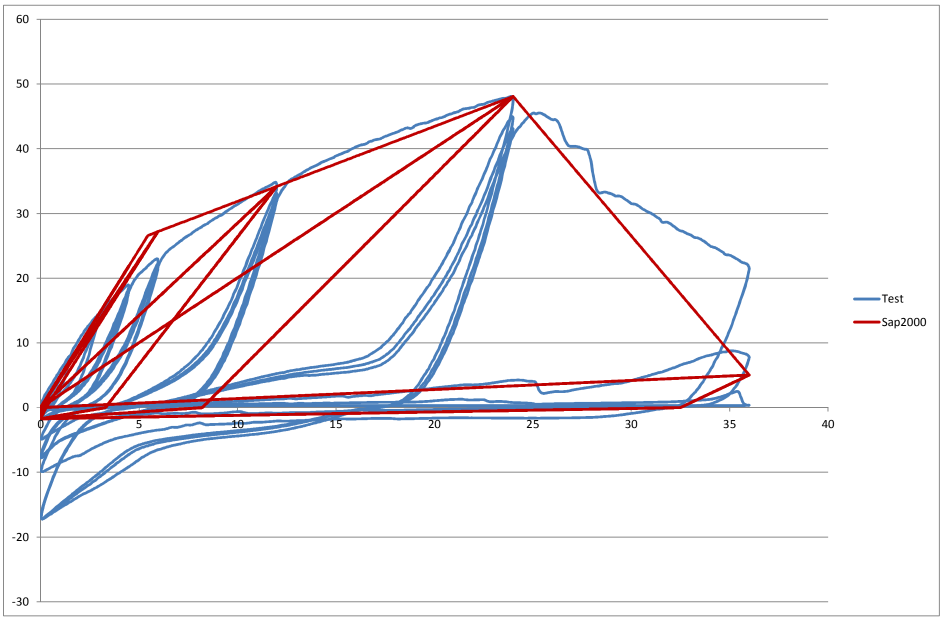


Figure 3-42 Calibration of hold-downs. Comparison in terms of hysteresis curves.

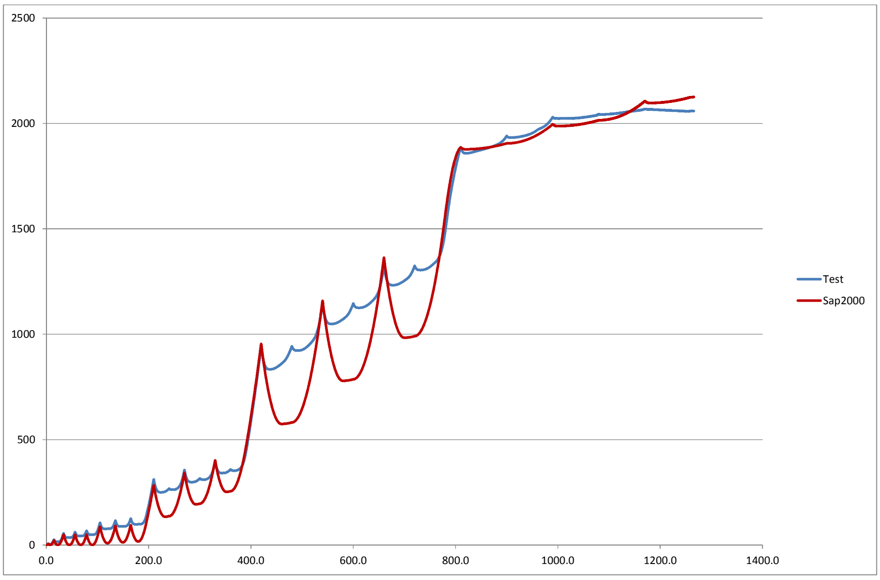


Figure 3-43 Calibration of hold-downs. Comparison in terms of dissipated energy.

The final difference in terms of dissipated energy was 3.2%. The calibration was obtained using the following values (see Figure 3-44) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements.

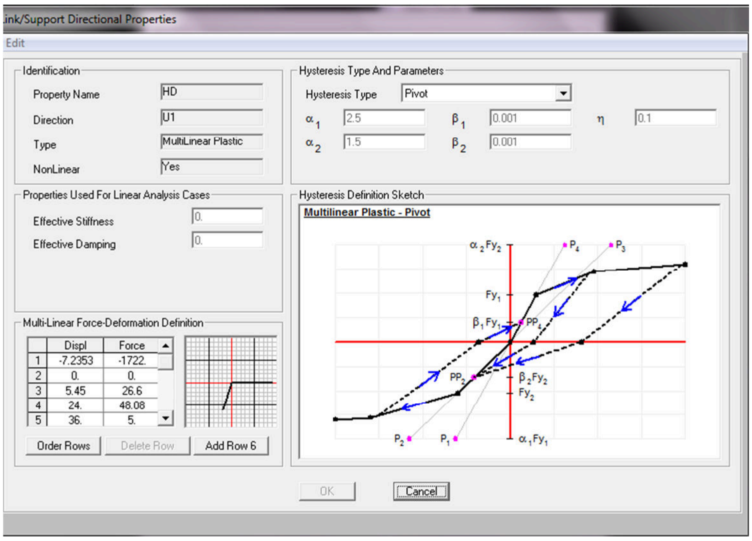


Figure 3-44 Values of hysteresis parameters used for hold-downs.

3.2.7.1.2 Hold-down WHT440 with 9 4x60 Anker nails (test 2-CS-01)

For the calibration of 2nd and 3rd floor hold-downs the WHT440 tests with 9 4x60 Anker nails were chosen. Figure 3-46 and Figure 3-47 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the shear resistance of angle brackets based on the force-uplift relationship showed in Figure 3-45. Hold-downs were schematized as two joints link elements.

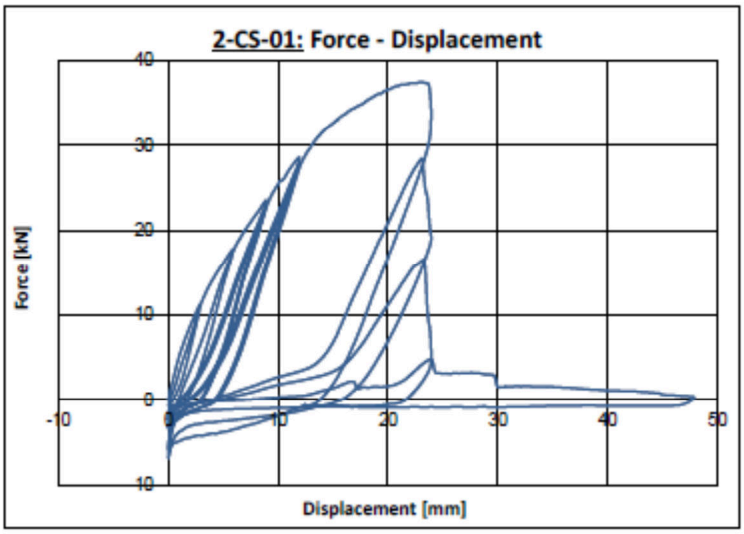


Figure 3-45 Force- slip behaviour of angle brackets taken into account for the calibration process (after Gavric, 2013a).

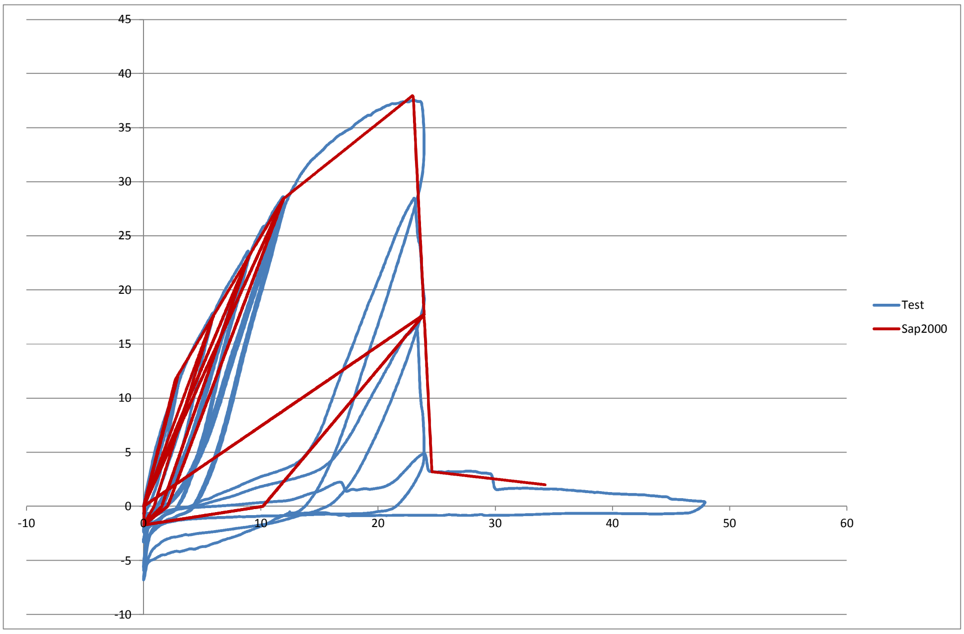


Figure 3-46 Calibration of hold-down. Comparison in terms of hysteresis curves.

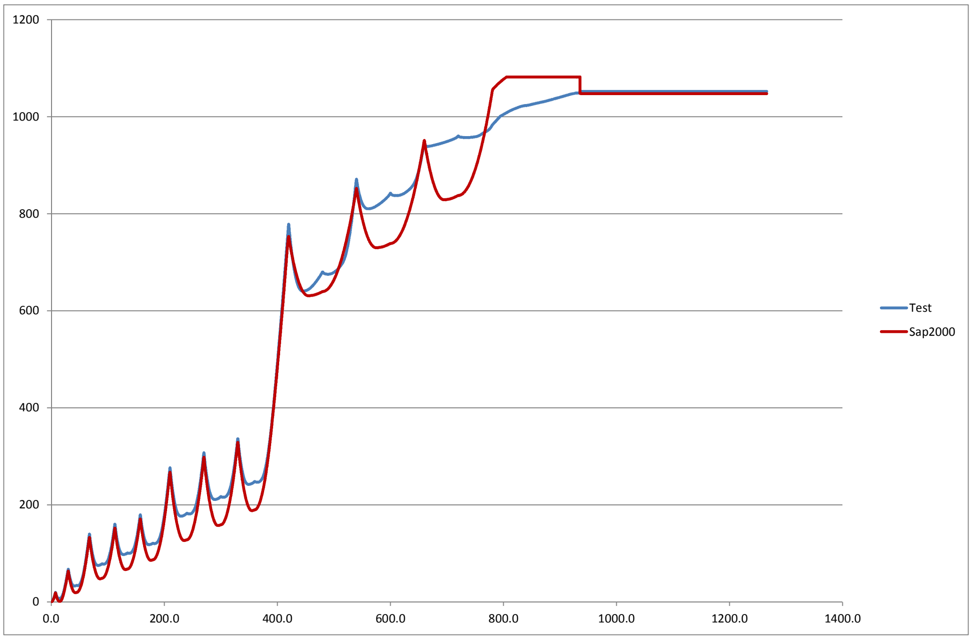


Figure 3-47 Calibration of hold-down. Comparison in terms of dissipated energy.

The calibration was obtained using the following values (see Figure 3-48) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements. The final difference in terms of dissipated energy was 0.42%.

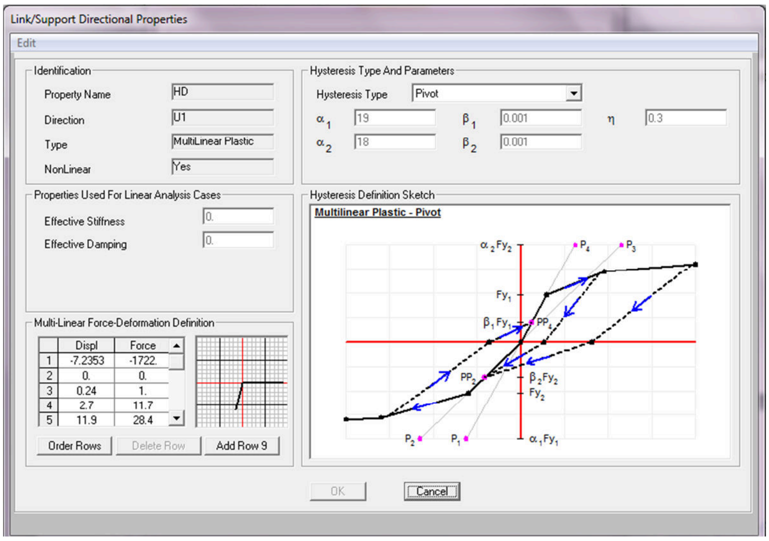


Figure 3-48 Values of hysteresis parameters used for hold-downs.

3.2.7.1.3 Angle brackets BMF 90x48x3x116 with 11 4x60 Anker nails (test 5-CN-03) - Uplift

Figure 3-49 and Figure 3-50 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the uplift resistance of angle brackets based on the force-uplift relationship showed in Figure 3-41 (center). Angle brackets were schematized as two joints link elements.

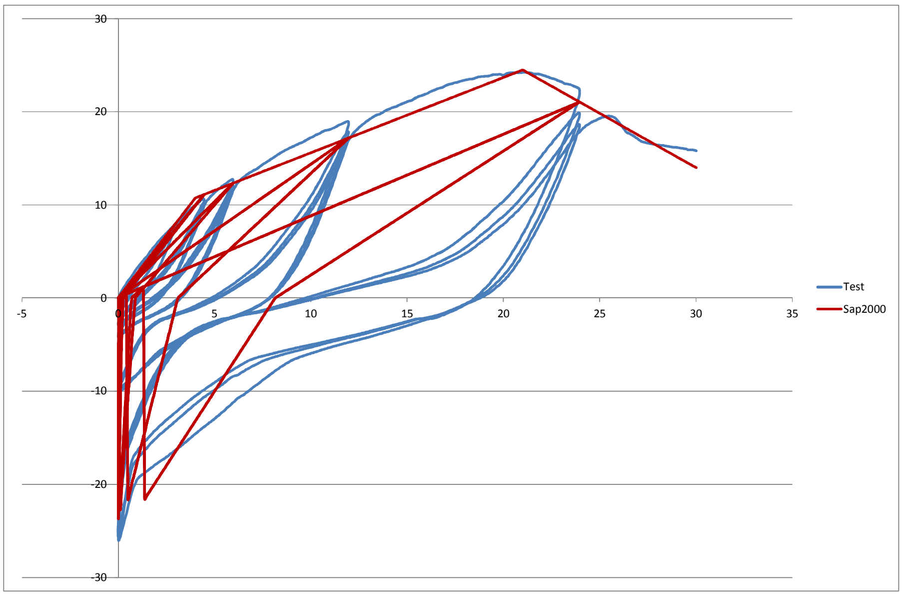


Figure 3-49 Calibration of angle brackets for uplift. Comparison in terms of hysteresis curves.

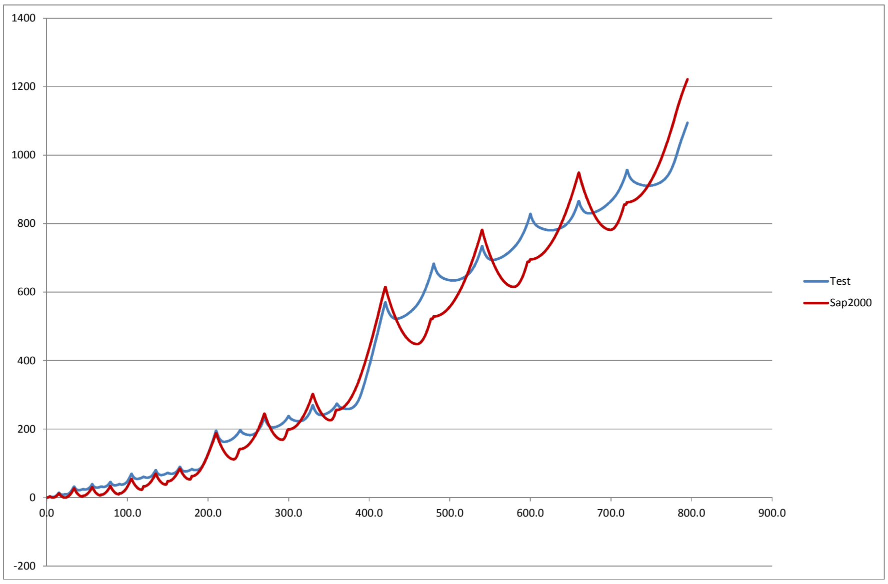


Figure 3-50 Calibration of angle brackets for uplift. Comparison in terms of dissipated energy.

The final difference in terms of dissipated energy was 11.6%.

The calibration was obtained using the following values (see Figure 3-51) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements.

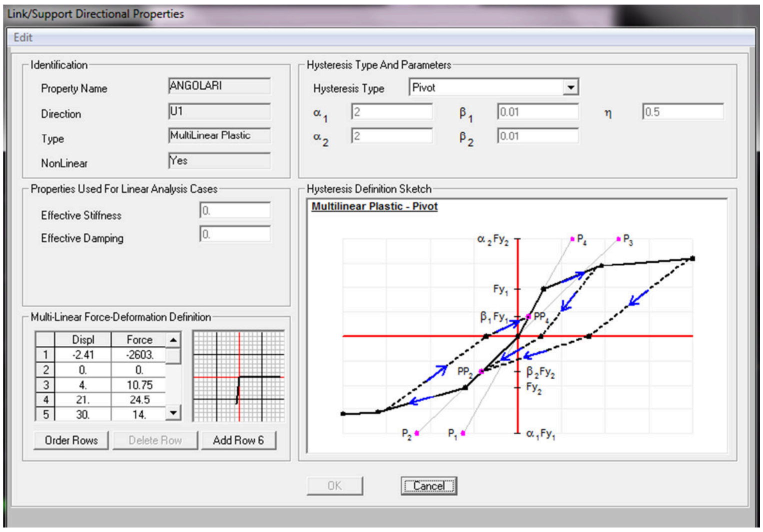


Figure 3-51 Values of hysteresis parameters used for uplift resistance of angle brackets.

3.2.7.1.4 Angle brackets BMF 90x48x3x116 with 11 4x60 Anker nails (test 7-CS-03) - Shear

Figure 3-52 and Figure 3-53 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the shear resistance of angle brackets based on the force-slip relationship showed in Figure 3-41 (right). Angle brackets were schematized as two joints link elements.

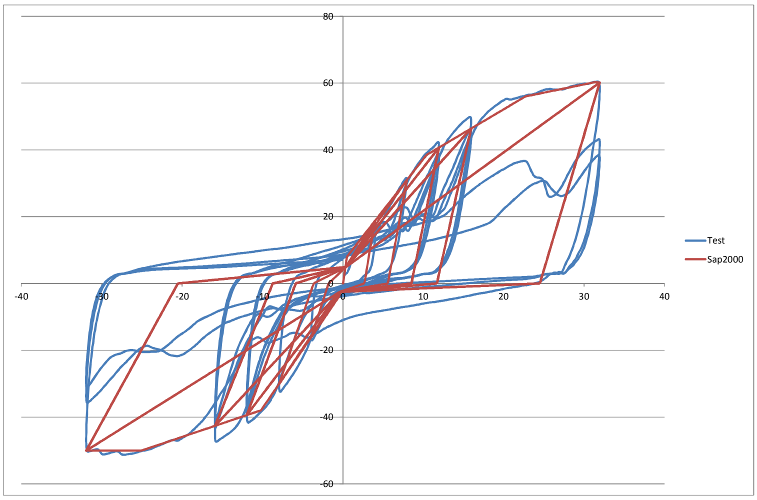


Figure 3-52 Calibration of angle brackets. Comparison in terms of hysteresis curves.

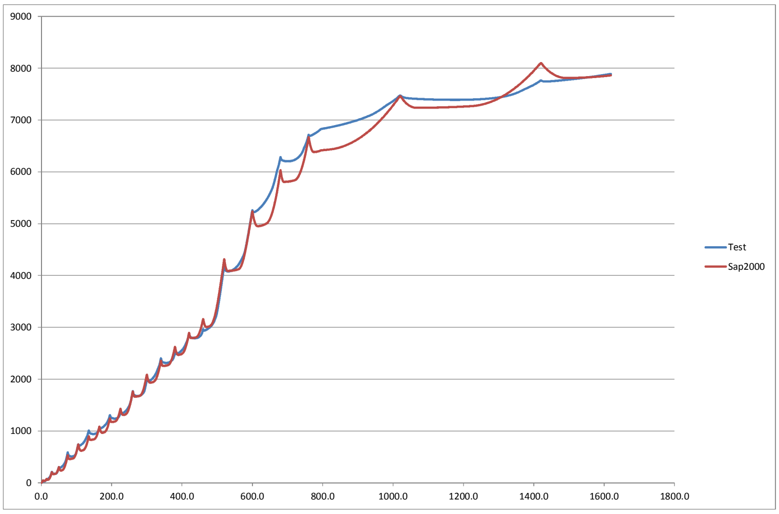


Figure 3-53 Calibration of angle brackets. Comparison in terms of dissipated energy.

The calibration was obtained using the following values (see Figure 3-54) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements. The final difference in terms of dissipated energy was 0.01%.

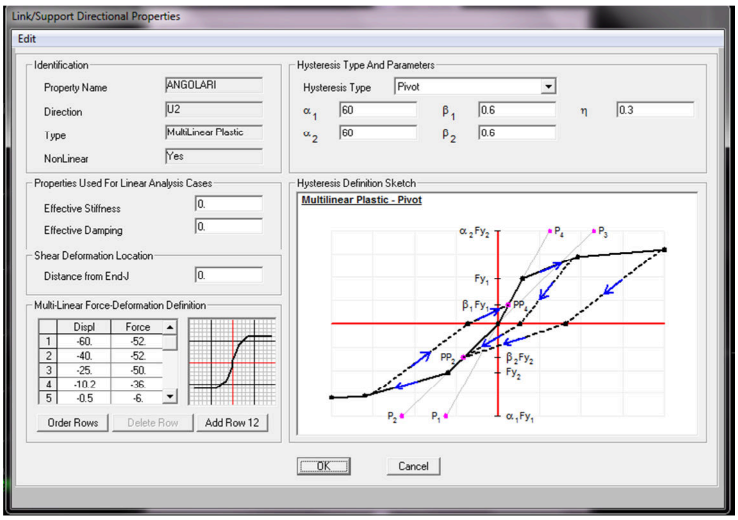


Figure 3-54 Values of hysteresis parameters used for angle brackets.

3.2.7.1.5 Angle brackets BMF 90x48x3x116 with 11 4x60 Anker nails (test 7-CS-06) - Shear

As a further attempt, Figure 3-52 and Figure 3-53 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the shear resistance of angle brackets based on the force-uplift relationship showed in Figure 3-55. Angle brackets were schematized as two joints link elements.

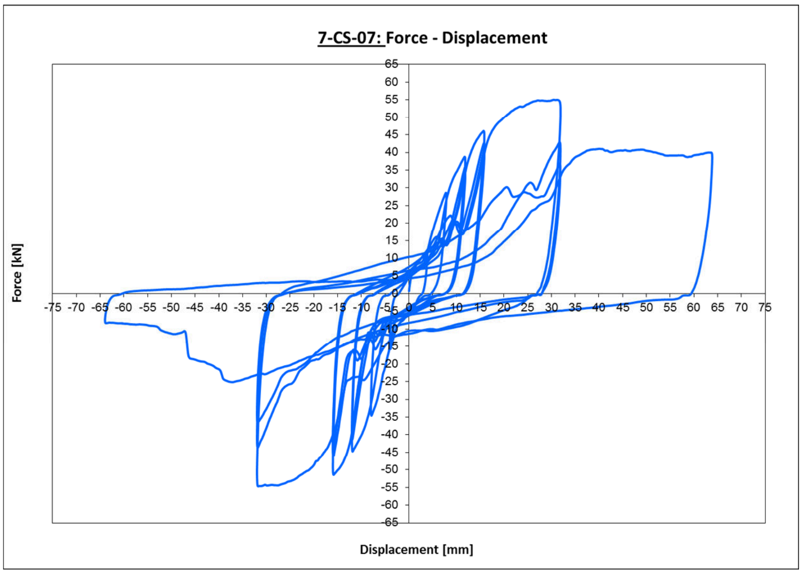


Figure 3-55 Force- slip behavior of angle brackets taken into account for the calibration process (after Gavric, 2013a).

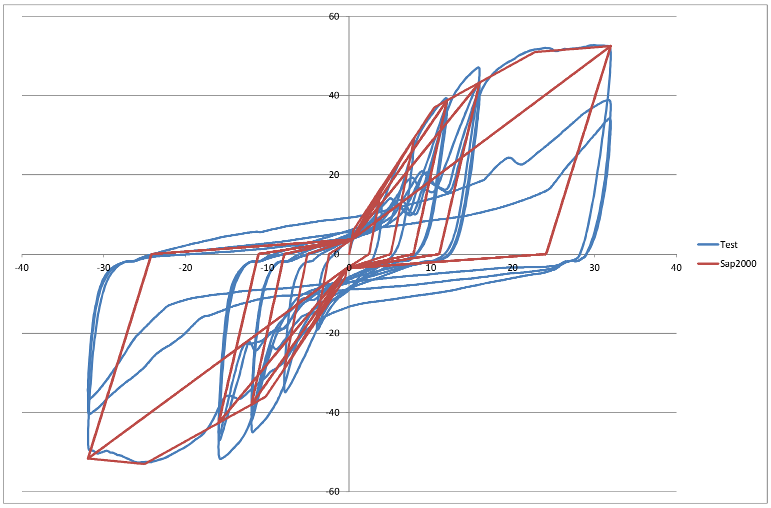


Figure 3-56 Calibration of angle brackets. Comparison in terms of hysteresis curves.

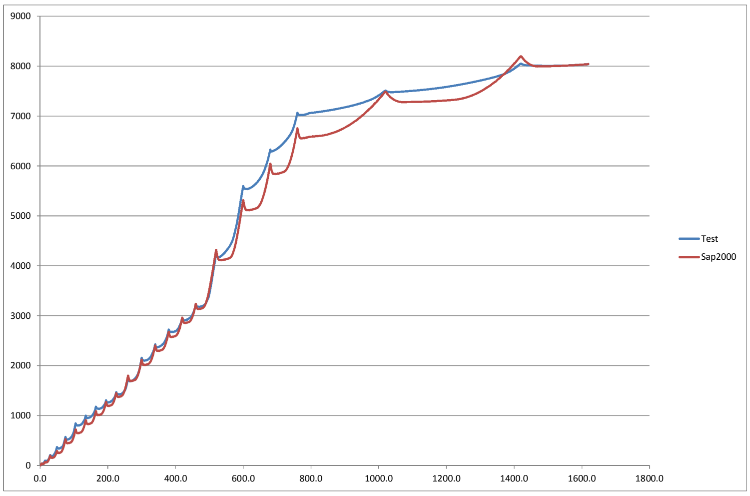


Figure 3-57 Calibration of angle brackets. Comparison in terms of dissipated energy.

The calibration was obtained using the following values (see Figure 3-58) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements. The final difference in terms of dissipated energy was 0.36%.

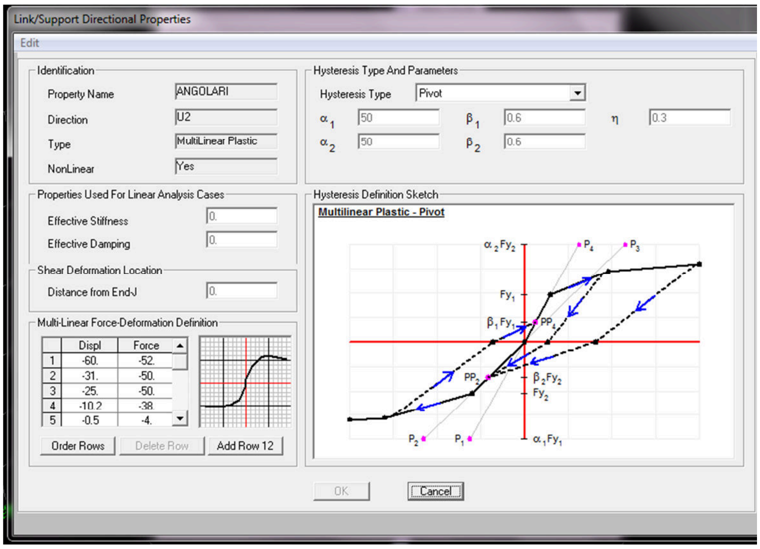


Figure 3-58 Values of hysteresis parameters used for angle brackets.

3.2.7.1.6 Angle brackets BMF 100x100x90x3 with 8 4x60 Anker nails (test 6-CS-01) - Uplift

Figure 3-60 and Figure 3-61 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the uplift resistance of angle brackets BMF 100x100x90x3 with 8 4x60 Anker nails based on the force-uplift relationship showed in Figure 3-59. Angle brackets were schematized as two joints link elements.

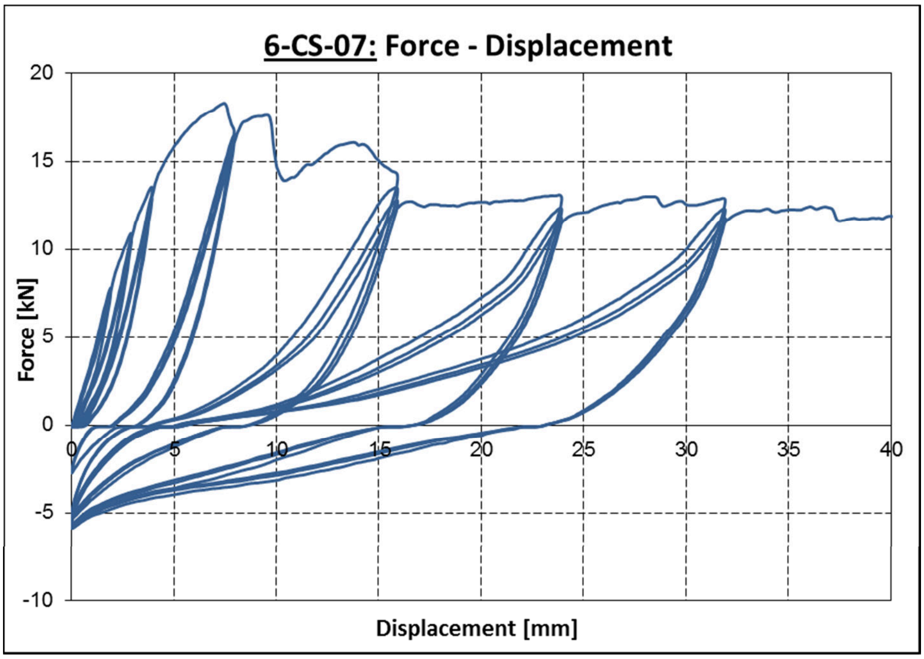


Figure 3-59 Force- slip behaviour of angle brackets taken into account for the calibration process (from Gavric, 2013a).

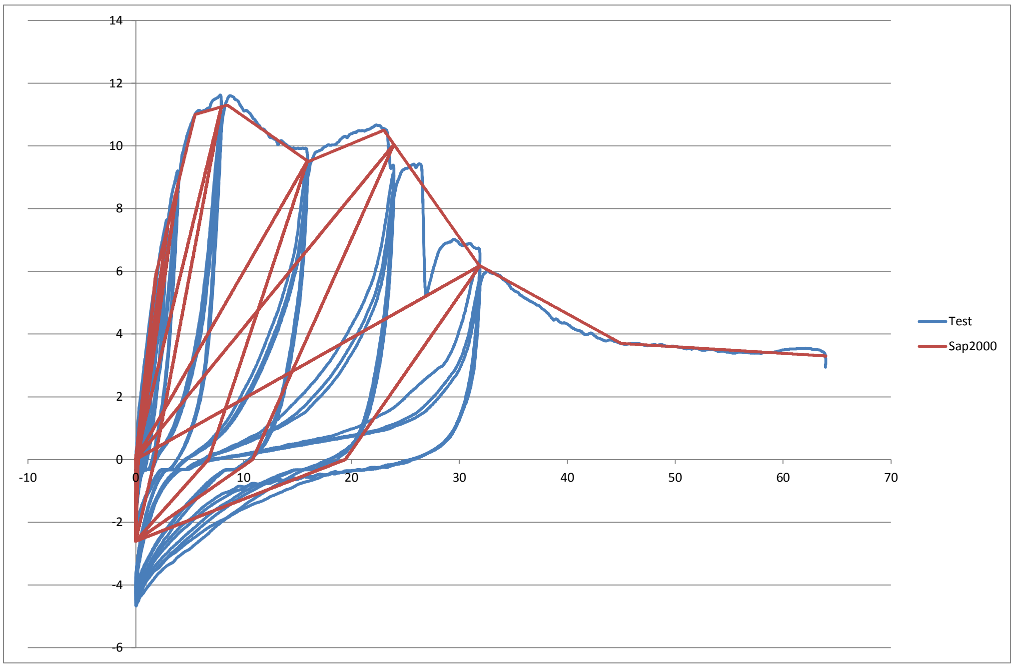


Figure 3-60 Calibration of angle brackets for uplift. Comparison in terms of hysteresis curves.

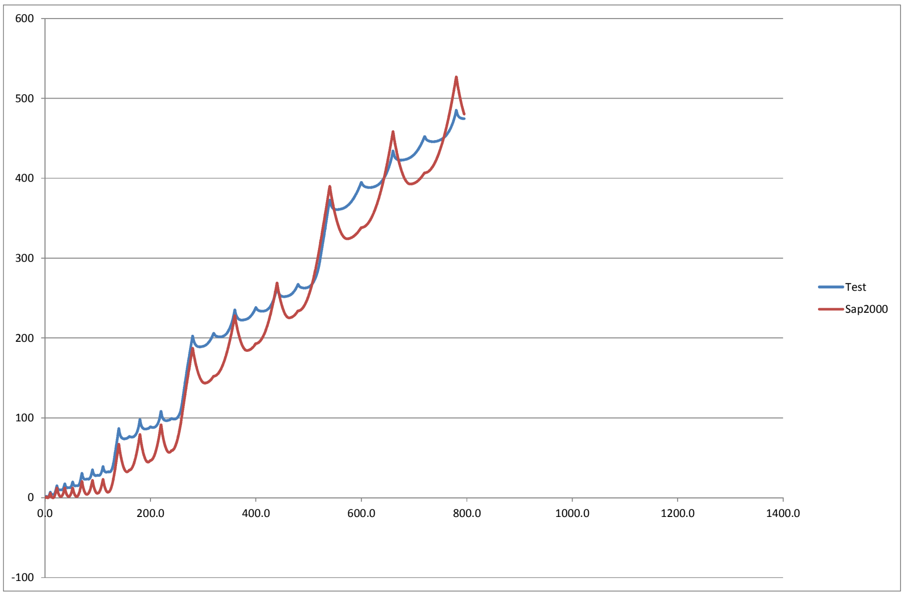


Figure 3-61 Calibration of angle brackets for uplift. Comparison in terms of dissipated energy.

The final difference in terms of dissipated energy was 1.2%.

The calibration was obtained using the following values (see Figure 3-62) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements.

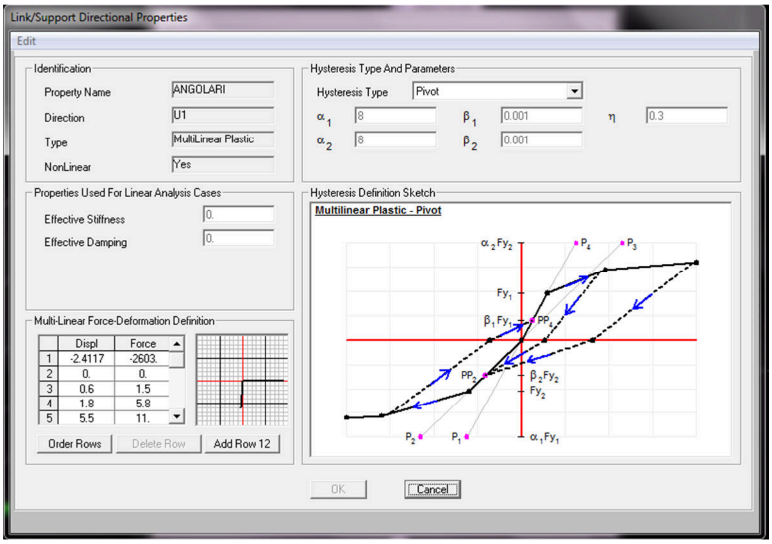


Figure 3-62 Values of hysteresis parameters used for uplift resistance of angle brackets.

3.2.7.1.7 Angle brackets BMF 100x100x90x3 with 8 4x60 Anker nails (test 8-CS-01) - Shear

Figure 3-64 and Figure 3-65 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the shear resistance of angle brackets based on the force-uplift relationship showed in Figure 3-63. Angle brackets were schematized as two joints link elements.

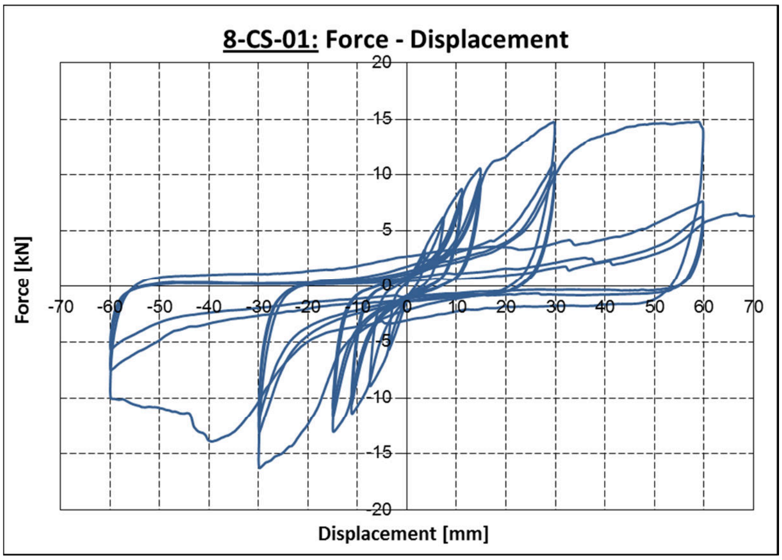


Figure 3-63 Force- slip behavior of angle brackets taken into account for the calibration process (after Gavric, 2013a).

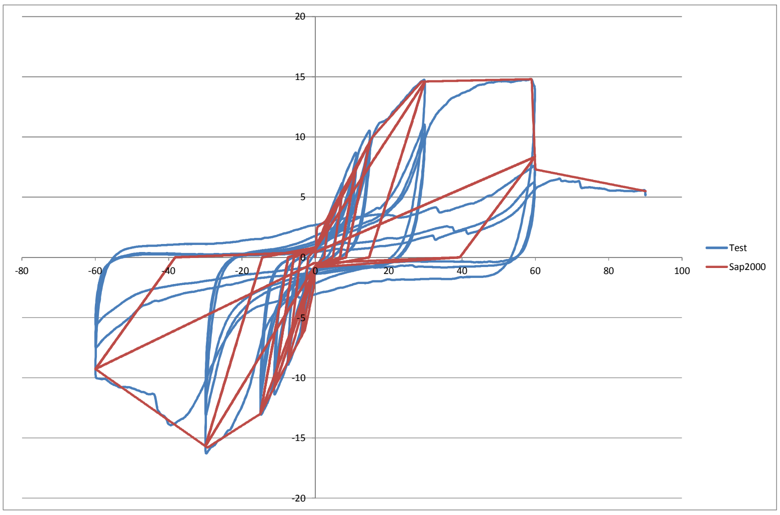


Figure 3-64 Calibration of angle brackets. Comparison in terms of hysteresis curves.

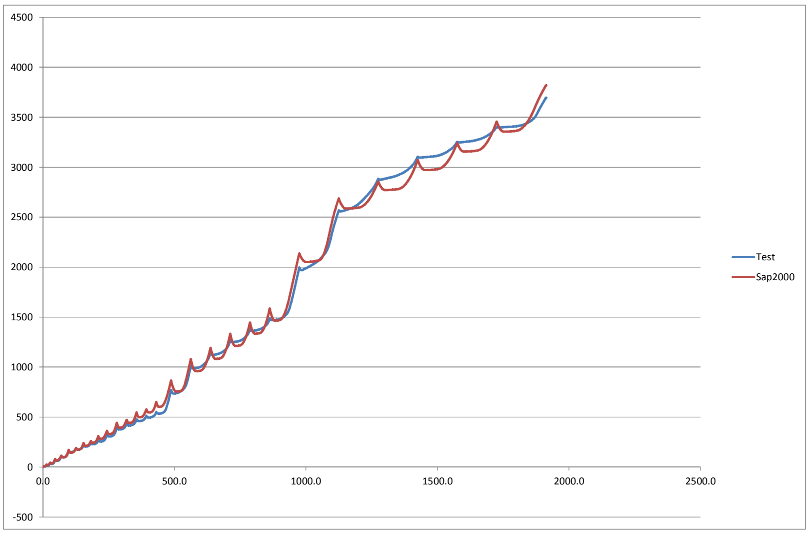


Figure 3-65 Calibration of angle brackets. Comparison in terms of dissipated energy.

The calibration was obtained using the following values (see Figure 3-66) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements. The final difference in terms of dissipated energy was 3.35%.

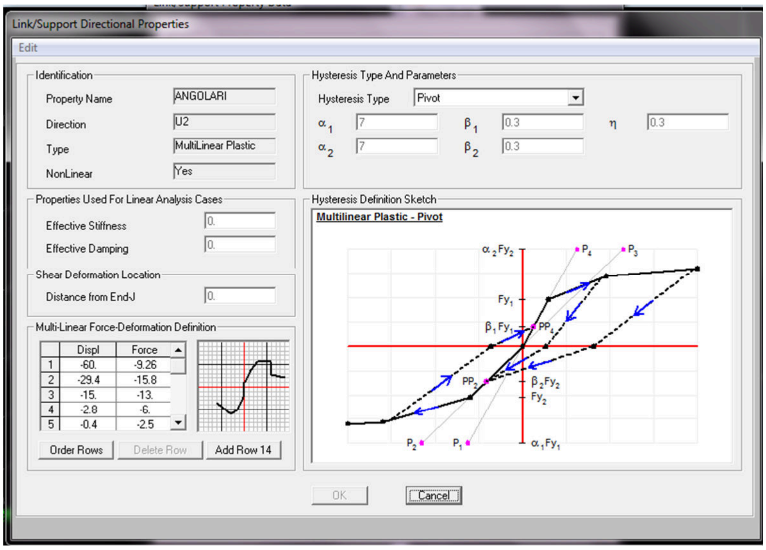


Figure 3-66 Values of hysteresis parameters used for angle brackets.

3.2.7.1.8 Step joint with self-tapping screws HBS 8x80 (test 10-CS-01)

Figure 3-68 and Figure 3-69 display the results of the calibration process with Sap2000 (CSI, 2000) using the Pivot hysteresis rule for the step joint, made with an outer strip of LVL of 27 mm of thickness and HBS 8x80 self-tapping screws, based on the force-slip relationship showed in Figure 3-67. The step joint was schematized as a two joints link element.

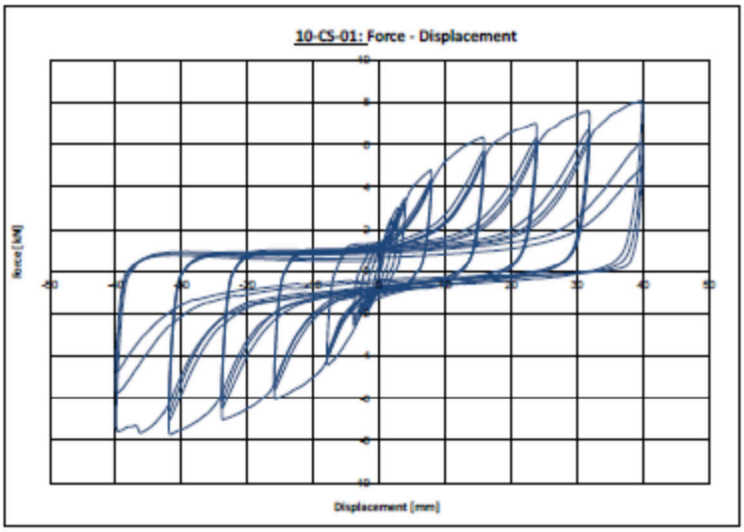


Figure 3-67 Force- slip behavior of step joints taken into account for the calibration process (after Gavric, 2013a).

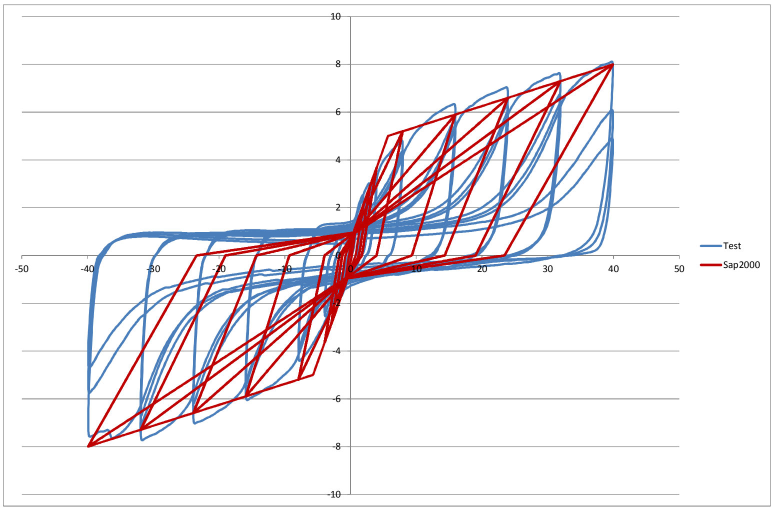


Figure 3-68 Calibration of step joint with self-tapping screws. Comparison in terms of hysteresis curves.

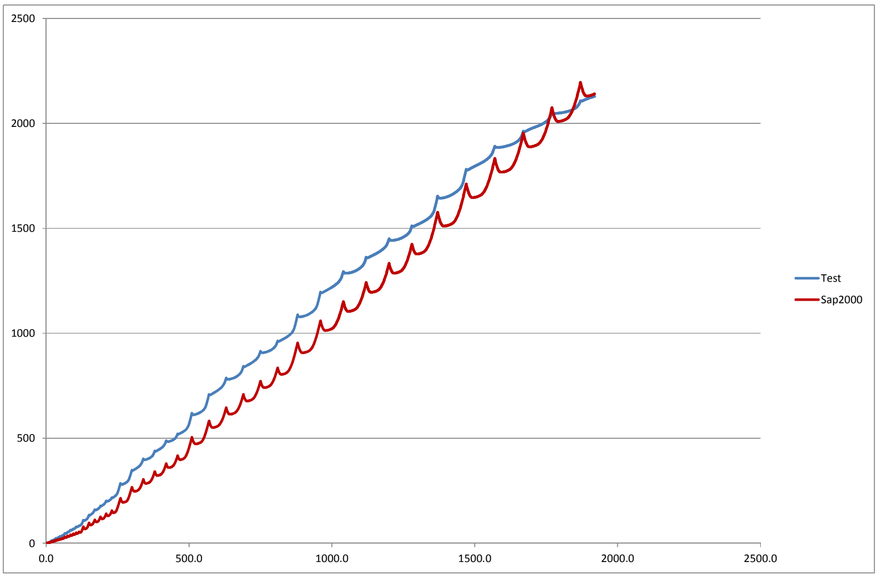


Figure 3-69 Calibration of step joint with self-tapping screws. Comparison in terms of dissipated energy.

The calibration was obtained using the following values (see Figure 3-70) for the unloading and reloading parameters α_1, α_2, β_1, β_2 and η for the non-linear link elements. The final difference in terms of dissipated energy was 0.59%.

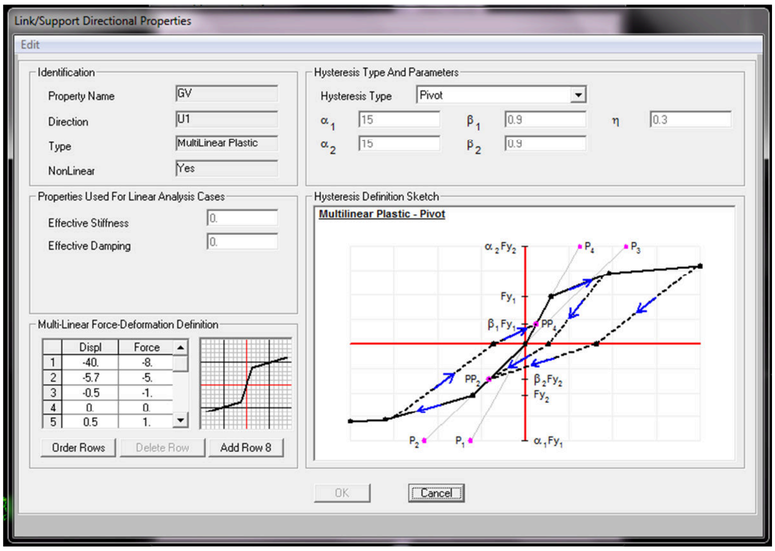


Figure 3-70 Values of hysteresis parameters used for angle brackets.

3.2.7.2 Calibration process for walls

Once completed the calibration procedure for hold-downs and angle brackets, a model for the wall calibration was prepared by making reference to the results of an experimental programme conducted by I. Gavric at CNR-Ivalsa laboratory in San Michele all'Adige (Gavric, 2013b). Shell elements were defined with the same length, height and thickness as the associated CLT wall panels and meshed with a mesh dimension of 0.5x0.5m.

The orthotropic model defined in §3.2.5 and equations 4-7 were used to schematize CLT walls. Hold-downs and angle brackets were placed in the same position used in the test set-up.

Non-linear Gap Link elements were also included at each base node to simulate the compression stiffness of the CLT panel.

The equivalent modulus of elasticity of CLT wall panels in the direction parallel to the grain of the outer layers was calculated for a 5-layer CLT panel as suggested by Blaß and Fellmoser (Blaß and Fellmoser, 2004) making reference to a C24 graded timber:

$$E_{0,eq} = \left[1-\left(1-\frac{E_{90,T}}{E_{0,L}}\right)\cdot\frac{a_3-a_1}{a_5}\right]\cdot E_{0,L} = \left[1-\left(1-\frac{370}{11000}\right)\cdot\frac{54-17}{85}\right]\cdot 11000 = 6373\,\text{N/mm}^2 \qquad (19)$$

where $E_{0,L}$ is the modulus of elasticity parallel to the grain of the longitudinal layers, $E_{90,T}$ is the modulus of elasticity perpendicular to the grain of the transversal layers and a_1, a_3, a_5 are shown in Figure 3-26.

The wall model is illustrated in Figure 3-71. The shell material was defined as Isotropic with the E modulus value calculated according to Eq. (19).

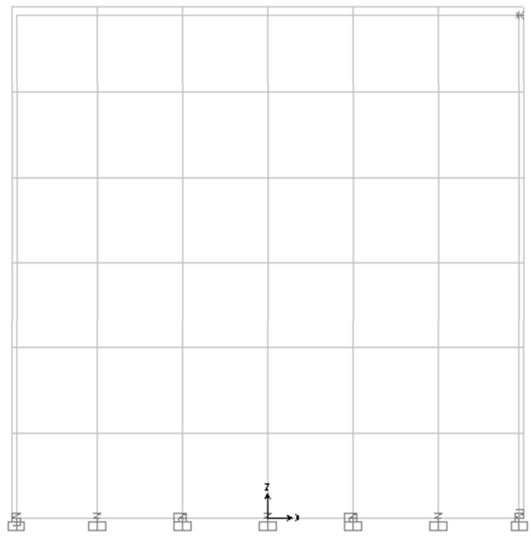
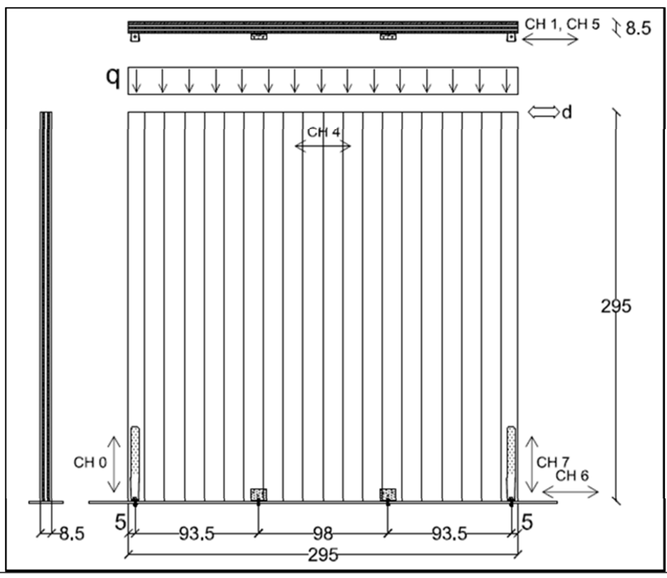


Figure 3-71 Model of the tested wall in Sap2000 (CSI, 2000).

3.2.7.2.1 Case 1 – Single wall calibration based on connection test results – Wall 1.1

The numerical comparison was made on the wall displayed in Figure 3-72 (wall 1.1 according to Gavric's report). Angle brackets were modelled taking into account their contribution both to the slip and uplift resistance of the wall, while hold-downs were modelled taking into account only the uplift resistance as their contribution to the slip resistance is considered negligible.



				Wall name	1.1
				Vertical load	18.5 kN/m
Connection type	Connection name	No. of conn.	Bolt	Nails type	No. of nails
HOLD-DOWN	Simpson HTT22	2	Φ16	Φ4x60 annual ringed	12
ANGLE BRACKET	BMF 90x48x3x116	2	Φ12	Φ4x60 annual ringed	11

Figure 3-72 Wall test set-up and specifications of hold-down and angle bracket connections (after Gavric, 2013b).

The shell material was defined as isotropic with the E modulus value calculated as in Eq. (19).

The results of the calibration process for the wall in terms of hysteresis curves and dissipated energy is shown in Figure 3-73 and Figure 3-74. The final difference in terms of dissipated energy was 4.1%.

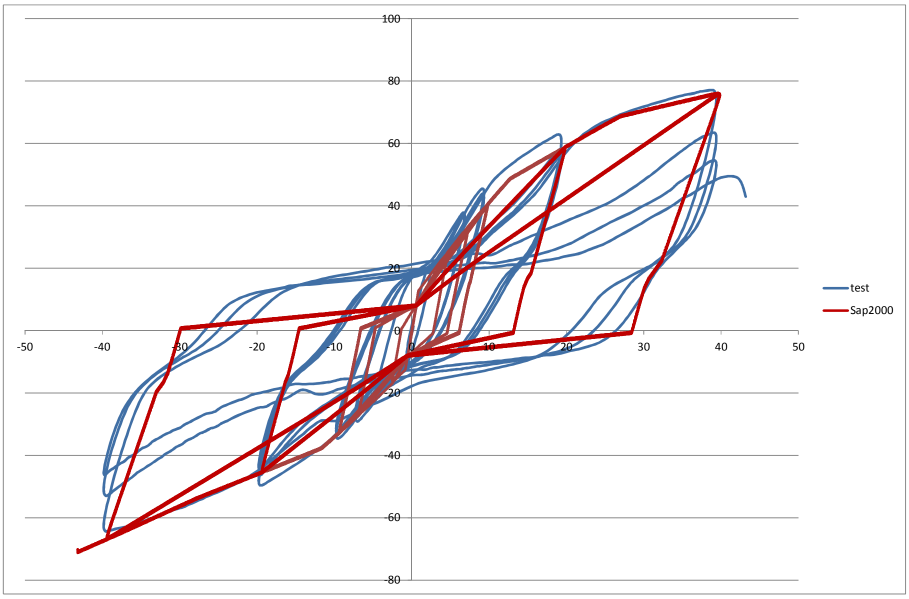


Figure 3-73 Calibration of CLT wall. Comparison in terms of hysteresis curves.

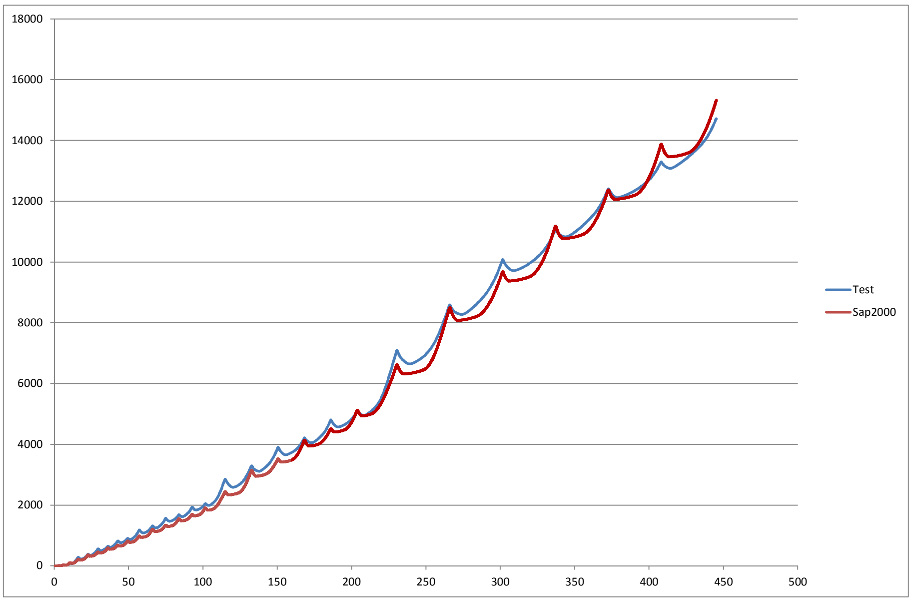
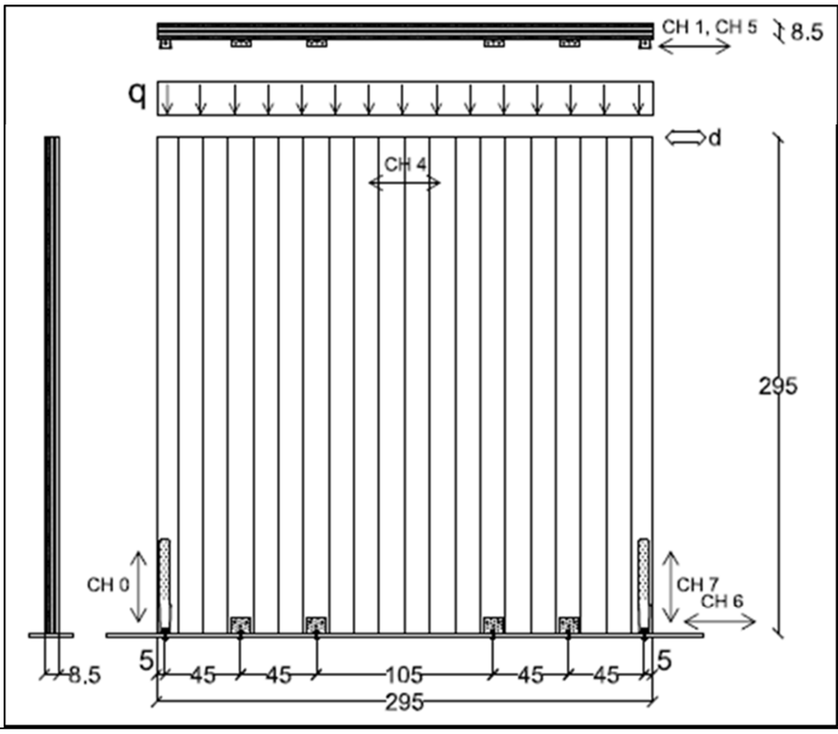


Figure 3-74 Calibration of CLT wall. Comparison in terms of dissipated energy.

The comparison in terms of hysteresis curves is not perfect but in terms of dissipated energy is quite satisfactory.

3.2.7.2.2 Case 2 – Single wall calibration based on connection test results – Wall 1.2

The numerical comparison was made this time on the wall 1.2, according to Gavric's report (Gavric, 2013b), showed in Figure 3-75. The hysteresis parameter for angle brackets and hold-downs are the same as in Case 2.



				Wall name	1.2
				Vertical load	18.5 kN/m
Connection type	Connection name	No. of conn.	Bolt	Nails type	No. of nails
HOLD-DOWN	Simpson HTT22	2	Φ16	Φ4x60 annual ringed	12
ANGLE BRACKET	BMF 90x48x3x116	4	Φ12	Φ4x60 annual ringed	11

Figure 3-75 Wall test set-up and specifications of hold-down and angle bracket connections (after Gavric, 2013b).

The results of the calibration process for the wall in terms of hysteresis curves and dissipated energy is shown in Figure 3-76 and Figure 3-77. The final difference in terms of dissipated energy was 4.4%.

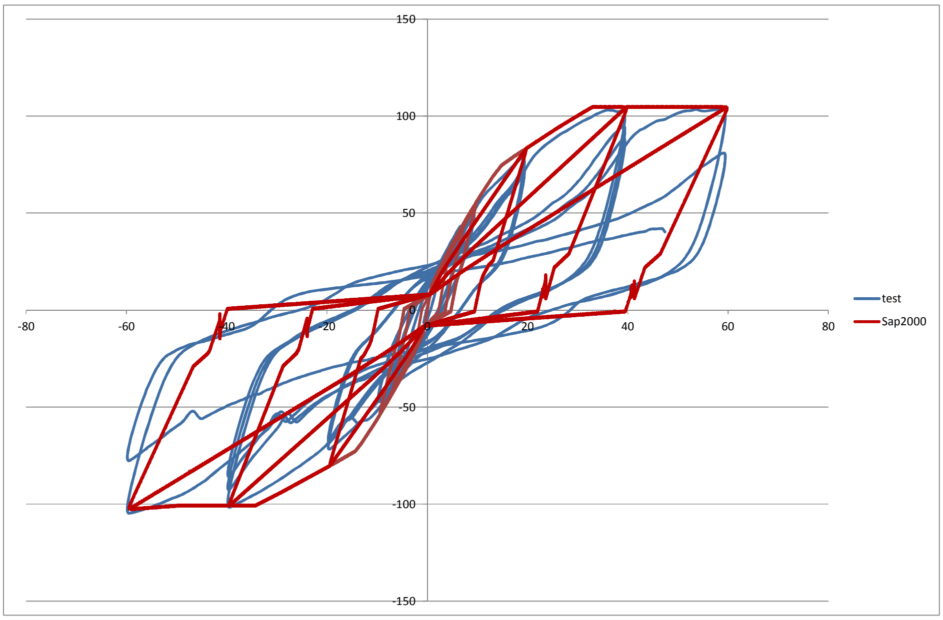


Figure 3-76 Calibration of CLT wall. Comparison in terms of hysteresis curves.

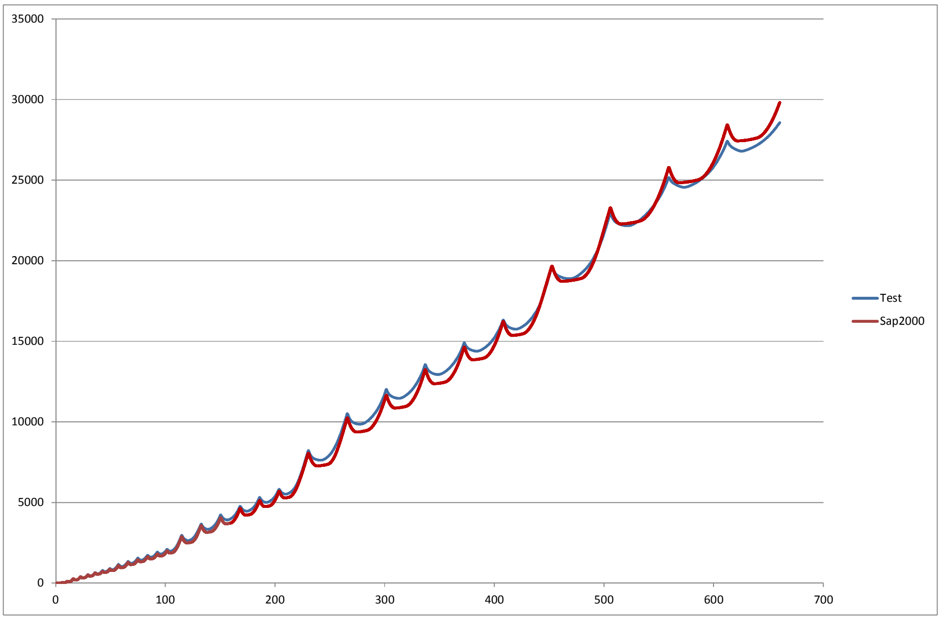
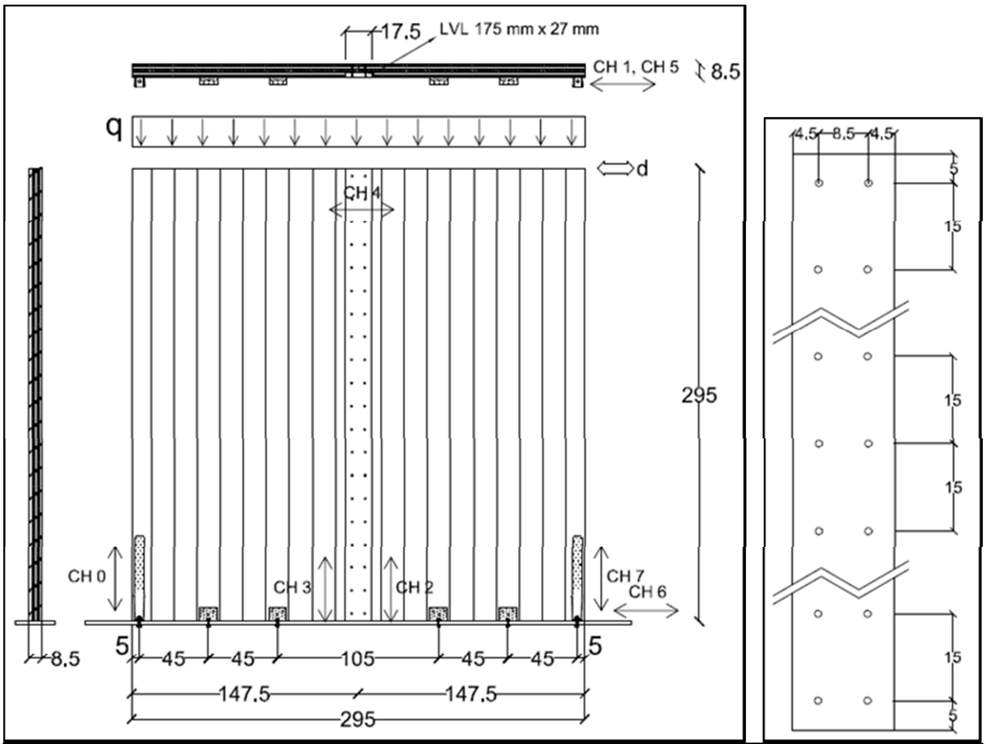


Figure 3-77 Calibration of CLT wall. Comparison in terms of dissipated energy.

Again the comparison in terms of hysteresis curves is not perfect but in terms of dissipated energy is quite satisfactory.

3.2.7.2.3 Case 3 – Single wall calibration based on connection test results – Wall 3.1

The numerical comparison was made this time on the wall 3.1, according to Gavric's report (Gavric, 2013b), showed in Figure 3-78. The hysteresis parameter for angle brackets and hold-downs are the same of Case 2.



Wall name	3.1
Vertical load	18.5 kN/m

Connection type	Connection name	No. of conn.	Bolt	Nails type	No. of nails
HOLD-DOWN	Simpson HTT22	2	Φ16	Φ4x60 annual ringed	12
ANGLE BRACKET	BMF 90x48x3x116	4	Φ12	Φ4x60 annual ringed	11

Connection type	Screw name	Inclination	No. of screws	Spacing
SCREW	HBS Φ8x100	35°	2x20	15 cm

Figure 3-78 Wall test set-up and specification of hold-down and angle bracket connections (after Gavric, 2013b).

The wall was modelled as two monolithic portions, left and right, connected in the middle by the step joint modelled as two non-linear cross springs going from the end base node of the left portion to the top node of the right portion and vice-versa, as shown in Figure 3-79.

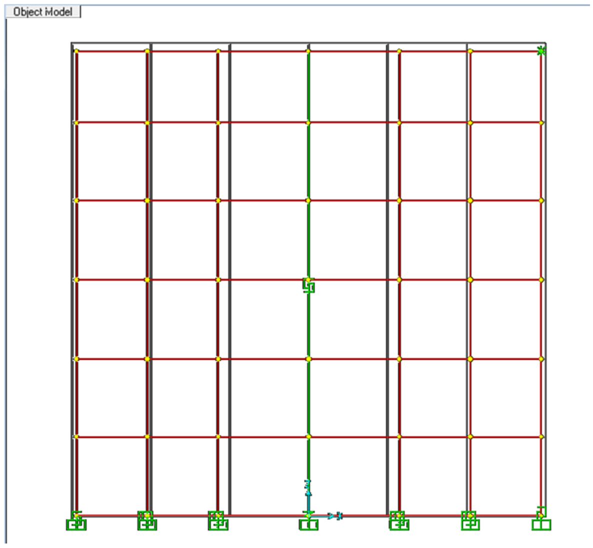


Figure 3-79 Model of the tested wall in Sap2000 (CSI, 2000).

The results of the calibration process for the wall in terms of hysteresis curves and dissipated energy is shown in Figure 3-80 and Figure 3-81. The difference in terms of dissipated energy was 3.4%.

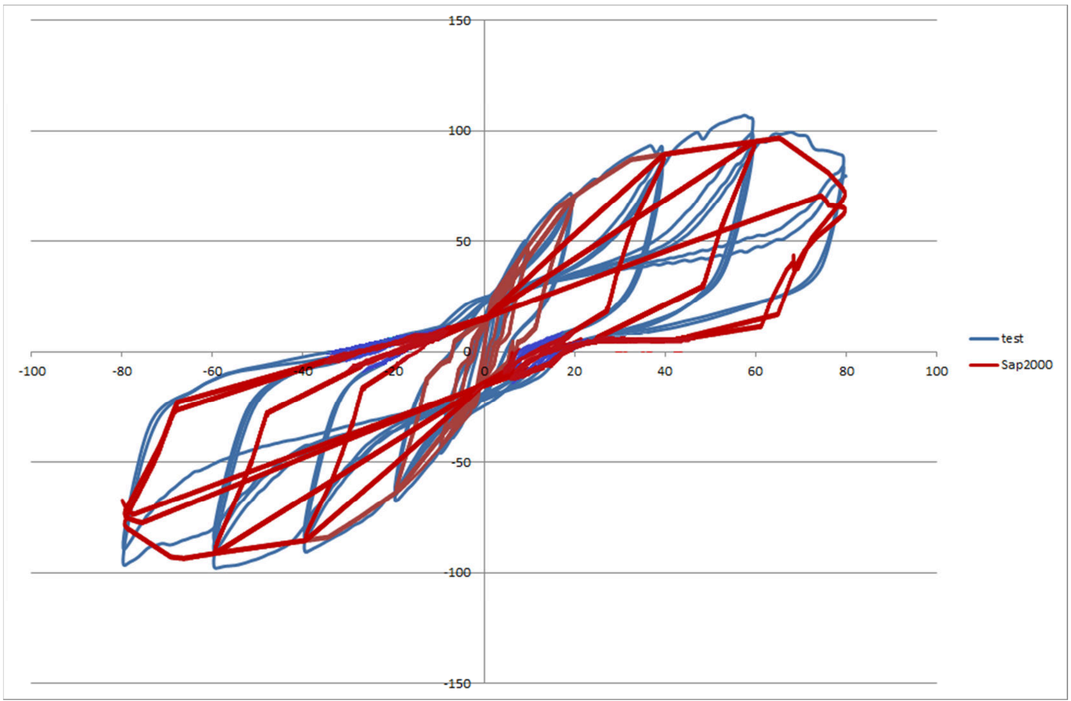


Figure 3-80 Calibration of CLT wall. Comparison in terms of hysteresis curves.

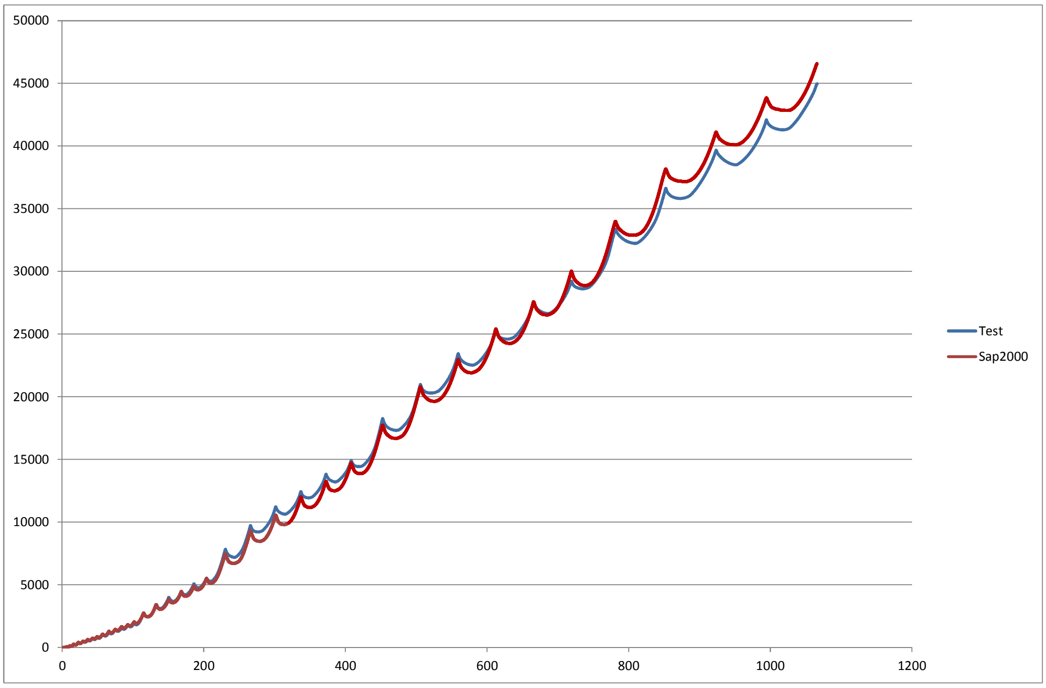


Figure 3-81 Calibration of CLT wall. Comparison in terms of dissipated energy.

Again the comparison in terms of hysteresis curves is not perfect but in terms of dissipated energy is quite satisfactory.

3.2.7.3 Numerical analysis on a full-scale 3 storey building

The next step after the calibration of connectors and walls is the model of a full scale 3 storey building and the comparison with experimental test results. The comparison is made again with the experimental results from shaking table tests of a three-storey full scale CLT building (Ceccotti and Follesa, 2006) made within the Sofie Project in 2006 and described in §3.2.6.2. The layout of the model is shown in Figure 3-82. CLT walls are modelled with the orthotropic model described in §3.2.5 with shell elements of the same length, height and thickness as the associated CLT wall panels and meshed with a mesh dimension of 0.5x0.5m as already described in §3.2.7.2. Hold-downs, angle brackets and vertical step joints are placed in the same position used in the tested building and are modelled by multi-linear plastic link elements calibrated as in the procedure described in §3.2.7.1. Non-linear Gap Link elements were also included at each base node to simulate the perpendicular to grain compression stiffness of the CLT floor panels.

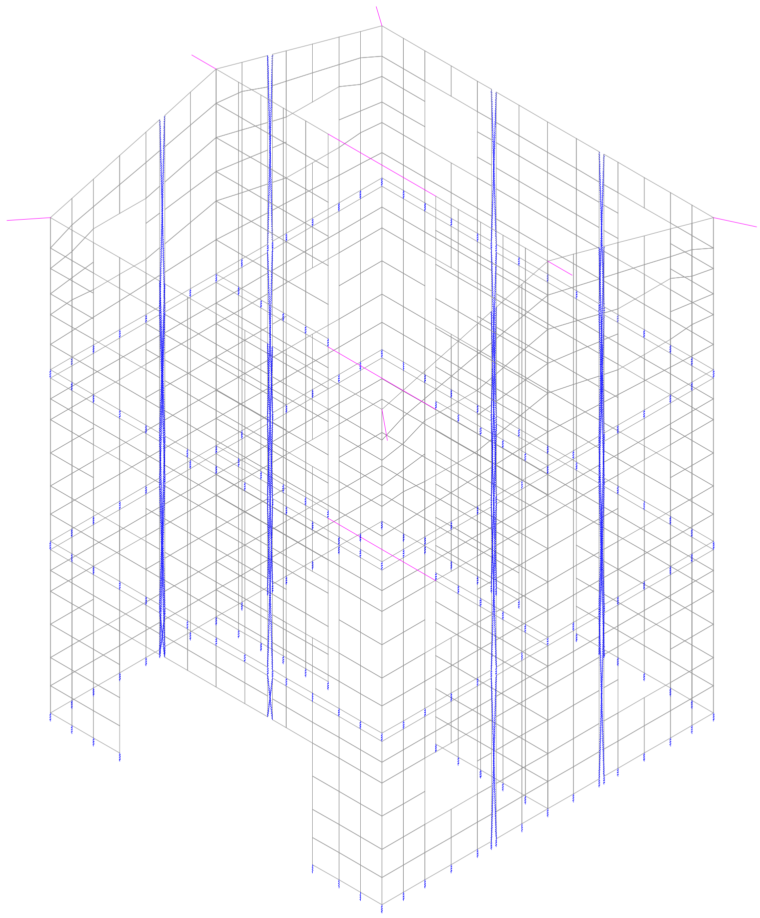


Figure 3-82 Layout of the 3D model used for the non-linear analysis. The mesh is showed with grey lines and non-linear link elements simulating vertical joints between CLT panels, hold-down anchors and angle brackets are showed with blue lines.

The validation was made with the 1995 JMA Kobe 100% test (0.82g, see Ceccotti and Follesa, 2006, for further details) for the asymmetric configuration of the building, i.e. Configuration C (see Figure 3-38, right).

Table 3-17 shows the comparison between the test results and the numerical model in terms of fundamental period, maximum displacements at each level in the North and South side of the building and in terms of maximum uplifts.

Table 3-17 Comparison between test results and numerical modelling.

	Period [s]	**Displacements [mm]**							
		0NE	0SE	1NE	1SE	2NE	2SE	3NE	3SE
Test	0.21	4.95	4.17	25.97	29.50	51.47	56.08	58.88	62.15
Model	0.195	2.55	2.08	20.95	23.61	48.31	51.15	62.35	65.34
Diff. [%]	8%	94%	100%	24%	25%	7%	10%	10%	1%
		Uplift [mm]							
Test		10.65	7.39						
Model		8.31	6.50						
Diff. [%]		6%	5%						

The results in terms of fundamental period and maximum displacements are quite satisfactory even if the latter are slightly underestimated by the model for the first two floors; therefore the proposed modelling method is suitable for the non-linear analysis of CLT buildings.

3.3 Seismic design of Light Frame buildings

The Light Frame is the world widest used construction system for low to mid-rise timber buildings, usually up to a maximum of four storeys. The system originated in North America in the 19th century during the colonization of the American continent by European settlers and nowadays represent the vast majority of the buildings stock in North America, but is also well-known and widely used in Asia, Australasia and Europe. The great diffusion of this system is also reflected by the large presence of design and detailing rules in a great number of international building codes.

Figure 3-83 Three story residential building for social housing built near Lucca, Italy.

3.3.1 Structural behaviour and features of Light-Frame buildings

Light Frame buildings are structures in which walls are composed of a timber frame, i.e. a frame composed by a bottom and top plate and generally equally spaced vertical studs, all of the same cross-section, and a sheathing. The sheathing, a thin wood-based panel (usually plywood or OSB of thicknesses varying from 9 to 15 mm) is connected to the timber frame by means of nails of small diameter (usually around 3 mm) equally spaced at 150 mm or less along the edges of the wood based panel and at 300 mm or less at internal supports. The studs are generally spaced at the same distance, usually 400 or 600 mm, or anyway a submultiple of the width of the wood-based sheathing which has commercial dimensions of 1,20x2,40 m or 1,22x2,44 m, or even 1,25x2,50.

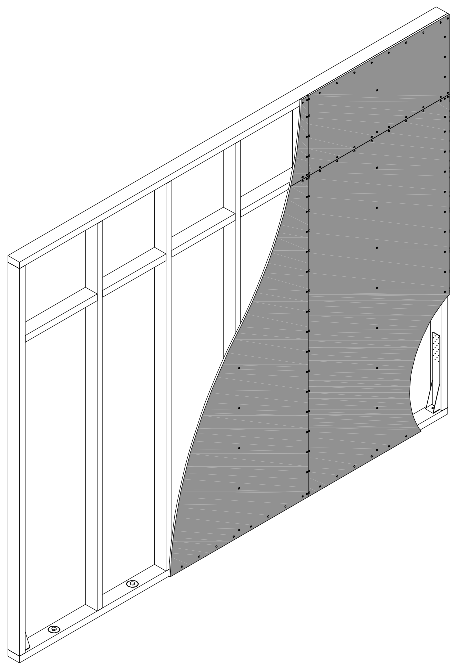


Figure 3-84 Typical Light-frame wall layup (after Follesa et al., 2011b).

Walls are connected to the foundation by means of hold-down at wall ends and at opening ends as uplift restraint connections, which are fixed to the end studs by means of nails and to the concrete foundation by means of anchor bolts connected to the foundation and uniformly distributed along the length of the wall as sliding restraint connections.

In modern Light-Frame construction, walls are often prefabricated within a factory by assembling the timber frame together with a double external and internal wood-based sheathing (OSB or Plywood usually connected to the framing with nailed connections, but recently also gypsum fibre panels connected to the framing with stapled connections). Prefabricated walls are then delivered to the construction site and erected with a platform type procedure very similar to the construction procedure used for CLT buildings and described in §3.2.2. In this case the shear restraint connection to the foundation and to the storey floors is made like for CLT construction with angle brackets placed on the internal side of the prefabricated wall. The uplift restraint connection is made again with hold-down anchors or with tie-downs connected on the internal or external (for tie-downs) side of the prefabricated panels (see Figure 3-85).

Figure 3-85 Pre-assembling of prefabricated Light-Frame walls.

The wall sheathing is usually placed with the long side in the vertical direction. Being in Europe the inter-storey height of buildings (e.g in Italy 2,70 m) usually higher than the sheathing panel height, it is necessary to place horizontal blockings between the wall studs in order to connect the sheathing panels along all their edges and transfer the shear force.

Wall studs resist vertical loads and horizontal out-of plane loads (e.g. wind load) and are braced in the wall plane by the nailed wood-based sheathing. The bottom plate of the wall frame is subjected to compression perpendicular to the grain, which is usually the crucial verification with respect to vertical loads.

Perpendicular walls at building corners are jointed by connecting the wall sheathing of both perpendicular walls to the corner stud or by connecting the end stud of each wall to the corner stud by means of screws. A screwed connection is used between walls studs of all perpendicular walls other than corner walls.

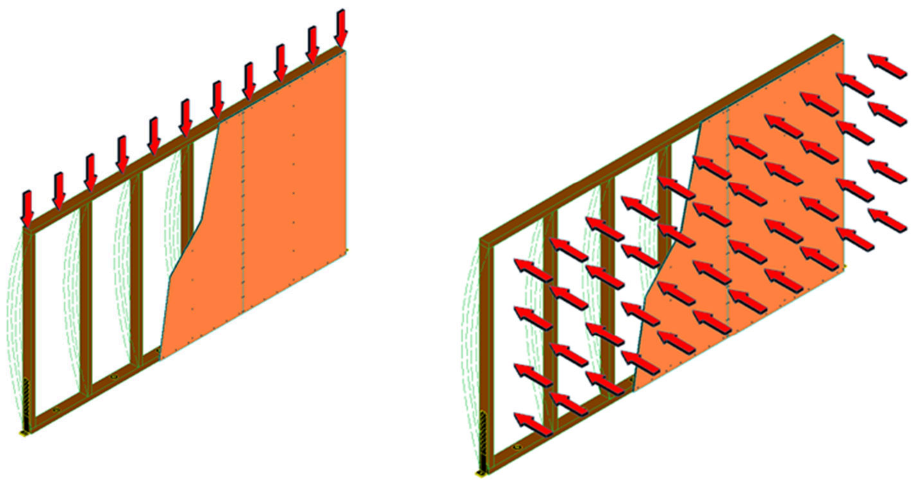


Figure 3-86 Double structural behaviour of wall studs: resistance to vertical (left) and horizontal out-of-plane external loads (right).

The floor construction is similar to the wall construction. The floor structure is usually made of glulam timber beams or joists, generally spaced at the same distance of wall studs. The wood based sheathing is placed above the timber joists, and is connected to the floor frame (joists and blockings) by means of the same type of nails used for the wall construction spaced at 150 mm c/c along the panel's edges and 300 mm c/c along internal supports. Between the floor beams, like for the wall studs, a timber bridging or blocking is placed in order to allow the edge-nailing of the floor sheathing. The floor frame is completed by an edge timber beam to which the sheathing is connected. The edge beam resists tension stresses when the floor is loaded in his plane by horizontal forces perpendicular to the edge beams.

Other types of horizontal diaphragms may be used, such as cross laminated timber or timber-concrete composite floors, provided that they are adequately connected to the lower and upper walls by means of mechanical fasteners. The concrete topping, in particular, shall be connected to the vertical panels to ensure the in-plane shear due to the diaphragm action is transferred to the walls and down to the foundations.

Figure 3-87 Timber-concrete composite floor construction in Light-Frame building.

The in-plane shear resistance of the shear walls subjected to horizontal loads like earthquake and wind is given by the nailed connection of the wall sheathing to the wall frame. When subjected to in-plane horizontal loads, the wood-based sheathing panels tend to rotate with respect to the wood frame and the collapse of the wall is attained when the shear failure of the nailed connection is reached.

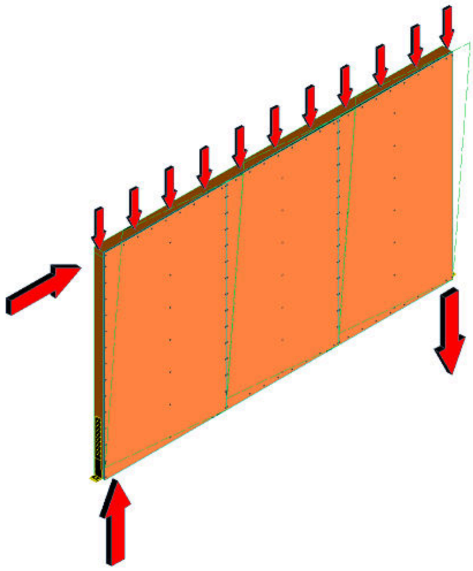


Figure 3-88 Slip behaviour of wood-based sheathing with respect to the timber frame and collapse reached in a light-Frame building subjected to a shaking table test.

The construction is a platform type, with upper storey walls built over the timber floor joists and connected to the floor and to the lower storey walls. The inter-storey uplift restraint connection is made, like for CLT system, by a double hold-down connection for inner walls and by a tie-down connection made with steel plates nailed to the upper and lower walls for external walls. The sliding restraint connection is usually made with screws or bolts connecting the upper walls to the floor frame and the floor frame to the lower walls.

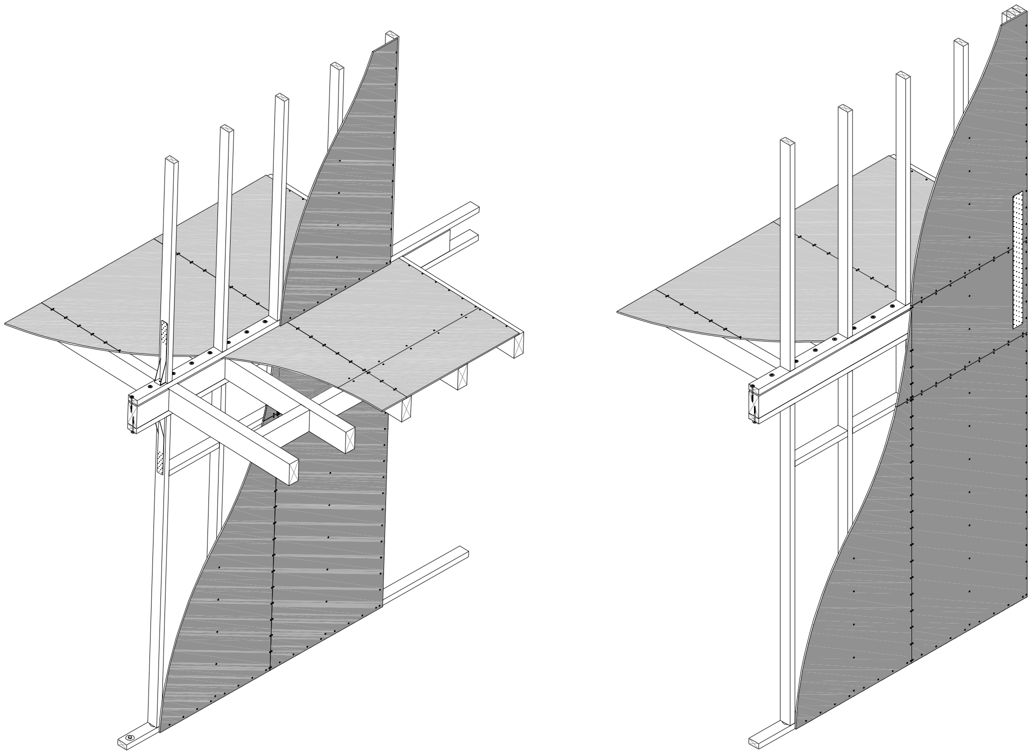


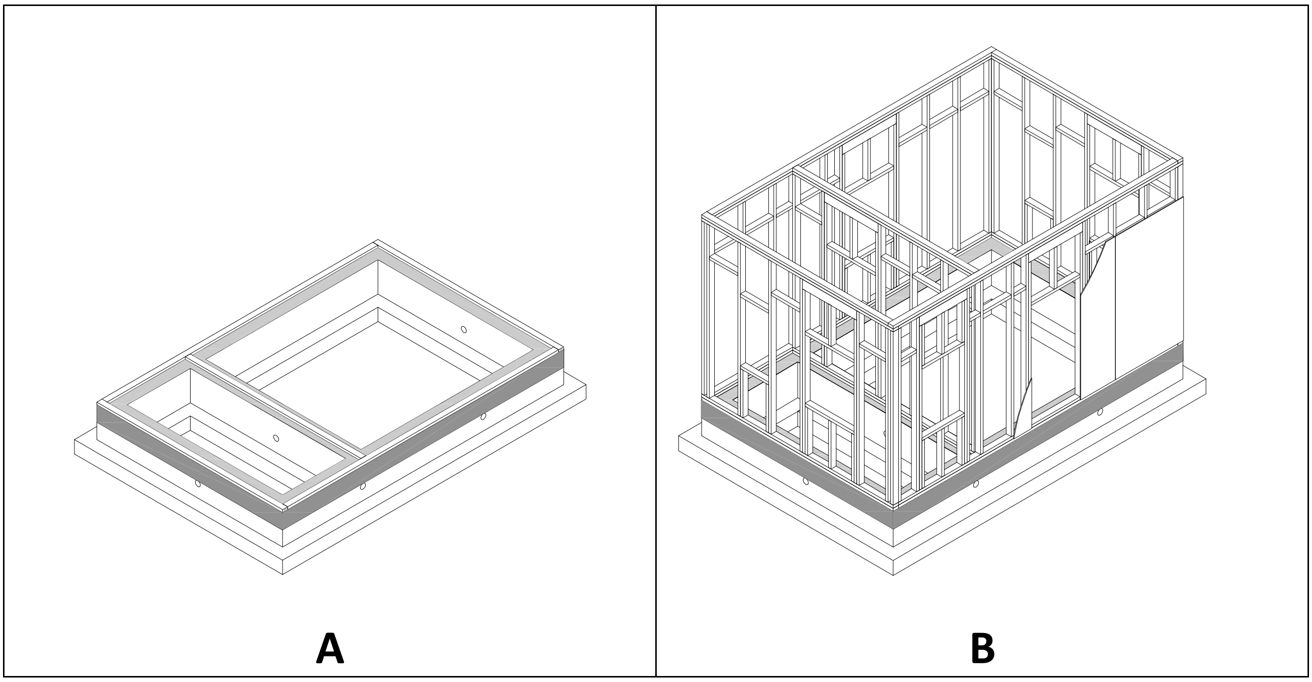
Figure 3-89 Inter-story connections for internal and external walls (after Follesa et al., 2011b).

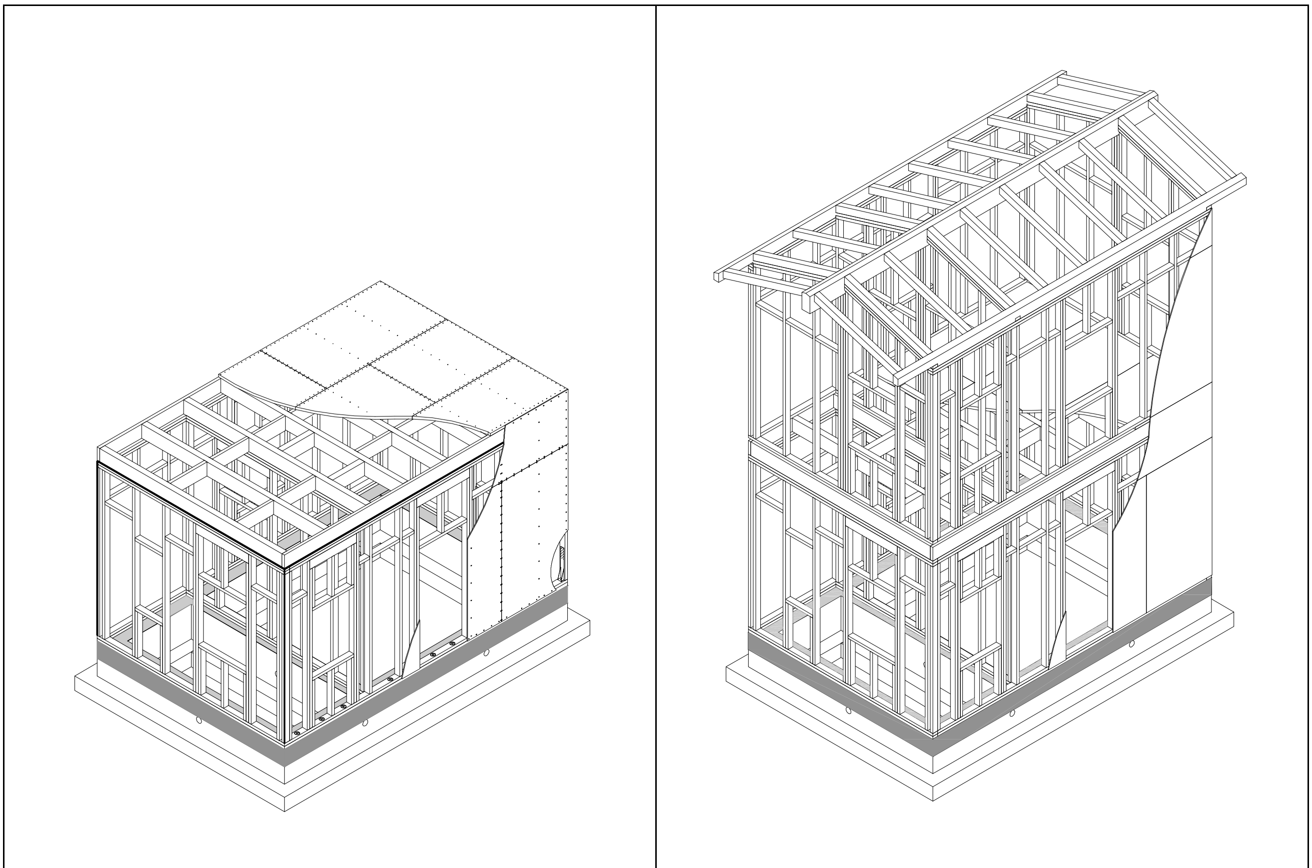
Finally the roof construction may be made similarly to the floor construction with (i) glulam timber beams and joists, blockings and wood-based sheathing panels, or (ii) with Light-Frame trusses usually made with the same dimension lumber employed for the wall skeleton construction connected with nailed plates.

Figure 3-90 Two different types of roof construction: with Light-Frame nailed trusses (left) or with glulam timber beams (right).

3.3.2 Construction process of a Light-Frame building

The construction process of a Light-Frame building follow a platform type procedure (the system is also known as Platform Frame) with the same sequence described in §3.2.2 for CLT buildings.





C D

Figure 3-91 Erection sequence of Light-Frame construction (after Follesa et al., 2011b): A: concrete foundation beams covered with bituminous waterproofing membrane; B: wall erection and connection to the foundation; C: floor construction; D: 2nd story wall erection over the 1st floor and roof construction (after Follesa et al., 2011b).

Walls and floor may be assembled directly on site or, as it is usually done for modern Light-Frame construction, pre-assembled in factory, delivered and erected on site (Figure 3-92).

Figure 3-92 Different type of Light Frame erection procedure: walls assembled directly on the building site (left) and assembly of prefabricated walls (right).

The wall construction is then completed on the internal side with gypsum plaster or fiber board finishing, a 4-5 cm cavity filled with soft insulation for the installations, internal gypsum, OSB or plywood sheathing nailed on the wall frame, insulation in the wall cavity, external OSB or plywood sheathing nailed on the wall frame, an external (usually 4-8 cm) insulation layer over which the external lining is made usually with plaster.

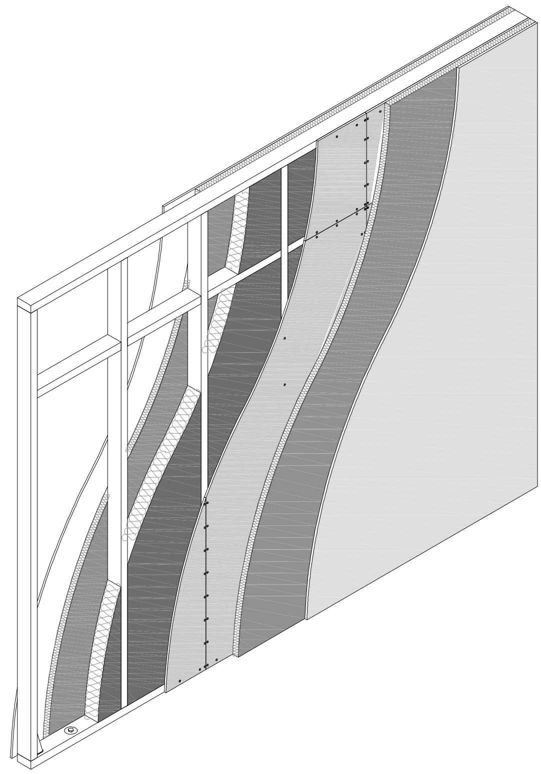


Figure 3-93 Light-Frame wall construction (after Follesa et al., 2011b).

The floor construction is completed, starting from below with the floor sheathing (usually again plywood or OSB) connected to the floor construction (usually glulam joists and blockings), a dust cover sheet, an insulation layer, a wax paper or polyethylene sheet, a 4-5 cm concrete or dry (perlite or marble chippings) screed for installations, underfloor heating, a second 4-5 cm concrete or dry screed and flooring.

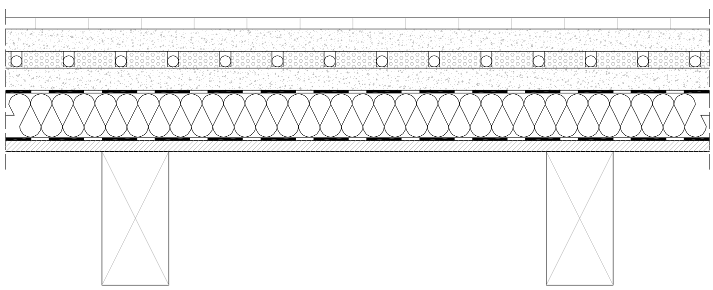


Figure 3-94 Light-Frame floor construction (after Follesa et al., 2011b).

3.3.3 Seismic design of Light-Frame buildings

Similarly to CLT construction, considering the seismic behaviour, a Light-Frame building may be schematized as a box-type structure in which vertical and horizontal diaphragms are made of timber frames to which a wood-based sheathing material (plywood or OSB) is connected by means of nails. Usually in practical design cases, Light-Frame floors are schematized as rigid in their plane; this assumption should of course carefully be checked by the designer, depending on the presence of openings and/or for high length-to-depth ratios of the floor plan, but in most cases in residential construction could be assumed as valid.

Like for CLT construction, according to Eurocode 8 (CEN, 2004) at each level the story shear is distributed to the walls underneath proportionally to their stiffness considering an accidental eccentricity of +/- 5% of the largest dimension of the building plan measured perpendicularly to the direction of the seismic action in addition to any actual eccentricity due to asymmetric distribution of masses and stiffnesses.

The horizontal deformation of a Light-Frame shear walls can be divided into five main components (Casagrande et al., 2012): (i) nail slip in sheathing-to-framing connection, (ii) wood-based sheathing shear deformation, (iii) sliding deformation due to slip in the anchor bolts or angle brackets used for the base connection of the shear wall, (iv) rocking deformation due to both hold-down stretching and wood squashing in the sill plate of the wall frame due to compression perpendicular to the grain. Moreover also the flexural deformation of the walls due to tension and compression forces in the outer studs should be considered. However, according to literature test results (Sartori and Tomasi, 2013) and considering the capacity design criteria given in §3.3.4, component (i) is predominant with respect to all the other components.

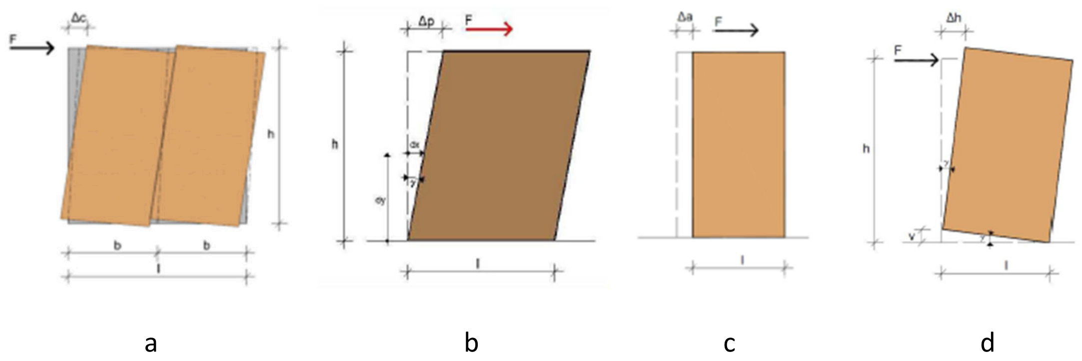


Figure 3-95 Deflection components of a Light-Frame shear wall (a) nail slip, (b) sheathing shear deformation, (c) sliding deformation, (d) rocking with hold-down deformation and compression perpendicular to the grain in base plate (after Casagrande et al., 2012).

Thus, for practical purposes, Light-Frame shear walls may be schematized with shell elements whose stiffness may be defined according to the corresponding layout of the sheathing-to-framing connection, as in the procedure described in §3.3.5. The numerical model should nevertheless include a proper schematization also of the uplift and sliding restraint connections by means of equivalent spring or truss elements like for the modelling procedure of CLT structures in order to better estimate the natural vibration period and the building deformations.

Regarding the value of the behaviour factor q to be adopted in the seismic design, as described in §2.4 Eurocode 8 (CEN, 2004) gives two different values, 3 and 5, depending on the type of horizontal diaphragm used for the floor construction, respectively glued or nailed. However, as explained above, horizontal diaphragms, even when nailed, are assumed as rigid in their plane in the seismic design without any possible ductile behaviour or energy dissipation capacity considered for the floor construction. Therefore, as the differentiation between glued and nailed diaphragms implicitly consider the possibility of energy dissipation behaviour at the floor level which is never realized, only a q factor of 3 should be considered.

3.3.4 Capacity-based design criteria

As reported in Casagrande et al. (2014) and as also underlined in §2.5, the Capacity Design Rules included in the current version of Eurocode 8 (CEN, 2004), even if referred mainly to the Light Frame system, do not properly address the global failure mechanism which should be achieved in order to reach the desired dissipative behaviour corresponding to the relevant Ductility Class. Few prescriptive ductility provisions are provided for the sheathing-to-framing connection, as reported in §2.4, however without any distinction between dissipative and non-dissipative connections or structural components and without any indication for the value of the overstrength factors to be used in the design. Consequently, in practical design applications, sheathing-to-framing connections, uplift restraint connections (hold-downs) and sliding restraint connections (anchor bolts, screws or angle brackets) are designed for the relevant forces derived from the seismic design, according to the analysis procedure used in the design.

Tomasi and Sartori (2013) and Humbert et al. (2014) showed that ductility properties of hold-down and angle bracket connections in Light-Frame construction are significantly lower with respect to sheathing-to-framing connections. Therefore, according to Follesa et al. (2011) the connections devoted to the dissipative behaviour in a Light-Frame building should only be the nailed connection between sheathing material and timber frame in shear walls.

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

- nailed connections between sheathing and timber joists/beams at each floor;
- shear connections between upper and lower walls, and between walls and foundation;
- connections against uplift placed at wall ends and at wall openings;
- connection between floors and underneath walls thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;
- connection between perpendicular walls, particularly at the building corners, so that the stability of the walls itself and of the structural box is always assured;
- sheathing panels under in-plane shear induced by seismic actions;

- timber framing members (studs, plates, and joists) under axial forces induced by seismic actions.

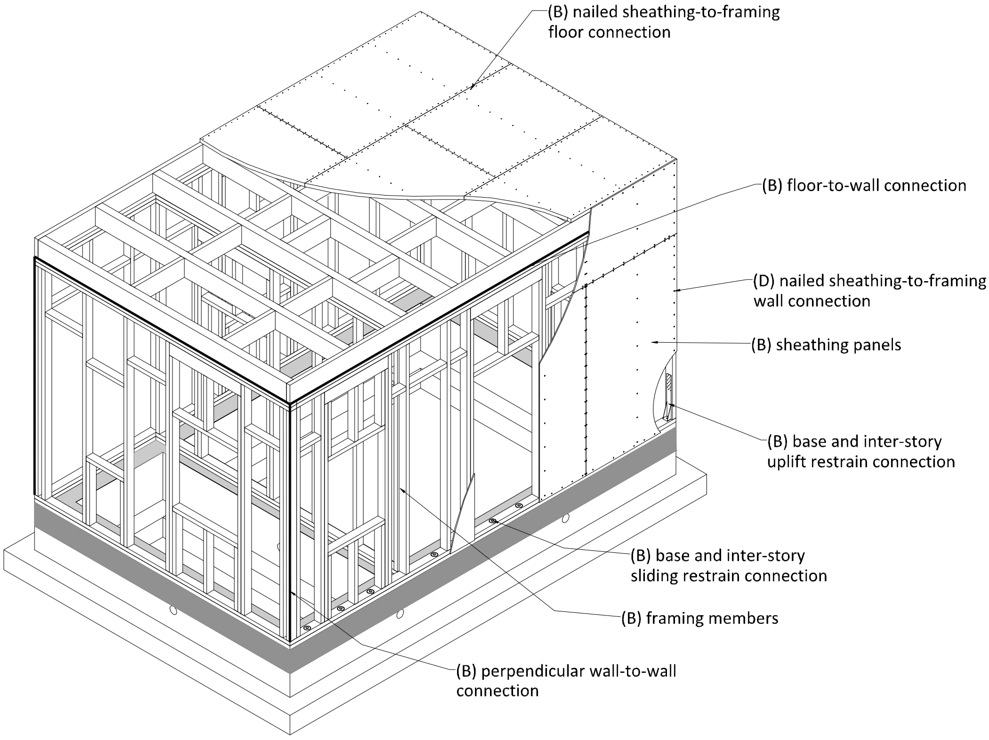


Figure 3-96 Dissipative (D) and non-dissipative (B) connections and structural components according to the capacity based design for Light-Frame construction.

The seismic resistance of shear walls should be higher at lower storeys and should decrease at higher storeys proportionally to the decrease of the storey seismic shear, thus leading to the simultaneous plasticization of the ductile connections in order to maximize the energy dissipation of the whole building.

Casagrande et al. (2014) proposed two different methods for the application of the capacity design applied to timber-frame building based on analytical formulations based on a linear elastic rheological model of a multi-storey timber-framed shear wall; the first one consistent with a Medium Ductility Class (DCM), the second one with a High Ductility Class (DCH). According to the DCH method the same capacity based design provisions listed above apply, whereas for the DCM method all sheathing-to-framing fasteners and connection devices (hold down and angle brackets) are considered as dissipative zones. An over-strength factor γ_{RD}=1.6 is suggested according to Schick et al., 2013.

3.3.5 Numerical modelling of Light-Frame buildings with Sap2000

In the following some suggestions about possible modelling methods of Light-Frame buildings with SAP2000 (CSI 2000) will be made, both for linear and non-linear analysis.

Casagrande et al., 2014, propose two different methods in order to evaluate the elastic behaviour of a framed timber wall loaded by a horizontal load with SAP2000, a “complete” method and a “simplified method". The "complete" method consist of pinned frame elements utilized to model the timber frame, and the sheathing panels, modelled by means of shell elements. The frame elements are connected to the shell points through linear elastic springs (Joint link) simulating the behaviour of nailed connections. The bottom chord is then connected to the foundation by means of linear link joints in order to model hold-downs and angle brackets and calibrated with parameters validated with the results of experimental data. The "simplified model" consists of four rigid pinned frame elements utilized to model the timber frame and diagonal or vertical linear link element to simulating the sheathing-to-framing stiffness and panel stiffness.

Rinaldin et al., 2013, propose two different methods for the non-linear modelling of Light-Frame walls. The first model, is a detailed model which describes the actual non-linear behaviour of Light-Frame walls using frame elements to model the timber frame, shell elements to model the sheathing panel and multi-linear link elements used to describe the non-linear behaviour of each single nail with an hysteretic shear-displacement law in two perpendicular directions in the wall plane, calibrated on the basis of experimental data on single nail tests using the Pivot hysteresis rule. The second model, used to model a full-scale building, is a simplified model composed of four rigid pinned frame elements and two diagonal non-linear springs as in the schematization of Figure 3-101, whose stiffness and strength properties are evaluated so that the backbone shear force-top displacement curve of the wall analysed with the detailed model is accurately described.

3.3.5.1 Light-Frame walls modelling

Light-Frame walls may be modelled as 2D elastic shell elements in one of the following ways:

a) By assigning the element an equivalent homogenized section, with the same dimensions of the actual Light-Frame wall and with different properties in different directions (orthotropic model) with the following procedure.

Again, as explained in §3.2.5.1 (b) the most important ones are: (*i*) $E_z = E_0$ that affects the vertical axial stiffness and the bending stiffness; and (*ii*) $G_{zx} = G_{eq}$ that affects the in-plane shear stiffness of the wall.

Assuming as explained in §3.3.3 that the predominant shear wall deformation component is due to nail slip in sheathing-to-framing connection and making reference to the Light-Frame wall schematization of Figure 3-97 and considering a rigid behaviour of the sheathing panels the following applies:

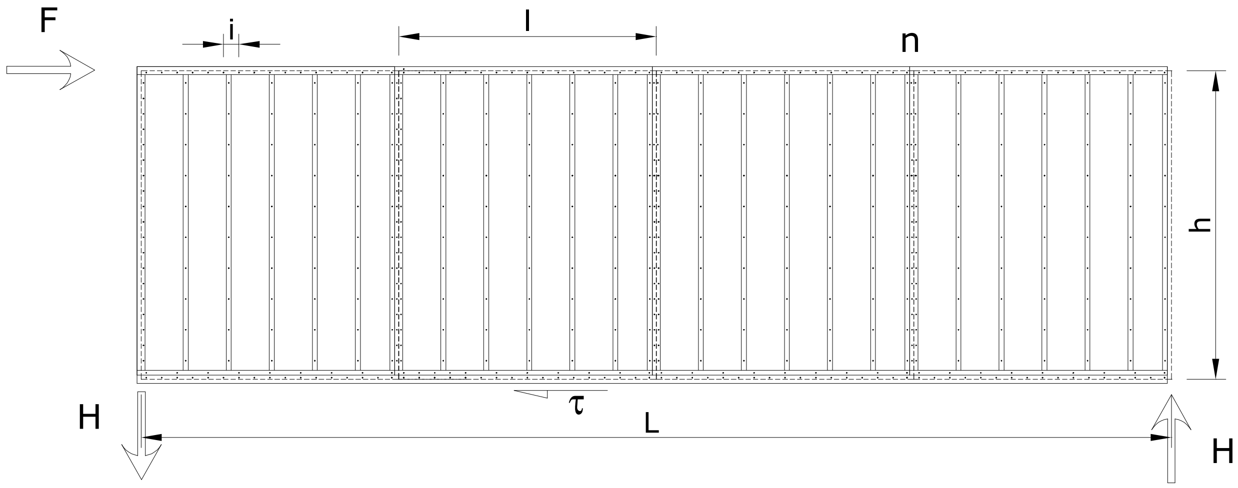


Figure 3-97 Light-Frame wall schematization.

n=total number of internal vertical edge joints between sheathing panels

m=total number of sheathing panels (1 or 2 for panels on one side or both sides)

i=nail spacing at panel's edges

L=total length of shear wall

h=height of shear wall

l=wood-based sheathing panel width

F=design seismic action acting on the shear wall

H=design uplift force acting on shear wall

τ=design shear per linear meter acting on the shear wall

According to Table 7.1 and Equation (7.1) of Eurocode 5 (CEN, 2009) the slip modulus k_n for nails without predrilling is the following:

$$k_n = \frac{\sqrt{\rho_{m1} \cdot \rho_{m2}}^{1.5} \cdot d^{0.8}}{30} \tag{20}$$

Where:

ρ_{m1}=mean density of timber frame lumber

ρ_{m2}=mean density of wood-based sheathing

Assuming a pure shear deformation, the design shear acting on each nail could be calculated as:

$$F_n = \frac{F \cdot i}{L \cdot m} \tag{21}$$

The total number of nails (not including nails at internal supports) of the shear wall is:

$$N = m \cdot \left(\frac{2 \cdot L}{i} + \frac{(2 \cdot n + 2) \cdot h}{i}\right) \tag{22}$$

With the same hypothesis, calling δ the nail slip, the total horizontal shear deformation of the wall will be:

$$\Delta = N \cdot \frac{\delta \cdot i}{L \cdot m} = N \cdot \frac{F_n \cdot i}{k_n \cdot L \cdot m} = N \cdot \frac{F \cdot i^2}{k_n \cdot L^2 \cdot m^2} \tag{23}$$

Therefore the total horizontal stiffness of the shear wall will be:

$$K = \frac{F}{\Delta} = \frac{L^2 \cdot m^2 \cdot k_n}{i^2 \cdot N} \tag{24}$$

The stiffness per linear meter k of the shear wall will be:

$$k = \frac{K}{L} = \frac{L \cdot m^2 \cdot k_n}{i^2 \cdot N} \tag{25}$$

Figure 3-98 shows the variation of shear wall stiffness per linear meter vs wall length. As it can be observed for shear wall sheathed on one side with wall height of 2.44 m and nail diameter of 3.1 mm and sheathing width of 1.22 m the wall stiffness per length tend to an average value of 0.9 kN/mm/m. This value is similar to the experimental values of 1.0-1.2 kN/mm/m found in Ceccotti and Karacabeyli (1996).

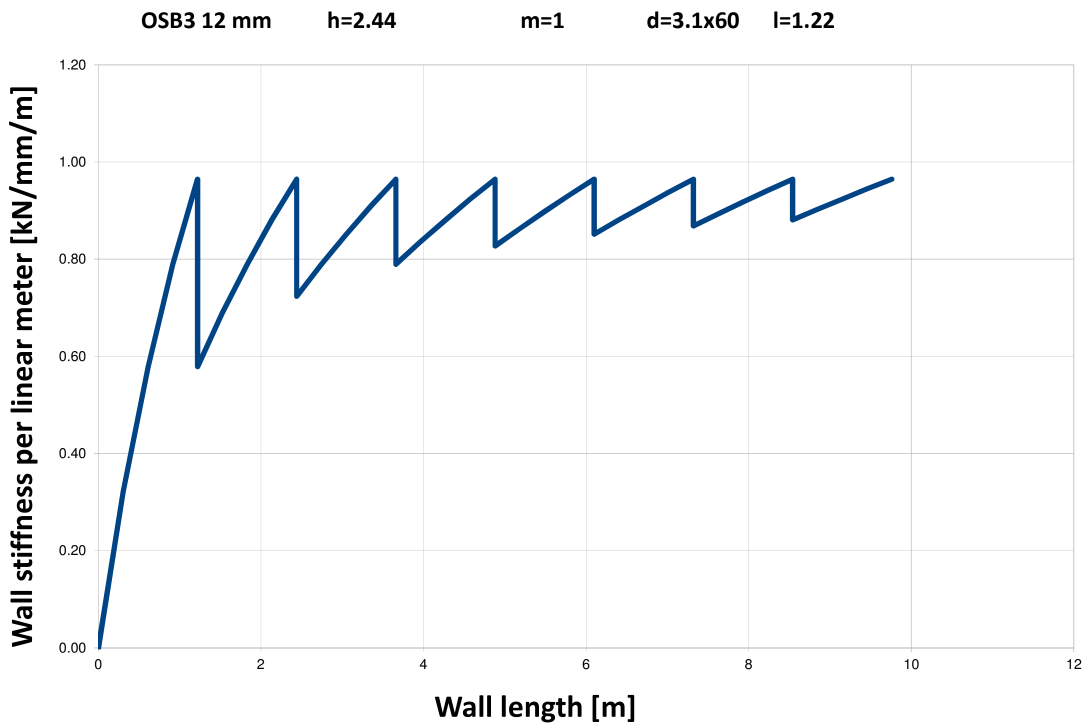


Figure 3-98 Shear wall stiffness per linear meter vs wall length.

Considering the shell element schematization of shear walls in Figure 3-99 where F is the horizontal force, δ is the ultimate displacement calculated as:

$$\delta = \chi \frac{F \cdot h}{G_{eq} \cdot A} = 1.2 \cdot \frac{F \cdot h}{G_{eq} \cdot b \cdot L} \tag{26}$$

And the wall stiffness K will be:

$$K = \frac{F}{\delta} = \frac{G_{eq} \cdot b \cdot L}{1.2 \cdot h} \tag{27}$$

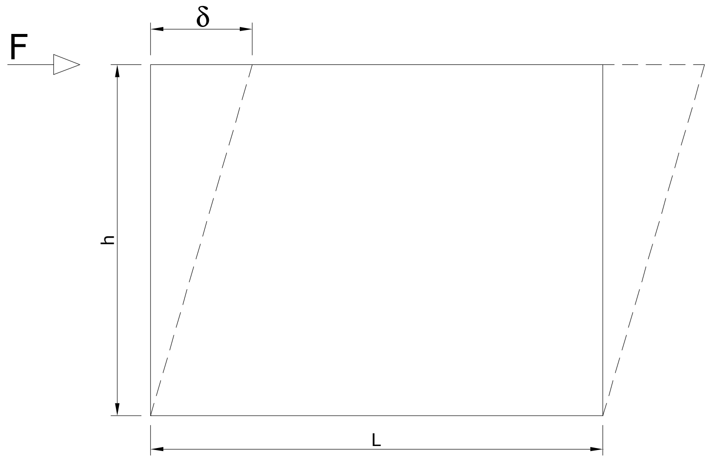


Figure 3-99 Shear wall schematization for the calculation of G_{eq}.

By equalizing the K value of equation (27) with equation (24), and assuming the value $k = \frac{K}{L} = \alpha$ of Eq. (24) as a constant, the G_{eq} value can be calculated as:

$$G_{eq} = \frac{1.2 \cdot h \cdot \alpha}{s \cdot L} \cdot L = \frac{1.2 \cdot h \cdot \alpha}{s} \text{ expressed in [N/mm}^2\text{]} \tag{28}$$

The calculation of the equivalent modulus of elasticity of Light-Frame walls in the vertical direction can be done by making reference to Figure 3-100.

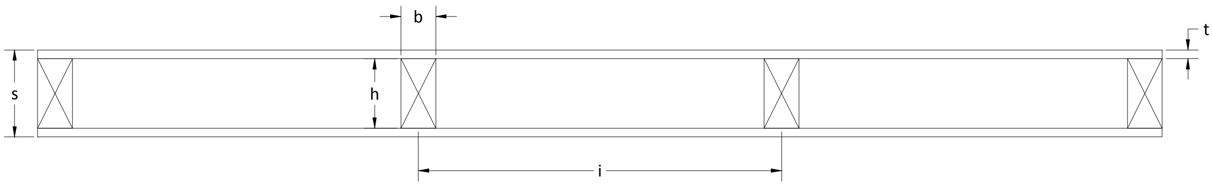


Figure 3-100 Shear wall cross-section.

$$A_{eq} = s \cdot i \tag{29}$$

$$A_{real} = b \cdot h + m \cdot t \cdot i \cdot \frac{E_{sh}}{E_{fr}} \tag{30}$$

$$E_{0,eq} = E_{fr} \cdot \frac{A_{real}}{A_{eq}} \tag{31}$$

Where:

m=total number of sheathing panels (1 or 2 for panels on one side or both sides)

i=studs spacing

s=shear wall total thickness

b=stud cross-section width

h=stud cross-section height

t=wood-based sheathing panel thickness

E_{fr}=modulus of elasticity parallel to the grain of wall studs

E_{sh}=modulus of elasticity parallel to the grain of wood-based sheathing

b) By using a pair of elastically deformable elements (links or springs) hinged on a frame of rigid hinged elements and calibrated on the bending stiffness and shear stiffness of the Light-Frame wall in the following way.

$$K_{spring} = \frac{K_{lat}}{2\,cos^2(\alpha)} \tag{32}$$

Where k_{lat} can be calculated according to Eq. (24)

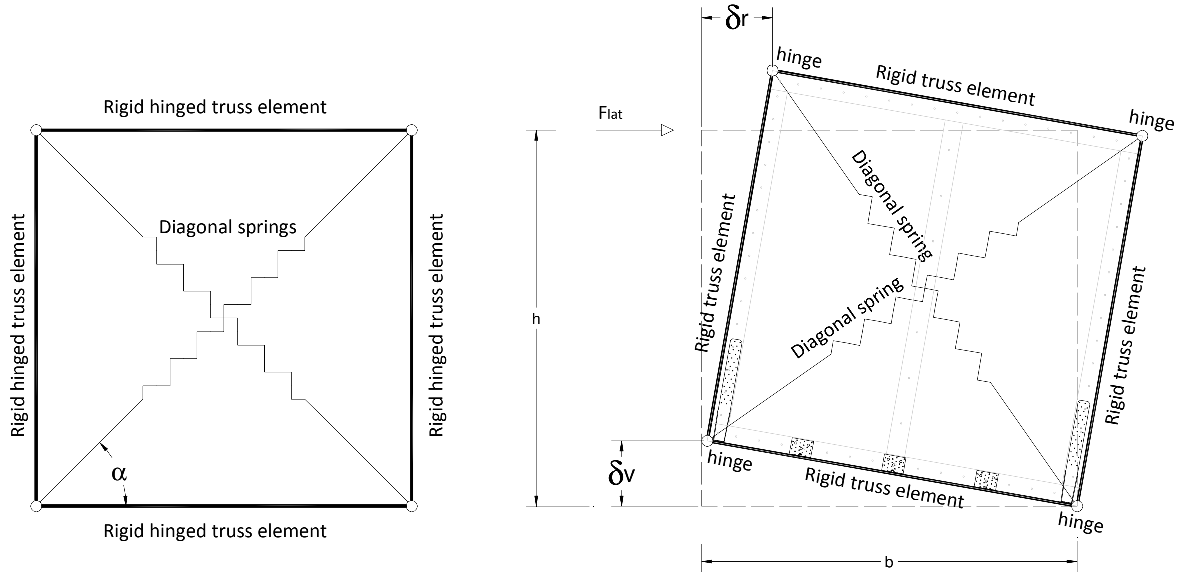


Figure 3-101 Schematization of a Light-Frame wall using a pair of diagonal link or spring elements.

3.3.5.2 Light-Frame floors modelling

Like for CLT floors also Light-Frame floors are usually modelled with a kinematic constraint as infinitely rigid in their plane. In SAP 2000 this can be done by applying a diaphragm constraint to all the floor nodes. Again such simplification is reliable for regular buildings with even distribution of shear walls spaced at regular distances like for residential buildings (usually not more than 6-7 m.). For other types of buildings (e.g. office or commercial buildings) with few shear walls and high span-to-depth ratios, this assumption should be carefully considered, and provisions should be taken in order to assure that joints between floor panels are rigid enough, e.g. by reducing the nail spacing and/or stiffening the floor with nailed steel straps or by applying a second layer of wood-based sheathing which could be nailed or glued to the first layer. Ceccotti et al. (2000) found anyway that even schematizing Light-Frame floors with their actual stiffness the seismic performance of Light-Frame buildings, even with high span-to-depth ratios, is not significantly affected. This may be explained with the fact that usually shear walls are regularly and closely-spaced.

It is also of course possible to model the floor deformability e.g. by schematizing the floor stiffness by means of equivalent cross-bracings with a similar procedure to the one described in §3.3.5.1 (b) for shear walls, or even to model the real beams included in the actual construction, rigidly connected to shell element schematizing the floor sheathing, whose stiffness properties may be calculated according to the procedure explained in §3.3.5.1 (a). However, this schematization of course greatly increases the complexity of the numerical model and thus the elaboration time, especially for multi-storey buildings.

3.3.5.3 Light-Frame connections modelling

Differently from CLT building where the connections modelling plays a crucial role in determining the correct lateral stiffness of the building, being the CLT panels rigid, in Light-Frame construction the shear walls deformation is predominant with respect to the other components due to the uplift restraint and sliding restraint connections which contribute to the rocking deformations of shear walls. Therefore a reliable modelling, even if simplified, may be done by schematizing the shear wall stiffness with the procedure described in §3.3.5.1 and the other connections as rigid.

However, of course, a more correct simulation of the actual structural behaviour may be done by modelling the hold-down and angle-brackets (or anchor bolts) behaviour with a similar procedure to the one described in §3.2.5.3 for CLT buildings.

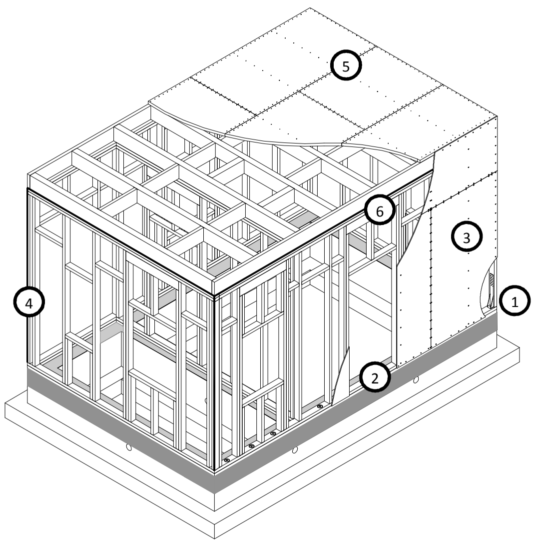


Figure 3-102 Connections in Light Frame buildings: 1 – Hold-downs, 2 – Anchor bolts, angle brackets or screws, 3 – Sheathing-to-framing wall connections, 4 – Vertical screwed joint between perpendicular walls, 5 – Sheathing-to-framing floor connections, 6 – Horizontal screwed or bolted connection between floor panels and walls underneath.

By making reference to Figure 3-102 we can identify 6 types of connections in a Light-Frame building, respectively:

1. Uplift restraint connection at the foundation and at floors made with hold-down anchors.
2. Slip (and also uplift) restraint connection made with anchor bolts, steel brackets or screws.

3. Sheathing-to-framing wall connections.
4. Vertical screwed joint between perpendicular walls.
5. Sheathing-to-framing floor connections.
6. Horizontal screwed or bolted connection between floor panels and walls underneath.

Considering the capacity based design criteria described in §3.2.4, connections #4, #5 and #6 could be modelled as rigid and connection 3 is already modelled in the wall schematization described in §3.3.5.1.

The remaining connections #1 and #2 may be schematized with 2 degrees of freedom link or spring elements as they restrain 2 translational degrees of freedom, which may or may not be located in the exact position of the real connection.

Like for **linear elastic static or dynamic analysis** of CLT buildings, the main problem is the correct evaluation of the connection stiffness, as it has a significant influence on the evaluation of the natural period of the building and of its overall stiffness (even if, as explained above, less important than for the case of CLT buildings).

Therefore there are two possibilities:

1. Making reference to available experimental data to evaluate the connection stiffness.

   Like for CLT construction there is a problem in modelling the real asymmetric behaviour of hold-down connections with linear elastic springs. Again there are two possibilities: either calibrating the stiffness of the linear spring with the procedure described in §3.2.5.3 and showed in Figure 3-30 or not to explicitly model the hold-down connection like in the example described in §3.2.6.

2. The second possibility is to calculate the stiffness of each connection according to the formula proposed by EN1995-1-1 for the calculation of the slip modulus of timber to timber and steel to timber connections like in the procedure described below, considered for an angle bracket connection.

   The horizontal stiffness K_H of the connection is computed based on the slip modulus at serviceability limit state of each fastener K_{con} used to connect the vertical metal plate of each connector to the wall panel and assuming the metal plate to concrete connection as rigid for the wall-foundation connection. This time the formula given in Table 7.1 and Equation (7.1) of Eurocode 5 (CEN, 2009) for the calculation of the slip modulus k_n for nails without predrilling should be used.

$$K_{con} = 2 \cdot \frac{\sqrt{\rho_{m1} \cdot \rho_{m2}}^{1.5} \cdot d^{0.8}}{30} \tag{33}$$

In Equation (10), ρ_{m1}, ρ_{m2} and d are the mean density of the sheathing panel and of the framing timber in [kg/m^3] and the fastener diameter in [mm], respectively, with the values of K_{con} in [N/mm]. It should be underlined however that this equation is valid for steel-to-timber connections. In modern Light-Frame construction with prefabricated walls, angle brackets are placed on the external side of the wall, externally to the wood-based sheathing. In this case therefore the nail connection is working in double shear and the connection is a steel-to-panel-to-timber connection. Therefore the stiffness calculated according to Equation (33) should be reduced.

Again as in §3.2.5.3, for n nails, the horizontal stiffness of the wall-foundation connection is given by:

$$K_H = n \cdot K_{con} \tag{34}$$

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

$$K_H = \frac{1}{\frac{1}{K_{H1}} + \frac{1}{K_{H2}}} \tag{35}$$

In case of **non-linear static or dynamic analysis** it is possible to schematize each connection as a multi-linear plastic link element as described in §3.2.5.3 with the same limits there referenced.

3.3.6 Linear static and dynamic analysis of Light-Frame buildings with SAP2000. Possible procedure and design example

Considering the numerical modelling procedure described in 3.3.5 for the schematization of the main structural components of a Light-Frame building, in this paragraph a numerical model for the linear static and dynamic analysis of Light-Frame buildings is proposed.

The proposed numerical model has been implemented in SAP2000 (CSI, 2000), with a similar procedure to the one described in §3.2.6 for the numerical modelling of CLT buildings and is utilized to perform the modal response-spectrum or the static analysis and to solve the associated equilibrium equations. Again the same pre- and post-processing software specifically developed by Tecnisoft (Modest, Ver. 8.6, 2015) was used to aid in the implementation.

Three types of elements are utilized, namely:

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

The model schematization is the same illustrated in Figure 3-32.

As for CLT walls schematization, the in-plane shear forces transmitted from the walls to the walls underneath can be directly obtained from the axial forces of the horizontal truss elements, while the uplifting vertical forces are obtained from the tensile forces in the vertical truss elements. A rigid diaphragm constraint is used to constrain all nodes at the same level with again a separate constraint is used to constraint the nodes at the bottom of the wall and at the top of the wall underneath.

Like for CLT buildings, the model described above is based on some simplified assumptions:

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

Shell elements are defined with the same length, height and thickness as the associated Light-Frame walls considering an orthotropic schematization as described in §3.3.5.1 (a). The shear connection of the panels to the foundation or to the floor underneath is schematized for each wall with a pair of truss elements whose axial stiffness is calculated with the same procedure described in §3.2.6 and Equations (13)

Vertical truss elements are also used to simulate the deformability of the floor diaphragms along their thickness, namely perpendicular to their plane, as the walls bear on the floor panels. Thus, the modulus of elasticity perpendicular to grain E_{90} of the corresponding grading lumber used for the sill plate in wall frames is selected for the isotropic material properties. Since the shell elements are meshed with a grid of 0.5 meters length, the cross sectional area of the vertical trusses is equal to 0.5 meters times the thickness of the wall above. At the foundation level, the modulus of elasticity of concrete is used for the isotropic material properties. Forces in the vertical truss elements of a wall are then utilized to calculate the tensile forces for the design of the hold-downs, typically installed at each end of the wall to resist overturning moments from horizontal seismic loads.

3.3.6.1 Design example of a two storey case-study building

The case study building considered in the design example is a 2-storey residential Light-Frame structure which was built and tested at the 1D shaking table facility at the University of California, San Diego within the CUREE-Caltech Wood Frame Project referenced in §1.3.1.

3.3.6.1.1 Geometric and structural configuration

The building has a square plan with dimensions of around 4.9x6.1 m and a total height of 6.45 m with a double-pitched roof. The floor plan is symmetric in the N-S direction (direction of shaking) with a large garage door opening of 3.0x2.20 m in the East outer wall and a 0.76x2.00 m door opening in the West outer wall, thus creating an asymmetrical configuration of the horizontal stiffness. East and West outer walls have a symmetric configuration of window openings (two at the first storey and three at the second storey). The net storey height is 2.46 m for the two storeys. Plan and elevation drawings are displayed in Figure 3-103.

The wall frames are made with 2x4 (50x100 mm) studs and bottom and top plates sheathed on one side with 9.5 mm OSB panels with 8d common nails (3.4x63 mm smooth nails) spaced at 150 mm at panel edges and at 300 mm at internal supports.

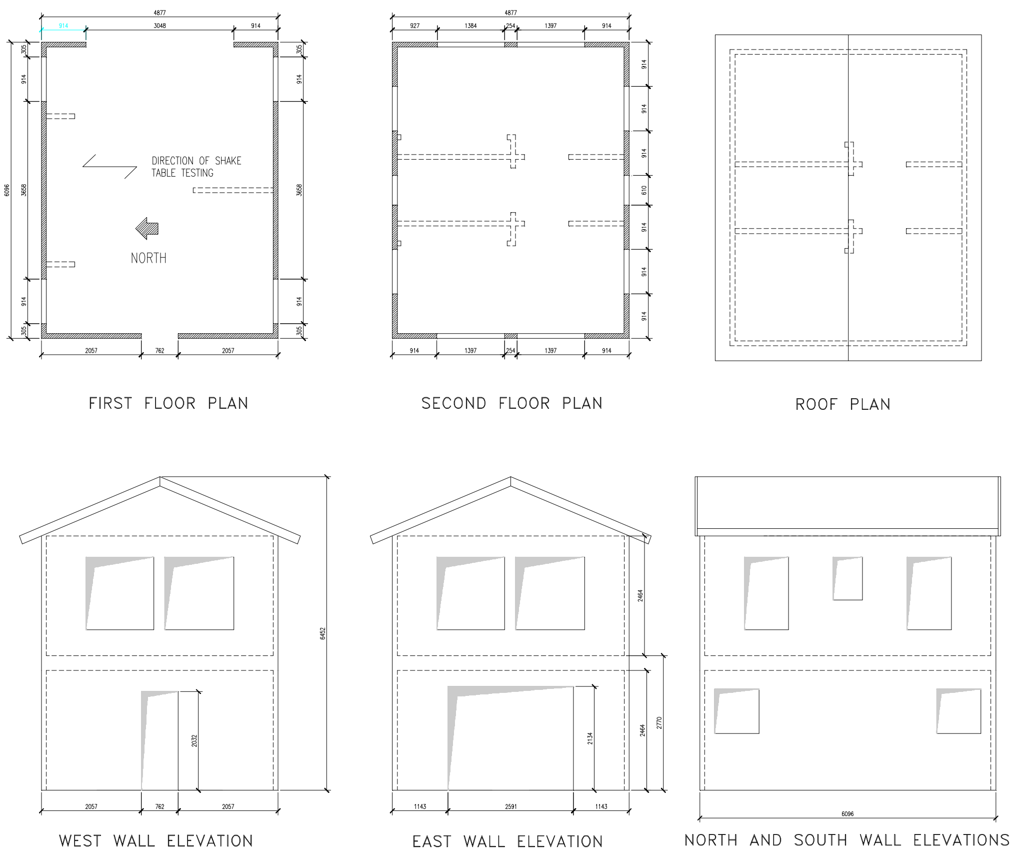


Figure 3-103 Plans and elevations of the case study building (dimensions in mm).

Figure 3-104 View of the test building over the shaking table (after Fischer and Filiatrault, 2000).

The total seismic weight added by means of steel blocks fixed to the floor frame was of 2.00 kN/m^2 for the 1st floor and of 0.9 kN/m^2 for the roof respectively, accounting for permanent and live loads in seismic combination.

3.3.6.1.2 Design spectrum

The same design response spectrum defined in §3.2.6.1.2, and illustrated in Figure 3-35 is defined from Section 3 of Eurocode 8 (CEN, 2004), based on a Type 1 elastic response spectrum with the following parameters:

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

3.3.6.1.3 Numerical model of the test building and preliminary design

Based on the procedure described in §3.3.5, a 3-dimensional numerical model has been developed for the 2-storey case study building. Floor and roof diaphragms are considered rigid in their plane and schematized by means of master-slave constraints applied to the reference nodes. Figure 3-105 shows the undeformed shape of the FE model and Figure 3-106 shows the Wall IDs of the building.

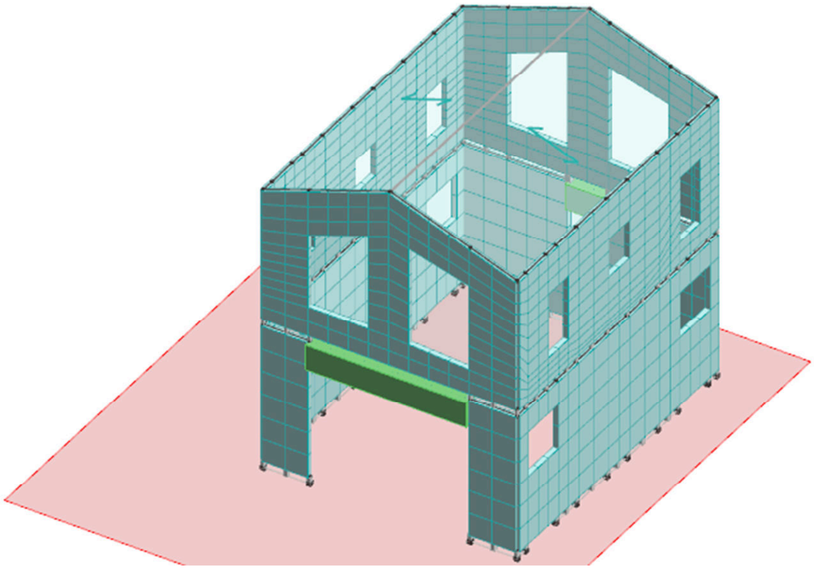


Figure 3-105 Undeformed shape of the building FE model.

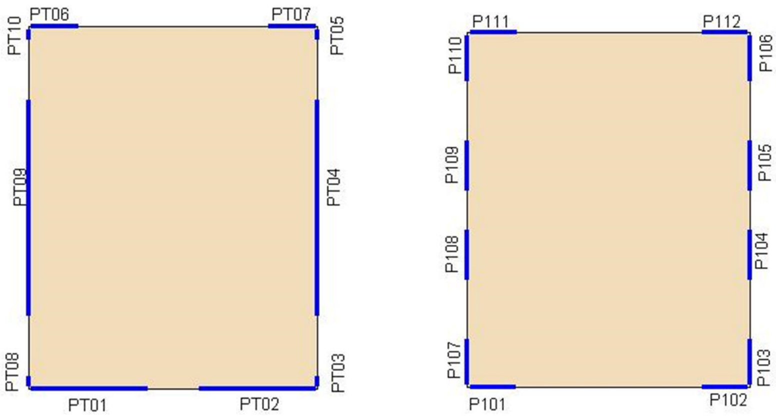


Figure 3-106 Wall IDs of structure for first (left) and second (right) floor.

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

$$T_1 = C_T \cdot h^{3/4} = 0.05 \cdot 6.45^{3/4} = 0.20\,\mathrm{s} \tag{36}$$

The lateral stiffness of the connections of the wall panels to the floor diaphragm below is assumed to be the same per linear meter for all wall assemblies of the structure.

The results of the preliminary analysis in terms of shear per unit length for each wall are presented in Table 3-18. The storey shear forces are 46.1 kN and 56.6 kN for the N-S and E-W direction at the first storey, and 22.2 kN and 22.7 kN for the N-S and E-W direction at the second storey respectively, with a correction factor λ = 0.85 applied according to Section 4.3.3.2.2 of EC8 (CEN, 2004).

Table 3-18 Preliminary analysis and design results.

Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Wall stiffness [kN/m]
PT01	2.01	X	16.8	8.36	d12 A.B.	1.6	3	15.00	0.7
PT02	2.01	X	16.8	8.36	d12 A.B.	1.6	3	15.00	0.7
PT03	0.25	Y	0.6	2.46	d12 A.B.	1.6	1	15.00	0.3
PT04	3.66	Y	27.0	7.39	d12 A.B.	1.6	4	15.00	0.8
PT05	0.25	Y	0.6	2.46	d12 A.B.	1.6	1	15.00	0.3
PT06	0.86	X	6.3	7.30	d12 A.B.	1.6	1	15.00	0.7
PT07	0.86	X	6.3	7.30	d12 A.B.	1.6	1	15.00	0.7
PT08	0.25	Y	0.6	2.47	d12 A.B.	1.6	1	15.00	0.3
PT09	3.66	Y	27.1	7.40	d12 A.B.	1.6	4	15.00	0.8
PT10	0.25	Y	0.6	2.47	d12 A.B.	1.6	1	15.00	0.3
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Wall stiffness [kN/m]
P101	0.86	X	5.5	6.42	d12 Screws	1.6	2	15.00	0.7
P102	0.86	X	5.5	6.42	d12 Screws	1.6	2	15.00	0.7
P103	0.86	Y	2.8	3.20	d12 Screws	1.6	1	15.00	0.7
P104	0.91	Y	2.9	3.20	d12 Screws	1.6	1	15.00	0.7
P105	0.91	Y	2.9	3.20	d12 Screws	1.6	1	15.00	0.7
P106	0.86	Y	2.8	3.20	d12 Screws	1.6	1	15.00	0.7
P107	0.86	Y	2.8	3.20	d12 Screws	1.6	1	15.00	0.7
P108	0.91	Y	2.9	3.20	d12 Screws	1.6	1	15.00	0.7
P109	0.91	Y	2.9	3.20	d12 Screws	1.6	1	15.00	0.7
P110	0.86	Y	2.8	3.20	d12 Screws	1.6	1	15.00	0.7
P111	0.86	X	5.6	6.52	d12 Screws	1.6	2	15.00	0.7
P112	0.86	X	5.5	6.40	d12 Screws	1.6	2	15.00	0.7

Two types of shear connections are used in the design of the building: $\phi 12$ Anchor bolts that are used for the shear-transferring connections of the 1st storey, and $\phi 12$ self-tapping screws that are used for the connections of the second storey. An over-strength factor of 1.6 was applied in the design of the shear connections according to the capacity-based criteria described in §3.3.4. Wall frame and nail diameter and spacing are specified in §3.3.6.1.1. Table 3-18 lists the required number of anchor bolts and screws for the shear connections and the lateral stiffness of each wall.

3.3.6.1.4 Final analysis and design of the building

Based on the results of the preliminary design, the numerical model is updated with the correct stiffness contribution of the shear connections to the horizontal pair of springs and nail spacing of walls, respectively. The building is analysed with the modal response spectrum analysis and Table 3-19 shows the fundamental periods and the mass participation factors for each mode shape of the structure. The first mode shape, with a period of 0.18 s that is similar to the 0.20 s computed with the code equation, is related to translation along the N-S direction of the building. The second mode shape, with a period of 0.11 s, is related to translation along the E-W direction of the building.

Table 3-19 Modal analysis results.

Mode	Period (sec)	Mass participation for translation along N-S direction (%)	Mass participation for translation along E-W direction (%)	Mass participation for rotation along vertical direction (%)
1	0.18	79.41	0.01	4.07
2	0.11	0.03	69.22	0.3
3	0.09	0.84	0.37	73.75
4	0.06	18.62	0.02	1.5
5	0.05	0.01	30.37	0.04
6	0.04	1.09	0.01	20.34
Sum		100.0	100.0	100.0

The results of the response spectrum analysis in terms of shear per unit length, uplift force for each wall, and shear per unit height for each vertical step joint are presented in Table 3-20 together with the required nail spacing for each wall and corresponding strength and stiffness and the required number of shear and uplift connections. The storey shear forces along the N-S direction are 49.0 kN and 25.3 kN for the first and second storey, respectively, while along the E-W direction are 61.1 kN and 23.6 kN. These values are very similar to the values obtained from the lateral force method of analysis.

HTT22 hold-down anchors were used both at the first and second storey to restrain the building uplift connected with 18 ϕ4x60 anker nails to the walls and with 1 class 4.6 ϕ16 steel rod to the foundation and 1 ϕ16 steel bolt for the inter-storey connection with a calculated design strength of 34.74 kN.

Finally Figure 3-107 show the mode shapes of the first, second and third periods corresponding to 0.18 s, 0.11s and 0.09 s. respectively.

Table 3-20 Final analysis and design results.

Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall shear strength [kN]	Wall stiffness [kN/m]
PT01	2.01	X	18.3	9.12	32.7	49.0	107.2	15.7	d12 A.B.	1.6	3	9.00	HTT22	3	18.5	1.2
PT02	2.01	X	18.3	9.12	52.0	16.0	80.4	8.8	d12 A.B.	1.6	3	9.00	HTT22	3	18.5	1.2
PT03	0.25	Y	0.7	2.64	14.6	8.9	30.1	3.1	d12 A.B.	1.6	1	5.00	HTT22	1	0.7	0.8
PT04	3.66	Y	29.2	7.98	21.2	11.6	90.6	41.5	d12 A.B.	1.6	4	10.00	HTT22	1	30.3	1.2
PT05	0.25	Y	0.7	2.76	15.9	13.0	32.1	4.3	d12 A.B.	1.6	1	5.00	HTT22	1	0.7	0.8
PT06	0.86	X	6.2	7.18	11.2	29.6	43.2	6.4	d12 A.B.	1.6	1	6.00	HTT22	2	7.2	1.5
PT07	0.86	X	6.2	7.18	24.7	13.0	50.4	6.2	d12 A.B.	1.6	1	6.00	HTT22	2	7.2	1.5
PT08	0.25	Y	0.5	2.02	15.5	9.4	28.9	3.4	d12 A.B.	1.6	1	7.00	HTT22	1	0.5	0.6
PT09	3.66	Y	29.5	8.06	14.7	10.9	76.1	35.1	d12 A.B.	1.6	4	10.00	HTT22	1	30.3	1.2
PT10	0.25	Y	0.5	1.97	13.7	11.2	36.3	4.7	d12 A.B.	1.6	1	7.00	HTT22	1	0.5	0.6
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall strength [kN]	Wall stiffness [kN/m]
P101	0.86	X	6.5	7.52	5.9	15.5	28.4	3.2	d12 Screws	1.6	2	7.00	HTT22	1	6.7	1.4
P102	0.86	X	6.5	7.53	18.2	7.5	31.2	5.0	d12 Screws	1.6	2	7.00	HTT22	1	6.7	1.4
P103	0.86	Y	3.1	3.59	10.0	11.2	29.5	2.5	d12 Screws	1.6	1	15.00	HTT22	1	3.4	0.7
P104	0.91	Y	2.8	3.06	12.4	10.9	33.8	5.9	d12 Screws	1.6	1	15.00	HTT22	1	3.7	0.7
P105	0.91	Y	2.8	3.03	12.9	10.4	30.6	3.7	d12 Screws	1.6	1	15.00	HTT22	1	3.7	0.7
P106	0.86	Y	3.0	3.48	26.8	6.9	35.4	4.4	d12 Screws	1.6	1	15.00	HTT22	2	3.3	0.7
P107	0.86	Y	3.0	3.47	11.5	8.3	22.1	2.2	d12 Screws	1.6	1	15.00	HTT22	1	3.4	0.7
P108	0.91	Y	2.9	3.17	10.3	11.8	31.7	5.9	d12 Screws	1.6	1	15.00	HTT22	1	3.7	0.7
P109	0.91	Y	2.8	3.06	10.2	12.4	32.1	5.9	d12 Screws	1.6	1	15.00	HTT22	1	3.7	0.7
P110	0.86	Y	3.2	3.71	26.9	12.6	46.7	2.9	d12 Screws	1.6	1	15.00	HTT22	2	3.3	0.7
P111	0.86	X	6.2	7.21	3.7	4.0	17.6	6.2	d12 Screws	1.6	2	7.00	HTT22	1	10.2	1.5
P112	0.86	X	6.1	7.07	3.9	3.7	17.3	5.5	d12 Screws	1.6	2	7.00	HTT22	1	8.9	1.4

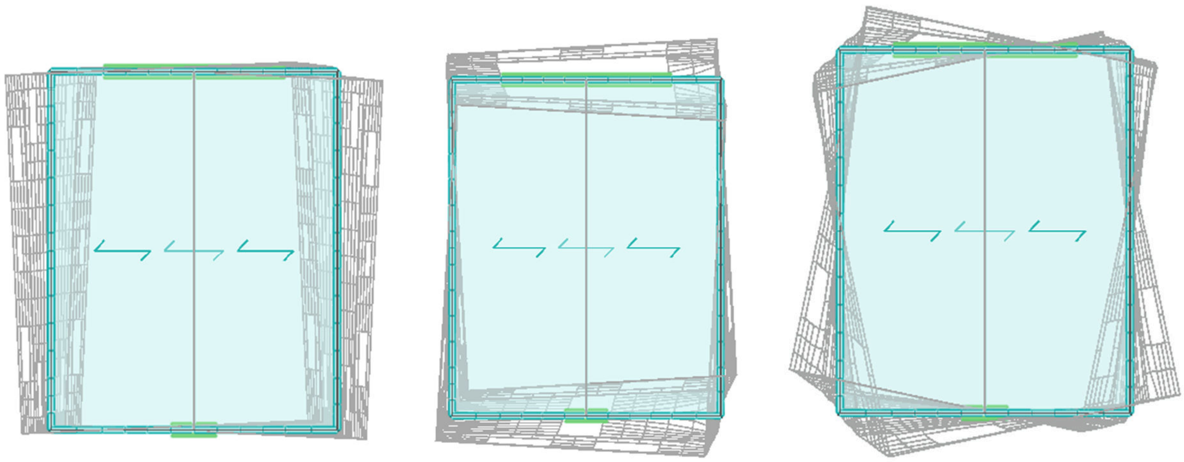


Figure 3-107 First, second and third mode shape of the building corresponding to periods of 0.18s, 0.11s and 0.09s respectively.

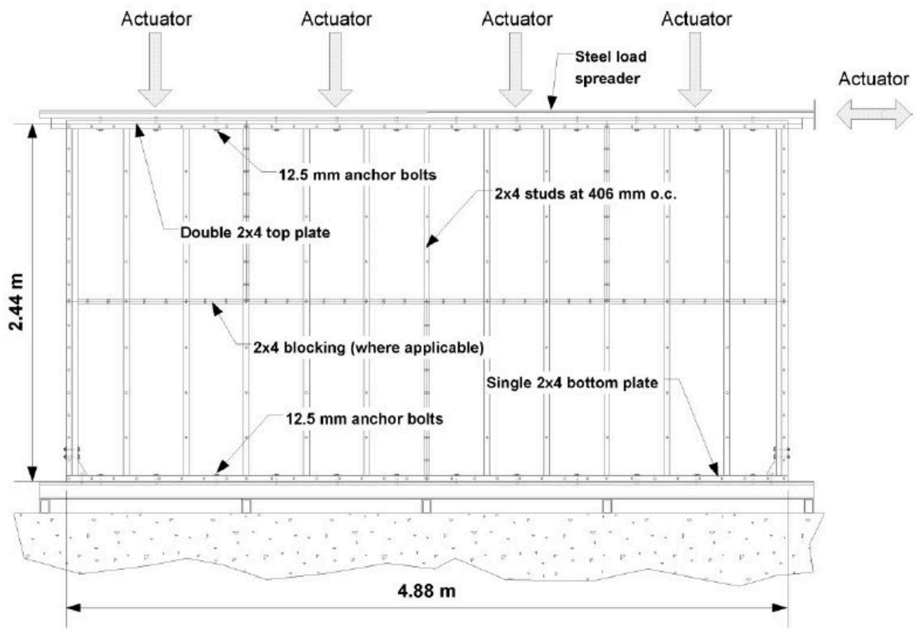
3.3.7 Non-linear dynamic analysis of Light-Frame buildings with SAP2000

Like in §3.3.6 for CLT buildings, SAP2000 (CSI, 2000) was used to perform non-linear analysis of Light-Frame buildings by modelling Light-Frame walls with an equivalent frame using a pair of elastically deformable elements (links or springs) hinged on a frame of rigid hinged elements and calibrated on the bending stiffness and shear stiffness of the Light-Frame wall similarly to the schematization provided in Figure 3-101. Hold-down and shear connections may be schematized by using the multi-linear plastic link elements to simulate the non-linear behaviour of mechanical joints in timber structures like in the procedure described in this paragraph.

3.3.7.1 Calibration process for walls

The calibration procedure for Light Frame walls is based on the results of cyclic tests conducted at FPInnovations (at that time Forintek Canada Corp.) in Vancouver on shear walls having different geometries and structural properties, sheathed with 9.5 mm and 12.5 mm OSB panels and with and without GWB panels on the other side and with different amount of vertical load (Karacabeyli and Ceccotti 1996, Karacabeyli and Ceccotti 1998). One wall specimen, named wall 46-02, was taken as example for the model calibration. The wall specimen has a length of 4.88 m and height of 2.44 and was loaded with a uniformly distributed constant load of 18.2kN/m applied on the top of the wall, as is illustrated in Figure 3-108 left. Common nails (3 mm in diameter and 65 mm in length) were used to nail sheathing to the framing; the nail spacing was 150 mm along panel edges and 300 mm elsewhere. Simpson hold-down HD2A were used at the ends of the shear wall and 12.5 mm anchor bolts spaced at 400 mm to connect the bottom sill plate were used.

The SAP2000 model consists of four pinned rigid straight members and two symmetric non-linear diagonal springs to simulate the wall behaviour, two asymmetric non-linear springs at the end of the wall to simulate the hold-down behaviour and two cross horizontal linear springs to simulate the anchor bolts connection. Figure 3-108 right shows the wall model and the SAP 2000 model while Figure 3-109 shows the comparison between test data and numerical analysis for a wall sheathed with 12.5 OSB panels on one side connected to the framing members with common nails as described above and the vertical load applied on the tests wall concentrated in the two upper model nodes.



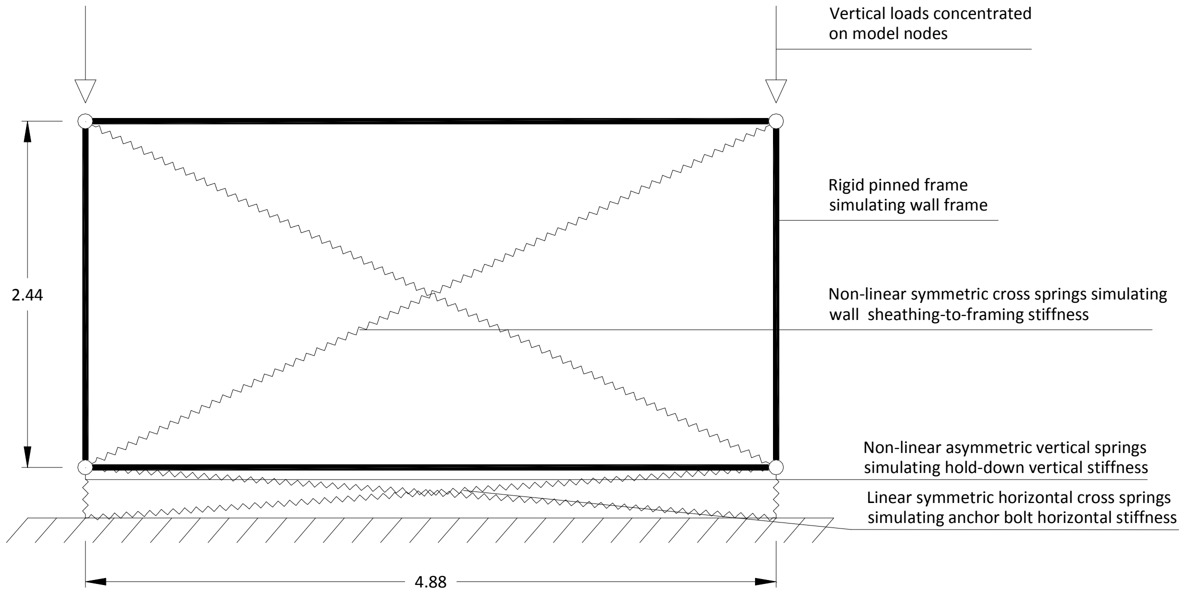


Figure 3-108 Tested wall (after Ceccotti and Karacabeyli, 2002) and model schematization.

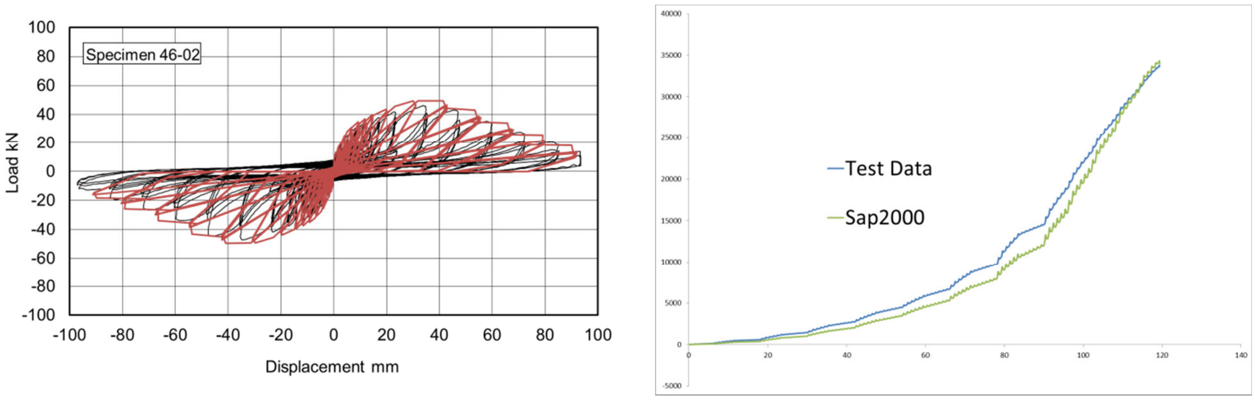


Figure 3-109 Comparison in terms of cyclic behaviour and dissipated energy between test and model.

The comparison both in terms of hysteresis curves and dissipated energy is by all accounts quite satisfactory.

3.3.7.2 Numerical analysis of a full-scale 3 storey building

The validation of the SAP2000 non-linear model for Light-Frame buildings was made, as in §3.2.7.3 for CLT structures, by comparing a 3D model with experimental test results of a full scale 2 storey building, tested within the CUREE-Caltech Woodframe Project at the 1D shaking table facility at the University of California, San Diego and already referenced in §1.3.1. The building is the same considered for the design example of §3.3.6.1, with a simple rectangular plan with dimensions of approximately 5x6m with openings on all the four sides and with a large garage door opening at the first level in order to introduce a torsional eccentricity in the lateral load resisting system. Detail of plans and elevation are illustrated in Figure 3-103. As already described in §1.3.1, the building was subjected to 5 shakes in total in the N-S direction, 4 of which were scaling reproduction of the 1994 Northridge Earthquake recorded at Canoga Park with PGA ranging from 0.05g to 0.5g and the fifth one the unscaled reproduction of the 1994 Northridge Earthquake recorded at Rinaldi with a maximum PGA of 0.89g.

Table 3-21 Details of the ground motions used for the seismic tests (after Folz et al., 2001).

Seismic test Level	Test designation	Ground motion	Amplitude scaling factor	Peak Ground Acceleration [g]
1	9.S.1	1994 Northridge Canoga Park	0.12	0.05
2	9.S.2	1994 Northridge Canoga Park	0.53	0.22
3 3R	9.S.3 9.S.3R	1994 Northridge Canoga Park	0.86	0.36
4	9.S.4	1994 Northridge Canoga Park	1.2	0.50
5	9.S.5	1994 Northridge Rinaldi Park	1.0	0.89

The natural period measured at the beginning of the whole series of shakings was 0.25 s and after the Level 5 test was 0.34 s with a 35% increase. As already described in §1.3.1 the test results were used for an international benchmark in order to assess the state-of-the-art of 3D numerical models to predict the inelastic behaviour of Light-Frame structures, by making a comparison between test results and model prediction for a number of instrument locations placed at the first and second storey and over the roof to measure displacement and accelerations relative to the shaking table.

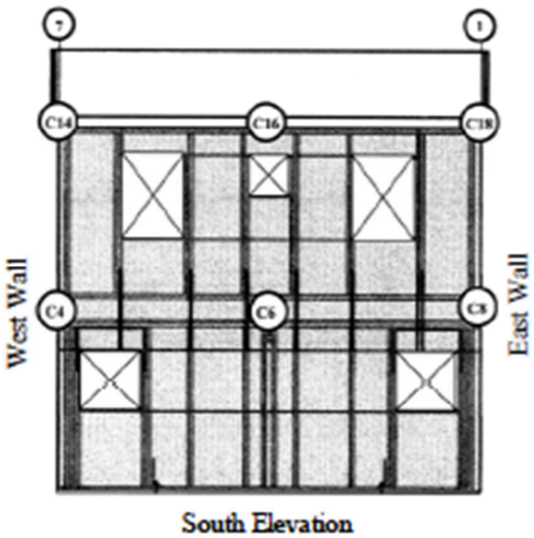


Figure 3-110 Location of relative displacement measurements along the South wall of the test structure (after Folz et al., 2001).

The layout of the model is shown in Figure 3-111. Light-Frame walls are modelled with the same schematization described in §3.3.7.1 with four pinned rigid straight members and two symmetric non-linear diagonal springs to simulate the wall behaviour, two asymmetric non-linear springs at the end of the wall to simulate the hold-down behaviour and two cross horizontal linear springs to simulate the anchor bolts connection. Floor and roof diaphragms are considered rigid in their plane and schematized by means of master-slave constraints applied to the reference nodes. Masses were concentrated on model nodes and a 5% Raleigh damping was applied on masses. A kinematic constraint of rigid diaphragm was applied also the wall base nodes.

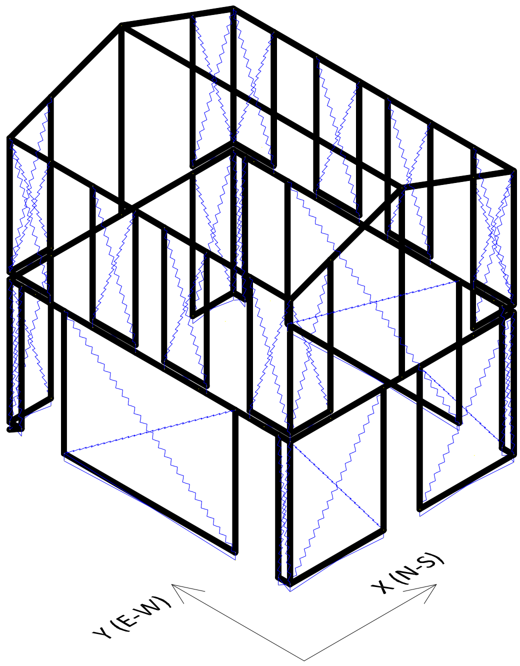


Figure 3-111 Layout of the 3D model used for the non-linear analysis. Black thick lines represent rigid frame elements and blue lines represent non-linear link elements simulating Light-Frame walls, hold-downs and shear connections behaviour.

The validation was made only with Level 5 test with the 1994 Northridge Rinaldi earthquake. Table 3-22 shows the comparison between the test results and the numerical model in terms of fundamental period, maximum displacements at each level in the North and South side of the building and in terms of maximum uplifts.

Table 3-22 Comparison between test results and numerical modelling.

	Period [s]	**Displacements [mm]**					
		C4	C6	C8	C14	C16	C18
Test	0.25	55.4	72.5	66.0	96.7	109.6	108.4
Model	0.25	32.3	55.6	81.4	70.5	90.5	109.7
Diff. [%]	0%	-42%	-24%	23%	-27%	-17%	1%

The results in terms of fundamental period and maximum displacements show a very good agreement. However the torsional behaviour observed with the numerical simulation was much more evident than the one observed during the test. In any case the proposed modelling method proved to be suitable for the non-linear analysis of Light-Frame buildings.

References – Chapter 3

Casagrande D., Rossi S., Sartori T., Tomasi R. [2012] "Analytical and numerical analysis of timber framed shear walls" World Conference on Timber Engineering 2012, WCTE 2012, 5, pp. 497-503.

Casagrande D., Sartori T., Tomasi R. [2014] "Capacity design approach for multi-storey timber-frame buildings." Proc., The International Network on Timber Engineering Research INTER, Bath, United Kingdom, paper 47-15-3.

Ceccotti A., Follesa M. [2006] "Seismic Behaviour of Multi-Storey X-Lam Buildings." Proceedings of 426 COST E29 International Workshop on Earthquake Engineering on Timber Structures, pages 81-95, Coimbra, Portugal.

Ceccotti, A., Follesa, M., Karacabeyli E. [2000] "3D seismic analysis of multi-storey wood frame construction" Proceedings of 6th World Conference on Timber Engineering (WCTE), Whistler Resort, Canada, 2000.

Ceccotti A., Follesa M., Lauriola M.P. [2007] "Quale fattore di struttura per gli edifici multipiano a struttura di legno con pannelli a strati incrociati?" XII Convegno ANIDIS L'ingegneria sismica in Italia, Pisa 2007.

Ceccotti A., Karacabeyli E. [1996] "Test Results on the Lateral Resistance of Nailed Shear Walls". International Wood Engineering Conference, New Orleans, USA, pp:V2,179-186, 1996.

Computers & Structures Inc. [2000]. "SAP2000—Integrated finite element analysis and design of structures. Ver. 14" Computers & Structures Inc.: Berkeley, CA.

European Committee for Standardization (CEN), [2009] "Eurocode 5 – Design of timber structures – Part 1-1: General rules and rules for buildings", Brussels, Belgium.

European Committee for Standardization (CEN), [2004] "Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings", Brussels, Belgium.

Fischer D., Filiatrault A. [2000] "Shake Table Tests of a Simplified Two-Story Single- Family House. Shake Table Test Descriptions, Test Phase Descriptions, Instrumentation, and Construction Drawings", CUREE-Caltech Woodframe Project, Task 1.1.1, 2000.

Follesa M., Lauriola M.P. et al. [2009] "Linee guida per l'edilizia in legno in Toscana" – Regione Toscana, Italy.

Follesa M., Fragiacomo M., Lauriola M.P., [2011a] "A proposal for revision of the current timber part (Section 8) of Eurocode 8 Part 1" Proceedings of 44th CIB W18 Meeting, Alghero 2011, paper n.44-15-1.

Follesa M., Maione F., Palanga G. [2011b] "Edifici a struttura di legno: progettazione e realizzazione" - Made Expo e FLA, 2011.

Follesa M., Christovasilis I., Vassallo D., Fragiacomo M., Ceccotti A. [2013] "Seismic design of multi-storey CLT buildings according to Eurocode 8." Ingegneria Sismica, Special Issue on Timber Structures, n. 04/2013, pp. 27-53.

Folz B., Filiatrault A., Ming Uang C., Seible F. [2001] "Blind predictions of the seismic response of a two-story woodframe house: an International Benchmark". Report No. SSRP – 2001/15. Division of Structural Engineering, University of California, San Diego, La Jolla, California 92093-0085.

Forest Products Laboratory. [2010]. "Wood handbook—Wood as an engineering material". General Technical Report FPL-GTR-190. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Fragiacomo M [2013] "Seismic behaviour of Cross-Laminated timber buildings: numerical modelling and design provisions". Cost Action FP1004. Focus Solid Timber Solutions – European Conference on Cross Laminated Timber (CLT), Graz 21-22 May 2013.

Fragiacomo M., Dujic B., Sustersic I. [2011] "Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions." Engineering Structures, Special Issue on Timber Structures, Vol. 33 No. 11, pp. 3043-3053.

Fragiacomo M, Rinaldin G. [2011] "Advanced models for seismic analyses of timber buildings." Proceedings of the "Grazer Holzbau-Fachtagung - 9. Gra FHT'11", Technical University of Graz, Austria, 30 pp.

Gavric I., Ceccotti A., Fragiacomo M. [2011] "Experimental cyclic tests on cross-laminated timber panels and typical connections." Proceedings of the 14th ANIDIS Conference, Bari (Italy), September 2011.

Gavric I., Fragiacomo M, Ceccotti A. [2012] Strength and deformation characteristics of typical X-Lam connections. Proceedings of the 12th World conference on timber engineering, Auckland (New Zealand), July 2012.

Gavric I., Fragiacomo M., Ceccotti A., [2013] "Capacity seismic design of x-lam wall systems based on connection mechanical properties" Proceedings of 46th CIB W18 Meeting, Vancouver 2013, paper n.46-15-2.

Gavric I. [2013a] "IVALSA X-Lam Connection Tests Report". Research report, CNR-IVALSA, San Michele all'Adige (TN), Italy.

Gavric I. [2013b] "IVALSA X-Lam Wall Tests Report". Research report, CNR-IVALSA, San Michele all'Adige (TN), Italy.

Gavric I. [2013c] "Seismic behaviour of cross-laminated timber buildings." Phd Thesis, Università degli Studi di Trieste, Italy.

Humbert J, Boudaud C, Baroth J, Hameury S., Daudeville L. [2014] “Joints and wood shear walls modelling I: Constitutive law, experimental tests and FE model under quasi-static loading” Engineering Structures 65 (2014) pp. 52–61.

Jorissen A., Fragiacomo M. [2011] “General notes on ductility in timber structures”, Engineering Structures, Vol. 33, N. 11, 2011, pp. 2987-2997.

Karacabeyli E., Ceccotti A. [1996] “Test results on the lateral resistance of nailed shearwalls”. Proceedings of the International Wood Engineering Conference (IWEC), New Orleans, USA, 1996. Vol. 2, pp. 179-186.

Karacabeyli E., Ceccotti A. [1998] “Nailed wood-frame shearwalls for seismic loads, test results and design considerations”. Proceedings of the Structural Engineers World Congress (SEWC), San Francisco, USA, 1996. Paper Reference T207-6.

Rinaldin G., [2013] “So.ph.i. software version 4.52”, http://giovanni.rinaldin.org/, Internet site.

Rinaldin G., Poh’sie G.H., Amadio C., Fragiacomo M. [2013] “Modelling of seismic behaviour of light-frame timber structures.” Ingegneria Sismica, Special Issue on Timber Structures, No. 4/2013, pp. 82-98.

Sartori T., Tomasi R. [2013] “Experimental investigation on sheathing-to-framing connections in wood shear walls” Engineering Structures, 56, pp. 2197- 2205.

Schick M., Vogt T., Seim W. [2013] “Connection and anchoring for wall and slab elements in seismic design”. Proceedings of the 46th CIB-W18-Meeting, paper 46-15-4. Vancouver, Canada, 2013.

Sustersic I., Dujic B., [2012]. “Simplified cross-laminated timber wall modelling for linear elastic seismic analysis”, 45th CIB W18, Vaxio, Sweden, paper 45-15-6.

Tecnisoft s.a.s., [2014]. “ModeSt – Version 8.6”, Prato, Italy.

Tomasi R., Sartori T. [2013] “Mechanical behaviour of connections between wood framed shear walls and foundations under monotonic and cyclic load” Construction and Building Materials, 44, pp. 682-690.

Uibel T, Blaß H.J., [2006]. "Load Carrying Capacity of Joints with Dowel Type Fasteners in Solid Wood Panels" 39th CIB W18, Firenze, Italy, paper 39-7-5.

Uibel T, Blaß H.J., [2007]. "Edge joints with dowel type fasteners in cross laminated timber" 40th CIB W18, Bled, Slovenia, paper 40-7-2.

4 – Design of mixed CLT-Light Frame buildings: numerical analysis and proposal for code implementation

4.1 Introduction

The two main structural systems widely used in Italy and Europe for the construction of multi-storey timber buildings are the Light-Frame and the CLT systems, which have been described in detail in Chapter 3 (see Figure 4-1).

Figure 4-1 Erection of a 4 storey CLT residential building near Varese, Lombardia, northern Italy (left), and of a 3-storey Light-Frame residential building near Lucca, Tuscany, central Italy (right).

Being both systems widely used, there is a growing demand by architects and designers about the possibility of using both the structural types together as the lateral-load resisting systems in the construction of multi-storey buildings, especially in earthquake prone areas. A typical example is the design of Light-Frame buildings with CLT stairs and lift cores or CLT buildings with Light-Frame partition walls. The possibility of using hybrid CLT/Light-Frame construction in the same building allows the best performance in terms of mechanical, thermal and acoustic properties of the CLT panels to be coupled with the lightness and ease of construction of Light-frame walls (Figure 4-2).

Another option, even if less common in European Countries where the construction of multi-storey timber buildings has a less enduring tradition, is the use of structural components of one of the two systems for the seismic retrofit of existing timber buildings, like e.g. the use of stiffening CLT walls inside existing Light-Frame buildings, as it has been investigated within the NEES-Soft Project (van de Lindt et al., 2014).

As explained in Chapters 1, 2 and 3, according to the force-based method of most seismic design codes, the design of ductile structures in earthquake-prone regions should be carried out via an elastic (static or dynamic) analysis where the seismic actions are reduced using an Action Reduction Factor (ARF, which in

Eurocode 8 (CEN, 2004) is called behaviour factor q) in order to account for the energy dissipation of the ductile structure. In this way a non-linear structure can be elastically designed for seismic actions, with the structural ductility only implicitly considered when evaluating the Action Reduction Factor. This method obviously cannot be used when two Lateral Load-Resisting Systems (LLRS) with different non-linear responses and therefore different ARF are used to resist the seismic actions in the same direction. If different systems are used in the same direction, e.g. Light-frame buildings with CLT walls, a more detailed investigation using non-linear static or dynamic analysis should be carried out to design the building.

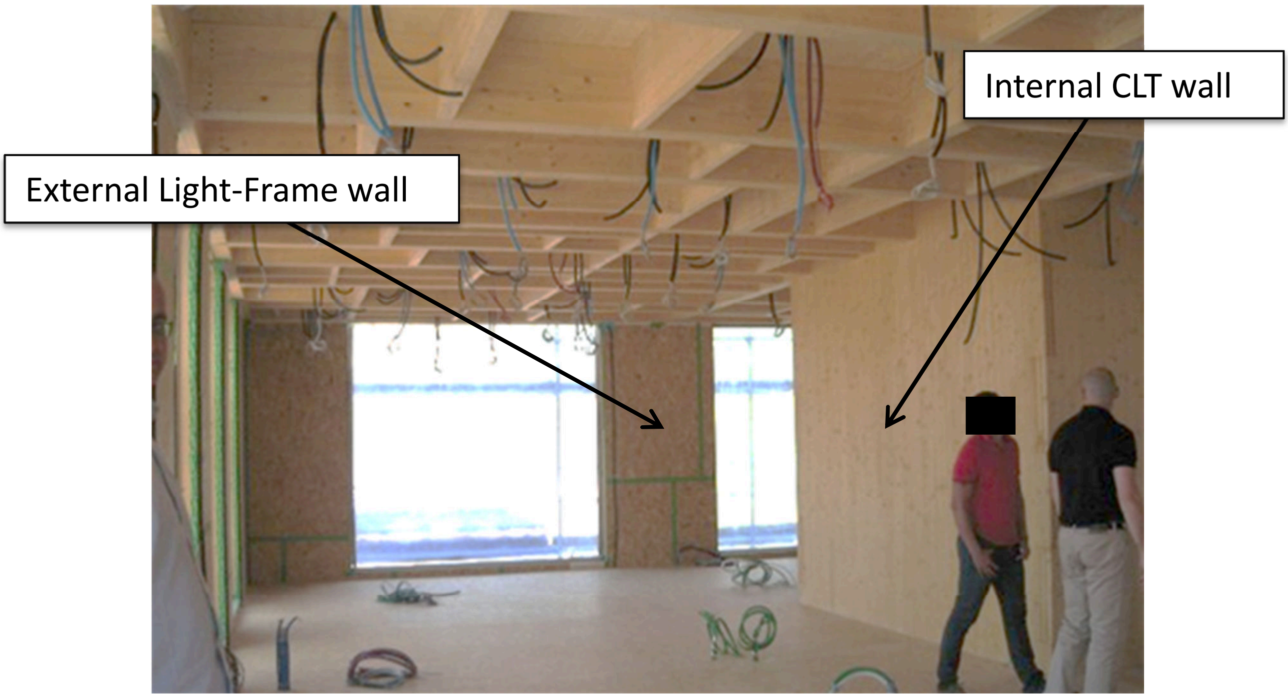


Figure 4-2 Construction of a mixed Light-Frame/CLT building in northern Italy.

However, as stated also in §3.1, non-linear analysis are a complex procedure which require the knowledge of the non-linear behaviour of the ductile structural components and the availability of a suitable finite element software incorporating a proper non-linear model which could accurately reproduce the hysteretic behaviour of dissipative zones. Moreover, too much complicated design methods could lead to possible mistakes in the modelling procedure and thus to non-conservative results.

Eurocode 8 (CEN, 2004) does not provide any specific provision for the seismic design of timber buildings with different LLRS. Some provisions are only given in the relevant chapter (Chapter 7) for composite steel-concrete structures, which are however treated as single structural system, according to the corresponding description provided, with their relevant value of the behaviour factor q. According to the National Building Code of Canada (NBCC 2010), in case of hybrid structures with combination of different type of seismic force modification factor (R_dR_0 according to NBCC 2010) acting in the same direction and in the same storey, the lowest value of R_dR_0 should be taken.

Therefore, besides non-linear design methods, simple design provisions which could be applied by means of linear analysis methods within the Eurocode 8 force-based philosophy are needed in order to safely allow the seismic of hybrid multi-storey timber buildings. This could be done either by (i) clearly defining new hybrid structural types with their associated values of the behaviour factor q or (ii) by defining simple design formula, based on extensive numerical simulations or analytical methods, which could be applied for the calculation of the behaviour factor q of the hybrid system.

4.2 Seismic design of buildings with different Lateral Load Resisting Systems: literature review

In view of the above considerations, several studies have been recently conducted in order to better investigate the seismic design of hybrid multi-storey timber buildings and find some provisions which could be incorporated inside code proposals. Chen et al. (2014) developed an idealized elasto-plastic mechanics-based model for the evaluation of the system ductility ratio of hybrid systems with two different types of LLRS. According to this simplified model, the ductility of the whole system can be interpreted as a function of stiffness ratio, strength ratio, ductility ratio and failure displacement of the two subsystems. Besides, an empirical model for estimating the ductility ratio μ, and the ductility-related force modification factor R_d according to the National Building Code of Canada (NBCC 2010), of hybrid buildings containing two types of LLRS is proposed and validated through non-linear time-history analysis conducted on a reference case-study hybrid buildings. The results showed that the current approach of NBCC 2010 of considering the lowest value of the force modification factor R_dR_0 between the two systems seems too conservative.

Guo et al. (2014) studied the seismic performance of hybrid podium structures with lower steel frames storeys and upper Light wood-frame structure up to a maximum of 3 storeys, according to the Chinese code for design of timber structures (GB50005), which is expected to be released in 2015. This new code will allows this type of hybrid structures up to a maximum of 7 storeys, with up to 3-storey wood frame structure on top of concrete, masonry or steel structures. A case study building with plan dimension of 9.6x13.2 m was studied with 12 different combinations of lower steel frame and upper light-frame structure analysed with linear static, response spectrum and non-linear time-history analysis. The analyses were conducted with SAP2000 and wood light-frame shear walls were simulated with a simplified equivalent model of lateral resistant element with three rigid beams and a pair of spring elements, similarly to the procedure described in §3.3.5.1 (b). The results demonstrated that this type of structure could be safely designed using the linear static method of analysis and the response spectrum analysis, even if no specification is given on the force modification factor used in the design. The recommendation is to improve the rigidity of lower steel frames by means of cross bracings when lower steel frame podium structure extends up to 4 storeys. The floor diaphragms were schematized both as rigid and flexible in the analysis. The results showed that no significant differences in the values of base shear were found between the two different assumption, therefore the assumption of rigid diaphragm could be considered acceptable.

Within the framework of the NEES-Soft Project, already described in §1.3.1, a slow pseudo-dynamic hybrid simulation on a 6.1x7.3 m plan dimension and 8.2 m high 3-storey wood Light-Frame structure was performed on strong floor at the NEES facility at the State University of New York at Buffalo (Pang et al., 2014). The hybrid structure was composed of a numerically simulated first storey with a large open-space in the first story for parking garage and two physically constructed residential upper storeys, thus making the building prone to weak first storey collapse during moderate to high intensity earthquakes. This hybrid testing technique allowed the evaluation of different types of seismic retrofit without the need to physically re-construct the first story after each test (for further details regarding the different type of retrofits employed see van de Lindt et al, 2014). The numerical substructure was modelled using the Timber3D software package developed within the NEES-Soft Project which incorporated the hysteretic models for the retrofit options being considered. The hybrid tests were performed using a testing controller with a program written in Matlab/Simulink which transferred the computed displacements to the actuator attached to the floor and roof diaphragms of the physical two storey structure and returned the restoring forces measured from the actuators to the numerical model for solving the displacements for the subsequent step. The results of the hybrid tests showed that this slow hybrid simulation procedure is a feasible and economical testing option for evaluating the structural efficiency of different retrofit techniques for soft-story wood-frame buildings.

Finally, Zhou et al. (2014) investigated the seismic force modification factor $R=R_dR_0$ and the value of the fundamental vibration period according to the National Building Code of Canada (NBCC 2010) of 67 multi-storey hybrid Light-Frame/masonry buildings with different combinations of (i) number of storeys, (ii) properties of the two sub-systems and (iii) properties of the connection between them through 2D non-linear dynamic analysis performed with the commercial software ABAQUS V6-10 incorporating a user-developed subroutine that includes the Bouc-Wen-Baber-Wen (BWBN) model to describe the hysteresis behaviour of various building components. The analysis results show that failure mode of hybrid building is influenced by the relative stiffness of the wood, masonry and connection system and by the ultimate deformation of the sub-systems, suggesting therefore that an R value larger than the lowest value of the two sub-systems can be assigned to design of the hybrid building. Empirical equations were proposed for the evaluation of R_d and R_0 of the hybrid building based on: (i) the different design method, (ii) the resistance ratio of the masonry core to hybrid building, (iii) the relevant R_d and R_0 values of the only wood-frame and only masonry building, (iv) the fundamental period T of the hybrid building, for which a formulation based on the stiffness ratio of masonry to the whole building stiffness, and the relevant fundamental periods of the only wood-frame and only masonry building was proposed.

4.3 Evaluation of the seismic performance of hybrid multi-storey CLT/Light Frame buildings

As discussed in §1.2, the design of timber structures according to Eurocode 8 (CEN, 2004) may be undertaken through a simple global linear elastic analysis where the elastic seismic actions are reduced by the value of the behaviour factor q, which incorporates the actual non-linear behaviour of the structure and the capacity to dissipate energy and withstand large deformations before the collapse is reached. This procedure, as already underlined in §1.2, is therefore strictly dependent on the need of establishing the correct value of the behaviour factor q associated with the reference structural type taken into consideration in order to undertake a reliable seismic design. According to Ceccotti and Sandhaas (2010), a coupled test-modelling approach can be used for the evaluation of the behaviour factor q of timber buildings. Monotonic and cyclic tests carried out on sub-assemblies such as connections or shear walls are used to calibrate a suitable hysteretic model capable to accurately reproduce the actual non-linear behaviour of such elements incorporated in a proper FE analysis program. The numerical model of the structure is then subjected to different seismic inputs with the peak ground accelerations (PGA) increased until a near-collapse criterion, previously established, is reached. The quantification of the q-value can be made by simply dividing the obtained $PGA_{near\text{-}collapse}$ values by the design PGA used to elastically design the building, according to Equation (1):

$$q = \frac{PGA_{near-collapse}}{PGA_{design}} \quad (1)$$

where $PGA_{near\text{-}collapse}$ is the PGA at the near collapse state and PGA_{design} is the design PGA with q=1.

According to Ceccotti and Sandhaas (2010), a second possible method is to assess the value of the behaviour factor q usingEquation (2):

$$q = \frac{V_{elastic}}{V_{plastic}} \quad (2)$$

where $V_{eslastic}$ is the seismic base shear calculated assuming linear-elastic behaviour and $V_{plastic}$ is the seismic base shear accounting for the real non-linear behaviour. This latter formulation, as stated also in Ceccotti and Sandhass (2010), is “code-independent” as it estimates a real q-value based on the earthquake frequency content and the building characteristics instead of the code-based formulation of Equation (1). Pozza et al. (2009) showed that by using the formulation given by Equation (2), a more uniform distribution of the results is obtained, with less variability compared to Equation (1) and with lower values of the behaviour factor q. On the other hand, this method requires higher computational time, since two dynamic analyses must be carried out in order to evaluate the corresponding base shear values.

Following the first method proposed by Ceccotti and Sandhaas (2010), the evaluation of the seismic design parameters of hybrid multi-storey CLT/Light Frame buildings is made according to Equation (1) with the following procedure:

1. A case study 4 storey building is selected (Figure 4-3), adapted from Vassallo et al., 2013, and designed in 4 different configurations of Light-Frame and CLT walls, starting from an all Light-Frame wall configuration and ending with an all CLT wall configuration (Figure 4-4).
2. The building is designed with the equivalent static force procedure with q=1 according to Eurocode 8 (CEN, 2004) and Eurocode 5 (CEN, 2009) for one of the most hazardous seismic regions of the Italian territory, i.e. Tolmezzo, Friuli, and for soil class B. The preliminary elastic design is made with SAP2000 software package according to the design methods proposed in §3.2.5 and §3.3.5.

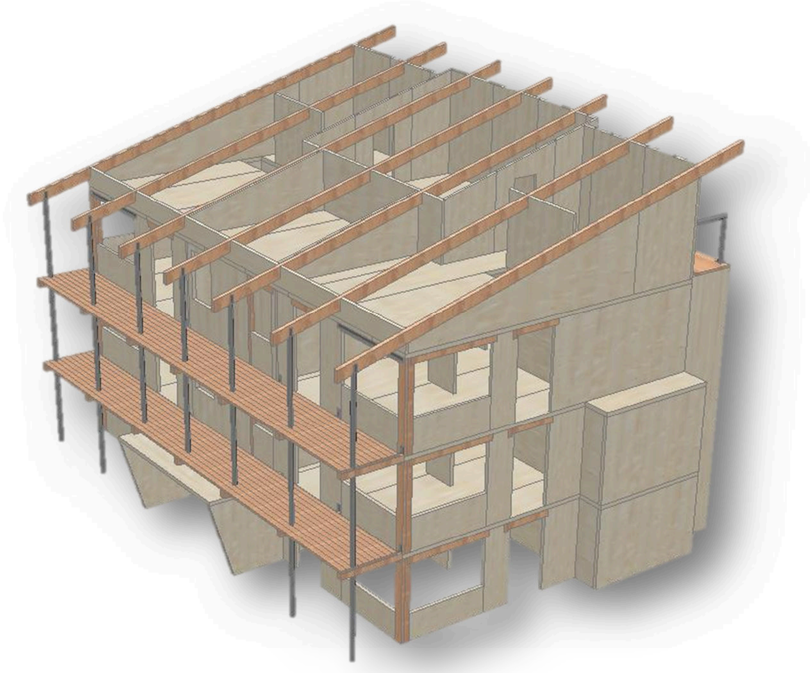


Figure 4-3 Four storey case study building selected for the evaluation of the seismic design parameters of hybrid multi-storey CLT/Light Frame buildings.

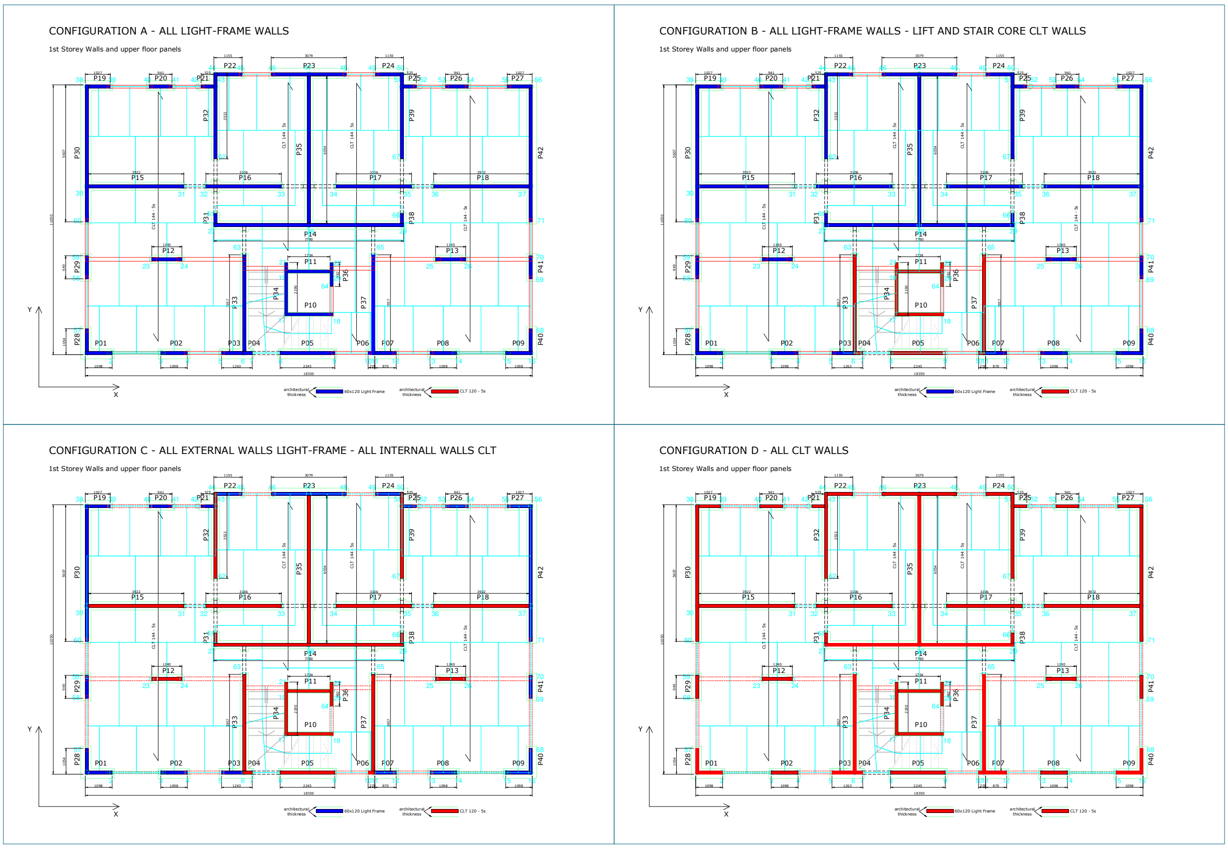


Figure 4-4 Four different first-storey wall configurations of Hybrid Light-Frame and CLT wall buildings. A - All Light-Frame walls. B - All Light-Frame walls with lift and stair core made with CLT walls. C – All external Light-Frame walls and all internal CLT walls. D – All CLT walls.

3. According to the preliminary design results, a 3D non-linear model of the 4 buildings is implemented in the FE program DRAIN-3DX (Prakash and Powell, 1994) which incorporates a suitable model developed to represent the hysteretic behaviour of semi-rigid mechanical joints in timber structures under reversed-cyclic loading (Ceccotti and Vignoli, 1989; Ceccotti et al., 2000). The model has been purposely enhanced with new features such as the strength and stiffness degradation and is described in §4.5.
4. The structural components (Light-Frame shear walls and connections of CLT walls) of the 3D non-linear building model are calibrated based on experimental results under fully reversed cyclic loading on structural sub-assemblies (shear walls) or connections, considering as calibration parameters maximum force, maximum displacement and amount of dissipated energy and then applied to the 3D model by simply adjusting the input parameters to the requirements of the design performed in step 2. This adjustment is made in different ways for the two structural systems. For Light-Frame walls is based on the assumption of a linear relationship between strength and stiffness and the lateral stiffness is assumed to be linearly proportional to the length of the walls; therefore if e.g., according the preliminary design results, the strength of the shear wall per meter will result 10 times the linear strength of the reference wall used in the calibration procedure, the stiffness parameters will be

multiplied by 10, with the displacement corresponding to yield, maximum load and collapse left unchanged. For the CLT system the procedure is more straightforward; from the preliminary design results will result a given number of the "basic" connection elements (hold-downs and angle brackets used in the calibration procedure), and therefore the input parameters of stiffness for each spring will be multiplied by the number of connector resulting from the preliminary design results (for the angle brackets connection divided by two as the connection is represented with two diagonal springs according to the schematization of CLT walls of Figure 4-17) with the displacement limits left unchanged.

5. A suitable set of 20 ground motion records is chosen, 6 of which being international historical quakes widely used in the non-linear dynamic analysis of structures (1995 JMA-Kobe, 1940 Imperial Valley-El Centro, 1994 Newhall-Northridge, 1989 Corralitos-Loma Prieta, 1999 Yapi Kredi-Kocaely and 1980 Brienza-Irpinia) and 14 regional earthquakes for type B soil recorded in different years near the design site location.
6. A "near-collapse" criterion is chosen as either the attainment of a 2% inter-storey drift or the failure in one of the structural components (shear walls or connections), whichever occurs first. The failure is assumed to occur in the structural component when the ultimate deformation calculated at 80% of the maximum force value in the post-peak portion of the 1st cycles backbone curve is attained.
7. Incremental Dynamic Analysis (IDA) has been performed for each of the 4 building configurations in Y (short) direction and for each ground motion records. In each IDA, several non-linear time-history analyses are performed by increasing progressively the PGA until the $PGA_{near\text{-}collapse}$ is reached.
8. For each building configuration, the q-value is assessed according to Equation (1).
9. Finally a discussion of the obtained results is provided together with a code proposal for the evaluation of the q-factor value to be used in the seismic design of hybrid CLT-Light-Frame building

In the following Sections, a detailed description of the chosen procedure is provided, together with details of the non-linear hysteretic model used and purposely modified in order to better reproduce the actual non-linear behaviour of CLT and Light-Frame components.

4.4 Four-storey case study building: description and preliminary design

The mixed CLT/Light Frame structure presented in this Thesis is a four-storey residential building which contains six apartments and has plan dimensions of 18 x 11 m and average height of 13 m. The building plan has been adapted from the case study building analysed by Vassallo et al., 2013 and designed in 4 different configurations: A - all Light Frame walls, B - all Light-Frame walls with lift and stair core made with CLT walls, C - all external Light-Frame walls and all internal CLT walls, and D - all CLT walls as illustrated in Figure 4-4. Figure 4-5 shows a schematic renderings of the four configurations with the CLT wall highlighted in red and the Light-Frame walls highlighted in blue, while Figure 4-6 illustrates a plan view of the 1st, 2nd and 3rd storey together with the location of CLT and Light Frame walls.

are 5-layer, 120 mm thick CLT panels are used at all stories as CLT walls. Hold-downs WHT HTT22 connected with 12 Anker nails 4x60 mm are used on the CLt walls to prevent uplift. Two types of angle brackets manufactured by Rotho Blaas were used as base and inter-storey shear connection: the WVS90110 (connected with 1ϕ12 steel rod class 4.6 to the foundation and 11 ϕ4x60 mm anker nails to the CLT wall) that was used for the shear-transferring connections of the 1st story, and the WB90 that was used for the connections of the upper stories. Light Frame walls are built with 60 mm × 120 mm C24 Spruce lumber with 3.01x65 mm common nails spaced at 150 mm on panel edges and at 300 mm on internal supports are used to fasten the 12mm OSB panels to the framing members. The floor diaphragms are built with 5-layer 144 mm CLT panels and the roof diaphragm with GLT beams and 3-layer 66 mm CLT panels.

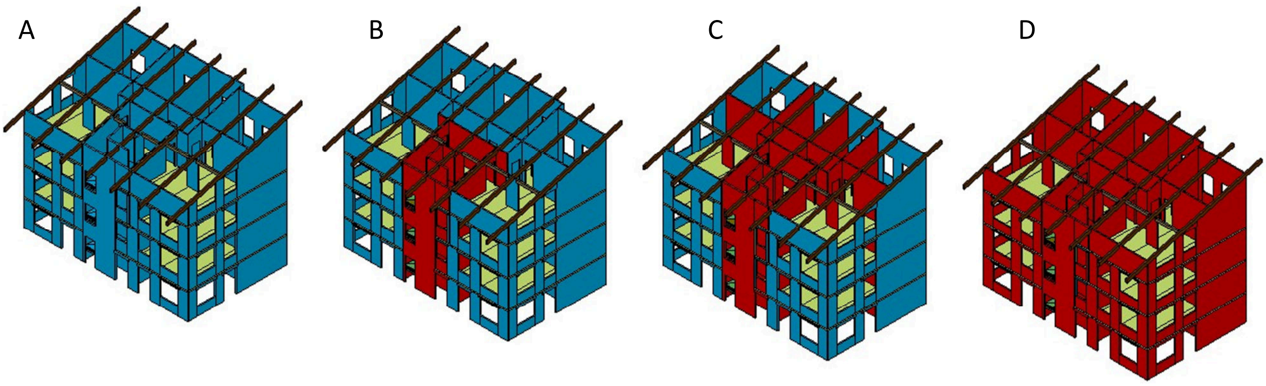
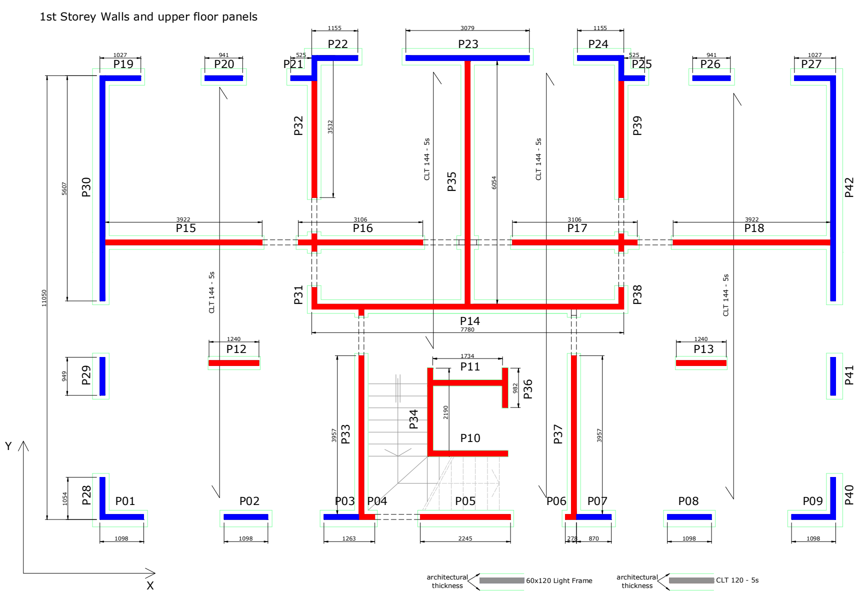


Figure 4-5 3D view of the structural components of the mixed CLT/Light Frame building for the four different configurations. In blue the Light Frame shear walls and in red the CLT shear walls. Roof panels are not showed for the sake of clarity.



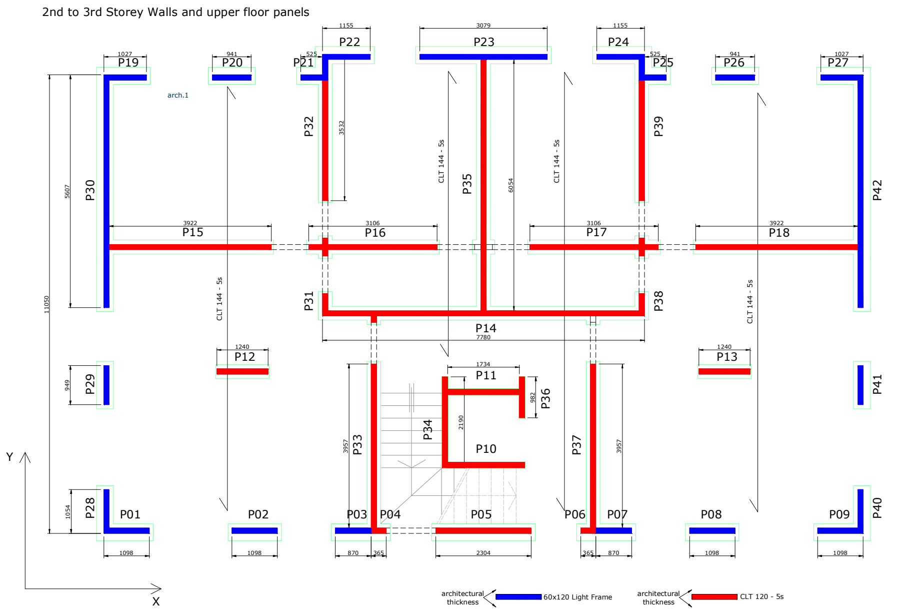


Figure 4-6 Plan view of the 1st and 2nd to 3rd storey together with the location of CLT (red) and Light Frame (blue) walls for configuration C.

4.4.1 Gravity loads

Gravity loads for the seismic combination were estimated based on the structural and non-structural elements. The dead loads G of external Light Frame walls and internal Light Frame and CLT walls are 0.91 kPa, 0.94 kPa and 0.63 kPa, respectively, while the dead loads of the floors and roof are 3.43 kPa and 1.28 kPa, respectively, where the roof loads refer to the inclined area. The live loads Q for the floors are 2.00 kPa for residential use; for the roof, no accidental load is considered for the seismic combination. Based on these gravity loads, Table 4-1 lists the total dead and live loads as well as the seismic weight of each floor of the building. The total seismic weight of the building is W = 3858.3 kN.

Table 4-1 Total dead load, live loads and seismic weight for each level of the building.

Story	Dead loads, G (kN)	Live loads, Q (kN)	Seismic weight, G + 0.3Q (kN)
1	981.4	413.3	1105.4
2	981.4	413.3	1105.4
3	995.1	413.3	1119.1
4	528.4	0.00	528.4
Sum	3486.3	1239.9	3858.3

4.4.2 Seismic preliminary design according to Italian Building Code and Eurocode 8

The design response spectrum, displayed in Figure 4-7, is calculated for Tolmezzo, near the epicentre of the 1976 Friuli Earthquake, for a 10% probability of exceedance in 50 years, according to the Italian National Building Code (Ministero delle Infrastrutture, 2008) with the following parameters:

- nominal life of the structure equal to 50 years;
- design ground acceleration α_g = 0.237 g;
- soil factor S = 1.17 for ground type B;
- amplification factor F_0 = 2.41
- lower limit of the period of constant spectral acceleration branch T_B = 0.151 s;
- upper limit of the period of constant spectral acceleration branch T_C = 0.453 s;
- value defining the beginning of the constant displacement response range of the spectrum T_D = 2.55 s;
- seismic force modification factor q = 1.

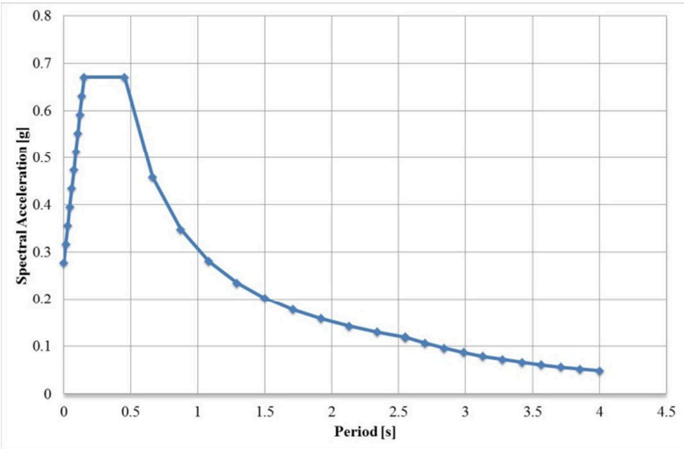


Figure 4-7 Design response spectrum considered in the design of the case study buildings. Maximum value of spectral acceleration 0.67g.

The preliminary design was performed by means of a three-dimensional numerical model of the four-story building implemented in the widespread software package for structural analysis SAP2000 (CSI, 2000), while a pre- and post-processing software specifically developed by Tecnisoft (Modest-Ver.8.6, 2014) was used to aid in the implementation. Figure 4-8 illustrates the numerical model of the building extracted from the pre-processing software. The numerical model for the preliminary design is based on the procedure described in §3.2.5 and §3.3.5 for CLT and Light-Frame linear models with SAP2000 (CSI, 2000).

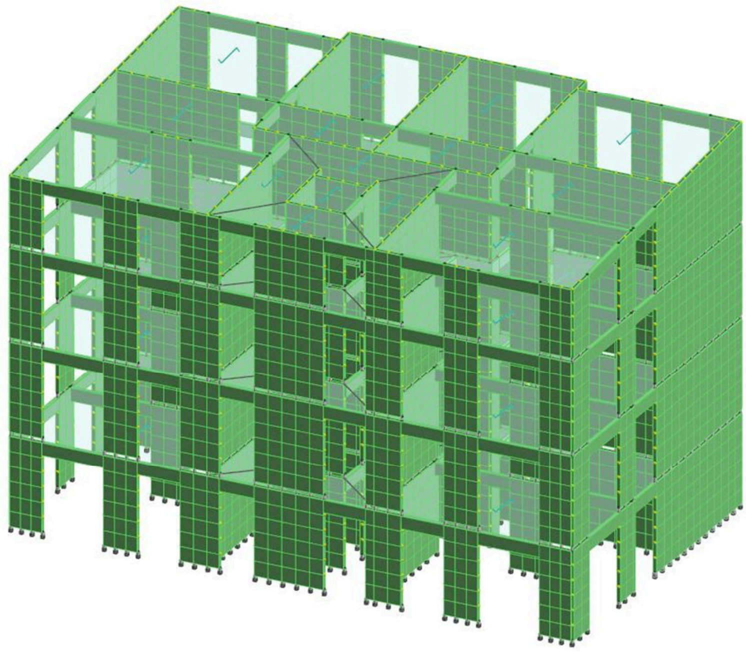


Figure 4-8 Undeformed shape of the preliminary-design numerical model.

The connections of CLT and Light Frame walls at each storey resulting from the preliminary design and based on common construction practice are summarized in Table 4-2. CLT walls are designed without vertical step joints as monolithic panel.

Table 4-2 Connections for CLT and Light-Frame walls considered in the preliminary design.

Story	CLT wall base connections	Light Frame walls base connection	Light Frame walls sheathing and nail spacing
1	WVS90110 steel brackets connected with 1 ϕ12 steel rod class 4.6 to the foundation and 11 ϕ4x60 anker nails to the CLT walls spaced at 300 mm	1 ϕ12 steel rod class 4.6 to the foundation spaced at 100 mm	12mm OSB panels connected to the framing members 3.01x65 mm common nails spaced at 75 mm
2	WB90 steel brackets connected with 8 ϕ4x60 anker nails to the CLT walls and 8 ϕ4x60 anker nails + 2 HBSϕ4x60 to the CLT floors spaced at 500 mm	ϕ12x140 screws to the CLT floors spaced at 100 mm	12mm OSB panels connected to the framing members 3.01x65 mm common nails spaced at 100 mm
3	WB90 steel brackets connected with 8 ϕ4x60 anker nails to the CLT walls and 8 ϕ4x60 anker nails + 2 HBSϕ4x60 to the CLT floors spaced at 700 mm	ϕ12x140 screws to the CLT floors spaced at 150 mm	12mm OSB panels connected to the framing members 3.01x65 mm common nails spaced at 150 mm
4	WB90 steel brackets connected with 8 ϕ4x60 anker nails to the CLT walls and 8 ϕ4x60 anker nails + 2 HBSϕ4x60 to the CLT floors spaced at 1000 mm	ϕ12x140 screws to the CLT floors spaced at 200 mm	12mm OSB panels connected to the framing members 3.01x65 mm common nails spaced at 150 mm

Table 4-3 shows the design strength for the connections and walls used in the seismic design. The design strength of Light-Frame walls is calculated according to §9.2.4 of Eurocode 5 (CEN, 2009). The design values were calculated considering k_{mod}=1.1 according to Eurocode 5 (CEN, 2009) and a partial safety factor γ_m=1.0 according to the current edition of Eurocode 8 (CEN, 2004). HTT22 hold-down anchors were used as uplift connections for both CLT and Light Frame walls, connected with 12 nails for CLT walls and 22 nails for Light-Frame walls. The reason for considering a different number of nails is explained by considering the capacity design rules at connection level described in §3.2.4 and §3.3.4 according to which, in order to obtain a ductile behaviour of the hold-down anchor both in CLT and Light-Frame construction, the maximum number of nails necessary to reach a nail-side failure is calculated. Being the resistance of the same connection greater for CLT panels than for Light-Frame connection, due to the lamination effect in steel-to-CLT connection, a lower number of nails is considered for the same hold-down anchor used in CLT walls.

Table 4-3 Connections for CLT and Light-Frame walls considered in the preliminary design.

Connection/wall type	Description	Design strength
Uplift connection for Light-Frame walls	HTT22 Hold-down Wall connection: 18 ϕ4x60 anker nails Floor connection: (foundation): with 1 class 4.6 ϕ16 steel rod Floor connection: (inter-storey): with 1 ϕ16 steel bolt	R_d=34.74 kN
Uplift connection for CLT walls	HTT22 Hold-down Wall connection: 12 ϕ4x60 anker nails. Floor connection: (foundation): with 1 class 4.6 ϕ16 steel rod Floor connection: (inter-storey): with 1 ϕ16 steel bolt	R_d=28.44 kN
Shear connection for CLT walls	WVS90110 steel brackets Wall connection: 11 ϕ4x60 anker nails. Floor connection: (foundation): 1 ϕ12 class 4.6 steel rod	R_d=11.50 kN
Shear connection for CLT walls	WB90 steel brackets Wall connection: 8 ϕ4x60 anker nails. Floor connection: (inter-storey): 8 ϕ4x60 anker nails + 2 HBSϕ4x60	R_d=5.8 kN
Shear connection for Light-Frame walls	1 ϕ12 steel rod class 4.6 (foundation)	R_d=10.12 kN
Shear connection for Light-Frame walls	ϕ12x140 screws to the CLT floors (inter-storey)	R_d=6.01 kN
Light-Frame walls	12mm OSB panels connected to the framing members with 3.0x65 mm common nails spaced at 150 mm	R_d=5.50 kN/m

4.4.3 Results of the seismic preliminary design

Tables 4-4 to 4-7 show the results of the preliminary design for the 4 building configurations in terms of shear and uplift forces acting on each wall together with the calculated stiffness and design strength for each wall, only for the first storey for brevity, according to the wall identification numbers given in Figure 4-9.

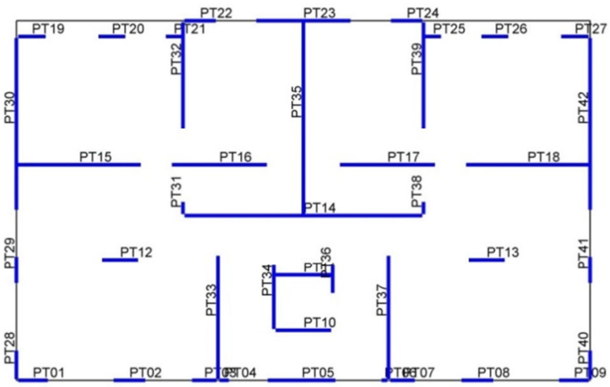


Figure 4-9 Wall identification numbers at first storey for the case study building.

Table 4-4 Results of the preliminary design for Configuration A (d12 A.B. stands for Anchor Bolts with 12 mm diameter, LF for Light-Frame wall).

PRELIMINARY DESIGN RESULTS CASE A - ALL LIGHT-FRAME WALLS																	
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Wall type	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall shear strength [kN]	Wall stiffness [kN/m]
PT01	1.03	X	17.6	17.01	39.8	40.1	142.2	96.3	LF	d12 A.B.	1.3	3	3.52	HTT22	2	17.6	3.0
PT02	1.10	X	18.7	17.01	25.8	29.3	83.8	139.7	LF	d12 A.B.	1.3	3	3.74	HTT22	2	18.7	3.0
PT03	0.87	X	14.8	17.01	102.2	105.2	194.1	63.6	LF	d12 A.B.	1.3	2	2.97	HTT22	4	14.8	3.0
PT04	0.39	X	2.2	5.67	84.0	84.0	191.1	19.1	LF	d12 A.B.	1.3	1	4.02	HTT22	4	2.2	1.3
PT05	2.25	X	89.1	39.69	209.7	207.5	53.5	89.1	LF	d12 A.B.	1.3	12	2.09	HTT22	8	89.1	5.4
PT06	0.28	X	1.6	5.67	76.3	76.3	173.6	17.6	LF	d12 A.B.	1.3	1	2.84	HTT22	3	1.6	1.4
PT07	0.87	X	14.8	17.01	104.9	106.0	193.5	63.7	LF	d12 A.B.	1.3	2	2.97	HTT22	4	14.8	3.0
PT08	1.10	X	18.7	17.01	29.5	25.7	83.8	139.7	LF	d12 A.B.	1.3	3	3.74	HTT22	2	18.7	3.0
PT09	1.03	X	17.6	17.01	40.2	39.1	139.3	95.7	LF	d12 A.B.	1.3	3	3.52	HTT22	2	17.6	3.0
PT10	1.86	X	61.5	33.02	127.8	144.9	273.2	116.6	LF	d12 A.B.	1.3	8	2.51	HTT22	6	61.5	4.0
PT11	1.86	X	59.4	31.91	166.7	119.5	281.7	121.6	LF	d12 A.B.	1.3	8	2.60	HTT22	7	59.4	3.9
PT12	1.24	X	26.1	21.06	0.5	3.6	182.4	304.1	LF	d12 A.B.	1.3	4	3.41	HTT22	1	26.1	3.5
PT13	1.24	X	26.1	21.06	3.8	0.3	182.4	304.1	LF	d12 A.B.	1.3	4	3.41	HTT22	1	26.1	3.5
PT14	7.65	X	978.9	127.93	657.3	653.6	832.4	404.3	LF	d12 A.B.	1.3	126	0.65	HTT22	25	978.9	17.2
PT15	3.99	X	265.1	66.50	318.8	402.3	397.0	398.7	LF	d12 A.B.	1.3	35	1.25	HTT22	16	265.1	8.5
PT16	3.11	X	158.9	51.15	201.3	205.3	210.5	339.1	LF	d12 A.B.	1.3	21	1.62	HTT22	8	158.9	6.7
PT17	3.11	X	158.9	51.15	205.9	200.7	210.4	339.1	LF	d12 A.B.	1.3	21	1.62	HTT22	8	158.9	6.7
PT18	3.99	X	265.1	66.50	402.9	317.2	397.4	397.9	LF	d12 A.B.	1.3	35	1.25	HTT22	16	265.1	8.5
PT19	0.96	X	15.7	16.25	87.3	134.4	217.0	75.2	LF	d12 A.B.	1.3	3	3.44	HTT22	6	15.7	3.0
PT20	0.94	X	15.3	16.25	25.0	27.7	75.0	125.0	LF	d12 A.B.	1.3	2	3.36	HTT22	2	15.3	2.8
PT21	0.59	X	6.4	10.83	66.2	19.8	106.2	64.9	LF	d12 A.B.	1.3	1	3.15	HTT22	3	6.4	2.1
PT22	1.09	X	17.8	16.36	48.3	100.5	189.3	90.8	LF	d12 A.B.	1.3	3	3.86	HTT22	4	17.8	2.9
PT23	3.08	X	167.9	54.54	182.7	183.5	293.1	220.2	LF	d12 A.B.	1.3	22	1.52	HTT22	7	167.9	7.1
PT24	1.09	X	17.9	16.36	99.8	47.9	189.3	91.2	LF	d12 A.B.	1.3	3	3.87	HTT22	4	17.9	2.9
PT25	0.59	X	6.4	10.83	19.2	66.6	106.2	65.0	LF	d12 A.B.	1.3	1	3.15	HTT22	3	6.4	2.1
PT26	0.94	X	15.3	16.25	27.7	25.0	75.0	125.0	LF	d12 A.B.	1.3	2	3.36	HTT22	2	15.3	2.8
PT27	0.96	X	15.7	16.25	133.3	84.1	216.4	75.7	LF	d12 A.B.	1.3	3	3.44	HTT22	5	15.7	3.0
PT28	0.99	Y	16.5	16.65	31.8	83.5	92.5	27.7	LF	d12 A.B.	1.3	3	3.45	HTT22	4	16.5	3.0
PT29	0.95	Y	15.8	16.65	41.6	25.0	72.5	112.7	LF	d12 A.B.	1.3	3	3.30	HTT22	2	15.8	3.1
PT30	5.54	Y	553.8	99.90	282.1	119.6	410.4	142.8	LF	d12 A.B.	1.3	72	0.83	HTT22	11	553.8	13.5
PT31	0.48	Y	2.3	4.93	39.8	54.0	89.6	8.4	LF	d12 A.B.	1.3	1	5.60	HTT22	3	2.3	1.1
PT32	3.47	Y	188.1	54.23	162.7	37.8	175.3	64.4	LF	d12 A.B.	1.3	25	1.53	HTT22	7	188.1	7.6
PT33	4.02	Y	250.9	62.39	255.9	387.7	298.1	216.8	LF	d12 A.B.	1.3	33	1.33	HTT22	15	250.9	8.0
PT34	2.13	Y	68.3	32.13	176.5	144.9	336.7	81.3	LF	d12 A.B.	1.3	9	2.58	HTT22	7	68.3	4.3
PT35	6.18	Y	554.7	89.73	197.0	512.2	506.8	282.9	LF	d12 A.B.	1.3	72	0.92	HTT22	20	554.7	12.9
PT36	0.98	Y	13.5	13.80	95.1	52.4	143.8	32.5	LF	d12 A.B.	1.3	2	4.13	HTT22	4	13.5	2.4
PT37	4.02	Y	251.7	62.59	269.4	384.4	289.6	216.0	LF	d12 A.B.	1.3	33	1.32	HTT22	15	251.7	8.0
PT38	0.48	Y	2.4	4.95	39.5	53.2	89.6	8.8	LF	d12 A.B.	1.3	1	5.58	HTT22	2	2.4	1.1
PT39	3.47	Y	188.8	54.44	161.3	38.0	173.3	64.2	LF	d12 A.B.	1.3	25	1.52	HTT22	7	188.8	7.6
PT40	0.99	Y	16.6	16.76	31.8	80.5	90.4	28.3	LF	d12 A.B.	1.3	3	3.42	HTT22	4	16.6	3.0
PT41	0.95	Y	15.9	16.76	40.4	23.6	72.4	112.6	LF	d12 A.B.	1.3	3	3.28	HTT22	2	15.9	3.1
PT42	5.54	Y	557.5	100.58	280.7	117.2	409.5	143.1	LF	d12 A.B.	1.3	72	0.82	HTT22	11	557.5	13.6

Table 4-5 Results of the preliminary design for Configuration B (d12 A.B. stands for Anchor Bolts with 12 mm diameter, WVS90110 for WVS90110 angle brackets, LF for Light-Frame wall, CLT for CLT walls).

PRELIMINARY DESIGN RESULTS CASE B - ALL LIGHT-FRAME WALLS, LIFT AND STAIR CORE CLT WALLS																	
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Wall type	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall shear strength [kN]	Wall stiffness [kN/m]
PT01	1.03	X	18.5	17.88	64.4	146.0	142.2	96.3	LF	d12 A.B.	1.3	2	3.35	HTT22	6	18.5	3.2
PT02	1.10	X	19.6	17.88	62.2	62.6	83.8	139.7	LF	d12 A.B.	1.3	3	3.56	HTT22	3	19.6	3.1
PT03	0.87	X	10.4	11.92	144.0	144.0	194.1	63.6	LF	d12 A.B.	1.3	2	4.23	HTT22	6	10.4	2.3
PT04	0.39	X	2.1	5.34	191.3	191.3	191.1	19.1	CLT	WVS90110	1	1	N/A	HTT22	7	11.5	21.5
PT05	2.25	X	84.0	37.41	608.4	605.3	53.5	89.1	CLT	WVS90110	1	11	N/A	HTT22	22	92.0	23.2
PT06	0.28	X	1.5	5.34	158.3	158.3	173.6	17.6	CLT	WVS90110	1	1	N/A	HTT22	6	11.5	23.3
PT07	0.87	X	10.4	11.92	141.4	141.4	193.5	63.7	LF	d12 A.B.	1.3	2	4.23	HTT22	6	10.4	2.3
PT08	1.10	X	19.6	17.88	52.8	50.5	83.8	139.7	LF	d12 A.B.	1.3	3	3.56	HTT22	2	19.6	3.1
PT09	1.03	X	18.5	17.88	132.6	52.1	139.3	95.7	LF	d12 A.B.	1.3	2	3.35	HTT22	5	18.5	3.2
PT10	1.86	X	57.8	31.05	396.8	622.9	273.2	116.6	CLT	WVS90110	1	8	N/A	HTT22	22	69.0	22.8
PT11	1.86	X	55.7	29.92	532.6	344.6	281.7	121.6	CLT	WVS90110	1	7	N/A	HTT22	19	57.5	21.8
PT12	1.24	X	27.3	22.00	15.2	15.9	182.4	304.1	LF	d12 A.B.	1.3	3	3.27	HTT22	1	27.3	3.6
PT13	1.24	X	27.3	22.00	9.9	8.8	182.4	304.1	LF	d12 A.B.	1.3	3	3.27	HTT22	1	27.3	3.6
PT14	7.65	X	915.0	119.58	899.3	869.2	832.4	404.3	CLT	WVS90110	1	112	N/A	HTT22	32	920.0	26.7
PT15	3.99	X	294.8	73.95	17.5	14.1	397.0	398.7	LF	d12 A.B.	1.3	31	1.12	HTT22	1	294.8	9.4
PT16	3.11	X	180.5	58.10	18.0	13.8	210.5	339.1	LF	d12 A.B.	1.3	19	1.43	HTT22	1	180.5	7.6
PT17	3.11	X	180.5	58.10	13.9	17.8	210.4	339.1	LF	d12 A.B.	1.3	19	1.43	HTT22	1	180.5	7.6
PT18	3.99	X	294.8	73.95	14.1	16.7	397.4	397.9	LF	d12 A.B.	1.3	31	1.12	HTT22	1	294.8	9.4
PT19	0.96	X	13.2	13.76	30.3	96.0	217.0	75.2	LF	d12 A.B.	1.3	2	4.06	HTT22	4	13.2	2.4
PT20	0.94	X	12.9	13.76	33.3	34.1	75.0	125.0	LF	d12 A.B.	1.3	2	3.97	HTT22	2	12.9	2.5
PT21	0.59	X	3.2	5.50	63.3	12.5	106.2	64.9	LF	d12 A.B.	1.3	1	6.21	HTT22	3	3.2	1.2
PT22	1.09	X	18.1	16.59	21.2	99.6	189.3	90.8	LF	d12 A.B.	1.3	2	3.81	HTT22	4	18.1	2.9
PT23	3.08	X	187.3	60.82	40.8	42.0	293.1	220.2	LF	d12 A.B.	1.3	20	1.36	HTT22	2	187.3	7.9
PT24	1.09	X	18.1	16.59	91.6	20.6	189.3	91.2	LF	d12 A.B.	1.3	2	3.81	HTT22	4	18.1	2.9
PT25	0.59	X	3.2	5.50	11.8	66.8	106.2	65.0	LF	d12 A.B.	1.3	1	6.21	HTT22	3	3.2	1.2
PT26	0.94	X	12.9	13.76	34.0	33.4	75.0	125.0	LF	d12 A.B.	1.3	2	3.97	HTT22	2	12.9	2.5
PT27	0.96	X	13.2	13.76	93.2	35.2	216.4	75.7	LF	d12 A.B.	1.3	2	4.06	HTT22	4	13.2	2.4
PT28	0.99	Y	13.5	13.69	53.0	154.0	92.5	27.7	LF	d12 A.B.	1.3	2	4.19	HTT22	6	13.5	2.4
PT29	0.95	Y	13.0	13.69	57.5	60.4	72.5	112.7	LF	d12 A.B.	1.3	2	4.02	HTT22	3	13.0	2.4
PT30	5.54	Y	591.7	106.76	111.5	28.9	410.4	142.8	LF	d12 A.B.	1.3	62	0.78	HTT22	5	591.7	14.5
PT31	0.48	Y	2.3	4.85	74.1	74.1	89.6	8.4	LF	d12 A.B.	1.3	1	5.70	HTT22	3	2.3	1.1
PT32	3.47	Y	201.7	58.16	85.5	20.8	175.3	64.4	LF	d12 A.B.	1.3	21	1.42	HTT22	4	201.7	8.1
PT33	4.02	Y	221.0	54.96	589.3	1017.8	298.1	216.8	CLT	WVS90110	1	28	N/A	HTT22	36	230.0	24.6
PT34	2.13	Y	60.1	28.27	429.7	407.0	336.7	81.3	CLT	WVS90110	1	8	N/A	HTT22	16	69.0	22.1
PT35	6.18	Y	584.3	94.51	0.0	16.6	506.8	282.9	LF	d12 A.B.	1.3	61	0.88	HTT22	1	584.3	13.6
PT36	0.98	Y	11.9	12.13	246.5	212.1	143.8	32.5	CLT	WVS90110	1	2	N/A	HTT22	9	23.0	20.2
PT37	4.02	Y	221.6	55.11	616.7	977.7	289.6	216.0	CLT	WVS90110	1	28	N/A	HTT22	35	230.0	24.6
PT38	0.48	Y	2.3	4.86	75.8	75.8	89.6	8.8	LF	d12 A.B.	1.3	1	5.68	HTT22	3	2.3	1.1
PT39	3.47	Y	202.3	58.34	82.5	20.6	173.3	64.2	LF	d12 A.B.	1.3	21	1.42	HTT22	4	202.3	8.1
PT40	0.99	Y	13.6	13.76	54.2	140.2	90.4	28.3	LF	d12 A.B.	1.3	2	4.17	HTT22	6	13.6	2.4
PT41	0.95	Y	13.1	13.76	50.8	50.9	72.4	112.6	LF	d12 A.B.	1.3	2	4.00	HTT22	2	13.1	2.5
PT42	5.54	Y	595.1	107.36	99.0	27.5	409.5	143.1	LF	d12 A.B.	1.3	62	0.77	HTT22	4	595.1	14.5

Table 4-6 Results of the preliminary design for Configuration C (d12 A.B. stands for Anchor Bolts with 12 mm diameter, WVS90110 for WVS90110 angle brackets, LF for Light-Frame wall, CLT for CLT walls).

PRELIMINARY DESIGN RESULTS CASE C - ALL EXTERNAL LIGHT-FRAME WALLS, ALL INTERNAL CLT WALLS																	
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Wall type	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall shear strength [kN]	Wall stiffness [kN/m]
PT01	1.03	X	20.0	19.36	37.2	86.6	151.8	63.8	LF	d12 A.B.	1.3	3	3.10	HTT22	4	20.0	3.4
PT02	1.10	X	21.3	19.36	37.4	37.9	62.2	103.7	LF	d12 A.B.	1.3	3	3.29	HTT22	2	21.3	3.3
PT03	0.87	X	11.2	12.90	95.7	160.3	237.7	38.6	LF	d12 A.B.	1.3	2	3.91	HTT22	6	11.2	2.4
PT04	0.39	X	2.3	5.79	129.2	129.2	286.1	23.1	CLT	WVS90110	1	1	N/A	HTT22	5	11.5	21.5
PT05	2.25	X	90.9	40.51	397.4	395.0	53.5	89.1	CLT	WVS90110	1	12	N/A	HTT22	14	92.0	23.2
PT06	0.28	X	1.6	5.79	104.5	104.5	233.6	20.5	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	23.3
PT07	0.87	X	11.2	12.90	154.7	97.7	255.3	49.9	LF	d12 A.B.	1.3	2	3.91	HTT22	6	11.2	2.4
PT08	1.10	X	21.3	19.36	28.2	27.1	83.8	139.7	LF	d12 A.B.	1.3	3	3.29	HTT22	2	21.3	3.3
PT09	1.03	X	20.0	19.36	71.1	29.4	159.4	83.9	LF	d12 A.B.	1.3	3	3.10	HTT22	3	20.0	3.4
PT10	1.86	X	62.9	33.76	260.6	478.4	604.9	106.9	CLT	WVS90110	1	8	N/A	HTT22	17	69.0	22.8
PT11	1.86	X	60.9	32.68	365.8	259.7	468.3	112.4	CLT	WVS90110	1	8	N/A	HTT22	13	69.0	22.8
PT12	1.24	X	26.8	21.59	83.2	83.4	158.5	264.1	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	21.3
PT13	1.24	X	26.8	21.59	71.9	71.2	187.9	313.2	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	21.3
PT14	7.65	X	1005.2	131.36	755.8	761.1	1048.3	377.3	CLT	WVS90110	1	123	N/A	HTT22	27	1012.0	26.9
PT15	3.99	X	271.1	68.00	464.6	579.9	392.5	407.1	CLT	WVS90110	1	34	N/A	HTT22	21	276.0	25.3
PT16	3.11	X	162.5	52.31	402.7	407.7	215.9	337.4	CLT	WVS90110	1	20	N/A	HTT22	15	172.5	24.5
PT17	3.11	X	162.5	52.31	404.3	396.4	215.2	337.3	CLT	WVS90110	1	20	N/A	HTT22	15	172.5	24.5
PT18	3.99	X	271.1	68.00	580.6	463.4	393.3	406.8	CLT	WVS90110	1	34	N/A	HTT22	21	276.0	25.3
PT19	0.96	X	14.7	15.26	11.5	55.6	127.2	72.9	LF	d12 A.B.	1.3	2	3.66	HTT22	3	14.7	2.7
PT20	0.94	X	14.4	15.26	23.5	24.3	75.0	125.0	LF	d12 A.B.	1.3	2	3.58	HTT22	1	14.4	2.7
PT21	0.59	X	3.6	6.10	110.0	182.8	285.8	54.8	LF	d12 A.B.	1.3	1	5.60	HTT22	7	3.6	1.3
PT22	1.09	X	20.1	18.41	140.4	140.4	371.8	75.9	LF	d12 A.B.	1.3	3	3.43	HTT22	6	20.1	3.2
PT23	3.08	X	207.8	67.50	71.4	73.9	267.9	209.3	LF	d12 A.B.	1.3	22	1.23	HTT22	3	207.8	8.7
PT24	1.09	X	20.1	18.41	140.5	140.5	374.0	77.4	LF	d12 A.B.	1.3	3	3.44	HTT22	6	20.1	3.2
PT25	0.59	X	3.6	6.10	195.7	107.3	281.3	55.6	LF	d12 A.B.	1.3	1	5.60	HTT22	8	3.6	1.3
PT26	0.94	X	14.4	15.26	24.2	23.6	75.0	125.0	LF	d12 A.B.	1.3	2	3.58	HTT22	1	14.4	2.7
PT27	0.96	X	14.7	15.26	54.6	11.4	123.6	73.5	LF	d12 A.B.	1.3	2	3.66	HTT22	3	14.7	2.7
PT28	0.99	Y	14.6	14.80	40.5	94.5	124.0	35.8	LF	d12 A.B.	1.3	2	3.88	HTT22	4	14.6	2.6
PT29	0.95	Y	14.0	14.80	29.7	32.1	59.2	94.6	LF	d12 A.B.	1.3	2	3.72	HTT22	2	14.0	2.6
PT30	5.54	Y	639.8	115.42	151.0	18.0	300.1	135.9	LF	d12 A.B.	1.3	67	0.72	HTT22	6	639.8	15.6
PT31	0.48	Y	2.3	4.74	98.7	98.7	219.5	18.4	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	20.4
PT32	3.47	Y	180.8	52.14	726.0	198.2	592.2	117.4	CLT	WVS90110	1	23	N/A	HTT22	26	184.0	24.3
PT33	4.02	Y	241.5	60.07	383.4	663.3	419.2	198.4	CLT	WVS90110	1	30	N/A	HTT22	24	253.0	25.0
PT34	2.13	Y	65.9	31.00	321.3	310.1	711.3	89.0	CLT	WVS90110	1	9	N/A	HTT22	12	69.0	22.1
PT35	6.18	Y	535.7	86.66	215.6	785.9	801.4	309.1	CLT	WVS90110	1	66	N/A	HTT22	28	540.5	26.0
PT36	0.98	Y	13.1	13.31	186.0	159.0	370.4	43.8	CLT	WVS90110	1	2	N/A	HTT22	7	23.0	20.2
PT37	4.02	Y	242.2	60.22	398.2	641.4	406.5	224.7	CLT	WVS90110	1	30	N/A	HTT22	23	253.0	25.0
PT38	0.48	Y	2.3	4.76	98.9	98.9	221.3	19.5	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	20.4
PT39	3.47	Y	181.4	52.31	728.2	192.6	598.8	115.3	CLT	WVS90110	1	23	N/A	HTT22	26	184.0	24.3
PT40	0.99	Y	14.7	14.88	38.4	89.8	126.8	41.8	LF	d12 A.B.	1.3	2	3.86	HTT22	4	14.7	2.6
PT41	0.95	Y	14.1	14.88	27.7	27.5	69.1	111.4	LF	d12 A.B.	1.3	2	3.70	HTT22	2	14.1	2.6
PT42	5.54	Y	643.5	116.10	148.4	17.7	295.7	136.0	LF	d12 A.B.	1.3	67	0.71	HTT22	6	643.5	15.7

Table 4-7 Results of the preliminary design for Configuration D (WVS90110 stands for WVS90110 angle brackets, CLT for CLT walls).

PRELIMINARY DESIGN RESULTS CASE D - ALL CLT WALLS																	
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Wall type	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall strength [kN]	Wall stiffness [kN/m]
PT01	1.03	X	18.3	17.73	84.4	204.0	248.3	62.3	CLT	WVS90110	1	3	N/A	HTT22	8	23.0	19.9
PT02	1.10	X	19.5	17.73	65.0	65.2	66.9	111.6	CLT	WVS90110	1	3	N/A	HTT22	3	23.0	19.5
PT03	0.87	X	15.4	17.73	190.2	113.5	281.4	45.4	CLT	WVS90110	1	2	N/A	HTT22	7	23.0	21.0
PT04	0.39	X	2.3	5.91	76.9	76.9	182.2	23.7	CLT	WVS90110	1	1	N/A	HTT22	3	11.5	21.5
PT05	2.25	X	92.9	41.36	314.9	313.7	53.5	89.1	CLT	WVS90110	1	12	N/A	HTT22	12	103.5	23.8
PT06	0.28	X	1.6	5.91	72.2	72.2	176.2	26.4	CLT	WVS90110	1	1	N/A	HTT22	3	11.5	23.3
PT07	0.87	X	15.4	17.73	108.2	177.1	285.1	57.3	CLT	WVS90110	1	2	N/A	HTT22	7	23.0	21.0
PT08	1.10	X	19.5	17.73	55.0	53.6	88.5	147.5	CLT	WVS90110	1	3	N/A	HTT22	2	23.0	19.5
PT09	1.03	X	18.3	17.73	186.5	65.7	227.3	79.5	CLT	WVS90110	1	3	N/A	HTT22	7	23.0	19.9
PT10	1.86	X	64.1	34.41	191.1	316.8	413.5	108.4	CLT	WVS90110	1	8	N/A	HTT22	12	69.0	22.8
PT11	1.86	X	61.9	33.25	236.4	213.7	354.5	111.3	CLT	WVS90110	1	8	N/A	HTT22	9	69.0	22.8
PT12	1.24	X	27.2	21.95	59.2	59.4	158.5	264.1	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	21.3
PT13	1.24	X	27.2	21.95	53.1	52.5	187.9	313.2	CLT	WVS90110	1	4	N/A	HTT22	2	34.5	21.3
PT14	7.65	X	1020.0	133.29	672.6	667.3	813.2	378.0	CLT	WVS90110	1	125	N/A	HTT22	24	1023.5	27.0
PT15	3.99	X	276.2	69.28	209.8	572.9	848.4	354.5	CLT	WVS90110	1	34	N/A	HTT22	21	287.5	25.4
PT16	3.11	X	165.5	53.29	326.9	333.8	213.7	337.6	CLT	WVS90110	1	21	N/A	HTT22	12	172.5	24.5
PT17	3.11	X	165.5	53.29	329.7	323.4	214.2	337.5	CLT	WVS90110	1	21	N/A	HTT22	12	172.5	24.5
PT18	3.99	X	276.2	69.28	574.1	169.3	813.4	352.3	CLT	WVS90110	1	34	N/A	HTT22	21	287.5	25.4
PT19	0.96	X	16.3	16.93	166.2	166.2	405.1	60.6	CLT	WVS90110	1	2	N/A	HTT22	6	23.0	20.4
PT20	0.94	X	15.9	16.93	57.6	57.8	79.4	132.3	CLT	WVS90110	1	2	N/A	HTT22	3	23.0	20.5
PT21	0.59	X	6.6	11.29	87.8	75.0	223.0	60.8	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	19.1
PT22	1.09	X	18.6	17.05	140.5	251.5	378.4	81.1	CLT	WVS90110	1	3	N/A	HTT22	9	23.0	19.6
PT23	3.08	X	174.9	56.82	447.8	508.2	367.2	196.9	CLT	WVS90110	1	22	N/A	HTT22	18	184.0	24.8
PT24	1.09	X	18.6	17.05	226.1	213.1	320.3	80.5	CLT	WVS90110	1	3	N/A	HTT22	8	23.0	19.6
PT25	0.59	X	6.6	11.29	76.1	88.6	224.7	60.3	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	19.1
PT26	0.94	X	15.9	16.93	57.8	57.6	79.4	132.3	CLT	WVS90110	1	2	N/A	HTT22	3	23.0	20.5
PT27	0.96	X	16.3	16.93	262.2	95.1	282.4	61.6	CLT	WVS90110	1	2	N/A	HTT22	10	23.0	20.4
PT28	0.99	Y	17.2	17.35	86.5	187.2	233.4	50.3	CLT	WVS90110	1	3	N/A	HTT22	7	23.0	20.2
PT29	0.95	Y	16.5	17.35	57.9	59.8	75.7	106.5	CLT	WVS90110	1	3	N/A	HTT22	3	23.0	20.4
PT30	5.54	Y	577.0	104.10	613.3	522.2	852.0	246.9	CLT	WVS90110	1	71	N/A	HTT22	22	586.5	26.5
PT31	0.48	Y	2.4	5.14	82.5	82.5	187.3	18.5	CLT	WVS90110	1	1	N/A	HTT22	3	11.5	20.4
PT32	3.47	Y	196.0	56.51	467.1	88.0	487.2	119.1	CLT	WVS90110	1	24	N/A	HTT22	17	207.0	24.8
PT33	4.02	Y	261.4	65.01	205.4	423.6	396.4	197.9	CLT	WVS90110	1	32	N/A	HTT22	15	264.5	25.1
PT34	2.13	Y	71.2	33.48	230.5	230.5	566.4	87.9	CLT	WVS90110	1	9	N/A	HTT22	9	80.5	22.9
PT35	6.18	Y	577.9	93.48	152.1	219.8	305.5	344.9	CLT	WVS90110	1	71	N/A	HTT22	8	586.5	26.2
PT36	0.98	Y	14.1	14.37	137.6	131.0	315.3	44.4	CLT	WVS90110	1	2	N/A	HTT22	5	23.0	20.2
PT37	4.02	Y	262.2	65.21	195.4	394.6	407.2	220.2	CLT	WVS90110	1	33	N/A	HTT22	14	264.5	25.1
PT38	0.48	Y	2.5	5.16	82.7	82.7	188.3	19.1	CLT	WVS90110	1	1	N/A	HTT22	3	11.5	20.4
PT39	3.47	Y	196.7	56.72	484.2	91.5	503.5	120.4	CLT	WVS90110	1	25	N/A	HTT22	18	207.0	24.8
PT40	0.99	Y	17.3	17.47	84.5	132.9	219.5	59.2	CLT	WVS90110	1	3	N/A	HTT22	5	23.0	20.2
PT41	0.95	Y	16.6	17.47	58.4	59.8	86.9	123.3	CLT	WVS90110	1	3	N/A	HTT22	3	23.0	20.4
PT42	5.54	Y	580.9	104.81	575.7	556.1	866.4	248.4	CLT	WVS90110	1	72	N/A	HTT22	21	586.5	26.5

The results in terms of nail spacing and number of connectors are of course unrealistic in practical cases where nail spacing lower than 5 cm are found for Light-Frame walls or great numbers of angle brackets or hold-down anchors are found. In real cases, stronger connections and different type of shear walls (e.g. midply walls) would be applied with such high seismic forces. However, the choice of the types of connectors and walls described in Table 4-3 is due to the availability of experimental data for such elements. For the purposes of the numerical analysis, it is irrelevant whether the same resistance is reached with a single stronger connector or with multiple weaker connectors.

The wall stiffness is calculated according to Equation 3 and 4 with the notation of Figure 4-10, where K_{LF} is the total Light-Frame wall horizontal stiffness, K_{WL} is the shear stiffness of the wall considering the sheathing-to-framing nailed connection according to the procedure describe in §3.3.5.1, $K_{AB}=\Sigma K_{ABi}$ is the total horizontal stiffness of the wall-to-foundation horizontal connection with 12 mm anchor bolts for Light-

Frame walls and angle brackets for CLT walls, and K_{CLT} is the total CLT wall horizontal stiffness, $K_{PA}=\infty$ is the CLT wall panel stiffness assumed as infinite.

$$K_{LF} = \frac{1}{\dfrac{1}{K_{WL}} + \dfrac{1}{K_{AB}}} \tag{3}$$

$$K_{CLT} = \frac{1}{\dfrac{1}{K_{PA}} + \dfrac{1}{K_{AB}}} = \frac{1}{\dfrac{1}{\infty} + \dfrac{1}{K_{AB}}} = K_{AB} \tag{4}$$

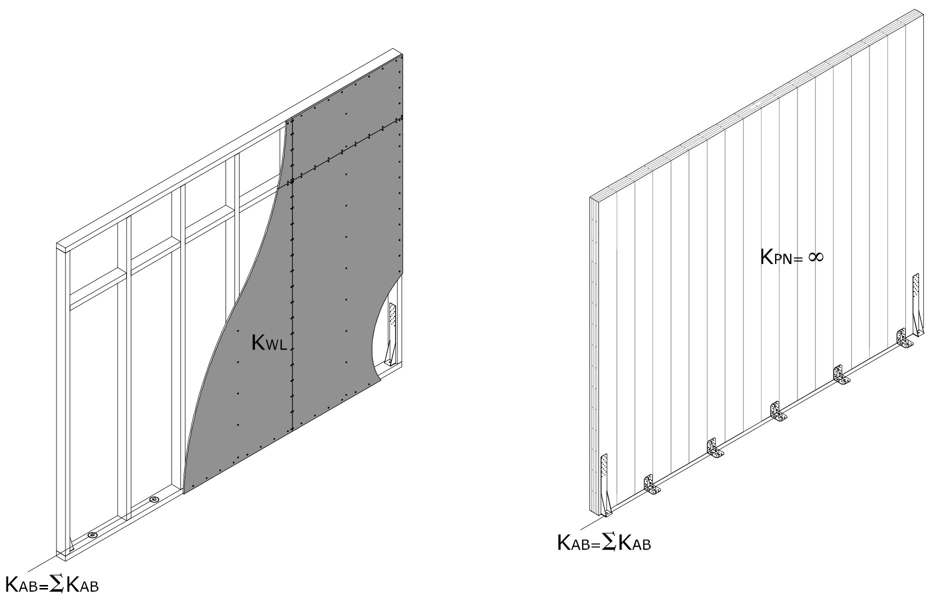


Figure 4-10 Light-Frame and CLT walls notation for the calculation of horizontal stiffness.

Table 4-8 summarizes the percentage ratio of horizontal strength and stiffness of CLT walls, respectively R_{CLT} and K_{CLT} with respect to the total horizontal strength and stiffness R_{TOT} and K_{TOT} in X and Y direction for each storey and for building configurations B and C. Obviously both values are 0 for Configuration A and 100 for configuration D.

Table 4-8 Ratio of horizontal strength and stiffness of CLT walls with respect to the total horizontal strength and stiffness in X and Y direction for each storey and for the four building configurations.

Configuration	Storey	R_{CLT-X}/R_{TOT-X} [%]	K_{CLT-X}/K_{TOT-X} [%]	R_{CLT-Y}/R_{TOT-Y} [%]	K_{CLT-Y}/K_{TOT-Y} [%]
B	1	45.6	65.2	19.8	46.0
	2	40.2	64.9	16.5	45.4
	3	42.5	72.1	17.8	53.0
	4	36.8	81.7	14.8	63.0
C	1	84.2	92.3	53.3	77.0
	2	81.0	91.7	48.1	76.3
	3	82.6	94.1	50.0	81.7
	4	76.6	95.7	43.7	87.4

As it can be observed, the horizontal stiffness of CLT wall with respect to the total horizontal stiffness ranges from approximately 65 to 80% in X and 46 to 63 in Y direction for Configuration B and even from 92 to 96% in X and 77 to 87 in Y direction for Configuration C.

Finally, Table 4-9 summarizes the fundamental periods and the mass participation factors for the first 6 vibration modes of the structure in the four building configurations.

Table 4-9 Modal analysis results of the preliminary design.

Configuration	Mode	Vibration period (sec)	Mass participation for translation along X direction (%)	Mass participation for translation along Y direction (%)	Mass participation for rotation along vertical direction (%)
A	1	1.32	0.04	87.18	0.16
	2	1.21	77.56	0.01	6.04
	3	1.12	5.66	0.14	78.47
	4	0.44	0.01	9.94	0.03
	5	0.39	10.43	0.02	2.23
	6	0.35	2.44	0	9.06
B	1	0.97	5.45	0.68	77.17
	2	0.74	0.23	80.71	0.43
	3	0.61	75.89	0.08	4.8
	4	0.32	1.46	0.26	11.21
	5	0.24	0.08	14.31	0.1
	6	0.20	11.93	0.02	0.5
C	1	0.82	0.3	0.1	80.51
	2	0.51	0.66	79.94	0.04
	3	0.50	78.63	0.65	0.25
	4	0.26	0.27	0.09	14.27
	5	0.17	0.02	15.1	0.01
	6	0.16	11.75	0	0.11
D	1	0.46	79.01	0.01	0.02
	2	0.42	0.02	22.51	58.13
	3	0.42	0	58.3	21.69
	4	0.14	0	14.45	0.65
	5	0.14	16.43	0	0.23
	6	0.13	0.06	0.54	14.53

4.5 Non-linear dynamic analysis: modelling method and calibration

As already discussed in §4.3, the chosen method to evaluate the value of the behaviour factor for a hybrid CLT/Light-Frame building is a mixed test-modelling approach, i.e. a procedure consisting in modelling the entire structure composed of structural components calibrated on the basis of test results and performing numerical non-linear dynamic analysis to evaluate the value of the behaviour factor q according to Equation (1), once a so-called “near collapse” status has been defined.

A discussion is made in Ceccotti and Sandhaas (2010) about the different modelling levels. The decision should be made based on the model scale. One possibility could be (a) to model the building at the material level, i.e. modelling the material (wood and wood-based components) and single fasteners (e.g. nails in sheathing-to-framing connections for Light-Frame buildings) based on material tests and tension tests on single fasteners. In principle, this modelling approach is very powerful, because based on simple and low expensive test at material level an entire building could be modelled and studied. However, due to the difficulties in correctly reproducing all the different mechanisms (e.g. wood embedding behaviour, yielding of fasteners, friction contribution etc.) which could be observed in experimental studies and to computational limitations, such models could hardly be used for a 3D model of an entire building and more often are limited to structure sub-assemblies like, e.g. nailed shear walls. Another possibility, described in Rinaldin et al. (2013a) and Rinaldin et al. (2013b) could be (b) a so-called “component approach” by which, in a higher hierarchical level, single structural components like hold-downs, angle bracket connections or screwed joints for CLT structures and nailed shear walls are modelled with non-linear springs reproducing the actual non-linear behaviour of such components in each direction observed during monotonic or quasi-static cyclic tests. The test scale is slightly bigger than for case (a), but nevertheless tests could be made with small test set-ups and again in an economical and practical scale. However again, even if this is a promising method to predict the global seismic behaviour of real buildings starting from small scale tests, when going from single components to real buildings the model should nevertheless take into account all the different mechanisms contributing to the global seismic behaviour of a real building other than connections (friction contribution, interaction between axial and shear resistance, bending, compression and shear deformation of timber elements and panels, etc.) which are difficult to be evaluated and modelled. Nevertheless this approach proved to be suitable even for the non-linear 3D modelling of multi-storey buildings (Rinaldin et al., 2014) even if with high computational time. In a greater scale a third possibility (c) is to model the entire building starting from “the biggest logical scale” as it is defined in Ceccotti and Sandhaas (2010), i.e. starting from test results on entire walls with all the typical connections used for the reference structural type (in this case CLT or Light-Frame) and modelling single walls with macro-elements simulated with few non-linear springs and rigid shell or frame elements simulating wood-based panels. With this simplified approach two results can be achieved: (i) the simplified model allow 3D modelling and non-linear analysis of real multi-storey buildings with reduced computational times with respect to method (a) and (b); (ii) starting from a simplified modelling of an entire wall, the different mechanisms contributing to the seismic behaviour and referenced above are already included in the macro-element. On the other hand, a bigger scale testing set-up are required, leading to a less economical procedure.

The modelling approach used in this case is something between method (b) and (c). CLT walls are modelled starting from test results on single connections, then an entire wall is modelled and calibrated based on wall test results and finally a simplified wall element is modelled with few non-linear springs which include non-linear properties of multiple connection elements acting in the same direction (e.g. steel angle brackets distributed at wall base are modelled as a single or double spring). The procedure chosen for Light-Frame walls is to start directly from wall test results and describe the entire wall behaviour with a simplified non-linear macro-element including only two springs, as described in §4.5.2.

4.5.1 DRAIN-3DX pinching hysteresis model: description and upgrade of the existing model

The numerical model used in this case for the non-linear dynamic analysis is a 3D space frame model created using the commercially available software Drain-3DX (Prakash and Powell, 1994) in which a hysteretic model with pinching behaviour developed at the University of Florence (Ceccotti and Vignoli, 1989) was implemented (Ceccotti et al., 2000). This model was successfully used in the prediction of the non-linear response of both Light Frame and CLT real buildings subjected to shake table tests (Ceccotti et al., 2000; Folz and Filiatrault, 2004; Follesa et al., 2010).

The hysteretic model is based on a piecewise tri-linear approximation of the non-linear cyclic behaviour of semi-rigid joints in timber structures obtained from test data with 4 or 6 different branch inclinations as shown in Figure 4-11. The model may have symmetric and asymmetric behaviour.

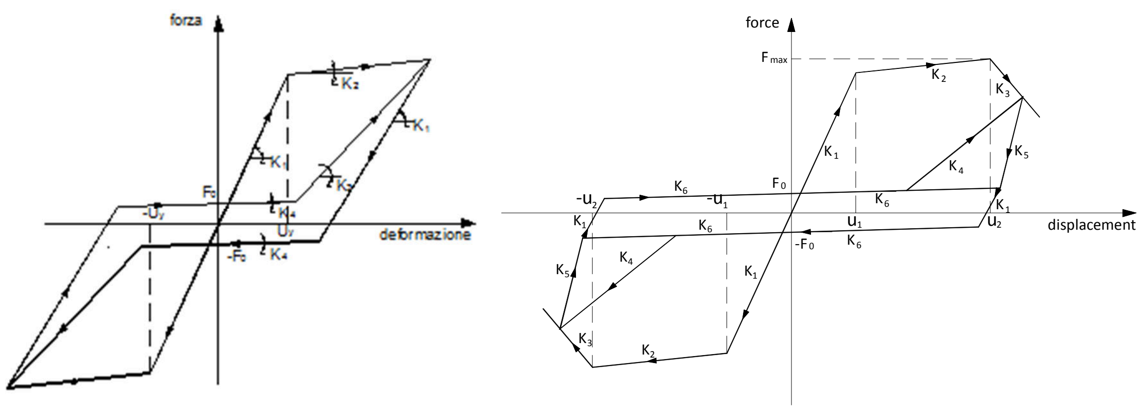


Figure 4-11 Tri-linear pinching hysteresis model for Drain 3DX with four and six branch inclinations.

The hysteretic model may be used for rotational or translational non-linear springs. The translational springs have six degrees of freedom (three displacement components at each end) and resist only axial forces. The element deformation is given by the relative displacement between the two ends. By defining dx_1, dx_2, dy_1, dy_2, dz_1, dz_2 as the displacement increases for the three spatial directions at each end, the element displacement increase dq may be defined as:

$$dq = [-dircos(X) \quad -dircos(Y) \quad -dircos(Z) \quad dircos(X) \quad dircos(Y) \quad dircos(Z)] \times \begin{bmatrix} dx_1 \\ dy_1 \\ dz_1 \\ dx_2 \\ dy_2 \\ dz_2 \end{bmatrix} \tag{5}$$

Which in contracted form is:

$$dq = [A] \times [dv]^T \tag{6}$$

Where

$$[A] = [-dircos(X) \quad -dircos(Y) \quad -dircos(Z) \quad dircos(X) \quad dircos(Y) \quad dircos(Z)]$$

$$[dv]=[dx_1 \quad dy_1 \quad dz_1 \quad dx_2 \quad dy_2 \quad dz_2]$$

Where dircos(X), dircos(Y) and dircos(Z) signify the three director cosines of the translational element with respect to the global X,Y and Z axis which can be written as:

$$dircos(X) = \frac{x_2 - x_1}{L} \tag{7}$$

$$dircos(Y) = \frac{y_2 - y_1}{L} \tag{8}$$

$$dircos(Z) = \frac{z_2 - z_1}{L} \tag{9}$$

With x_1, y_1, z_1, and x_2, y_2, z_2 representing the global coordinates of the two element ends and L the element length.

The relationship between the axial force increase dF and the deformation increase dq is:

$$dF = K_t \times dq \tag{10}$$

With K_t representing the tangent stiffness.

The tangent stiffness matrix of the element can be written as:

$$[K_t] = [A]^T \times K_t \times [A] \tag{11}$$

Considering the six branch inclinations, 9 parameters are needed to define the model:

1. The elastic stiffness K_1
2. The first inelastic stiffness K_2
3. The second inelastic stiffness K_3
4. The pinching reloading stiffness K_4
5. The pinching unloading stiffness K_5
6. The pre and post pinching stiffness K_6. If this value is set to -1 the stiffness is variable depending on the maximum displacement previously reached during the time-history u_{max}
7. The yield displacement u_1
8. The displacement corresponding to the peak force u_2
9. The residual force F_0

The above described hysteretic model proved to be reliable in the simulation of the real non-linear behaviour of mechanical joints in timber structures, especially if the fitting to the test data is made by minimizing the difference in energy dissipation. However, the model is rather simplified as it does not take into account two well-known phenomena that occur in the cyclic behaviour of semi-rigid timber joints: strength and stiffness degradation.

Stiffness and strength degradation have therefore been implemented in the hysteretic model according to the laws defined in Rinaldin et al. (2013a) with the following relationship:

$$K_{deg} = K_4 \cdot \left[1 - \frac{u_{max}}{u_u}(1-\beta)\right] \tag{12}$$

where

K_{deg}= degraded stiffness

u_{max} maximum displacement reached during the time-history

u_u ultimate displacement

β stiffness degradation linear parameter varying between 0 and 1

To implement the strength degradation, an energy related law, simplified with respect to the one defined in Rinaldin et al. (2013a), has been used in order to calculate the extra displacement part due to the strength degradation in the reloading branch.

$$\Delta d = \gamma \cdot d_{el} \cdot E_{dis}^{\alpha} \tag{13}$$

where

Δd additional displacement at reloading due to strength degradation

γ linear strength degradation parameter

E_{dis} dissipated energy

α exponential strength degradation parameter

Therefore, according to Equations (12) and (13) in the new version of the hysteretic model four new input parameters must be defined:

10. The ultimate displacement u_u
11. The linear stiffness degradation parameter β
12. The linear strength degradation parameter γ
13. The exponential strength degradation parameter α

For the asymmetric model, developed in order to describe the asymmetric behaviour of hold-down and tie-down connections in CLT and Light-Frame construction, a 14th parameter has been defined:

14. The elastic stiffness for the compression negative side K_{11}

4.5.2 Calibration to test data

In order to validate the new hysteretic model of DRAIN-3DX, a calibration procedure is made by fitting the model to test results on CLT connections, CLT walls and Light-Frame walls tested under full reversed cyclic loading. The calibration parameters considered are again, like in the procedure described in Chapter 3, maximum force, maximum displacement and amount of dissipated energy.

4.5.2.1 Calibration procedure for CLT connections

The calibration started from the fitting to the cyclic test data obtained on the same type of connectors used for the design of the 4 storey case-study building conducted by I. Gavric at CNR-Ivalsa laboratory in San Michele all'Adige (Gavric, 2013a) and referenced in §3.2.7.1. Figure 4-12 and Figure 4-13 show the calibration process in terms of comparison of hysteresis cycles and dissipated energy for the WVS90110 steel brackets connected with 1 ϕ12 steel rod class 4.6 to the foundation and 11 ϕ4x60 anker nails to the CLT walls and for hold-downs HTT22 connected with 1 ϕ12 steel rod class 4.6 to the foundation and 12 ϕ4x60 anker nails to the CLT walls.

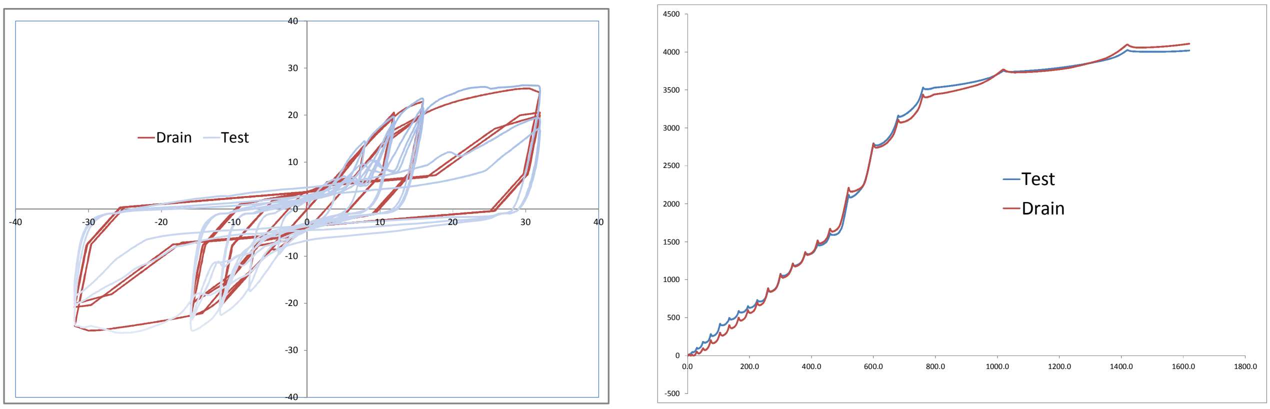


Figure 4-12 Calibration procedure for WVS90110 steel brackets and comparison in terms of dissipated energy.

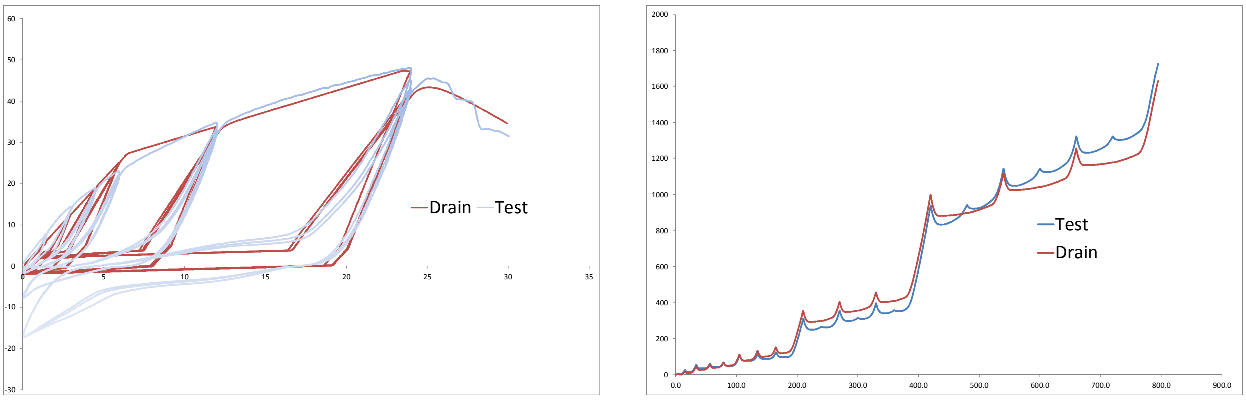


Figure 4-13 Calibration procedure for HTT22 hold-downs and comparison in terms of dissipated energy.

Figure 4-14 show the same calibration process for the angle brackets referenced in Figure 4-12 with the old and new model with the same values of the common fitting parameters between the two models. As it can be observed the fitting to the test data is greatly improved.

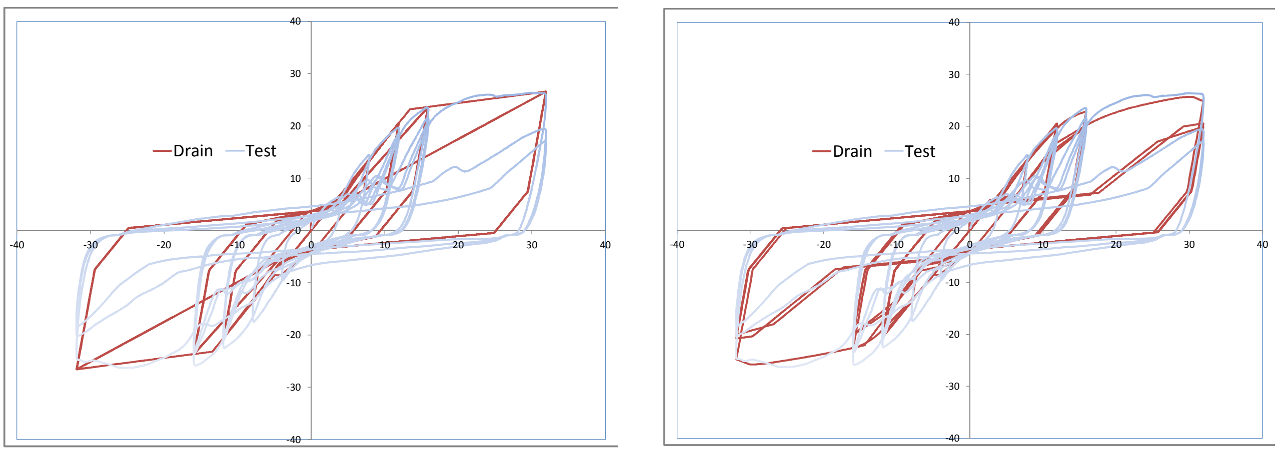


Figure 4-14 Comparison between old and new model.

4.5.2.2 Calibration procedure for CLT walls

Once the calibration of the single connectors was performed, the next step was the calibration of a single wall connected with the same type of connections and subjected to a cyclic displacement history and the comparison with test results on CLT walls tested with the same type of connections referenced in §4.5.2.1 conducted by I. Gavric at CNR-Ivalsa laboratory in San Michele all'Adige (Gavric, 2013b). The wall behaviour is simulated by a plane frame composed of rigid straight members rigidly connected and non-linear springs simulating the actual behaviour of the single connectors. Figure 4-14 shows the CLT wall tested under cyclic loading and the model schematization of the wall, composed by rigid straight members simulating the CLT panel behaviour, a couple of horizontal non-linear springs simulating the total shear stiffness of all the angle brackets, and vertical non-linear springs simulating the vertical asymmetric behaviour of hold-downs and angle brackets. Figure 4-16 shows the comparison between the test data and the model in terms of hysteresis cycles and dissipated energy. Vertical loads were lumped on the model top nodes. In order to obtain a better fitting also the fundamental contribution of friction was taken into account by means of a friction element (type 05) already included into the DRAIN-3DX library and considering a friction coefficient of 0.4.

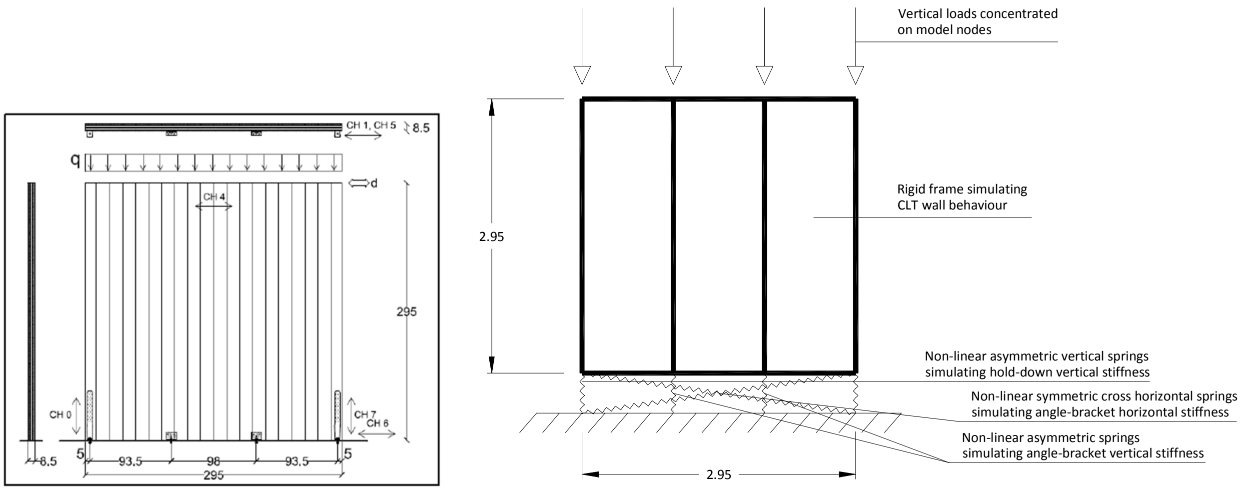


Figure 4-15 Tested wall and model schematization.

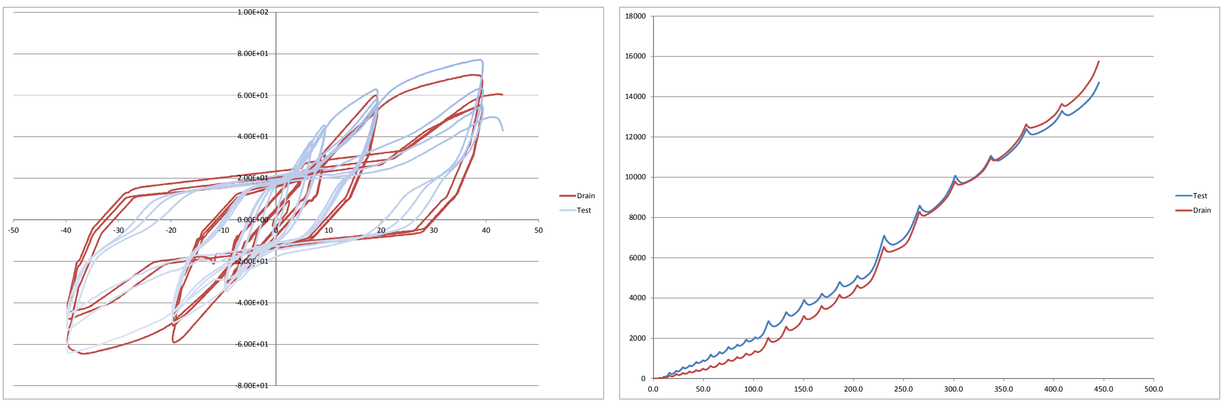
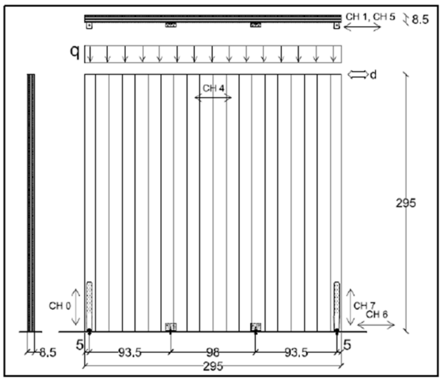


Figure 4-16 Comparison in terms of cyclic behaviour and dissipated energy between test and model.

In order to reduce the computational time of the 3D-model, a further simplified model has been developed, schematized by four rigid straight members simulating the CLT panel behaviour, a couple of horizontal non-linear springs simulating the total shear stiffness of all the angle brackets, and two vertical non-linear springs simulating the vertical asymmetric springs accounting for the vertical stiffness of hold-downs and angle brackets. Vertical loads are concentrated on the top model nodes. Friction elements have been placed on the base model nodes. Figure 4-17 show the simplified model schematization and Figure 4-18 shows the comparison between the test data and the model in terms of hysteresis cycles and dissipated energy. Even if less accurate than for the case of the model described in Figure 4-15, the comparison in terms of wall cyclic behaviour and dissipated energy is still satisfactory.



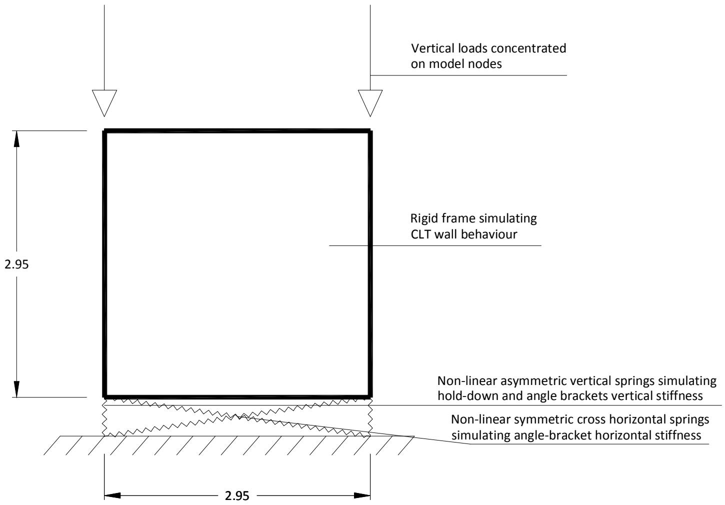
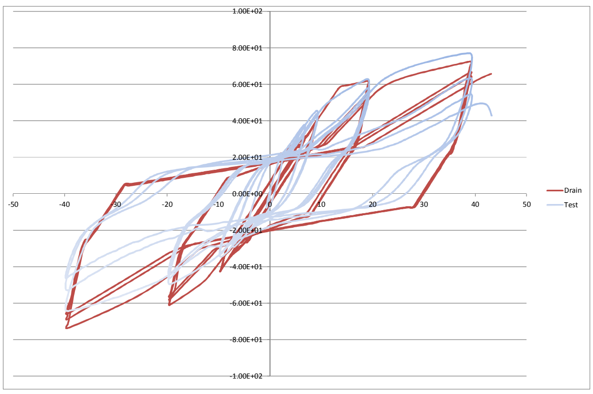


Figure 4-17 Tested wall and simplified model schematization.



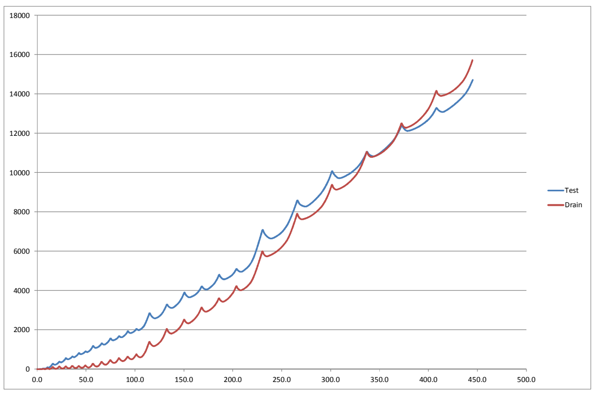


Figure 4-18 Comparison in terms of cyclic behaviour and dissipated energy between test and simplified model.

4.5.2.3 Calibration procedure for Light-Frame walls

The calibration procedure for Light Frame walls is based on the results of cyclic tests conducted at FPInnovations (at that time Forintek Canada Corp.) already referenced in §3.3.7.1 (Karacabeyli and Ceccotti 1996, Karacabeyli and Ceccotti 1998) and used for the non-linear dynamic analysis performed with SAP2000. For the sake of clarity the description of same wall described in §3.3.7.1 is here referenced. The wall specimen has a length of 4.88 m and height of 2.44 and was loaded with a uniformly distributed constant load of 18.2kN/m applied on the top of the wall, as is illustrated in Figure 4-19 left. Common nails (3 mm in diameter and 65 mm in length) were used to connect the sheathing to the framing; the nail spacing was 150 mm along panel edges and 300 mm elsewhere. Simpson hold-down HD2A was used at the ends of the shear wall and 12.5 mm anchor bolts spaced at 400 mm to connect the bottom sill plate were used.

The simplified model in DRAIN-3DX, similarly to the same model used for the analysis perform with SAP2000 and described in Chapter 3, consists of four pinned rigid straight members and two symmetric non-linear diagonal springs to simulate the wall behaviour, two asymmetric non-linear springs at the end of the wall to simulate the hold-down behaviour and two cross horizontal linear springs to simulate the anchor bolts connection. Figure 4-19 right shows the wall model and the DRAIN-3DX model while Figure 4-20 shows the comparison between test data and numerical analysis for a wall sheathed with 12.5 OSB panels on one side connected to the framing members with common nails as described above and the vertical load applied on the tests wall concentrated in the two upper model nodes.

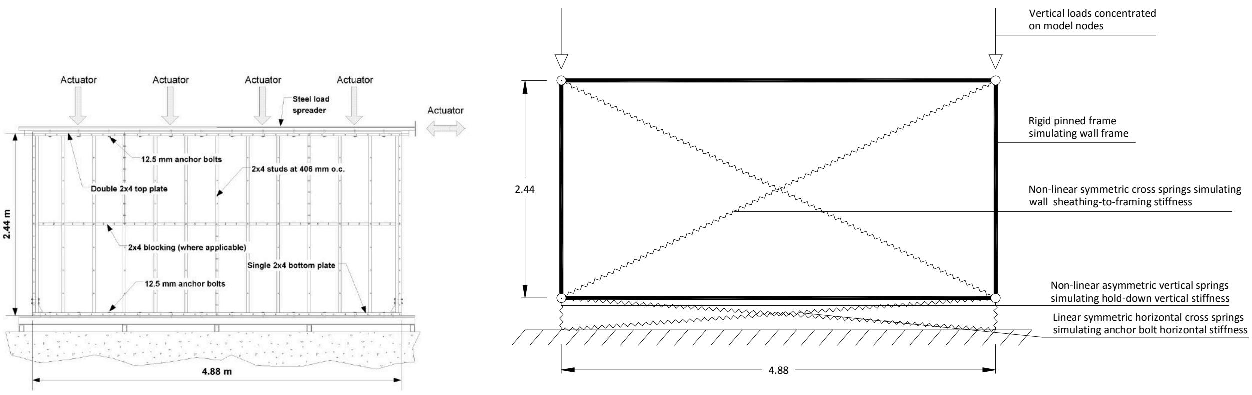
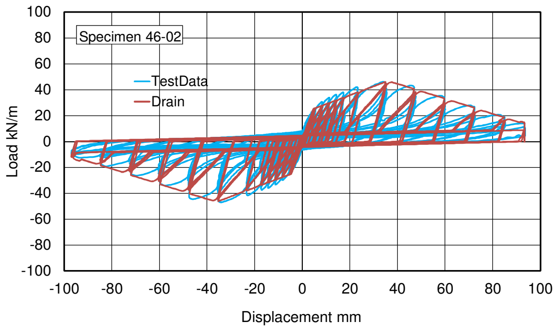


Figure 4-19 Tested wall (after Ceccotti and Karacabeyli, 2002) and model schematization.



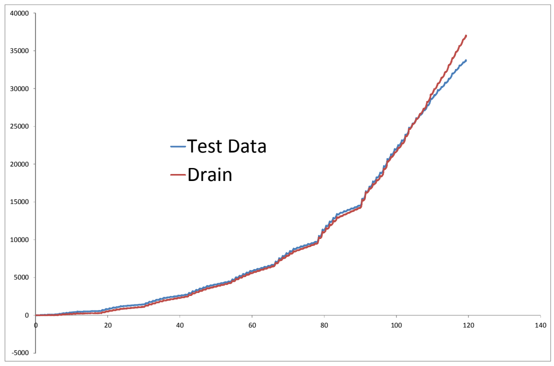


Figure 4-20 Comparison in terms of cyclic behaviour and dissipated energy between test and model.

4.6 Non-linear dynamic analysis: analysis and results

Based on the results of the preliminary design described in §4.4 and on the modelling and calibration procedure described in §4.5.1 and §4.5.2, a 3D model of the case study building has been built with DRAIN-3DX (Prakash and Powell, 1994) for the four building configurations.

4.6.1 3D-Model of the case-study building

The 3D-model of the case study building for Configuration C is illustrated in Figure 4-21 (similar schematizations are used for the other three configurations with different layout of walls like in Figure 4-5). CLT walls are composed of rigid straight members, shear connections (WVS90110 and WB90 angle brackets) are schematized with horizontal symmetric non-linear springs and hold-down anchors with vertical asymmetric non-linear springs as in the schematization of Figure 4-17. Light-Frame walls are modelled with four pinned rigid straight members and two symmetric non-linear diagonal springs to simulate the wall behaviour, two asymmetric non-linear springs at the end of the wall to simulate the hold-down behaviour and two cross horizontal linear springs to simulate the anchor bolts and screwed connection. Masses and vertical loads are lumped on the top storey model nodes. Floors are schematized as rigid by means of equivalent, very stiff cross-bracings schematized with truss elements, not showed in Figure 4-21 for the sake of clarity.

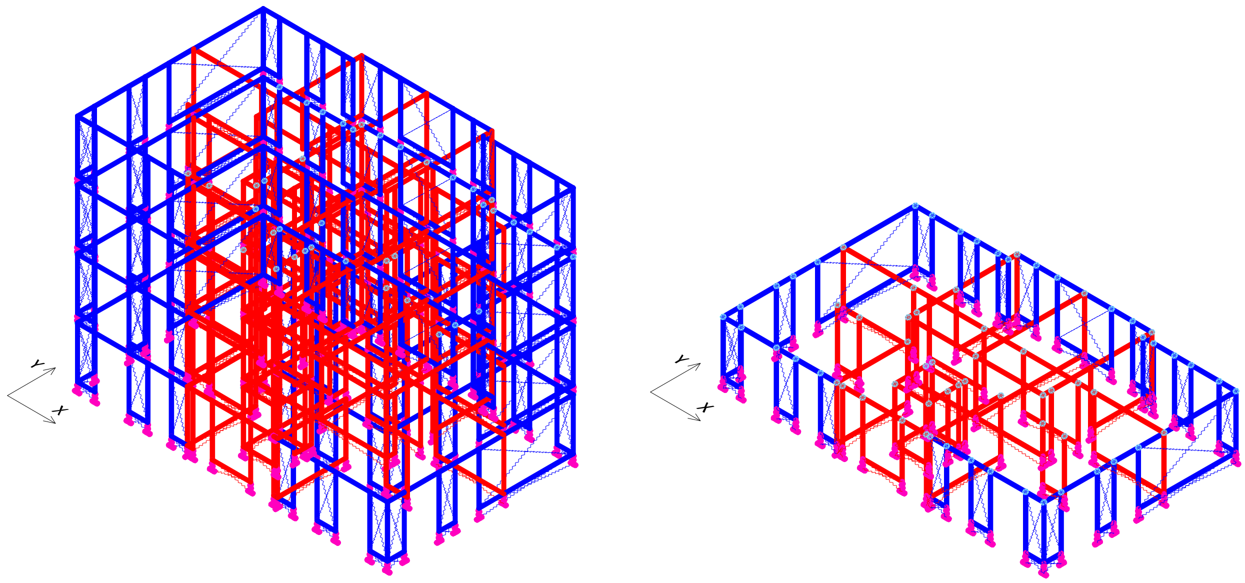


Figure 4-21 Drain-3DX 3D-model of the case study 4 storey building and of the first storey only for Configuration C. Like for Figure 4-5, Light-Frame walls are schematized with blue lines and CLT walls are schematized with red lines. Masses are lumped on model nodes represented with grey dots and hold-down anchors are represented with pink lines. Floors are schematized as rigid by means of equivalent rigid cross bracings, not showed in the figure for the sake of clarity.

The non-linear stiffness values of each spring are calculated by multiplying the unit stiffness values of the wall elements and connections considered in the calibration procedure described in §4.5.2 by the resulting multiplying factors obtained from the preliminary design results showed in §4.4.3 considering the actual wall length and nail spacing for Light-Frame walls and number of connectors.

For the time-history non-linear analysis, a time step of 0.005 sec and a stiffness-related Raleigh damping of 5% have been considered.

4.6.2 Choice of the earthquakes ground motions for the non-linear dynamic analysis

For the non-linear time-history analysis, a set of 12 earthquake ground motion records have been chosen, 6 of which are international historical earthquakes and 6 are spectrum-based earthquakes for the same type of soil considered in the seismic preliminary design, occurred in the same region (Friuli) in Tolmezzo and close to the epicentre of two consecutive earthquakes occurred in 1976.

The details of the chosen ground motion records are listed in Table 4-10.

Table 4-10 Details of the original ground motion records used for the non-linear time-history analysis.

Record number	Record Name	Country	Date	Station	Component	Duration (s)	PGA (g)
1	Kobe	Japan	1995/01/16	JMA	N-S	24.00	0.820
2	El Centro	California	1940/05/19	Imperial Valley	N-S	29.00	0.313
3	Brienza	Italy	1980/11/23	Brienza	N-S	20.00	0.220
4	Northridge	California	1994/01/17	Newhall	E-W	19.98	0.600
5	Loma Prieta	California	1989/10/18	Corralitos	E-W	39.98	0.644
6	Kocaeli	Turkey	1999/08/17	Yapi Kredi	N-S	27.88	0.168
7	Friuli 1	Italy	1976/05/06	Tolmezzo	N-S	21.00	0.370
8	Friuli 2	Italy	1976/05/06	Tolmezzo	SE-NW	21.00	0.480
9	Friuli 3	Italy	1976/09/11	Folgaria-San Rocco	E-W	18.58	0.227
10	Friuli 4	Italy	1976/09/11	San Rocco	N-S	13.03	0.090
11	Friuli 5	Italy	1976/09/15	Folgaria-San Rocco	N-S	22.07	0.259
12	Friuli 6	Italy	1976/09/15	San Rocco	E-W	13.03	0.120

4.6.3 Near-collapse criterion

The choice of the near-collapse criterion is crucial for the evaluation of the behaviour factor q through non-linear time-history analysis, especially in this case where a hybrid building with different types of lateral load resisting systems are employed. Different criteria have been proposed in literature both for CLT and Light-Frame buildings. For CLT structures, Ceccotti et al., 2007 proposed a maximum uplift value based on experimental research for one or more hold-down anchors in the analysed building. For Light-Frame structures another possibility is given by establishing a maximum value of the inter-story drift, e.g. 2 or 2.5%, which represent the maximum ultimate drift displacement according to the reference seismic design code requirements representing the state of “near collapse” equivalent to extensive damage or based on experimental results observations.

However, considering the different seismic behaviour of the two structural systems, the choice of the near-collapse criterion based on a single deformation parameter appear too limiting and may not be representative of the hybrid building seismic behaviour. Therefore the choice is to consider the deformation corresponding to the 80% of the maximum load reached in the descending part of the first cycle envelope curve of any connection or CLT or Light-Frame wall sub-systems as defined in EN 12512 (CEN, 2001) and shown in Figure 4-22.

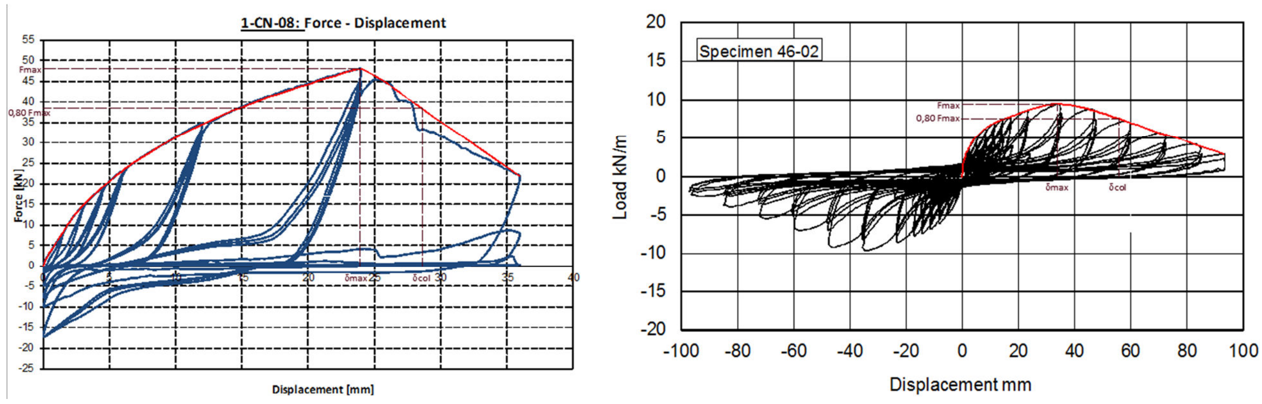


Figure 4-22 Definition of the near-collapse criterion for hold-downs and Light-Frame walls.

4.6.4 Analysis results

For each configuration, the case study building was analysed both in X and Y direction. For the dynamic analysis, a time-step of 0.005 sec. was used and a 5% stiffness proportional Raleigh damping was applied in order to account for the dissipating contribution of non-structural elements such as partition walls.

Table 4-11 summarizes the fundamental periods and the mass participation factors for the first 3 mode shapes of the structure in the four building configurations.

Table 4-11 Modal analysis results of the DRAIN-3DX non-linear model.

Configuration	Mode	Period (sec)	Mass participation for translation along X direction (%)	Mass participation for translation along Y direction (%)	Mass participation for rotation along vertical direction (%)
A	1	0.85	18.55	65.79	0.02
	2	0.79	66.32	18.65	0.05
	3	0.65	0.54	0.14	49.61
B	1	0.72	4.30	80.48	0.02
	2	0.65	80.15	4.42	0.06
	3	0.59	0.54	0.22	58.63
C	1	0.60	29.40	54.92	0.04
	2	0.59	55.05	29.71	0.02
	3	0.46	0.35	0.26	54.37
D	1	0.41	22.30	61.60	0.02
	2	0.37	61.79	22.52	0.05
	3	0.31	0.45	0.16	49.67

The periods as it was expected are lower than the ones listed in Table 4-9 since the non-linear springs characteristics have been designed according to the preliminary design results made considering a behaviour factor q=1, whereas the periods given in Table 4-9 are derived from a first hypothesis of connection layouts for the four configurations based on common practice.

Tables 4-12 to 4-15 show the analysis results for the 12 input ground motion records respectively for building configurations A, B, C and D in X and Y direction in terms of ultimate Peak Ground Acceleration PGA_u and the corresponding calculated values of the behaviour factor q.

Table 4-12 Analysis results for Configuration A.

	X Direction		Y Direction	
Record Name	**PGA_u [g]**	**q**	**PGA_u [g]**	**q**
Kobe	0.95	4.01	1.21	5.09
El Centro	0.88	3.73	0.98	4.12
Brienza	0.86	3.63	0.92	3.87
Northridge	0.82	3.46	0.89	3.76
Loma Prieta	0.99	4.19	1.01	4.27
Kocaeli	0.93	3.91	1.01	4.25
Tomezzo N-S	0.94	3.99	1.22	5.15
Tolmezzo E-W	0.99	4.18	1.08	4.57
Folgaria 1 E-W	1.10	4.66	1.17	4.94
San Rocco 1 N-S	1.12	4.72	1.15	4.87
Folgaria 2 N-S	1.31	5.54	1.47	6.19
San Rocco 2 E-W	1.04	4.37	1.08	4.55
Average		**4.20**		**4.64**

Table 4-13 Analysis results for Configuration B.

	X Direction		Y Direction	
Record Name	**PGA_u [g]**	**q**	**PGA_u [g]**	**q**
Kobe	0.93	3.94	1.15	4.85
El Centro	0.87	3.66	0.94	3.97
Brienza	0.75	3.16	0.78	3.29
Northridge	0.76	3.21	0.65	2.74
Loma Prieta	0.71	3.00	0.73	3.08
Kocaeli	0.91	3.82	1.01	4.10
Tomezzo N-S	0.85	3.59	0.92	5.05
Tolmezzo E-W	0.79	3.33	0.82	4.20
Folgaria 1 E-W	0.82	3.46	1.14	4.70
San Rocco 1 N-S	1.03	4.33	1.06	4.48

Folgaria 2 N-S	1.22	5.15	1.26	5.31
San Rocco 2 E-W	0.91	3.82	0.99	4.18
Average		**3.71**		**4.16**

Table 4-14 Analysis results for Configuration C.

	X Direction		**Y Direction**	
Record Name	**PGA_u [g]**	**q**	**PGA_u [g]**	**q**
Kobe	0.69	2.92	0.78	3.29
El Centro	0.62	2.60	0.63	2.68
Brienza	0.64	2.69	0.68	2.87
Northridge	0.62	2.62	0.61	2.57
Loma Prieta	0.69	2.91	0.69	2.91
Kocaeli	0.67	2.82	0.69	2.90
Tomezzo N-S	0.68	2.89	0.83	3.51
Tolmezzo E-W	0.66	2.78	0.77	3.27
Folgaria 1 E-W	0.81	3.43	0.77	3.27
San Rocco 1 N-S	0.73	3.06	0.67	2.83
Folgaria 2 N-S	0.66	2.79	0.78	3.30
San Rocco 2 E-W	0.83	3.49	0.80	3.36
Average		**2.92**		**3.06**

Table 4-15 Analysis results for Configuration D.

	X Direction		**Y Direction**	
Record Name	**PGA_u [g]**	**q**	**PGA_u [g]**	**q**
Kobe	0.47	2.00	0.69	2.93
El Centro	0.57	2.42	0.50	2.09
Brienza	0.66	2.78	0.60	2.52
Northridge	0.45	1.91	0.44	1.85
Loma Prieta	0.51	2.15	0.57	2.42
Kocaeli	0.59	2.51	0.60	2.54
Friuli 1	0.63	2.66	0.77	3.27
Friuli 2	0.64	2.72	0.64	2.69
Friuli 3	0.70	2.97	0.59	2.49
Friuli 4	0.61	2.58	0.55	2.31

Friuli 5	0.52	2.21	0.63	2.66
Friuli 6	0.66	2.79	0.71	2.99
Average		**2.47**		**2.56**

The first remark about the analysis results is that the proposed Eurocode 8 values for the current behaviour factors q of the two structural systems, i.e. q=3 for Light Frame buildings with rigid diaphragms and q=2 for CLT buildings are confirmed by all the analysed cases. For configuration A, i.e. all Light-Frame walls, the average value of q was 4.20 in X and 4.64 in Y direction which are far above the proposed code values. Also for configuration D the average q values in X and Y directions are above the code proposed values, and only for Northridge earthquake are lower than 2. This is a bit surprising if we consider that CLT walls were modelled and designed without vertical step screwed joints (therefore higher q values were expected for the case of CLT walls as the dissipative performance of the building increases), but anyway confirms the results found in Ceccotti et al. (2007), Pozza et al. (2009) and Pozza et al. (2013). According to the analysis results a q value respectively of 4.0 for Light-Frame structures and 2.0 for CLT structures seem reasonably acceptable and largely conservative. The q values found for the two hybrid configurations are closer to the reference configuration with all Light-Frame walls (Configuration A) for configurations B, were only the lift and stair case walls are made with the CLT system, and to the reference configuration with all CLT walls (Configuration D) for configuration C were all the internal walls are made with the CLT system.

The results for the four configurations show a general decrease of the calculated q factor from the configuration A (only Light-Frame walls) to configuration D (only CLT walls). The collapse occurred for configuration A in most cases due to the collapse displacement reached for Light Frame walls at first storey both in X and Y direction while only in few cases for hold-down failures. For configuration B, the collapse was mainly due to hold-down failures in both CLT and Light-Frame walls and only in some cases occurred for Light-Frame walls collapse. For configurations C and D, the collapse occurred mainly for hold-down failures in CLT walls and only in a few cases for angle brackets failure in CLT walls. Being the building designed in all the four configurations at each storey for the reference seismic shear, the collapse occurred not always in the first storey but also in the second and third storey, especially for configurations C and D. Figure 4-27 and Figure 4-28 show the results for the four building configurations in X and Y direction respectively. As it can be observed, the calculated values in two cases (Brienza and Tolmezzo N-S records) are higher in configuration D than in configuration C. This may be due to the fact that the layout of walls in X direction was not symmetric like in Y direction and consequently a slight torsional behaviour induced the collapse in external walls PT02 and PT26 in X direction (see Figure 4-9 for reference) due to hold-down failures.

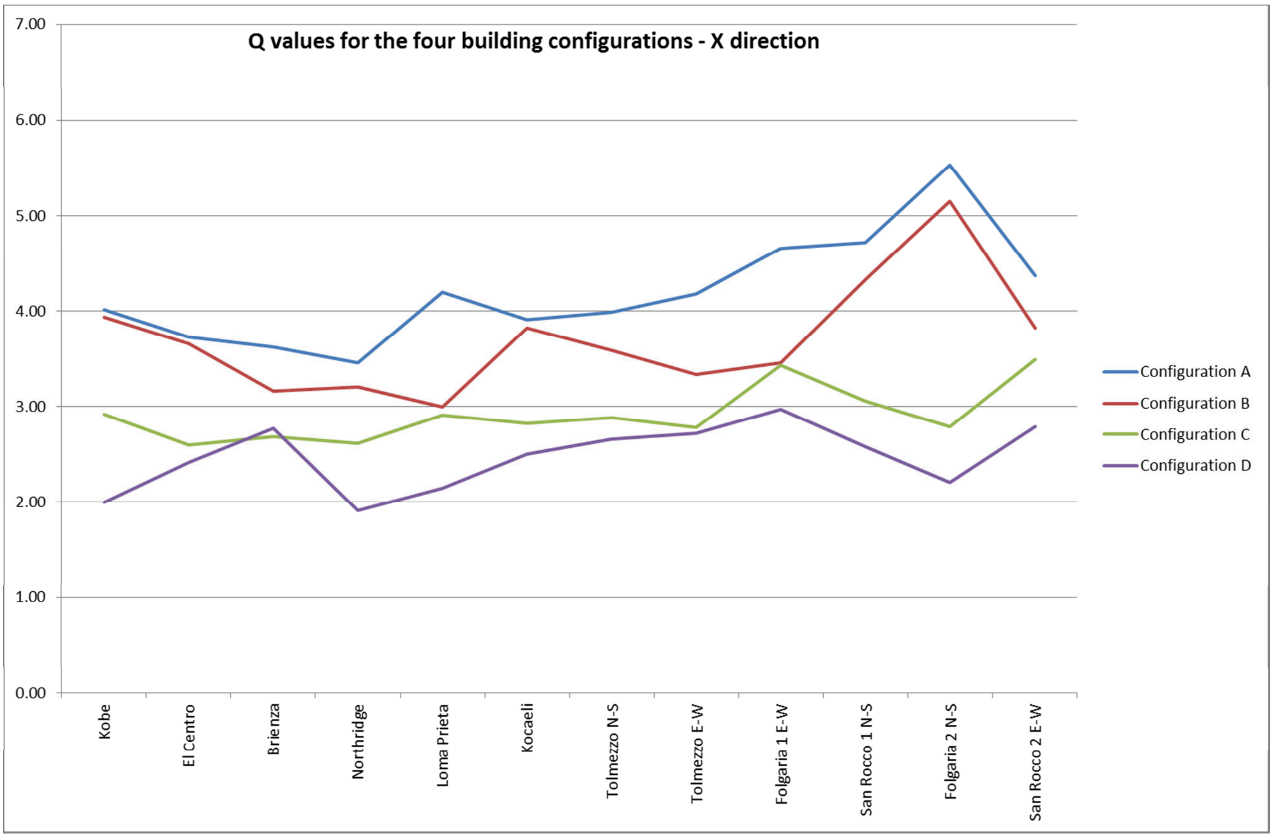


Figure 4-23 Comparison of the analysis results for the four building configurations in X direction.

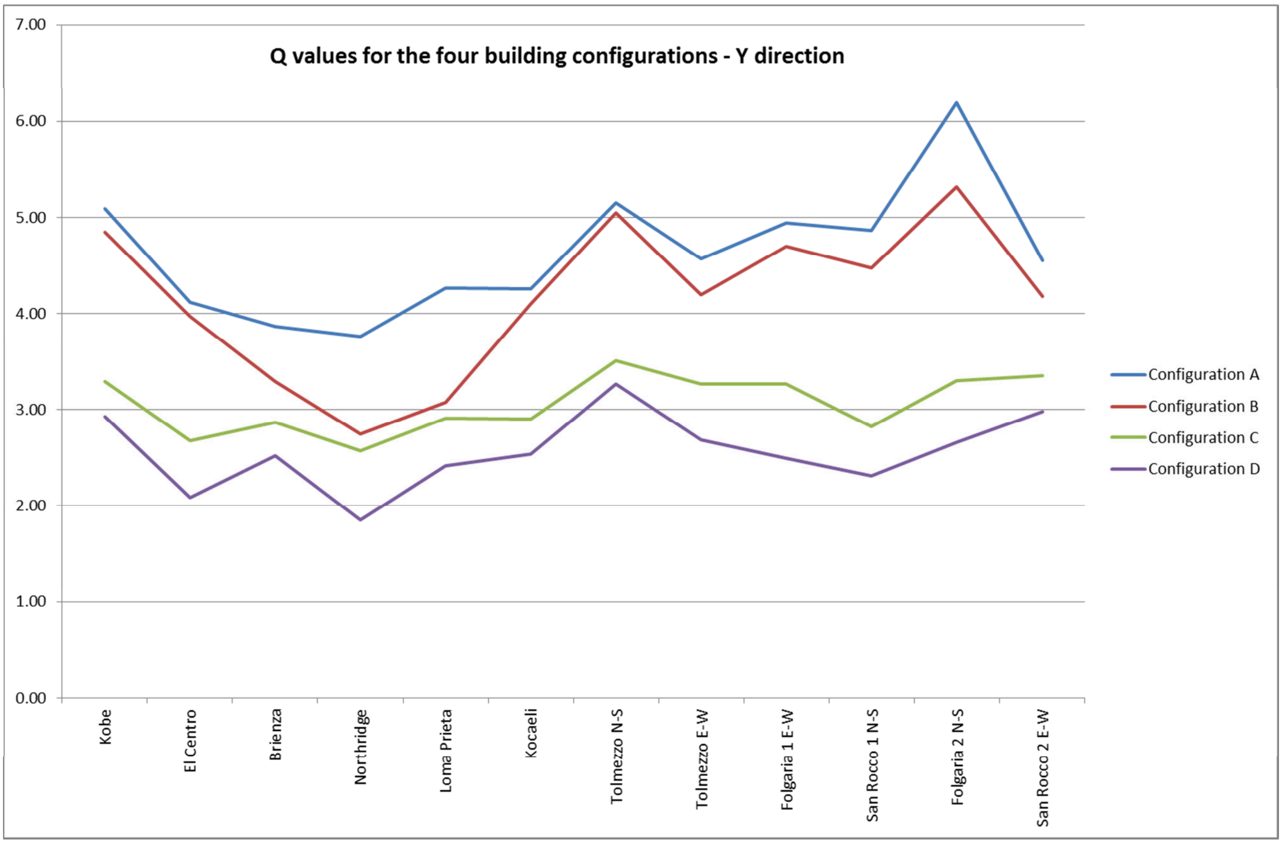


Figure 4-24 Comparison of the analysis results for the four building configurations in Y direction.

Figure 4-25 show the deformed shape at the time-step when the PGA_u for Kobe record, direction Y, was reached. The "near collapse" state was due to the collapse displacement reached for one of the two hold-down of wall 29 (see Figure 4-9 for reference). Finally Figure 4-26, Figure 4-27 and Figure 4-28 show the recorded time history in three different cases were the collapse was reached for hold-down failure, Light-Frame wall failure and angle brackets failure in CLT walls respectively.

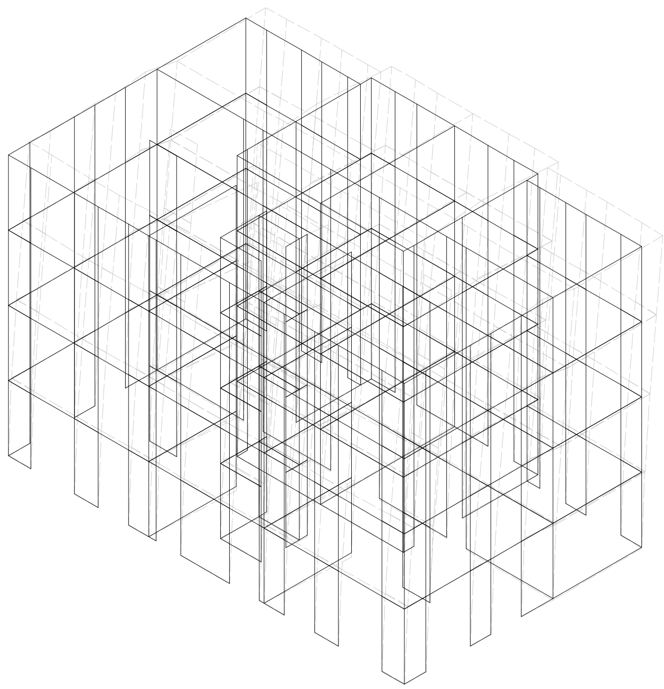


Figure 4-25 Deformed shape of the building for configuration C at the collapse for the Kobe record in Y direction. Collapse was due to hold-down failure in wall n. 29, storey 2.

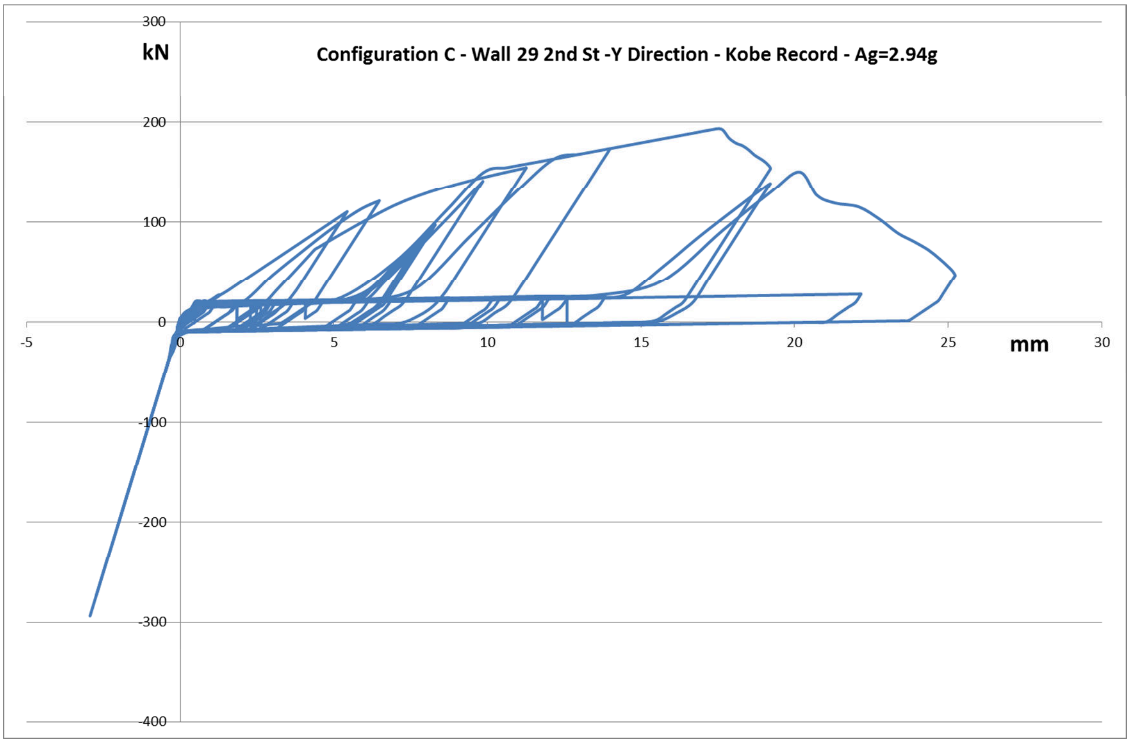


Figure 4-26 Time-history for hold-down asymmetric non-linear spring which attained the "near-collapse" limit state referenced in Figure 4-25.

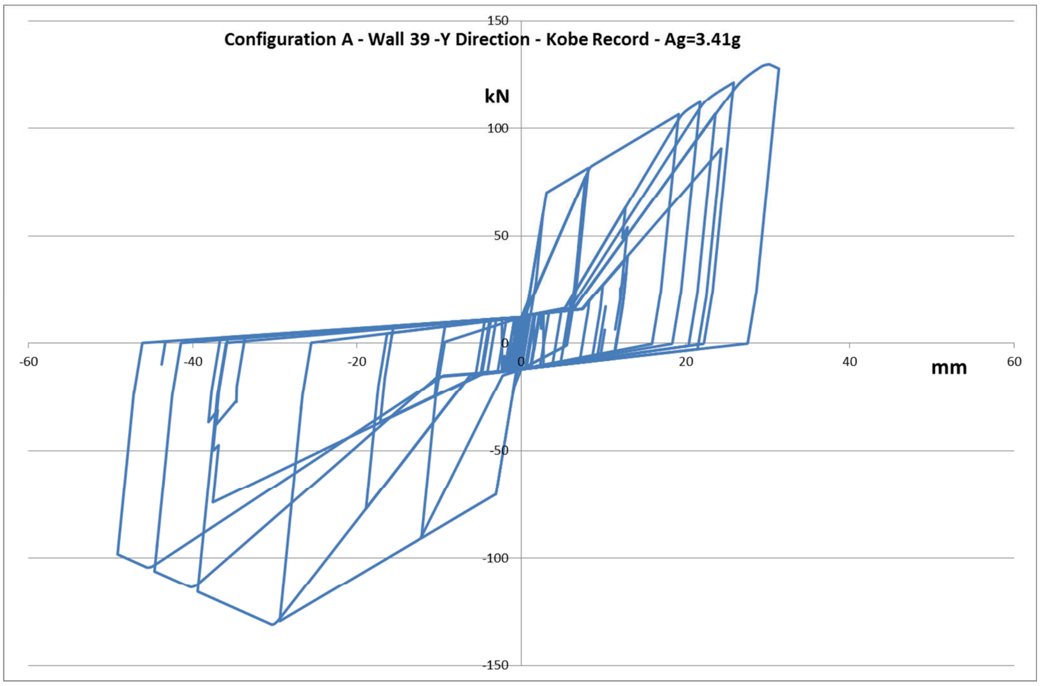


Figure 4-27 Time-history for the Light-Frame n.39-1st storey wall which attained the "near-collapse" limit state for Configuration A, Y direction, Kobe record.

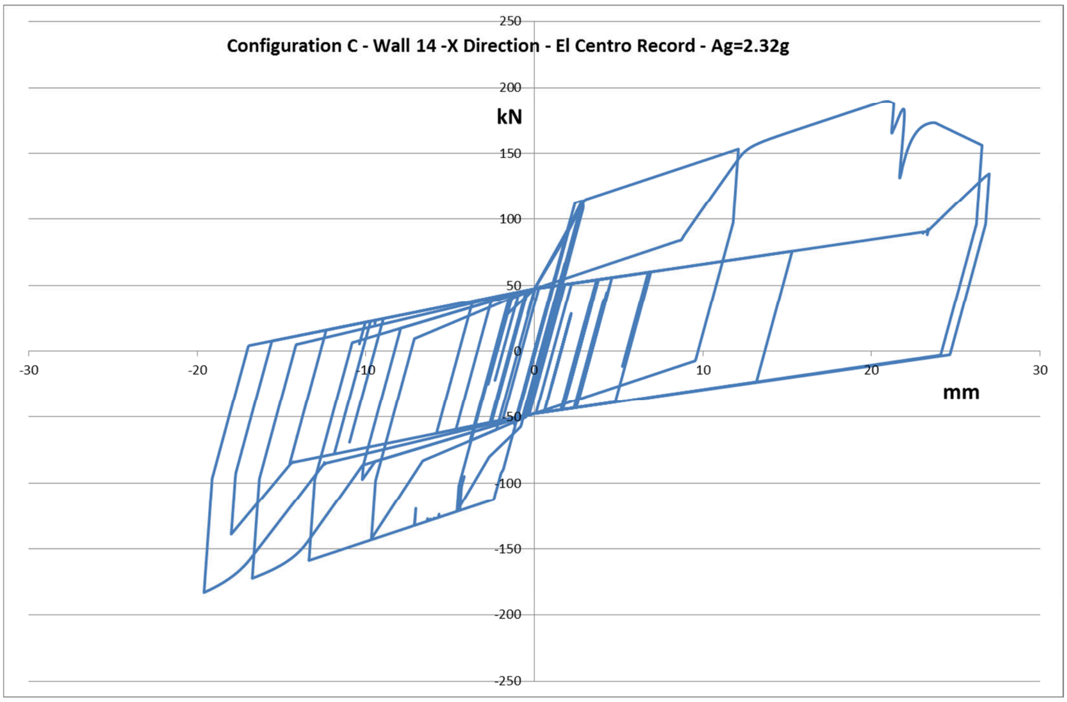


Figure 4-28 Time-history for CLT wall n.14 non-linear spring which attained the “near-collapse” limit state for Configuration D, X direction, El Centro record.

4.7 Analytical formulation proposal for the calculation of q factor for hybrid buildings

The analysis results described in §4.6 for the four building configurations lead to some considerations about the seismic behaviour of hybrid CLT-Light-Frame structures. First of all, it is confirmed by this analytical study that the values of the behaviour factors found for the hybrid configurations B and C are always higher than the lowest behaviour factor of the two structural systems (except for one case, i.e. Brienza record in X direction were the resulting q value was higher in configuration D than the one found for configuration C). Therefore the current approach proposed by some seismic design codes, like e.g. the National Building code of Canada (NBCC 2010), of considering the lowest value of the seismic force modification factor between the two systems is confirmed. Secondly, the results found in Zhou et al. (2014) for mixed Masonry-Light-Frame structures are confirmed, i.e. the q value of the hybrid system could be assumed as something in between the proposed code values for the two reference systems, and related to the total lateral resistance of the less ductile system with respect to the total lateral resistance of the building in the reference direction. However, a slight modification is proposed to the reference analytical formulation proposed by Zhou et al. (2014) by making reference to the total horizontal stiffness of the less ductile system with respect to the total calculated horizontal stiffness of the building in the same direction. According to this proposal, the following formulation can be proposed:

$$q_H = (q_{HD} - q_{LD}) \times (1 - k) + q_{LD} \tag{14}$$

Where:

q_H is the reference behaviour factor q for the hybrid system

q_{HD} is the reference behaviour factor q for the High Ductility system (Light-Frame in this case)

q_{LD} is the reference behaviour factor q for the Low Ductility system (CLT in this case)

k is the lowest horizontal stiffness ratio among all storeys of the Low Ductility system (CLT in this case) with respect to the total horizontal stiffness of the building in the reference direction

According to the results showed in Table 4-8, the lowest horizontal stiffness among all storeys of CLT walls in Y direction with respect to the total horizontal stiffness in the same direction is 77.0% for configuration C. Therefore, considering a reference q value of 4.0 for the Light-Frame system and 2.0 for the CLT system, the calculated q value for the hybrid system should be:

$$q_H = (4.0 - 2.0) \times (1 - 0.77) + 2.0 = 2.46 \tag{15}$$

According to the q value found in Equation 15, a new preliminary design was made for the building configuration C with the same procedure described in §4.4.3. The results are showed in Table 4-16.

Table 4-16 Results of the new preliminary design for Configuration C (d12 A.B. stands for Anchor Bolts with 12 mm diameter, WVS90110 for WVS90110 angle brackets, LF for Light-Frame wall, CLT for CLT walls).

PRELIMINARY DESIGN RESULTS CASE C - ALL EXTERNAL LIGHT-FRAME WALLS, ALL INTERNAL CLT WALLS - q=2.46																	
Wall Name	Length [m]	Wall direction	Sei. Shear [kN]	Sei. Shear [kN/m]	Sei. Uplift 1 [kN]	Sei. Uplift 2 [kN]	Sei. Axial [kN]	Sta. Axial [kN]	Wall type	Shear connection type	Overstrength factor	N Shear connections	Nail spacing s [cm]	Uplift connection type	N Uplift connections at each end	Wall shear strength [kN]	Wall stiffness [kN/m]
PT01	1.03	X	8.9	8.62	16.6	35.4	67.6	63.8	LF	d12 A.B.	1.3	1	6.96	HTT22	2	8.9	1.6
PT02	1.10	X	8.9	8.12	15.7	16.1	26.1	103.7	LF	d12 A.B.	1.3	1	7.84	HTT22	1	8.9	1.5
PT03	0.87	X	5.9	6.83	52.8	75.7	125.8	38.6	LF	d12 A.B.	1.3	1	7.39	HTT22	3	5.9	1.3
PT04	0.39	X	2.7	6.78	149.7	151.4	335.2	23.1	CLT	WVS90110	1	1	N/A	HTT22	6	11.5	21.5
PT05	2.25	X	18.7	8.31	82.3	81.0	11.0	89.1	CLT	WVS90110	1	3	N/A	HTT22	3	23.0	14.6
PT06	0.28	X	2.7	9.59	168.8	171.5	386.9	20.5	CLT	WVS90110	1	1	N/A	HTT22	7	11.5	23.3
PT07	0.87	X	5.9	6.83	78.9	51.7	135.1	49.9	LF	d12 A.B.	1.3	1	7.39	HTT22	3	5.9	1.3
PT08	1.10	X	8.9	8.12	11.8	10.9	35.1	139.7	LF	d12 A.B.	1.3	1	7.84	HTT22	1	8.9	1.5
PT09	1.03	X	8.9	8.62	28.7	13.1	71.0	83.9	LF	d12 A.B.	1.3	1	6.96	HTT22	2	8.9	1.6
PT10	1.86	X	15.5	8.35	64.4	112.4	149.5	106.9	CLT	WVS90110	1	2	N/A	HTT22	4	23.0	15.9
PT11	1.86	X	15.0	8.08	87.5	64.2	115.8	112.4	CLT	WVS90110	1	2	N/A	HTT22	4	23.0	15.9
PT12	1.24	X	9.9	8.01	30.9	30.5	58.8	264.1	CLT	WVS90110	1	2	N/A	HTT22	2	11.5	13.9
PT13	1.24	X	9.9	8.01	26.4	26.4	69.7	313.2	CLT	WVS90110	1	2	N/A	HTT22	1	11.5	13.9
PT14	7.65	X	60.4	7.90	44.9	45.8	63.0	377.3	CLT	WVS90110	1	8	N/A	HTT22	2	69.0	13.7
PT15	3.99	X	31.3	7.85	53.6	67.4	45.3	407.1	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	13.4
PT16	3.11	X	24.1	7.75	54.5	58.7	32.0	337.4	CLT	WVS90110	1	3	N/A	HTT22	3	34.5	15.2
PT17	3.11	X	24.1	7.75	60.5	58.7	31.9	337.3	CLT	WVS90110	1	3	N/A	HTT22	3	34.5	15.2
PT18	3.99	X	31.3	7.85	68.3	53.5	45.4	406.8	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	13.4
PT19	0.96	X	7.0	7.28	5.5	23.4	60.7	72.9	LF	d12 A.B.	1.3	1	7.67	HTT22	1	7.0	1.3
PT20	0.94	X	7.0	7.46	11.5	11.4	36.6	125.0	LF	d12 A.B.	1.3	1	7.32	HTT22	1	7.0	1.3
PT21	0.59	X	2.8	4.76	85.9	135.8	223.1	54.8	LF	d12 A.B.	1.3	1	7.17	HTT22	6	2.8	1.0
PT22	1.09	X	8.5	7.76	58.5	59.2	156.8	75.9	LF	d12 A.B.	1.3	1	8.14	HTT22	3	8.5	1.4
PT23	3.08	X	31.0	10.08	10.7	12.4	40.0	209.3	LF	d12 A.B.	1.3	4	8.22	HTT22	1	31.0	1.3
PT24	1.09	X	8.5	7.76	54.4	59.2	157.6	77.4	LF	d12 A.B.	1.3	1	8.15	HTT22	3	8.5	1.4
PT25	0.59	X	2.8	4.76	135.7	83.7	219.6	55.6	LF	d12 A.B.	1.3	1	7.17	HTT22	6	2.8	1.0
PT26	0.94	X	7.0	7.46	11.3	11.5	36.6	125.0	LF	d12 A.B.	1.3	1	7.32	HTT22	1	7.0	1.3
PT27	0.96	X	7.0	7.28	24.5	5.4	59.0	73.5	LF	d12 A.B.	1.3	1	7.67	HTT22	1	7.0	1.3
PT28	0.99	Y	6.8	6.88	18.8	38.9	57.6	35.8	LF	d12 A.B.	1.3	1	8.35	HTT22	2	6.8	1.2
PT29	0.95	Y	6.8	7.18	14.4	16.7	28.7	94.6	LF	d12 A.B.	1.3	1	7.67	HTT22	1	6.8	1.3
PT30	5.54	Y	53.1	9.58	13.4	1.5	24.9	135.9	LF	d12 A.B.	1.3	6	8.64	HTT22	1	53.1	1.3
PT31	0.48	Y	2.2	4.58	93.5	95.4	212.2	18.4	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	20.4
PT32	3.47	Y	24.0	6.92	98.5	26.3	78.6	117.4	CLT	WVS90110	1	3	N/A	HTT22	4	34.5	14.4
PT33	4.02	Y	27.6	6.87	43.9	67.4	48.0	198.4	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	13.3
PT34	2.13	Y	14.3	6.71	63.7	67.1	153.9	89.0	CLT	WVS90110	1	2	N/A	HTT22	3	23.0	15.0
PT35	6.18	Y	39.9	6.45	16.0	54.6	59.6	309.1	CLT	WVS90110	1	5	N/A	HTT22	2	46.0	12.3
PT36	0.98	Y	6.1	6.23	87.1	78.6	173.6	43.8	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	15.5
PT37	4.02	Y	27.7	6.89	46.7	73.4	46.5	224.7	CLT	WVS90110	1	4	N/A	HTT22	3	34.5	13.3
PT38	0.48	Y	2.2	4.60	98.7	95.7	213.9	19.5	CLT	WVS90110	1	1	N/A	HTT22	4	11.5	20.4
PT39	3.47	Y	24.1	6.94	93.5	36.7	79.5	115.3	CLT	WVS90110	1	3	N/A	HTT22	4	34.5	14.4
PT40	0.99	Y	6.9	6.92	19.5	38.8	59.0	41.8	LF	d12 A.B.	1.3	1	8.29	HTT22	2	6.9	1.2
PT41	0.95	Y	6.9	7.22	13.1	13.3	33.5	111.4	LF	d12 A.B.	1.3	1	7.62	HTT22	1	6.9	1.3
PT42	5.54	Y	53.4	9.64	13.4	1.5	24.6	136.0	LF	d12 A.B.	1.3	6	8.59	HTT22	1	53.4	1.3

Based on the upgraded preliminary design, the non-linear model was updated with the new values of stiffness for the non-linear springs simulating hold-downs, angle brackets and Light-Frame walls and a new set of non-linear analysis was performed only for configuration C in Y direction for the verification of the formula proposed in Equation 14. Table 4-17 show the analysis results and Figure 4-29 the trend of q values for the 12 records in Y direction.

Table 4-17 Analysis results for Configuration D.

	Y Direction	
Record Name	**PGA_u [g]**	**q**
Kobe	0.74	3.12
El Centro	0.59	2.49
Brienza	0.67	2.83
Northridge	0.59	2.49
Loma Prieta	0.65	2.74
Kocaeli	0.67	2.83
Friuli 1	0.75	3.16
Friuli 2	0.68	2.87
Friuli 3	0.67	2.83
Friuli 4	0.61	2.57
Friuli 5	0.70	2.95
Friuli 6	0.68	2.87
Average		**2.81**

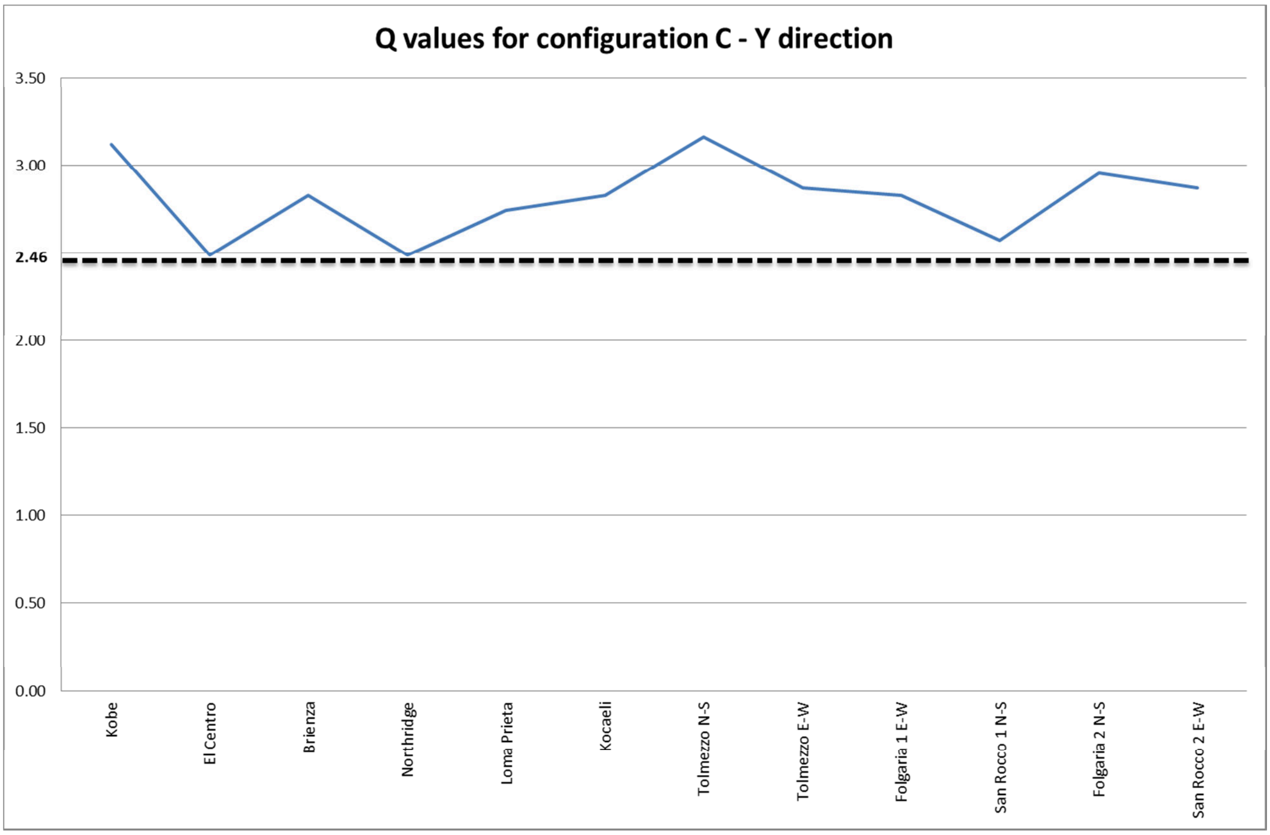


Figure 4-29 Analysis results for configuration C, Y direction.

As it can be observed, the calculated q-value for the hybrid configuration was confirmed by all case, even if with a small scatter for two cases (El Centro and Northridge record). More analyses should be of course performed in order to confirm the validity of the proposed formulation, also with other types of hybrid buildings with different types of lateral load resisting systems. However the formulation proved to be reliable for the analysed cases even if the results are limited to the chosen building configurations and the ductility properties of the two structural systems analysed in this case. More analyses should be performed also with other non-linear models proposed in literature to confirm the validity of the results.

References – Chapter 4

Ceccotti, A., Follesa, M., Karacabeyli E. [2000] "3D seismic analysis of multi-storey wood frame construction" Proceedings of 6th World Conference on Timber Engineering (WCTE), Whistler Resort, Canada, 2000.

Ceccotti, A., Karacabeyli, E. [2002]. "Validation of seismic design parameters for wood-frame shearwall systems." Canadian Journal of Civil Engineering, 29(3), 484-498.

Ceccotti A., Follesa M., Lauriola M.P. [2007] "Quale fattore di struttura per gli edifici multipiano a struttura di legno con pannelli a strati incrociati?" XII Convegno ANIDIS L'ingegneria sismica in Italia, Pisa 2007.

Ceccotti A., Sandhaas C. [2010] "A proposal for a standard procedure to establish the seismic behaviour factor q of timber buildings", Proceedings of 11th World Conference on Timber Engineering (WCTE), Riva del Garda, Italy, 2010.

Ceccotti A., Vignoli A. [1989] "A pinching hysteresis model for semi-rigid joints." European Earthquake Engineering Journal, Vol.3 pages 3–9, 1989.

Chen M., Chui Y.H., Doudak G. [2014] "An approach for estimating seismic force modification factor of hybrid building systems". Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Computers & Structures Inc. [2000]. "SAP2000—Integrated finite element analysis and design of structures. Ver. 14" Computers & Structures Inc.: Berkeley, CA.

European Committee for Standardization (CEN), [2009] "Eurocode 5 – Design of timber structures – Part 1-1: General rules and rules for buildings", Brussels, Belgium.

European Committee for Standardization (CEN), [2004] "Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings". Brussels, Belgium.

European Committee for Standardization (CEN), [2001] "EN12512— Timber structures – Test methods – Cyclic testing of joints made with mechanical fasteners". Brussels, Belgium.

Follesa M., Ni C., Popovski M., Karacabeyli E. [2010] "Blind prediction of the seismic response of the neeswood capstone building", Proceedings of the World Conference on Timber Engineering, Riva del Garda, Italy, 2010.

Folz B., Filiatrault A. [2004] "Blind predictions of the seismic response of a woodframe house: an international benchmark study". Earthquake spectra, 20(3):825-851, 2004.

Gavric I. [2013a] “IVALSA X-Lam Connection Tests Report”. Research report, CNR-IVALSA, San Michele all’Adige (TN), Italy.

Gavric I. [2013b] “IVALSA X-Lam Wall Tests Report”. Research report, CNR-IVALSA, San Michele all’Adige (TN), Italy.

Guo S., He M., Ni C. [2014] “Seismic analysis of hybrid multi-story light wood frames in China”. Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Karacebeyli E., Ceccotti A. [1996] “Test results on the lateral resistance of nailed shearwalls”. Proceedings of the International Wood Engineering Conference (IWEC), New Orleans, USA, 1996. Vol. 2, pp. 179-186.

Karacebeyli E., Ceccotti A. [1998] “Nailed wood-frame shearwalls for seismic loads, test results and design considerations”. Proceedings of the Structural Engineers World Congress (SEWC), San Francisco, USA, 1996. Paper Reference T207-6.

Ministero delle Infrastrutture [2008] “Decreto del Ministero delle Infrastrutture 14/01/08 - Norme tecniche per le costruzioni".

NRC [2010] “National Building Code of Canada 2010”. Canadian Commission on Building and Fire Code, National Research Council of Canada, Ottawa, Ont.

Pang W., Ziaei E., Jennings E., Shao X., van De Lindt J.W., Gershfeld M, Pryor S. [2014] “Analytical formulation based on extensive numerical simulations of behavior factor q for CLT buildings”. Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Pozza L., Scotta R., Trutalli D, Ceccotti A., Polastri A. [2013] “A non-linear numerical model for the assessment of the seismic behaviour and ductility factor of X-Lam timber structures”. Proceedings of the International Symposium on Timber Structures, Istanbul, Turkey, 2009, pp 151–162.

Pozza L., Scotta R., Vitaliani R. [2009] “A non-linear numerical model for the assessment of the seismic behaviour and ductility factor of X-Lam timber structures”. Proceedings of the International Symposium on Timber Structures, Istanbul, Turkey, 2009, pp 151–162.

Prakash V., Powell G. H. [1994] “DRAIN-3DX: Base program description and user guide.” Report No. UCB/SEMM-94/08. Department of Civil Engineering, University of California, Berkeley, August 1994.

Rinaldin G., Amadio C., Fragiacomo M. [2013a] “A component approach for the hysteretic behaviour of connections in cross-laminated wooden structures.” Earthquake Engineering and Structural Dynamics, Vol. 42 No. 13, pp. 1885–2042, doi: 10.1002/eqe.2310.

Rinaldin G., Poh'sie G.H., Amadio C., Fragiacomo M. [2013b] "Modelling of seismic behaviour of light-frame timber structures." Ingegneria Sismica, Special Issue on Timber Structures, No. 4/2013, pp. 82-98.

Rinaldin G., Poh'sie G.H., Fragiacomo M. Amadio C., Pontarin F. [2014] "Non-linear modelling of the three and seven storey x-lam buildings tested within the Sofie project." Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

Tecnisoft s.a.s., [2014]. "ModeSt – Version 8.6", Prato, Italy.

Van de Lindt J.W., Bahmani P., Mochizuki G., Pryor S. E., Gershfeld M., Tian J., Michael D. Symans M. D., Rammen D. [2014] "Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Wood-Frame Building with Retrofits. II: Shake Table Test Results." Journal of Structural Engineering, E4014004, doi: 10.1061/(ASCE)ST.1943-541X.0001206.

Vassallo D., Christovasilis I. P., Follesa M., Fragiacomo M. [2013], "Design of a four-story Cross Laminated Timber Building in Northern Italy" –, Wood Design Focus Volume 23 Issue 4, 2013.

Zhou L., Chui Y.H., Ni C. [2014] "Seismic performance of mid-rise light wood frame building connected to a stiff core". Proceedings of 13th World Conference on Timber Engineering (WCTE), Quebec City, Canada, 2014.

5 – Proposal for a new background document of the Timber Part of Eurocode 8

5.1 Introduction

In this Chapter a proposal for a Background Document containing possible changes to Chapter 8 “Specific rules for timber buildings” of ENV1998-1:2004 (Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings – December 2004) is presented based on current literature, considerations and research outcomes discussed in the preceding chapters of this thesis.

In the following paragraph §5.2 the current code text is reported in italic, together with the proposed changes which includes the removal or the modification of existing parts as well as the introduction of new parts. The removed parts of the text code will be marked with a strikethrough line and the added parts will be highlighted in grey.

All the modifications to the code test will be referenced and the motivation and scientific background behind each proposal will be given in §5.3, together with the related references. Simple editorial changes will not be referenced.

5.2 Proposal of revision of Chapter 8 “Specific rules for timber buildings” of ENV1998-1:2004

8 SPECIFIC RULES FOR TIMBER BUILDINGS

8.1 General

8.1.1 Scope

(1)P For the design of timber buildings EN 1995 applies. The following rules are additional to those given in EN 1995.

8.1.2 Definitions

(1)P The following terms are used in this section with the following meanings:

Static ductility

ratio between the ultimate deformation and the deformation at the end of elastic behaviour, calculated according to EN12512, evaluated in quasi-static cyclic tests (see 8.3.2(3)P); ***[a]***

Semi-rigid joints

joints with significant flexibility, the influence of which has to be taken into account in structural analysis in accordance with EN 1995 (e.g. dowel-type joints);

Rigid joints

joints with negligible flexibility in accordance with EN 1995 (e.g. glued solid timber joints);

Dowel-type joints

joints with dowel-type mechanical fasteners (nails, staples, screws, dowels, bolts etc.) loaded perpendicular to their axis;

~~Carpenter~~ Carpentry joints

joints, where loads are transferred by means of ~~pressure~~ compression areas and without mechanical fasteners (e.g. skew notch, tenon, half joint).

8.1.3 Design concepts

(1)P Earthquake-resistant timber buildings shall be designed in accordance with one of the following concepts:

a) High- or Medium-dissipative structural behaviour;

b) Low-dissipative structural behaviour.

(2) In concept a) the capability of parts of the structure (dissipative zones) to resist earthquake actions out of their elastic range is taken into account. When using the design spectrum defined in 3.2.2.5, the behaviour factor q may be taken as being greater than 1,5. The value of q depends on the ductility class (see 8.3).

(3)P Structures designed in accordance with concept a) shall belong to structural ductility classes M or H. A structure belonging to a given ductility class shall meet specific requirements in one or more of the following aspects: structural type, type and ~~rotational~~ ductility capacity of connections.

(4)P Dissipative zones shall be located in joints and connections, whereas the timber members themselves shall be regarded as behaving elastically. The energy dissipation is provided by plasticization of metal

fasteners, by embedment of timber at the interface with the fasteners, and for some systems also by friction.

(5) The properties of dissipative zones should be determined by tests either on single joints, on whole structures or on parts thereof in accordance with ~~pr~~EN 12512.

(6)P As an alternative to the design concept provided in (4)P, dissipative zones could be located outside of joints and connections in purposely developed energy dissipaters (e.g. lead extruded or hydraulic dampers, dog-bone steel plates, etc.). In this case, both the timber members and the joints and connections shall be regarded as behaving elastically. In order to ensure the correct behaviour of the energy dissipaters, their connections to the timber members should be as stiff as possible. These connections, the other joints and connections between timber members, and all the timber members shall be overdesigned with sufficient over-strength. The appropriate behaviour factor q should not be determined according to Table 8.2 but will depend on the mechanical properties of the energy dissipaters and the geometrical properties of the structure.

(~~6~~7) In concept b) the action effects are calculated on the basis of an elastic global analysis without taking into account non-linear material behaviour. When using the design spectrum defined in 3.2.2.5, the behaviour factor q should not be taken greater than 1,5. The resistance of the members and connections should be calculated in accordance with EN 1995-1~~:2004~~ without any additional requirements. This concept is termed ductility class L (low) and is appropriate only for certain structural types (see Table 8.1). Other structural types, classified in ductility class M (medium) or H (high) may be designed with concept b) provided that the corresponding provisions given in the reference parts of this section for the capacity based design are satisfied. ***[b]***

8.2 Materials and properties of dissipative zones

(1)P The relevant provisions of EN 1995 apply. ~~With respect to the properties of steel elements, EN 1993 applies.~~

(2)P When using the concept of dissipative structural behaviour, the following provisions apply:

a) only materials and mechanical fasteners providing appropriate low cycle fatigue behaviour according for example to EN409 may be used in joints regarded as dissipative zones;
b) timber or wood-based elements and glued joints shall be considered as non-dissipative zones;

c) carpenter joints may only be used, with the provisions given in 8.5.2(1)P, when they can provide sufficient energy dissipation capacity, without presenting risks of brittle failure in shear or tension perpendicular to the grain. The decision on their use shall be based on appropriate test results.

(3) (2)P a) of this subclause is deemed to be satisfied if 8.3(3)P is fulfilled.

(4) For sheathing-material in shear walls and diaphragms of Light-Frame structures, (2)P a) is deemed to be satisfied, if the following conditions are met:

a) particleboard-~~panels~~ sheathing ~~have~~ has a density of at least 650 kg/m^3;
b) plywood-sheathing is at least 9 mm thick;
c) particleboard - and fibreboard-sheathing are at least 13 mm thick;
*d) Oriented Strand Board sheathing (OSB) type2, 3 or 4 according to EN300 is at least 12 mm thick;**[c]***
*e) Gypsum Fibre boards (GF) sheathing according to EN 15283-2 is at least 12 mm thick **[d].***

*(5) For CLT panels according to prEN16351 used in shear walls and diaphragms of solid construction, (2)P a) is deemed to be satisfied if the panels are at least 60mm thick **[e].***

(~~5~~6)P Steel material for connections shall conform to the following conditions:

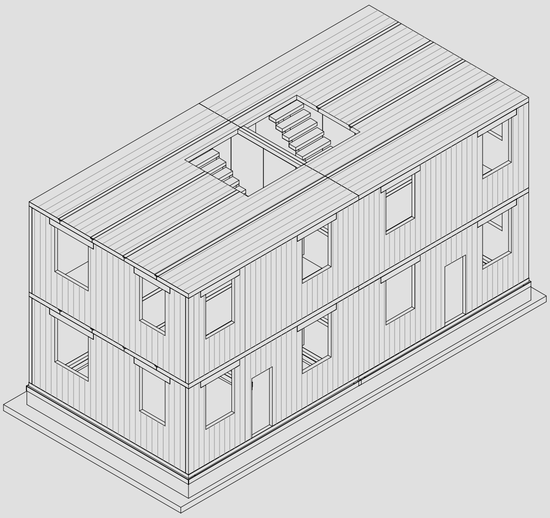
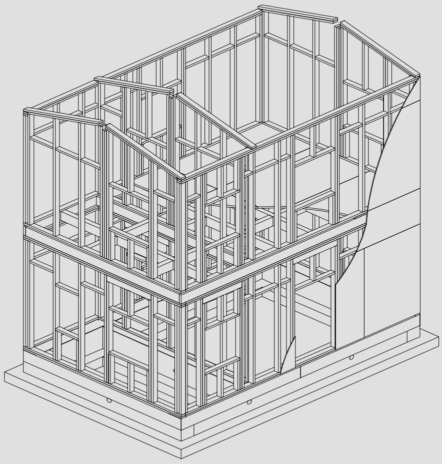
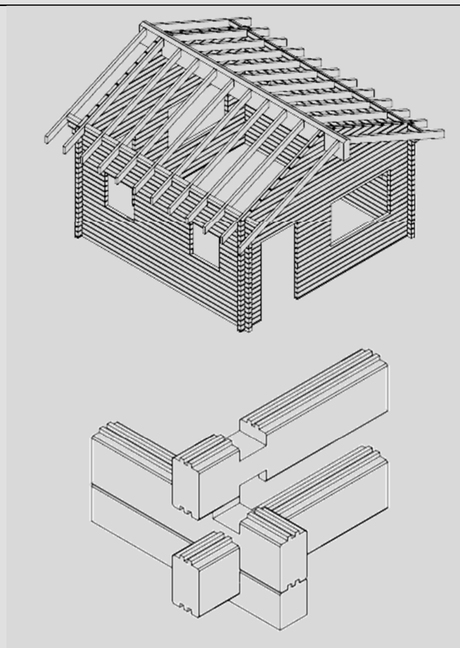
a) ~~all~~ ~~connection~~ steel plate elements ~~made of steel~~ shall fulfil the relevant requirements in EN 1993;
b) steel fasteners shall fulfil the relevant requirements in EN409, with special regard to resistance to low-cycle fatigue for dissipative connections;
~~b~~c) The ductility properties of the dissipative connections ~~in trusses and between the sheathing material and the timber framing~~ in Ductility Class M or H structures (see (8.3)) shall be tested for compliance with 8.3.2(3)P by cyclic tests on the relevant combination of the connected parts and fastener.

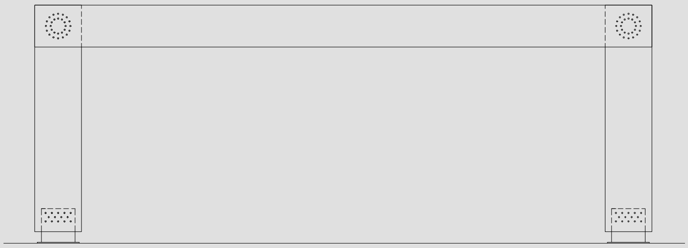
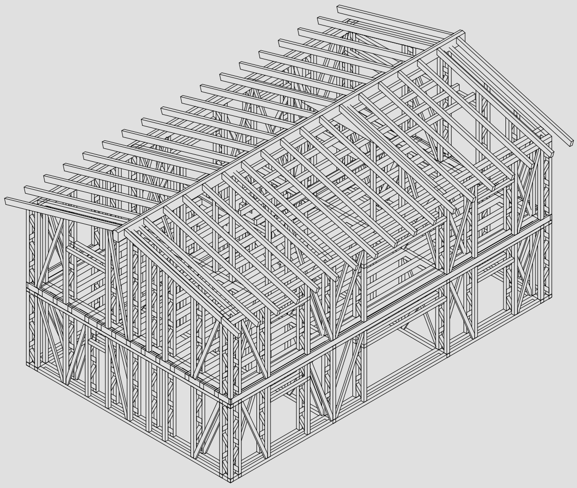
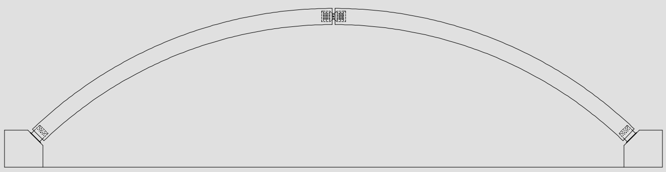
8.3 Structural types, ductility classes and behaviour factors

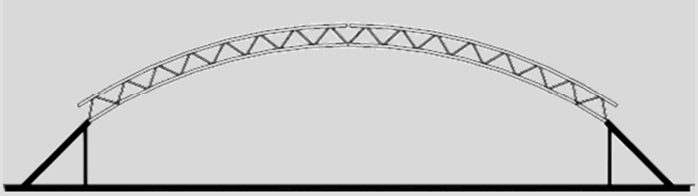
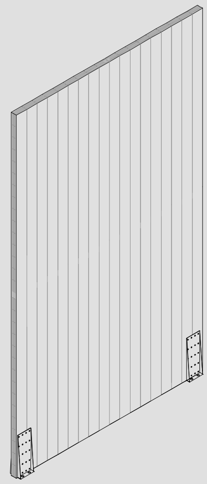
8.3.1 Structural types

(1)P Timber buildings shall be classified into one of the following structural types according to their behaviour under horizontal seismic actions:

Table 8.1: Structural types for timber buildings. [f]

Structural type	**Example**
1. Cross Laminated Timber (CLT) system, i.e. buildings composed of walls and floors made of solid CLT panels according to 8.2.(5) with the specifications given in 8.3.3.1	
2. Light wood-frame system, i.e. structures in which walls, floors and roofs are made of timber frames to which a wood-based or other type of sheathing material according to 8.2.(4) are connected according to the specifications given in 8.3.3.2	
3. Log House building system, i.e. structures in which walls are made by the superposition of rectangular or round solid or glulam timber elements, prefabricated with carpentry joints at their ends and with upper and lower grooves according to specifications given in 8.3.3.3	

4. Moment-resisting frames, i.e. frames composed of timber elements with semi-rigid joints between the members and with the foundation made with mechanical fasteners according to the specifications given in 8.3.3.4	
5. Post and beam timber systems, namely systems of timber columns and beams pinned-connected, with vertical bracings made of trussed frame structures according to the specifications given in 8.3.3.5	
6. Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill.	
7. Large span arches with two or three hinged joints according to the specifications given in 8.3.3.6	

8. Large span trusses with nailed, screwed, doweled and bolted joints according to the specifications given in 8.3.3.6	
9. Vertical cantilever systems made with glulam or CLT wall elements according to the specifications given in 8.3.3.7	

(2)P In principle, all seismic forces in one direction shall be resisted by one system type. If different lateral load resisting systems are used in the same direction, the lower value of the behaviour factor q between the two systems shall be used. In order to use higher values of the behaviour factor q (anyway not higher than the highest value between the two systems), non-linear static (push-over) or non-linear dynamic (time-history) analyses shall be carried out to design the system. **[g]**

(3) Different structural systems not listed above may be used provided that the properties of dissipative zones should be determined by tests either on single joints, on whole structures or on parts thereof in accordance with EN 12512. The appropriate behaviour factor q should be determined based on non-linear dynamic numerical simulations of the structure by implementing the non-linear cyclic behaviour of the dissipative zones obtained from the experimental tests **[h].**

8.3.2 Ductility classes and behaviour factors

(1)P Depending on their ductile behaviour and energy dissipation capacity under seismic actions, and on the capacity design criteria and detailing rules defined in 8.3.3, timber buildings that are regular in elevation shall be assigned to one of the three ductility classes L, M or H as given in Table 8.1, where the corresponding upper limit values of the behaviour factors are also given.

NOTE Geographical limitations on the use of ductility classes M and H may be found in the relevant National Annex.

Table 8.~~1~~2: ~~Design concept,~~ Structural types and upper limit values of the behaviour factors for the three ductility classes for buildings regular in elevation [i].

Structural type	**DCL**	**DCM**	**DCH**
CLT buildings	1,5	2,0	3,0
Light-Frame buildings	1,5	2,5	4,0
Log House buildings	1,5	2,0	-
Moment resisting frames	1,5	2,5	4,0
Post and beam timber buildings	1,5	2,0	-
Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill	1,5	2,0	-
Large span arches with two or three hinged joints	1,5	-	-
Large span trusses with nailed, screwed, doweled and bolted joints	1,5	2,0	-
Vertical cantilever systems made with glulam or CLT wall elements	1,5	2,0	-

~~Design concept and ductility class~~	~~q~~	~~Examples of structures~~
~~High capacity to dissipate energy - DCH~~	~~3,0~~	~~Nailed wall panels with glued diaphragms, connected with nails and bolts; Trusses with nailed joints.~~
	~~4,0~~	~~Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).~~
	~~5,0~~	~~Nailed wall panels with nailed diaphragms, connected with nails and bolts.~~

~~Medium capacity to dissipate energy - DCM~~	~~2,0~~	~~Glued wall panels with glued diaphragms, connected with nails and bolts; Trusses with doweled and bolted joints; Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill.~~
	~~2,5~~	~~Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).~~
~~Low capacity to dissipate energy - DCL~~	~~1,5~~	~~Cantilevers; Beams; Arches with two or three pinned joints; Trusses joined with connectors.~~

(2) If the building is non-regular in elevation (see 4.2.3.3) the q-values listed in Table 8.1 should be reduced by 20%, but need not be taken less than q = 1,5 (see 4.2.3.1(7) and Table 4.1).

~~(3)P In order to ensure that the given values of the behaviour factor may be used, the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class M structures and at a static ductility ratio of 6 for ductility class H structures, without more than a 20% reduction of their resistance.~~

(3)P In order to ensure that the given values of the behaviour factor may be used, the structural sub-assemblies representative of the structural type and incorporating the dissipative zones according to the capacity based design criteria given in 8.3.3 (a shear wall for Light-Frame, CLT, Log House and vertical cantilever system; a portal frame for moment-resisting frame; a single span truss for large trusses) shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class M structures and at a static ductility ratio of 6 for ductility class H structures, without more than a 20% reduction of their resistance between the first and third cycles envelope curve. ***[a].***

(4) The provisions of (3)P of this subclause and of ~~8.2(2) a) and~~ 8.2(6) c) may be regarded as satisfied in the dissipative zones of all structural types classified in ductility class H if the following provisions are met:

a) in doweled, bolted and nailed timber-to-timber and steel-to-timber joints, the minimum thickness of the connected members is 10·d and the fastener-diameter d does not exceed 12 mm;
b) In shear walls and diaphragms of Light-Frame construction, the sheathing material is wood-based with a minimum thickness of 4d, where the nail diameter d does not exceed 3,1 mm.

If the above requirements are not met, but the minimum member thickness of 8d and 3d for case a) and case b), respectively, is assured, the dissipative zones of all structural types can be regarded as ductility class M.~~reduced upper limit values for the behaviour factor q,~~

As an alternative, the above provisions may be regarded as satisfied for the dissipative zones of all ductility class M structural types and of the ductility class H CLT system with segmented wall according to 8.3.3.1.3., if a ductile failure mechanism characterized by the formation of one or two plastic hinges in the mechanical fasteners is attained. Referring to 8.2.2 of EN 1995-1-1 for timber-to-timber and panel-to-timber connections, failure modes a, b and c for fasteners in single shear, and g and h for fasteners in double shear should be avoided. Referring to 8.2.3 of EN 1995-1-1 for steel-to-timber connections, failure modes a, c for fasteners in single shear, and f, j and l for fasteners in double shear should be avoided. Special care should be taken in avoiding brittle failures characterized by splitting, shear plug, tear out and tensile fracture of wood in the connection regions.

~~Table 8.2: Structural types and reduced upper limits of behaviour factors~~

~~Structural types~~	~~Behaviour factor *q*~~
~~Hyperstatic portal frames with doweled and bolted joints~~	~~2,5~~
~~Nailed wall panels with nailed diaphragms~~	~~4,0~~

(5) For structures having different and independent properties in the two horizontal directions, the q factors to be used for the calculation of the seismic action effects in each main direction should correspond to the properties of the structural system in that direction and can be different.

8.3.3 Capacity design rules *[k]*

(1) In the seismic design, the resistance of shear walls shall be higher at lower storeys and shall decrease at higher storeys proportionally to the decrease of the storey seismic shear.

(2)P In order to achieve the attainment of the capacity design rules defined in the following subclauses for all the structural types listed in Table 8.1, the design strength of the brittle parts $F_{Rd,b}$ should be greater than or equal to the design strength of the ductile parts $F_{Rd,d}$ multiplied by an overstrength factor γ_{Rd} according to the following equation:

$$\gamma_{Rd} \cdot F_{Rd,d} \leq F_{Rd,b} \quad (1)$$

where the values of γ_{Rd} are given in Table 8.3.

Table 8.3: Values of the over-strength factor γ_{Rd} for DCM and DCH.

Structural type	**DCM**	**DCH**
CLT buildings	1,3	1,6
Light-Frame buildings	1,3	1,6
Log House buildings	1,3	-
Moment resisting frames	1,3	1,6
Post and beam timber buildings	1,6	-
Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill	1,3	-
Large span arches with two or three hinged joints	-	-
Large span trusses with nailed, screwed, doweled and bolted joints	1,6	-
Vertical cantilever systems made with glulam or CLT wall elements	1,6	-

8.3.3.1 Capacity design rules for CLT buildings

8.3.3.1.1 General

(1) Cross laminated (CLT) timber buildings are structures in which walls and floors are composed of cross laminated timber panels according to 8.2(5).

(2) The connection of the walls to the foundation shall be made by means of mechanical fasteners (hold-down anchors, steel brackets, anchoring bolts, nails and screws) and shall adequately restrain the wall against uplift and sliding. Uplift connections should be placed at wall ends and at opening ends, while sliding connections should be distributed uniformly along the wall length (Figure 8-1).

(3) Walls shall have heights equal to the inter-storey height and may be made of a unique element up to the maximum transportable length or may be composed of more than one panel, of widths not lower than 1.2m, connected together by means of vertical joints made with mechanical fasteners such as screws or nails (segmented wall). Perpendicular walls are connected by means of joints made with mechanical fasteners (usually screws).

(4) Horizontal diaphragms are made of CLT timber panels connected together by means of horizontal joints made with mechanical fasteners (screws or nails). The floor panels bear on the wall panels and on timber beams if present, and are connected with mechanical fasteners (screws or nails).

(5) Other types of horizontal diaphragms may be used, provided that their in-plane rigidity is ensured by means of sheathing wood-based panels according to 8.2(4) a-b-c-d. Timber-concrete composite floors may

be used provided that they are adequately connected to the lower and upper walls by means of mechanical fasteners. The concrete topping, in particular, shall be connected to the vertical panels to ensure the in-plane shear due to the diaphragm action is transferred to the walls and down to the foundations.

(6) The upper walls will bear on the floor panels (platform construction), and will be connected to the lower walls using mechanical fasteners similar to those used for the wall-foundation connection. Tie-down connections nailed to the CLT walls may be used for the external walls uplift restraint.

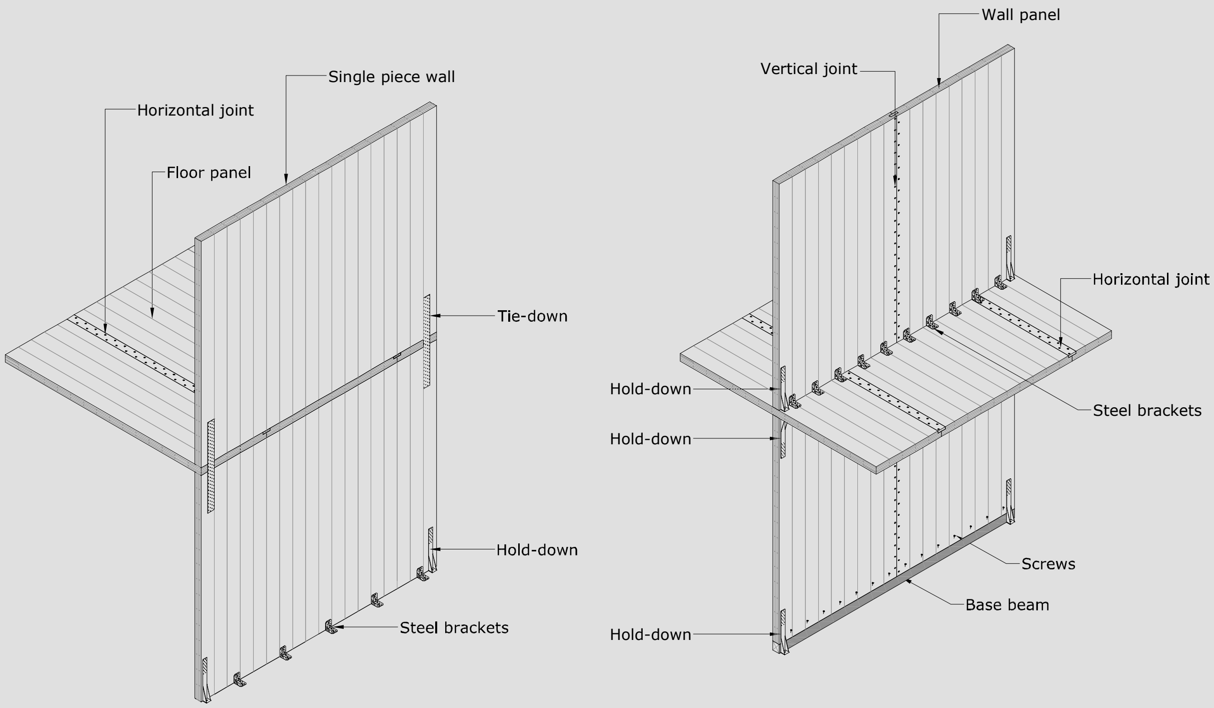


Figure 8-1: Walls and floors in monolithic, left and segmented, right Cross Laminated Timber buildings.

8.3.3.1.2 Capacity design rules for DCM

(1) In CLT buildings designed for DCM, walls may be composed by either only one CLT panel or by more than one panel connected with rigid vertical joints.

8.3.3.1.2.1 Capacity design rules at building level

(1)P CLT buildings should be regarded as box-type structures. In order to achieve this behaviour some structural components should be considered as dissipative zones and some other should be considered as non-dissipative and properly designed with sufficient overstrength to avoid any possible brittle failure mechanism. The structural elements which should be over-designed with sufficient overstrength in order to ensure the development of cyclic yielding in the dissipative zones shall be (Figure 8-2):

- *all CLT wall and floor panels;*

- *connections between adjacent floor panels (or connection of other type of sheathing material like in 8.3.3.1.1(5)) in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;*
- *connections between floors and walls underneath thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;*
- *connections between perpendicular walls, particularly at the building corners, so that the stability of the walls themselves and of the structural box is always ensured.*

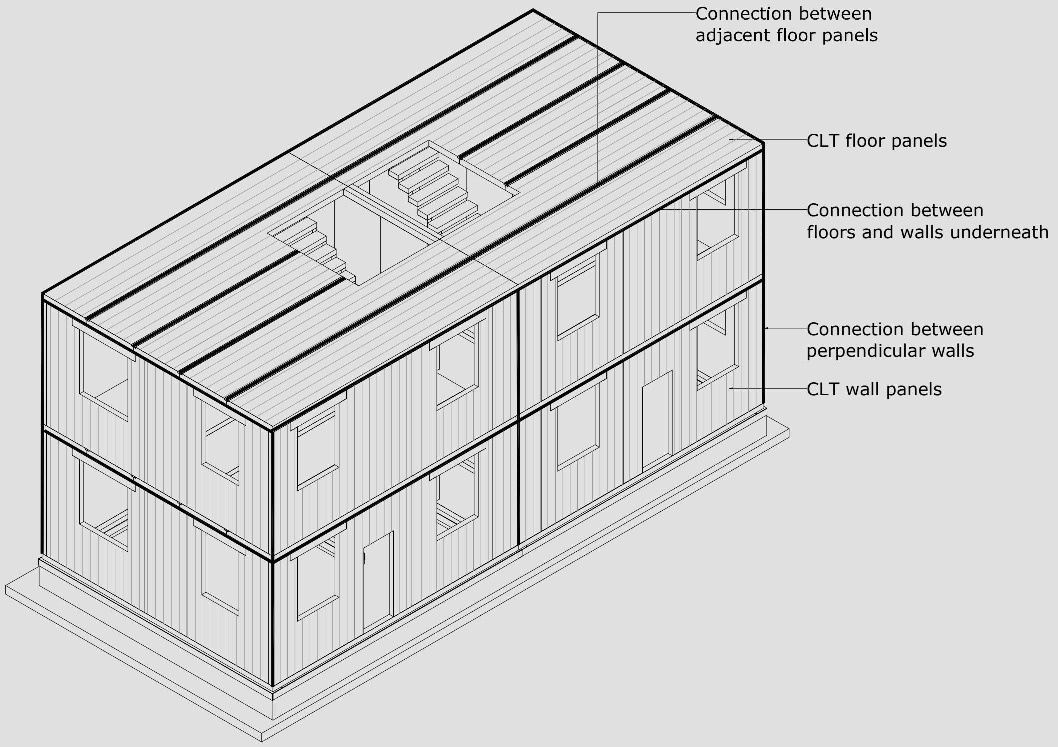


Figure 8-2: Connections to be designed with overstrength criteria in order to fulfil the capacity design criteria in Cross Laminated Timber buildings in DCM.

(2)P The connections devoted to the dissipative behaviour in a CLT building shall be:

- *connections against sliding between walls and floor underneath, and between walls and foundation (usually steel brackets or screwed connections);*
- *anchoring connections against uplift placed at wall ends and at wall openings (usually hold-down anchors).*

8.3.3.1.2.2 Capacity design rules at connection level

(1)P When designing connections as defined in 8.3.3.1.2.2(P) a ductile failure mode characterized by yielding of fasteners (nails or screws) in steel-to-timber or timber-to-timber connections should be achieved and

brittle failure mechanisms should be avoided. In order to achieve this behaviour, parts of the connection should be designed with sufficient overstrength to avoid any possible brittle failure mechanism. These overdesign requirements apply to:

- *tensile and pull through strength of anchor bolts;*
- *steel plate resistance in the weaker section of hold-down and angle brackets connections.*

Other brittle failures such as splitting, shear plug, tear out and tensile fracture of wood in the connection regions should be always avoided.

8.3.3.1.3 Capacity design rules for DCH

(1) In CLT buildings designed for DCH wall shall be composed of more than one CLT panel or by more than one panel of widths not lower than 1.2m and not greater than 2.6m connected joints made with mechanical fasteners (screws or nails).

8.3.3.1.3.1 Capacity design rules at building level

(1)P The same provisions of 8.3.3.1.2.1(P) apply.
(2)P In addition to the provisions of 8.3.3.1.2.2(P) the following connections should be considered and designed for dissipative behaviour:

- *vertical screwed or nailed step joints between parallel wall panels in case of walls composed of multiple CLT panels;*

8.3.3.1.3.2 Capacity design rules at connection level

(1)P The provisions of 8.3.3.1.2.2 apply.

8.3.3.2 Capacity design rules for Light-Frame buildings

8.3.3.2.1 General

(1) Light-frame buildings are structures in which walls, floors and roofs are made of timber frames to which a wood-based sheathing material (plywood or OSB) or other type of sheathing material according to 8.2.4(P) is connected by means of screws, nails or staples (Figure 8-3).

(2) Shear walls are composed of a top and bottom plate and equally spaced vertical studs which the sheathing material is connected to on one or both sides.

(3) Horizontal diaphragms are composed of equally spaced beams or joists and timber bridging in between, usually spaced at the same distance of wall studs, on top of which the sheathing material is connected by means of screws, nails or staples. At each floor, a perimeter edge beam should be provided to resist the tension forces which arise from the diaphragm action when the floor is loaded by horizontal forces acting in its plane.

(4) The connection of the walls to the foundation should be made by means of mechanical fasteners (steel brackets, anchor bolts, nails and screws) and should adequately restrain the wall against overturning and sliding. Overturning connections should be placed at wall ends and at opening ends, while sliding connections should be distributed uniformly along the wall length.

(5) Walls have heights equal to the inter-storey height. Perpendicular walls are connected by joining together two vertical studs with mechanical fasteners (usually nails or screws).

(6) Other types of horizontal diaphragms may be used, such as cross laminated timber floors, provided that their in-plane rigidity is assured. Timber-concrete composite floors may be used provided that they are adequately connected to the lower and upper walls by means of mechanical fasteners. The concrete topping, in particular, shall be connected to the vertical panels to ensure the in-plane shear due to the diaphragm action is transferred to the walls and down to the foundations.

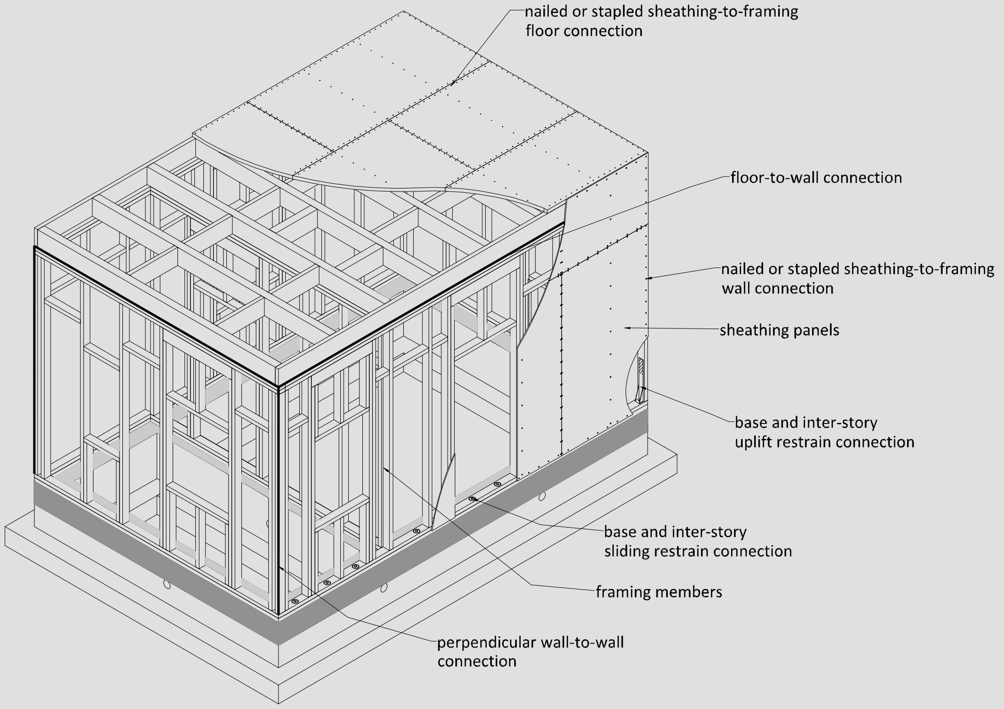


Figure 8-3: Walls and floors in Light-Frame timber buildings and examples of typical connections.

8.3.3.2.2 Capacity design rules for DCM

(1) In Light-Frame buildings designed for DCM, shear walls may be sheathed with all the types of sheathing material defined in 8.2.4(P) except for CLT panels, connected to the wall framing by means of screws, nails or staples.

8.3.3.2.2.1 Capacity design rules at building level

(1) Light-frame buildings shall act at the greater possible extent as box-type structures. To achieve this, it is important to ensure that local failures which may compromise the box-type behaviour will not occur.

(2) The connections devoted to the dissipative behaviour in a light-frame building are:

- *screwed, nailed or stapled connection between sheathing material and timber frame in shear walls;*
- *shear connections between upper and lower walls, and between walls and foundation;*
- *connections against uplift placed at wall ends and at wall openings.*

(3) In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength so as to avoid anticipated brittle failure. This overstrength requirement applies especially to:

- *nailed connections between sheathing and timber joists/beams at each floor;*
- *connection between floors and walls underneath thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;*
- *connection between perpendicular walls, particularly at the building corners, so that the stability of the walls itself and of the structural box is always ensured;*
- *sheathing panels under in-plane shear induced by seismic actions;*
- *timber framing members (studs, plates, and joists) under axial forces induced by seismic actions.*

8.3.3.2.2.2 Capacity design rules at connection level

(1)P The provisions of 8.3.3.1.2.2 apply.

8.3.3.2.3 Capacity design rules for DCH

(1) In Light-Frame buildings designed for DCH shear walls only plywood or OSB panels as defined in 8.2.4(P) shall be used as sheathing material connected to the wall framing only by means of nails (stapled connections are not allowed).

8.3.3.2.3.1 Capacity design rules at building level

(1) Provision 8.3.3.2.2.1(1) applies.

(2) The connections devoted to the dissipative behaviour in a light-frame building are nailed or stapled connection between sheathing material and timber frame in shear walls.

(3) In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength so as to avoid anticipated brittle failure. This overstrength requirement applies especially to:

- *nailed connections between sheathing and timber joists/beams at each floor;*
- *shear connections between upper and lower walls, and between walls and foundation;*
- *connections against uplift placed at wall ends and at wall openings;*
- *connection between floors and underneath walls thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;*
- *connection between perpendicular walls, particularly at the building corners, so that the stability of the walls itself and of the structural box is always ensured;*
- *sheathing panels under in-plane shear induced by seismic actions;*
- *timber framing members (studs, plates, and joists) under axial forces induced by seismic actions.*

8.3.3.2.3.2 Capacity design rules at connection level

(1)P The provisions of 8.3.3.1.2.2 apply.

8.3.3.3 Capacity design rules for Log House buildings

8.3.3.3.1 General

(1) Log House buildings are structures in which walls are made by the superposition of rectangular or round solid or glulam timber elements ('logs'), prefabricated with upper and lower grooves in order to ease the overlapping and improve the stability of the wall.

(2) The connection between perpendicular walls is made by means of carpentry joints obtained by notching the logs of the two walls or by means of screws.

(3) Horizontal diaphragms may be made according to 8.3.4.1(3).

(4) The connection of the walls to the foundation should be made by means of mechanical fasteners (tie rods, anchor bolts, nails and screws) and should adequately restrain the wall against overturning and

sliding. Overturning connections should be placed at wall ends and at door ends, while sliding connections should be distributed uniformly along the wall length.

(5) Special care shall be taken to ensure possible undesired uplift of the logs due to overturning moment is prevented. To this aim, it shall be ensured that the stabilizing moment due to gravity loads is greater than or equal to the overturning moment due to seismic action multiplied by the overstrength factor γ_{Rd}. Alternatively, adequate restrain connections such as steel tie-rods or screws shall be used.

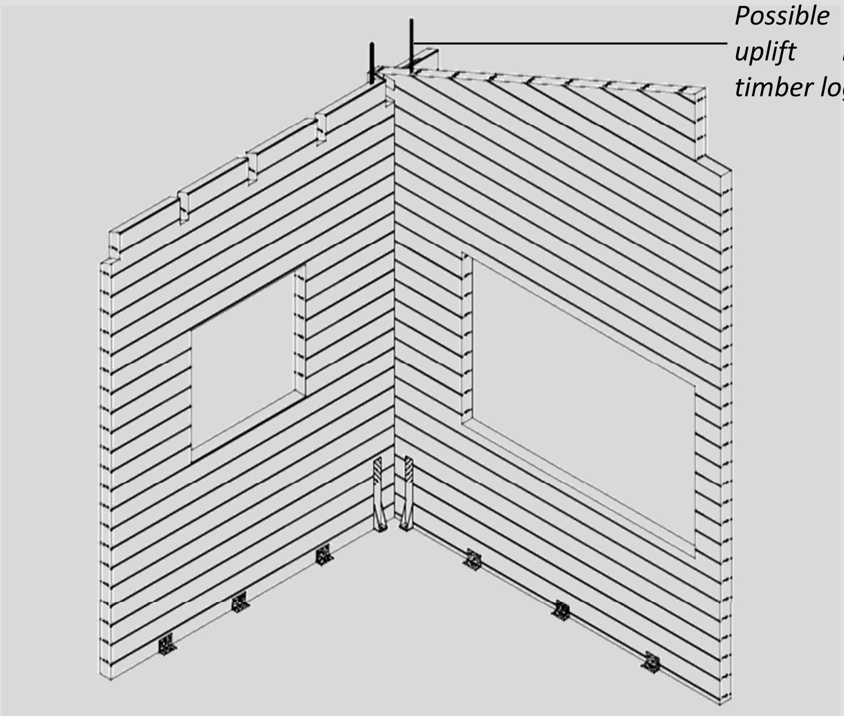


Figure 8-4: Typical connection details in Log House buildings.

8.3.3.3.2 Capacity design rules

8.3.3.3.2.1 Capacity design rules at building level

(1) Log house buildings shall act at the greater possible extent as box-type structures. To achieve this it is important to ensure that local failures which may compromise the box-type behaviour will not occur. Energy dissipation takes place at the interface between the logs due to friction, and in the carpentry joints between perpendicular walls due to compression perpendicular to grain. In order to ensure the development of the energy dissipation in the dissipative zones, the brittle failure mechanisms such as shear in the carpentry joints should be prevented with sufficient overstrength. All timber logs and all the connections to the foundation or between any massive sub-element should be also designed with sufficient overstrength.

8.3.3.3.2.2 Capacity design rules at connection level

(1)P The provisions of 8.3.3.1.2.2 apply.

8.3.3.4 Capacity design rules for moment resisting frames

8.3.3.4.1 General

(1) Moment-resisting frames are frames composed of timber elements with semi-rigid joints between all members made with mechanical fasteners composed of steel tubes, dowels, bolts, screws or nails. The column-foundation connections could be pinned or semi-rigid.

8.3.3.4.2 Capacity design rules for DCM

(1) In order to ensure plasticization of the fasteners, the timber members shall be overdesigned with sufficient over-strength.

8.3.3.4.3 Capacity design rules for DCH

(1) Provision given in 8.3.3.4.2(1) applies. In addition, only moment-resisting frames with high-ductility joints, i.e. a special system which incorporate beam-column joints specifically designed to attain high ductility behaviour are allowed for design according to DCH. An example is the use of densified veneer wood reinforced joints with expanded tube fasteners. The upper limit value of the behaviour factor listed in Table 8.2 can be used only if provision of 8.3.2(3)P for ductility class H structures is satisfied for a typical portal frame subassembly.

8.3.3.5 Capacity design rules for post and beam buildings

8.3.3.5.1 General

(1) Post and beam timber buildings are buildings composed by timber column and beams pinned-connected with mechanical joints made with dowel-type fasteners and with vertical bracings made of trussed frame structures with mechanical fasteners in the joints of the frame and/or the connections of the bracing elements.

(2) Other types of vertical bracings such as Light-Frame shear walls or CLT walls may be used. In this case the corresponding provisions for DCM apply.

(3) Horizontal diaphragms may be made according to 8.3.3.2.1(3) and 8.3.3.2.1(6).

8.3.3.5.1 Capacity design rules

(1) In order to ensure plasticization of the fasteners, the timber members shall be overdesigned with sufficient over-strength.

8.3.3.6 Capacity design rules for large span arches or trusses

(1) In order to ensure plasticization of the fasteners, the timber members shall be overdesigned with sufficient over-strength.

(2) Only large span trusses connected with nailed, screwed, doweled and bolted joints are allowed for DCM provided that the provisions given in 8.3.2(4) are fulfilled.

8.3.3.7 Capacity design rules for vertical cantilever systems

8.3.3.7.1 General

(1) Vertical cantilever timber buildings are buildings composed by walls made with CLT of glulam elements connected to the foundation and floors by means of mechanical connectors (usually metal plate connectors) which shall adequately restrain the wall against uplift and sliding. Unlike CLT buildings, the walls are continuous across the floors, and are designed as a vertical cantilever subjected to the seismic actions.

(3) Horizontal diaphragms may be made according to 8.3.3.2.1(3) and 8.3.3.2.1(6).

8.3.3.7.2 Capacity design rules

(1) In order to ensure plasticization of the fasteners, the timber walls and the metal plates used in the connections shall be overdesigned with sufficient over-strength.

8.4 Structural analysis

(1)P In the analysis the slip in the joints of the structure shall be taken into account.

~~*(2)P An E0-modulus-value for instantaneous loading (10% higher than the short term one) shall be used.*~~

(2) 3D numerical finite element models used for the linear and non-linear analysis of timber buildings should represent the mechanical connections used in the real construction, modelled with the correct stiffness calculated according to EN 1995-1-1. Numerical models which include only timber elements (columns, beams or walls) with rigid connections are not allowed since they may lead to non-conservative results, both in the estimation of the natural period and of the deformation characteristics of the building.

(3) Floor diaphragms may be considered as rigid in the structural model without further verification, if both of the following conditions are met:

a) the detailing rules for horizontal diaphragms given in the relevant sections of 8.3 are applied;
and
b) their openings do not significantly affect the overall in-plane rigidity of the floors.

8.5 Detailing rules

8.5.1 General

(1)P The detailing rules given in 8.5.2 and 8.5.3 apply for earthquake-resistant parts of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M and H).

(2)P Structures with dissipative zones shall be designed so that these zones are located mainly in those parts of the structure where yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

8.5.2 Detailing rules for connections

(1)P Compression members and their connections (e.g. carpenter joints), which may fail due to deformations caused by load reversals, shall be designed in such a way that they are prevented from separating and remain in their original position.

(2)P Bolts and dowels shall be tightened and tight fitted in the holes. Large bolts and dowels ~~(d > 16 mm)~~ shall not be used in timber-to-timber and steel-to-timber connections if the provisions given in 8.2(4) are not satisfied, except in combination with timber connectors.

*(3) Dowels, smooth nails and staples should not be used without additional provision against withdrawal. They are however admissible in diaphragms for the connection of sheathings to the timber framing (see 8.3.2.(4)) and in secondary members. **[I].***

(4) In the case of tension perpendicular to the grain, additional provisions should be met to avoid splitting (e.g. nailed metal or plywood plates).

8.5.3 Detailing rules for Light-Frame horizontal diaphragms

(1)P For horizontal diaphragms under seismic actions EN 1995-1-1~~:2004~~ applies with the following modifications:

a) the increasing factor 1,2 for resistance of fasteners at sheet edges shall not be used;

b) when the sheets are staggered, the increasing factor of 1,5 for the nail spacing along the discontinuous panel edges shall not be used;

c) the distribution of the shear forces in the diaphragms shall be evaluated by taking into account the in-plan position of the lateral load resisting vertical elements.

(2)P All sheathing edges not meeting on framing members shall be supported on and connected to transverse blocking placed between the wooden beams. Blocking shall also be provided in the horizontal diaphragms above the lateral load resisting vertical elements (e.g. walls) ***[m].***

(3)P The continuity of beams shall be ensured, including the trimmer joists in areas where the diaphragm is disturbed by holes.

(4)P Without intermediate transverse blocking over the full height of the beams, the height-to-width ratio (h/b) of the timber beams should be less than 4.

(5)P If $a_g.S \geq 0,2g$ the spacing of fasteners in areas of discontinuity shall be reduced by 25%, but not to less than the minimum spacing given in EN 1995-1~~:2004~~.

(6)P When floors are considered as rigid in plan for structural analysis, there shall be no change of span-direction of the beams over supports, where horizontal forces are transferred to vertical elements (e.g. shear-walls) ***[m].**.*

8.6 Safety verifications

(1)P The strength values of the timber material shall be determined taking into account the k_{mod}-values for instantaneous loading in accordance with EN 1995-1-1:2004.

(2)P For ultimate limit state verifications of structures designed in accordance with the concept of low-dissipative structural behaviour (Ductility class L), the partial factors for material properties γ_M for ~~fundamental~~ accidental load combinations from EN 1995 apply.

(3)P For ultimate limit state verifications of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M or H), the partial factors for material properties γ_M for ~~accidental~~ fundamental load combinations from EN 1995 apply. Alternatively, if the strength degradation due to cyclic loading is appropriately accounted for in the evaluation of the connection resistance, the partial factors for material properties γ_M for accidental load combinations from EN 1995 apply. A strength degradation of 20% shall be considered if no experimental values are available. This provision applies only

to the dissipative zones of each structural type according to the capacity based design criteria given in 8.3.3. Other structural elements shall be designed according to 8.6(2)P ***[n].***

(4)P In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength.

~~*This overstrength requirement applies especially to:*~~

- ~~*anchor-ties and any connections to massive sub-elements;*~~
- ~~*connections between horizontal diaphragms and lateral load resisting vertical elements.*~~

(5) Carpenter joints do not present risks of brittle failure if the verification of the shear stress in accordance with EN 1995 is made with an additional partial factor of 1,3.

(6) In order to meet the requirements for the Damage Limit State under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement" in accordance with 2.1(1)P and 3.2.1(3), the following limits applies:

a) for timber buildings having non-structural elements of brittle materials attached to the structure (e.g. Autoclaved Aerated Concrete or brick partition walls):

$d_r \nu \leq 0.005h$;

b) for timber buildings having ductile non-structural elements (e.g. gypsum plasterboard):

$d_r \nu \leq 0.010h$;

where:

d_r *is the design interstorey drift as defined in 4.4.2.2(2);*

h *is the storey height;*

ν *is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.* ***[o].***

8.7 Control of design and construction

(1)P The provisions given in EN 1995 apply.

(2)P The following structural elements shall be identified on the design drawings and specifications for their special control during construction shall be provided:

- *anchor-ties and any connections to foundation elements;*

- *diagonal tension timber or steel trusses used for bracing;*
- *connections between horizontal diaphragms and lateral load resisting vertical elements;*
- *connections between sheathing panels and timber framing in horizontal and vertical diaphragms.*

(3)P The special construction control shall refer to the material properties and the accuracy of execution.

5.3 Explanation and scientific background for the proposed changes

(a) – Definition of static ductility and ductility classification

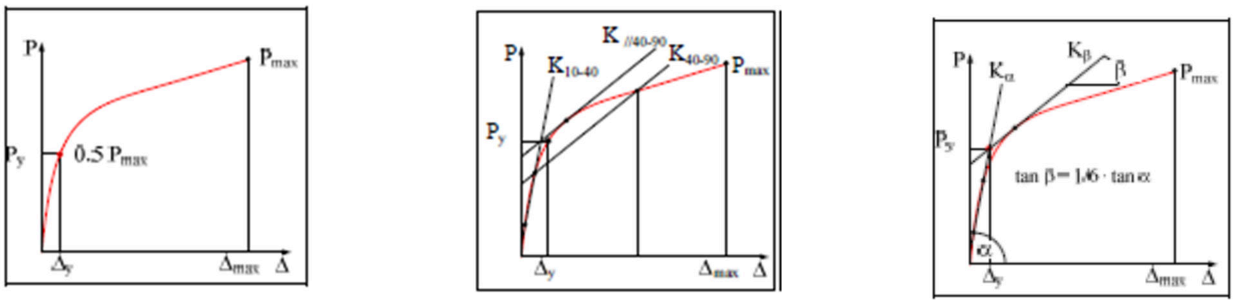
There are a number of recent scientific experiences which demonstrate that the current provisions given within section 8 of Eurocode 8 (CEN, 2004) in order to classify timber structures in Medium and High ductility class are not always met both for Light-Frame (Boudaud et al, 2010; Vogt et al., 2012; Sartori and Tomasi, 2013; Tomasi and Sartori, 2013) and CLT (Gavric et al., 2011, Gavric, 2013a, Gavric 2013b) shear walls and connections both in terms of values of static ductility and/or strength degradation.

In some cases (especially for CLT shear walls or CLT connections) static ductility values lower than 4 (sometimes ranging from 2 to 2,5) are observed and in some other cases, even when static ductility values of 6 are reached, the strength degradation is higher than 20%, in some cases even higher than 50%. For Light-Frame structures in some cases static ductility values lower than 3 and strength degradation due to cyclic loading higher than 30% were found. Therefore, according to that current definition, both Light-Frame and CLT structures should be probably classified in Medium or Low ductility class and consequently the values of the behaviour factors provided should be reduced.

Nevertheless results of both test on full-scale buildings and numerical investigations conducted in recent years demonstrated the good overall dissipative behaviour of both structural systems, justifying the classification of these structural types as Medium and High ductility systems.

Munoz et al. (2008), by comparing six different methods used in the calculations of the yield point and ductility ratio in various types of connections and wall assemblies, demonstrated that differences up to 100% can be found in the calculations of the ductility ratio. While there is an international agreement about the definition of the ultimate displacement (defined as the displacement corresponding to 80% of the maximum load in the descending portion of the 1st cycle backbone curve in a cyclic test), different methods are proposed for the evaluation of the yield displacement of mechanical joints in timber structures and of the loading protocol for cyclic testing. Out of these, two methods are taken into account for comparing the calculated values of yield displacements and static ductility with the procedure suggested in EN12512 (CEN, 2001). Karacabeyli and Ceccotti (1996) proposed a simple method to calculate the yield point, derived as the point in the 1st cycle backbone curve corresponding to 50% of the maximum capacity. Yasumura and Kawai (1998) calculate the yield point as the displacement corresponding to the horizontal projection in the 1st cycle backbone curve of the intersection of the initial stiffness, calculated as the line between the 10% and 40% of the maximum capacity and the line tangent to the 1st cycle backbone curve and parallel to the secant between 40% and 90% of the maximum capacity (see Figure 5-1).

According to Munoz et al. (1998) the method proposed by Karacabeyli and Ceccotti presented a highly dependent load-deformation curve shape approaches which for Light-Frame shear walls produced realistic values. However the same does not apply to single connections with high initial stiffness. The Yasumura and Kawai method produced reasonable estimates of the yield points for all the evaluated systems irrespective of the load-deformation curve shape and yields generally lower values of the yield displacement, at least for Light-Frame walls and consequently higher values of the static ductility with respect to the EN12512 procedure.



Karacabeyli and Ceccotti (1996) Yasumura and Kawai (1998) EN12512 (2001)

Figure 5-1: Three different methods proposed for the evaluation of the yield point: Karacabeyli and Ceccotti (1996), Yasumura and Kawai (1998) and the EN12512 method (after Munoz et al., 2008).

Table 5-2 show a comparison between the three methods for five tests, two of which on Light-Frame walls and three on CLT walls, in terms of yield displacements, static ductility ratios and strength degradation, calculated according to EN12512 between the first and third cycle envelope curve, at a ductility level of 4 and 6. Details of walls are referenced in Table 5-1. The results of Table 5-2 which do not match the current provisions given in Eurocode 8 for DCM and DCH ductility classes are highlighted in bold letters.

Table 5-1 Properties of Light-Frame and CLT walls considered in Table 5-2.

Type of wall	Id.	Dimensions [m], material and connection types	Vertical load applied [kN/m]
Light-Frame	23-02	4.88x2.44 sheathed with 12.5 mm OSB on one side connected with 3x65 common nails spaced at 150/300 mm	-
Light-Frame	46-02	4.88x2.44 sheathed with 12.5 mm OSB on one side connected with 3x65 common nails spaced at 150/300 mm	18.2
CLT	1-1	2.95x2.95 CLT panel 85mm 5-layers connected with two HTT22 hold-	18.5

		down anchors and 2 BMF 90x48x3x116	
CLT	1-2	2.95x2.95 CLT panel 85mm 5-layers connected with two HTT22 hold-down anchors and 2 BMF 90x48x3x116	18.5
CLT	3-1	2.95x2.95 CLT panel 85mm 5-layers connected with four HTT22 hold-down anchors and 2 BMF 90x48x3x116 and 1 vertical sep joint connected with 8x100 self-tapping screws spaced at 150 mm	18.5

Table 5-2 Comparison between three different methods for the evaluation of yield displacements, static ductility and strength degradation for Light-Frame and CLT walls.

Type of wall	Yield displacement dy [mm]	Static ductility Ds	Strength degradation at Ds=4	Strength degradation at Ds=6	Method
Light-Frame 23-02	7.4	7.3	16.8%	**24.5%**	Yasumura and Kawai
	5.8	9.2	15.4%	**21.3%**	EN 12512
	5.1	10.5	12.0%	18.5%	Karacabeyli and Ceccotti
Light-Frame 46-02	5.6	9.2	16.0%	18.3%	Yasumura and Kawai
	4.3	11.9	11.1%	9.9%	EN 12512
	5.2	9.9	15.6%	18.0%	Karacabeyli and Ceccotti
CLT 1-1	8.8	**4.5**	**28.4%**	-	Yasumura and Kawai
	8.5	**4.6**	**28.0%**	-	EN 12512
	7.4	**5.3**	**25.2%**	-%	Karacabeyli and Ceccotti
CLT 1-2	10.4	**5.4**	**22.9%**	-	Yasumura and Kawai
	15.5	**3.7**	**37.8%**	-	EN 12512
	9.2	6.0	17.4%	**38.6%**	Karacabeyli and Ceccotti
CLT 1-2	11.2	6.8	12.9%	13.3%	Yasumura and Kawai
	12.5	6.3	13.5%	14.5%	EN 12512
	9.9	7.6	12.6%	13.7%	Karacabeyli and Ceccotti

Analysing in detail the results of Table 5- 2 it should be noted that the method proposed by Karacabeyli and Ceccotti yields lower values of yield displacements and consequently higher values of static ductility for most of the walls considered. The Yasumura and Kawai method yields higher yield displacements compared to the EN12512 procedure for Light-Frame walls and lower values for CLT walls. In general however the current provisions given for DCM and DCH ductility classes are not met in one case for Light-Frame walls regarding the strength degradation and in most cases for CLT walls even for DCM ductility class.

According to Ceccotti, 1995, in order to ensure sufficient ductility, it is required that the ductility obtained from cyclic tests should be greater than the assumed behaviour factor q multiplied a factor of 3 for moment-resisting joints. This value is reduced to 2 for Light-Frame shear walls, because of the highly positive effect in reducing inertia forces due to damping caused by friction, and due to compression perpendicular to the grain between parts.

However, regarding moment-resisting frames, Wrzesniak et al., 2013, by performing Incremental Dynamic Analysis (IDA) on an industrial portal frame and a three-story, five-bay frame, found that for a joint ductility of at least 6 and 4, q-factors of respectively 2 and 1.5 could be suggested for the portal frame, while the recommended q-factor values could be 2.5 and 2 for the 3-storey, 5-bay frame, which are markedly different for the q values suggested by the current version of Eurocode 8.

Therefore more research is needed in order to confirm or change the current ductility provisions given in Eurocode 8 for DCM and DCH ductility classes. Moreover the results in terms static ductility are markedly different if the evaluation is made on test performed single connections (giving higher values) or on structural subassemblies (giving lower values), and also for an entire building, depending on the number of storeys. According to Casagrande et al., 2014, the global ductility of a shear wall depends only on the ductility of the weakest component. Hence, the provisions about ductility should be applied only to the weakest connection component whereas an elastic design can be used for other devices Thus the code should specify at which scale the evaluation of the ductility properties of the dissipative zones should be made, if (i) at connection level (single fastener in Light-Frame systems; hold-down/angle bracket and screw connection in CLT systems; beam-column joint in moment-resisting frames) or (ii) at sub-assembly level (e.g. shear walls, portal frames or single span trusses according to the structural type).

A possible better formulation could be proposed based on the following procedure:

1. evaluate the static ductility according to the EN12512 procedure by means of cyclic tests performed on dissipative structural elements (e.g. metal connectors, beam-column joints) or sub-

assemblies (e.g. shear walls, portal frames or single span trusses) representative of the reference structural type;

2. build one or more 3D models (e.g. with different number of storeys) of representative buildings using a the FE program which incorporates a suitable model to represent the hysteretic behaviour of the above referenced ductile structural elements or sub-assemblies under reversed-cyclic loading;
3. perform a suitable set (at least with 20 different earthquakes records) of non-linear dynamic analysis on each building configuration of interest in order to establish the value of the behaviour factor q;
4. establish according to the calculated values of the behaviour factor q the classification of the structural type DCM and DCH ductility classes (e.g. DCM for q values ranging from 1.5 to 2.5 and DCH for q values greater than or equal to 3)
5. evaluate the corresponding demand of static ductility values according to DCM and DCH for the dissipative zones.

As an example, a possible formulation of the above procedure could be summarized in the code text by a Table similar to Table 5- 3.

Table 5- 3 Possible proposal of static ductility values of dissipative zones tested according to EN12512 without more than a 20% reduction of their resistance between the first and third cycles envelope curve for all structural types.

Structural type	**Dissipative sub-assembly/element**	**DCM**	**DCH**
CLT buildings	Shear wall	3,0	4,0
CLT buildings	Hold-downs, angle brackets, screws	8,0*	12,0*
Light-Frame buildings	Shear wall	3,0	5,0
Light-Frame buildings	Fastener (nail/screw)	10,0*	16,0*
Log House buildings	Shear wall	2,0	-
Moment resisting frames	Portal Frame	2,0	3,0
Moment resisting frames	Beam-column joint	6,0	10,0
Post and beam timber buildings	Braced Frame	2,0	-
Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill	Shear wall	2,0	-

Large span trusses with nailed, screwed, doweled and bolted joints	Single span truss	2,0	-
Vertical cantilever systems made with glulam or CLT wall elements	Shear wall	2,0	-

The values proposed in Table 5- 3 are only a possible example. More research is needed for all structural types in order to provide reliable values. The value marked with asterisk (*) have been calculated by multiplying the corresponding behaviour factor q by 4 as recommended by Ceccotti, 1995.

(b) – Design for DCL

The possibility of designing every structural type for DCL is given in the relevant chapters of all other materials in Eurocode 8..

(c) – OSB sheathing

OSB is probably the most used wood-based sheathing material in Light-Frame construction. There are a large number of experimental results about the good dissipation properties of Light-Frame shear walls sheathed with OSB panels (Karacabeyli and Ceccotti, 1998, Ceccotti and Karacabeyli, 2002, Sartori et al., 2013).

(d) – Gypsum Fibre boards (GF) sheathing

Light-Frame buildings sheathed with Gypsum Fibre boards (GF) sheathing and stapled connections are becoming more and more used in the current construction practice. Moreover recent research conducted at the University of Trento, Italy (Sartori and Tomasi, 2013) and within the SERIES Project (Piazza et al., 2013) have proved the suitability of Gypsum Fibre Panels (PF-GF) connected to the timber framing with staples as a sheathing material for shear walls in Light-Frame construction.

(e) – CLT panels

Cross Laminated Timber (CLT) buildings are widely spread all over Europe for the construction of medium to high-rise buildings in seismic areas. Currently the qualification process of the structural panels is made by CE marking each panel according to European Technical Approvals (ETA) which are specific of each single producer and are generally based on the procedure described in CUAP 03.04/06. At present a European Standard for CLT (prEN 16351 Timber structures — Cross Laminated Timber — Requirements) is under formal vote and will be published soon. After the publication, the CE marking of CLT panels will be possible only according to the procedure established in this standard and all the single ETA’s will expire. The limitation of 60 mm of panel thickness is given according to current production specification of most of the European producers.

(f) – Structural types

Table 8.1 is completely new with respect to the current version of Eurocode 8. New structural systems for timber buildings already widely used in seismic regions such as the Cross Laminated Timber (CLT) system and the Log House system were introduced. With respect to the current version, all the structural types referred to structural assemblies for building roofs like trusses with nailed, doweled or bolted joints or with connectors were removed. The reason for this change is that the timber trusses were introduced in the 2004 edition probably overlooking the meaning of timber trusses given in the previous 1995 ENV edition where this system referred to vertical bracing systems used in buildings (even large span glulam roofs, where the timber elements are directly connected to the foundation and resist vertical and horizontal loads). As this chapter refers to lateral load resisting systems in timber building, there is no reason to make reference to structural assemblies used for roofs. The structural type referenced in 2004 edition as “Hyperstatic portal frames” is here referenced with the most common definition of “Moment resisting frames” and two values of the behaviour factor q are given for DCM and DCH. Also the vertical cantilever system is a new structural type not referenced in the 2004 edition which is nevertheless widely used in seismic regions. The graphic description was re-introduced like in the 1995 ENV edition.

(g) – Buildings with different lateral load resisting systems

The possibility of using hybrid construction with different lateral load resisting systems acting at the same level, such as for example mixed Light-Frame/CLT structures, is increasingly sought by architects and structural engineers as it allows sometime the best performance in terms of mechanical, thermal, acoustic and fire resistance properties of multi-storey timber buildings. According to the National Building Code of Canada (NBCC 2010), in case of hybrid structures with combination of different type of seismic force modification factor (R_dR_0 according to NBCC 2010) acting in the same direction and in the same storey, the lowest value of R_dR_0 should be taken. The same provision is suggested herein as a first simplified and conservative approach, with the option of performing non-linear static (push-over) or non-linear dynamic (time-history) analyses to derive the actual higher values of the behaviour factor q (not higher anyway of the highest value between the two systems) to be used in the seismic design. A possible method and a code proposal is given in Chapter 4.

(h) – New structural systems

The possibility of using new structural systems not yet included within the structural types listed in Table 8.1 has been better specified in order to allow the development and use of new structural types for timber buildings.

(i) – Behaviour factors

New values for the behaviour factors were introduced, specifying two different values, if applicable, for DCM and DCH ductility classes. The values given for CLT structures are based on research results and numerical investigations conducted with the Sofie Project for buildings designed according to the capacity design rules given in 8.3.3 and referenced in Ceccotti and Follesa, 2006, Ceccotti et al., 2007, Ceccotti et al., 2013, Pozza et al., 2009, Fragiacomo et al., 2011, Pozza et al., 2013.

For Light-Frame structures two different values of the behavior factor q are given for DCM and DCH. The highest values of 5.0 given in the 2004 edition, even if given also in the National Building Code of Canada (NBCC 2010) is not confirmed by other international codes (e.g. New Zealand, NZS 3603, 1993) and by all the numerical investigations conducted so far (see Chapter 4 as a reference). Therefore a more conservative value of 4.0 is proposed. For the seismic design according to DCM a value of 2.5, given in Campos Costa et al., 2013, is proposed in order to include Light-Frame buildings sheathed with gypsum fibre boards and stapled connections. Unlike the 2004 edition, and according to the provisions given in the previous 1995 ENV edition, no distinction is made between glued and nailed diaphragms.

For Log House buildings no reference could be found on behavior factor proposals; however studies are in progress (Bedon et al. 2014, 2015). Therefore the proposal is to use a value of 2.0 for DCM in order to take into account the dissipative contribution of frictions between logs, anyway further research is needed to confirm this value.

For moment resisting frames the lower q value (2.5) is confirmed for DCM. However a more conservative value of 4.0 is proposed for DCH, according to recent research results based on extensive numerical investigations on moment resisting frames with densified veneer wood reinforced joints with expanded tube fasteners (Wrzesniak et al., 2013).

Large span trusses with nailed, screwed, doweled and bolted joints are proposed with a value of the behaviour factor q of 2.0 for DCM according to the value proposed in 2004 edition. No values are proposed for DCH, as the value of 3.0 proposed in 2004 edition only for nailed wood trusses do not find proper justifications in literature.

Finally for mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill the proposed value of 2.0 was confirmed and large span glulam arches and vertical cantilevers were considered as Low Ductility class structural types, according to 2004 edition (where only glulam arches were referenced).

(j) – Ductility rules for dissipative zones

According to Ceccotti, 1995, the ductility properties of dowel-type fasteners of mechanical joints in timber structures are related to the slenderness of the fastener. Slenderness is defined as the ratio between the thickness of the wood member and the diameter of the dowel-type fastener. Slender fasteners tend to dissipate more energy as the possible development of plastic hinges in the steel will be increased whereas if large diameter steel fasteners are used, they will perform elastically without dissipation of energy in the steel. Moreover, splitting is better prevented if the thickness of the timber member is increased in relation to the diameter of the fasteners. According to Ceccotti and Touliatos, 1995, the provision for the minimum thickness of the connected members of at least 8d and the dowel diameter not exceeding 12 mm, is given order to reach a Mode III failure (corresponding to failure mode *f* for timber-to-timber connection in single shear and failure mode *k* for timber-to-timber connection in double shear according to §8.2.2 of EN1995-1-1 with the formation of two plastic hinges in the steel fastener). This rule leads to values of $\frac{t_1}{\sqrt{\frac{M_{yd}}{f_{hd} \cdot d}}}$ and $\frac{t_2}{\sqrt{\frac{M_{yd}}{f_{hd} \cdot d}}}$ greater than 3.5, which according the Mohler diagrams of Figure 5- given in Hilson, 1995, that represent the Johansen formulas calculated for $\beta = \frac{f_{h2k}}{f_{h1k}} = 1$, ensure the attainment of a Mode III failure. Therefore, even the rule of minimum thickness of the connected members of at least 8d which is currently given for DCM ductility class in Eurocode 8 (whereas a 10d values is given for DCH), would lead to the most ductile failure modes according to Johansen theory.

However, by applying the usual values of timber embedding strength and fastener steel yielding strength (considering a characteristic timber density of 350 kg/m^3 and a characteristic tensile strength of steel of 800 N/mm^2), with the above referenced values of thickness of the timber members and diameter of steel fastener, the values obtained for $\frac{t_1}{\sqrt{\frac{M_{yd}}{f_{hd} \cdot d}}}$ and $\frac{t_2}{\sqrt{\frac{M_{yd}}{f_{hd} \cdot d}}}$ are 4.27, which is a high conservative value. Even for CLT connections, by applying the modified Johansen theory proposed by Uibel and Blaß, 2006, a value of 4.05 is obtained.

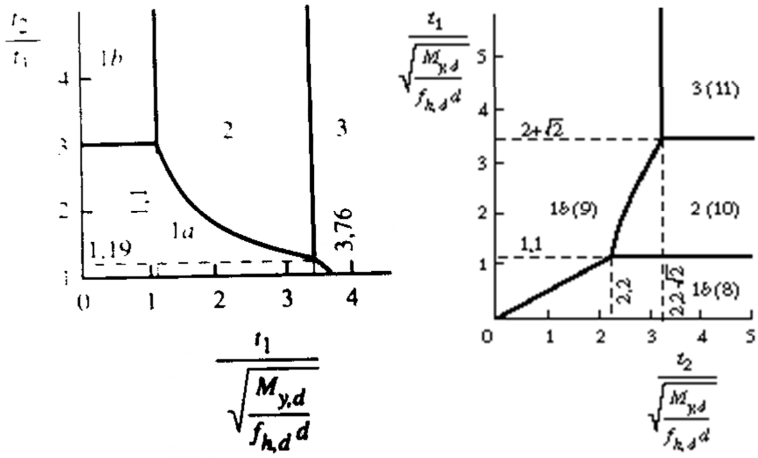


Figure 5-2: Modified Mohler diagram for single shear (left) and double shear (right) calculated for β=1 (after Hilson, 1995).

Therefore, while keeping the same provisions currently given in Eurocode 8, a new formulation of the detailing rules for dissipative zones is proposed, integrating them with new provisions towards “performance based” design rules. The new provisions require the attainment of a certain ductile failure mode according to the Johansen theory as given in Eurocode 5, in accordance with the basic philosophy of Eurocodes as an alternative to the "prescriptive" provisions given in 2004 and previous editions, where the fastener diameter is limited to 12 mm and the thickness of the timber members to 10 times the fastener diameter for DCH.

(k) – Capacity based design rules and over-strength factors

The capacity based design rules here provided for CLT buildings are mainly based on the research experience of the Sofie Project and are referenced in Chapter 3 and in Ceccotti and Follesa, 2006, Ceccotti et al., 2007, Follesa et al., 2013, Gavric et al., 2013. The proposed values of over-strength factors of 1.3 and 1.6 for DCM and DCH ductility classes are referenced in Fragiacomo et al., 2011, Gavric et al., 2011 and Gavric et al., 2012.

The capacity based design rules here provided for Light-Frame buildings buildings are mainly based on research experience and are referenced in Chapter 3 and in Follesa et al., 2011 and Casagrande et al. 2014. The proposed value of over-strength factors 1.6 DCH ductility classes are referenced in Casagrande et al., 2014 and Schick et al., 2013, while a value of 1.3 for DCM is proposed according to the proposal currently included in the draft document for the new Italian Building Code (Norme Tecniche delle Costruzioni “NTC 2015” – Bozza 10/2014). An analytical method for the application of such provisions for single storey and multi-storey Light-Frame buildings could be found in Casagrande et al., 2014.

The capacity based design rules and over-strength factors provided for Log House building are based on construction practice and are referenced in Follesa et al., 2011.

The capacity based design rules and over-strength factors provided for other structural types are based on construction practice and partly based on previous background documents provided for previous editions of this chapter. The values of over-strength factors proposed for post and beam structures, moment resisting frames, large span trusses and vertical cantilevers are based on the values proposed for dowel-type connections in Jorissen and Fragiacomo, 2011.

(l) – Smooth nails and stapled connections

This sentence, which was included in the 1995 ENV version and deleted in the 2004 edition, has been re-introduced also for compliance with the provisions included in 8.3.3.2.2.

(m) – Detailing rules for Light-Frame horizontal diaphragms

It should be underlined that, in the construction practice of European Countries and differently from North America for example, often horizontal diaphragms are made with glulam timber beams or joists, a single layer of timber boards with the sheathing nailed above the timber boards. Blockings are almost never used for esthetical reasons, since the timber beams or joists are left uncovered. Therefore further research is needed in order to verify is the rigid diaphragm assumption is still valid with this or similar type of floor construction.

Moreover it should be also pointed out that the rule of no change of span-direction in horizontal diaphragms is almost never applied in practical applications.

(n) – Partial safety factors for material properties

The values of the partial safety factors given for the ultimate limit state verifications of structures designed according to the concept of dissipative and non-dissipative structural behaviour were inverted in 2004 edition with respect to the previous 1995 ENV edition.

In principle, it is correct to use in seismic design the partial safety factor γ_M=1.0 given for the accidental load combination, as seismic actions could be regarded as an accidental load case. However, the reference strength considered when designing according to the dissipative behaviour concept, should be adequately reduced to take into account the strength degradation due to cyclic loading. For timber structures, the dissipative elements are mechanical connections, whose strength is calculated according to Eurocode 5 (EN 1995-1, CEN, 2009) using the Johansen theory. This strength is calculated for monotonic loading and no allowance for degradation due to cyclic loading is made. By analogy with the other material chapters, it is therefore prescribed that the partial safety factor γ_M=1.0 given for the accidental load combination could be used only if the strength degradation due to cyclic loading is appropriately accounted for in the evaluation of the connection resistance. Otherwise the material safety coefficient given for fundamental load combinations should be applied, in order to approximately account for strength degradation under cyclic load in dissipative structures. This is based on the assumption that the ratio between the partial safety factors for the material properties for fundamental load combinations (i.e. γ_M=1.3 according to Eurocode 5) and the partial safety factors for the material properties for accidental load combinations (i.e. γ_M=1.0 according to Eurocode 5) is roughly equal to the ratio between the strength for monotonic loading and the degraded strength due to cyclic loading in the dissipative zones. However this rule should be applied only to the dissipative zones of each structural type, while for non-dissipative structural elements the same value of the partial safety factor given for the design according to the concept of non-dissipative behaviour should be used.

For non-dissipative timber structures (e.g. structures with only glued joints), since there is no degradation due to cyclic loading considering the linear-elastic stress/strain relationship for timber structures until collapse is reached, the partial safety factors for the material properties for accidental load combinations (i.e. γ_M=1.0 according to Eurocode 5) could be used. It should be noted that this provision, which was included also in the previous 1995 ENV version of Chapter 8, is different from the case of other material properties of Eurocode 8.

(o) –Inter-storey drift limits

This provision, based on design practice experience and on results of experimental research conducted on different structural types, is also referenced in Follesa et al., 2011.

References – Chapter 5

Bedon, C., Fragiacomo, M., Amadio, C., and Sadoch, C. [2014] "Experimental study and numerical investigation of "Blockhaus" shear walls subjected to in-plane seismic loads." ASCE Journal of Structural Engineering, ASCE, published online, doi: 10.1061/(ASCE)ST.1943-541X.0001065.

Bedon, C., Rinaldin, G., and Fragiacomo, M. [2015] "Non-linear modelling of the seismic behaviour of 'Blockhaus' structures." Engineering Structures, in print.

Boudaud C., Haumeury S., Faye C., Daudeville L. [2010] "European seismic design of shear walls: experimental and numerical tests and observations", Proceedings of the 11th World Conference on Timber Engineering, Riva del Garda (TN), Italy.

Campos Costa A., Candeias P. X., Piazza M., Tomasi R., Grossi P. [2013] "Seismic performance of multi-storey timber buildings - RubnerHaus building -- Final Report" – SERIES. Work Package [WP9-TA5 LNEC].

Casagrande D., Sartori T., Tomasi R. [2014] "Capacity design approach for multi-storey timber-frame buildings." Proc., The International Network on Timber Engineering Research INTER, Bath, United Kingdom, paper 47-15-3.

Ceccotti, A. [1995] "Timber connections under seismic actions". In: Timber engineering–STEP 1. 1st Edition. STEP/EUROFORTECH. The Netherlands, ISBN 90-5645-001-08. Pp. C17/1-C17/10.

Ceccotti A., Follesa M. [2006] "Seismic Behaviour of Multi-Storey X-Lam Buildings." Proceedings of 426 COST E29 International Workshop on Earthquake Engineering on Timber Structures, pages 81-95, Coimbra, Portugal.

Ceccotti A., Follesa M., Lauriola M.P. [2007] "Quale fattore di struttura per gli edifici multipiano a struttura di legno con pannelli a strati incrociati?" XII Convegno ANIDIS L'ingegneria sismica in Italia, Pisa 2007.

Ceccotti, A., Karacabeyli, E. [2002]. "Validation of seismic design parameters for wood-frame shearwall systems." Canadian Journal of Civil Engineering, 29(3), 484-498.

Ceccotti A., Sandhaas C., Okabe M., Yasumura M., Minowa C., Kawai N., [2013] "SOFIE project – 3D shaking table test on a seven-storey full-scale Cross-Laminated building", Earthquake Engineering & Structural Dynamics, DOI: 10.1002/eqe.2309.

Ceccotti, A., Touliatos P. [1995] "Detailing of timber structures in seismic areas". In: Timber engineering–STEP 2. 1st Edition. STEP/EUROFORTECH. The Netherlands, ISBN 90-5645-001-08. Pp. D10/1-D10/13.

European Committee for Standardization (CEN), [2001] - EN 12512: Timber structures- Test methods. Cyclic testing of joints made with mechanical fasteners.

European Committee for Standardization (CEN), [2009] "Eurocode 5 – Design of timber structures – Part 1-1: General rules and rules for buildings", Brussels, Belgium.

European Committee for Standardization (CEN), [2004] Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, Brussels, Belgium.

Follesa M., Christovasilis I., Vassallo D., Fragiacomo M., Ceccotti A. [2013] "Seismic design of multi-storey CLT buildings according to Eurocode 8." Ingegneria Sismica, Special Issue on Timber Structures, n. 04/2013, pp. 27-53.

Fragiacomo, M., Dujic, B., and Sustersic, I. (2011). "Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions." Engineering Structures, Special Issue on Timber Structures, Vol. 33 No. 11, pp. 3043-3053.

Gavric I. [2013a] "IVALSA X-Lam Connection Tests Report". Research report, CNR-IVALSA, San Michele all'Adige (TN), Italy.

Gavric I. [2013b] "IVALSA X-Lam Wall Tests Report". Research report, CNR-IVALSA, San Michele all'Adige (TN), Italy.

Gavric I. [2013c] "Seismic behaviour of cross-laminated timber buildings." Phd Thesis, Università degli Studi di Trieste, Italy.

Gavric I., Ceccotti A., Fragiacomo M. [2011] "Experimental cyclic tests on cross-laminated timber panels and typical connections." Proceedings of the 14th ANIDIS Conference, Bari (Italy), September 2011.

Gavric I., Fragiacomo M, Ceccotti A. [2012] Strength and deformation characteristics of typical X-Lam connections. Proceedings of the 12th World conference on timber engineering, Auckland (New Zealand), July 2012.

Gavric I., Fragiacomo M., Ceccotti A., [2013] "Capacity seismic design of x-lam wall systems based on connection mechanical properties" Proceedings of 46th CIB W18 Meeting, Vancouver 2013, paper n.46-15-2.

Gavric, I., Fragiacomo, M., and Ceccotti, A. (2014). "Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures". RILEM Materials and Structures, published online, doi: 10.1617/s11527-014-0278-7.

Gavric, I., Fragiacomo, M., and Ceccotti, A. (2015). "Cyclic behaviour of typical screwed connections for cross-laminated (CLT) structures." European Journal of Wood and Wood Products, Vol. 73 No. 2, pp. 179-191, doi: 10.1007/s00107-014-0877-6.

Gavric, I., Fragiacomo, M., and Ceccotti, A. (2015). "Cyclic behavior of cross-laminated timber (CLT) wall systems: Experimental tests and analytical prediction models." ASCE Journal of Structural Engineering, in print.

Hilson B. O.. [1995] "Joints with dowel-type fasteners - Theory". In: Timber engineering–STEP 1. 1st Edition. STEP/EUROFORTECH. The Netherlands, ISBN 90-5645-001-08. Pp. C3/1-C3/11.

Jorissen A., Fragiacomo M. [2011] "General notes on ductility in timber structures", Engineering Structures, Vol. 33, N. 11, 2011, pp. 2987-2997.

Karacabeyli E., Ceccotti A. [1998] "Nailed wood-frame shearwalls for seismic loads, test results and design considerations". Proceedings of the Structural Engineers World Congress (SEWC), San Francisco, USA, 1996. Paper Reference T207-6.

Norme Tecniche delle Costruzioni "NTC 2015" – Bozza 10/2014.

NRC [2010] "National Building Code of Canada 2010". Canadian Commission on Building\ and Fire Code, National Research Council of Canada, Ottawa, Ont.

NZS 3603 [1993] "Timber structures standard". Wellington, New Zealand.

Munoz W., Mohammad M., Salenikovich A., Quenneville P. [2008]: "Determination of yield point and ductility of timber assemblies: in search for a harmonised approach". Engineered Wood Products Association.

Karacabeyli, E., Ceccotti A. [1996]: "Quasi-static reversed cyclic testing of nailed joints". Proceedings of International Council for Building and Research Studies and Documentation Working Commission W18 – Timber Structures. Pap. 29-7-7. Karlsruhe, Germany.

Piazza M., Tomasi R., Grossi P., Campos Costa A., Candeias P. X. [2013] "Seismic performance of multi-storey timber buildings - RubnerHaus building – Final Report" - Seismic Engineering Research Infrastructures For European Synergies (SERIES), Work package [WP9 – TA5 LNEC].

Pozza L., Scotta R., Trutalli D, Ceccotti A., Polastri A. [2013] "A non-linear numerical model for the assessment of the seismic behaviour and ductility factor of X-Lam timber structures". Proceedings of the International Symposium on Timber Structures, Istanbul, Turkey, 2009, pp 151–162.

Pozza L., Scotta R., Vitaliani R. [2009] "A non-linear numerical model for the assessment of the seismic behaviour and ductility factor of X-Lam timber structures". Proceedings of the International Symposium on Timber Structures, Istanbul, Turkey, 2009, pp 151–162.

Sartori T., Tomasi R. [2013] "Experimental investigation on sheathing-to-framing connections in wood shear walls" Engineering Structures, 56, pp. 2197- 2205.

Schick M., Vogt T., Seim W. [2013] "Connection and anchoring for wall and slab elements in seismic design". Proceedings of the 46th CIB-W18-Meeting, paper 46-15-4. Vancouver, Canada, 2013.

Sustersic, I., Fragiacomo, M., and Dujic, B. [2015] "Seismic analysis of cross laminated multistorey timber buildings using linear and nonlinear static and dynamic methods." Submitted to ASCE Journal of Structural Engineering, Special issue on Seismic Resistant Timber Structures.

Tomasi R., Sartori T. [2013] "Mechanical behaviour of connections between wood framed shear walls and foundations under monotonic and cyclic load" Construction and Building Materials, 44, pp. 682-690.

Uibel T, Blaß H.J., [2006]. "Load Carrying Capacity of Joints with Dowel Type Fasteners in Solid Wood Panels" 39th CIB W18, Firenze, Italy, paper 39-7-5.

Vogt T, Hummel J., Seim W. [2012] "Timber framed wall elements under cyclic loading", Proceedings of the 12th World Conference on Timber Engineering, Auckland, New Zealand.

Yasumura, M., Kawai N. [1998] "Estimating seismic performance of wood-framed structures" Proceedings of 1998 I.W.E.C. Switzerland. Vol.2. pp. 564-571.

Wrzesniak D., Amadio C., Rinaldin G., Fragiacomo M. [2013] "Proposal for the q-factor of moment-resisting timber frames with high ductility dowel connectors" Proceedings of the 46th CIB-W18-Meeting, paper 46-15-6. Vancouver, Canada, 2013.

Wrzesniak D., Rodgers G. W., Fragiacomo M., Chase J.G. [2014] "Damage avoidance design of timber structures using high-force-to-volume damping devices" Proceedings of the 13th World Conference on Timber Engineering, Quebec City, Canada.

6 – Conclusions and recommendations

6.1 Preface

The research projects carried out so far and referenced in the previous chapters brought a huge amount of experimental data and useful information which has been used to develop the additional aspects related to the seismic design of timber buildings listed in the previous chapters. At the same time, due also to the development of powerful software packages for structural analysis, new models for the linear and non-linear analysis of timber structures have been developed and used for research purposes especially in the evaluation of the seismic performance of medium to high rise timber buildings referenced in the previous chapters (Fragiacomo et al. 2011, Rinaldin et al. 2013a, b, Follesa et al. 2013).

In recent years a relevant number of medium to high-rise timber buildings have been built in different low, moderate and high seismicity European zones. The experimental results have shown that, when correctly designed and properly detailed, not only are multi-storey timber buildings able to withstand severe earthquakes, but have also the additional benefit, depending on the structural type and the connection detailing, of a limited damage mainly concentrated to some connection elements which are easily replaceable. However, the rules currently included in Section 8 of EC8 are not sufficient to properly address the seismic design of such tall buildings.

6.2 Recommendations for the current evolution of EC8

It is common sense that a standard should be as concise and accurate as possible, and too many and unnecessarily complex provisions should generally be avoided. However, also a lack of design rules or the incorrect application of the existing ones due to a misinterpretation may lead to a possible non-conservative design, which could be particular dangerous for medium to high rise buildings.

Therefore, while keeping the same concise approach of the current version of the timber Section, it is very important to give guidance to the structural designers for a proper analysis, design and detailing of the different timber structural types in seismic zones, within the basic principles of EC8. This is the reason why a comprehensive review of the current version of this Section is needed as explained in detail in the previous chapters.

Moreover, many parts of the current provisions included in Chapter 8 of Eurocode 8 need a comprehensive review and more research is needed for a better formulation of existing provisions. The current provisions given in order to classify timber structures in Medium and High ductility class are not satisfied for the most common structural types used nowadays in the construction practice and therefore a new formulation should be proposed based on the ductility properties of structural sub-assemblies representative of each structural types. Further research is also needed in order to provide appropriate values of the over-strength factor to be used according to the capacity based design rules given for each structural type. The choice of the correct value of the over-strength factor to be applied in the seismic design is of the outmost importance in capacity based design philosophy as an incorrect evaluation of this coefficient may lead to an undesirable less ductile behaviour of the entire building with possible catastrophic consequences.

Further investigation should also be carried out in order to address new provisions for the force-based design procedure of building with different lateral load resisting system (e.g. mixed steel-timber buildings or concrete-timber buildings) as this type of construction is used more and more for different reasons (energy efficiency, architectural issues, fire resistance), by performing experimental studies and non-linear static and dynamic analysis of different case-study buildings with different combinations of structural elements designed to resist the horizontal loads.

Finally more experimental and numerical investigations should be performed in order to better investigate the inter-storey drift limits to be given for the different structural types both in case of presence of brittle or ductile non-structural partitions.

6.3 Recommendations for the third generation of EC8

Due to a great number of advantages over more traditional structural systems widely used in Europe like masonry or concrete (sustainability, speed of construction, energy efficiency), timber systems are being used more and more worldwide for the construction of residential multi-storey buildings. The new frontiers are now the “tall wood buildings” with a number of storeys ranging from 10 to 30 (Pei et al. 2014). A 10-storey building has been recently built in Australia and a 14-storey building is already under construction in Norway, even if in a non-seismic area, and there are projects for the construction of buildings up to 20 storey in Austria and 30 storeys in Canada (Green, 2012) and USA.

Sustainability issues are more and more important in the construction field and National and Local regulations of most of the European Countries are nowadays based on methods for the environmental and energy-efficiency assessment of new buildings. Furthermore, the Construction Products Regulation EU/305/2011 added a 7th Basic Requirement for the construction products: the "sustainable use of natural resources". From this point of view wood as construction material plays a crucial role in the fight against climate change. Forests, producing wood, reduce the amount of carbon dioxide in the atmosphere, fixing the carbon through the process of photosynthesis. By increasing the use of wood in construction, carbon dioxide is basically displaced from the atmosphere into the buildings. Furthermore, the use and transformation of wood into construction products requires less CO_2 emissions compared to the same process for other materials such as concrete, bricks, glass or steel.

Figure 6-1: Nine storey CLT building built in London in 2008. Structures were built in nine weeks. Design: Waugh-Thistleton Architects.

Another advantage of wood construction is the speed of erection as there is no need to wait for casting and hardening of concrete, and the timber components can be prefabricated off-site, transported to the building site and quickly connected using modern connectors. For this reason, in a number of cases timber was preferred to reinforced concrete (e.g. in the 9-storey building built in 2008 in London, see Figure 6-1). Last but not least, timber systems proved to be suitable for the construction of buildings which could be dismounted, re-used and/or easily repaired and retrofitted. In 2012 in Florence, Italy, medium seismicity area, two completely demountable 3-storey residential timber buildings were built. These buildings will be temporarily used for 3-4 years for social housing, and then demounted and remounted in a new location for the same purpose (see Figure 6-2).

Figure 6-2: Two 3-storey residential demountable buildings built in Florence in 2012. Structural design: dedaLEGNO.

Therefore, considering these new trends for the next few years, a third generation of EC8 for timber structures should address the following issues:

1. More detailed provisions about non-linear static and dynamic analysis methods should be provided in order to foster their use in seismic design. However, the non-linear behaviour of timber structural systems is essentially based on the non-linear properties of connections. Furthermore, structural designers do not have usually easy access to experimental data (which should refer to the same connection with the same type, number and diameter of fasteners used in the actual design). Therefore, in order to improve the ease of use of these methods, the products certification (ETA, CE marking based on product standards) for connections and fasteners should contain also details about the non-linear properties of such elements.
2. Provisions should be given for the use of displacement-based design (direct displacement based design and N2 method) as an alternative to the force-based procedure (van de Lindt et al. 2012). Particularly

for the design of tall wood buildings or for the design of buildings with different lateral load resisting systems, this approach seems to be more suitable for the evaluation of the correct seismic performance.

3. Some guidance should be also given for the seismic design of demountable buildings and for the retrofitting of existing timber (van de Lindt et al. 2014) and non-timber (e.g. masonry, Sustersic and Dujic 2012) buildings using wood-based products.
4. Guidelines on the design of tall (10 storeys and more) timber buildings should also be given so as to account for the specific behaviour of timber (e.g. the influence of the higher vibration modes in the design due to the low modulus of elasticity of timber).
5. Provisions should also be given for the seismic design of innovative low-damage structural systems, such as rocking walls and frames with energy dissipaters prestressed using unbonded tendons (Buchanan et al. 2008) or with gravity loads (Wrzesniak et al. 2014); the use of Tuned Mass Dampers (Poh'sie et al. 2014) and advanced materials such as superelastic shape memory alloys (van de Lindt and Potts 2008); and the use of passive base isolation systems for timber buildings (Sancin et al. 2014).
6. Recommendations for the estimation of the connection ductility in the dissipative regions should also be provided, together with detailing rules such as the use of specific reinforcement to avoid brittle failure modes such as shear plug, splitting, etc.

6.4 Recommendations for further research

With the aim of investigating the seismic performance of tall timber buildings, new types of connections and/or new design approaches should be provided. For instance, the hold-down connectors commonly available for the construction of timber buildings have a maximum characteristic strength of 100 kN. However, it is not unusual to calculate uplift forces up to 500-700 kN even in low seismicity areas for medium-rise buildings (6-7 storeys). Therefore, in case these uplift forces are resisted only by hold-down connectors, this may lead to an excessively great number of connectors to be placed at the same position, with risk of brittle failure (e.g. splitting) within the connected timber parts. So there is a demand for stronger connection systems for medium to high-rise buildings in seismic areas (Polastri and Angeli, 2014) or alternative design methods which yields smaller seismic forces in the connections.

This is the reason why new approaches for the seismic design of such tall buildings, including alternative design procedures such as the use of new types of dissipative steel connections, innovative energy dissipaters (Wrzesniak et al. 2014) and tuned mass dampers (Poh'sie et al. 2014), deformable floor diaphragms, pre-stressed re-centring walls (Buchanan et al. 2008), or multi-storey segmental rocking walls should be further investigated (Pei et al., 2014).

Furthermore, more research is needed in order to better estimate: (i) the seismic performance of non-structural elements; (ii) the building techniques for the construction of demountable buildings; and (iii) techniques for the seismic retrofitting of existing buildings.

Research into the prediction of the ductility of dissipative connection systems will also be crucial. Due to the scatter of experimental results, it will be important to develop suitable details (e.g. the use of an appropriate reinforcement in the timber part) to ensure the brittle failure modes can occur only once the connection has developed the required target ductility needed to attain the behaviour factor q tabled for a certain structural type.

Last but not least, an investigation into the relationship between the behaviour factor q of the entire building, the geometry of the building, and the ductility of the dissipative zones should be undertaken for the different structural systems currently used in practice. This study would allow the provision of better information in the choice of the behaviour factor q in the Eurocode 8.

References – Chapter 6

Buchanan A., Deam B., Fragiacomo M., Pampanin S., Palermo A. [2008] "Multi-storey prestressed timber buildings in New Zealand." Structural Engineering International, IABSE, Special Edition on Tall Timber Buildings, Vol. 18 No. 2, pp. 166-173.

European Committee for Standardization (CEN), [2004] Eurocode 8—Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, Brussels, Belgium.

Follesa M., Christovasilis I., Vassallo D., Fragiacomo M., Ceccotti A. [2013] "Seismic design of multi-storey CLT buildings according to Eurocode 8." Ingegneria Sismica, Special Issue on Timber Structures, n. 04/2013, pp. 27-53.

Fragiacomo M., Dujic B., Sustersic I. [2011] "Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions." Engineering Structures, Special Issue on Timber Structures, Vol. 33 No. 11, pp. 3043-3053.

Green M. C. [2012] "The case for tall wood buildings." Research Report.

Pei S., Berman J., Dolan D., Van De Lindt J. W., Ricles J., Sause R., Blomgren H. E., Popovski M., Rammer D. [2014] "Progress on the development of seismic resilient tall CT buildings in the Pacific Northwest." Proc., WCTE 2014, Quebec City, Canada, August 10-14.

Poh'sie G.H., Rinaldin G., Fragiacomo M., Amadio C., Ceccotti A. [2014] "Application of translational tuned-mass dampers on the seven storey building tested within the SOFIE project." Proc., WCTE 2014, Quebec City, Canada, August 10-14.

Polastri A., Angeli A. [2014] "An innovative connection system for CLT structures: experimental – numerical analysis". Proc., WCTE 2014, Quebec City, Canada, August 10-14.

Rinaldin G., Amadio C., Fragiacomo M. [2013a] "A component approach for the hysteretic behaviour of connections in cross-laminated wooden structures." Earthquake Engineering and Structural Dynamics, Vol. 42 No. 13, pp. 1885–2042, doi: 10.1002/eqe.2310.

Rinaldin G., Poh'sie G.H., Amadio C., Fragiacomo M. [2013b] "Modelling of seismic behaviour of light-frame timber structures." Ingegneria Sismica, Special Issue on Timber Structures, No. 4/2013, pp. 82-98.

Sancin L., Rinaldin G., Fragiacomo M., Amadio C. [2014] "Seismic analysis of an isolated and a non-isolated light-frame timber building using artificial and natural accelerograms." Bollettino di Geofisica Teorica e Applicata/Bulletin of Theoretical and Applied Geophysics, Vol. 55 No. 1, pp. 103-118, doi: 10.4430/bgta0093.

Sustersic I., Dujic B. [2012] “Seismic Strengthening of existing buildings with cross laminated timber panels.” Proc., WCTE2012, Auckland, New Zealand, July 16-19.

van de Lindt J. W., Potts A. [2008] “Shake Table Testing of a Superelastic Shape Memory Alloy Response Modification Device in a Wood Shearwall.” Journal of Structural Engineering, Vol. 134 No. 8, pp. 1343 – 1352.

van de Lindt, J., Rosowsky, D., Pang, W., and Pei, S. [2012]. “Performance-based seismic design of midrise wood frame buildings” Journal of Structural Engineering, 139(8), 1294‑1302.

van de Lindt J. W., Bahmani P., Mochizuki G., Pryor S. E., Gershfeld M., Tian J., Michael D. Symans M. D., Rammer D. [2014] “Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Wood-Frame Building with Retrofits. II: Shake Table Test Results.” Journal of Structural Engineering, E4014004, doi: 10.1061/(ASCE)ST.1943-541X.0001206.

Wrzesniak D., Rodgers G.W., Fragiacomo M., Chase J.G. [2014] “Damage avoidance design of timber structures using high-force-to-volume damping devices.” Proc., WCTE 2014, Quebec City, Canada, August 10-14.