



UNIVERSITÀ DEGLI STUDI  
DI TRENTO

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COMPORAMENTO STRUTTURALE DI SISTEMI  
COSTRUTTIVI IN LEGNO REALIZZATI CON  
PARETI PORTANTI INTELAIATE

STRUCTURAL BEHAVIOR  
OF TIMBER FRAMED BUILDINGS

Novembre, 2012

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Prof. Roberto Tomasi  
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UNIVERSITA' DEGLI STUDI DI TRENTO

Dottorato di Ricerca in Ingegneria dei Sistemi Strutturali Civili e Meccanici XXV ciclo

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# SOMMARIO

Questo lavoro di tesi ha come obiettivo principale quello di caratterizzare meccanicamente il comportamento strutturale di edifici in legno a pareti portanti intelaiate, con particolare riferimento alle sollecitazioni derivanti da azioni sismiche. Per poter raggiungere tale obiettivo è stata portata a termine una ampia campagna sperimentale suddivisa in tre fasi. Nella prima fase si è proceduto allo studio della connessione fra pannelli controventanti a base di legno e montanti costituenti le singole pareti portanti. Successivamente si è investigato il comportamento delle connessioni utilizzate per ancorare le pareti intelaiate a terra. Nella seconda fase sono state sottoposte a test delle pareti a grandezza reale realizzate mediante differenti materiali e diversamente connesse a terra. I risultati emersi hanno permesso lo studio delle singole componenti strutturali che costituiscono un intero edificio, dal singolo connettore fino alla parete vera e propria. Al fine di conoscere il comportamento delle componenti stesse all'interno di un edificio reale, e dunque la loro interazione reciproca in caso di sisma, nella terza fase è stata eseguita una prova su tavola vibrante di un edificio a tre piani in scala reale. Tutti i test effettuati hanno permesso di raccogliere una moltitudine di dati con i quali popolare un database da utilizzare sia per poter comprendere nel dettaglio il funzionamento del sistema costruttivo in esame e sia per poter validare le formulazioni proposte. Dal punto di vista analitico è stata sviluppata una formulazione in grado di prevedere lo spostamento orizzontale di una parete intelaiata soggetta ad una forza orizzontale in sommità. Tale formulazione è stata poi validata con successo, confrontandone i risultati con quelli ottenuti dai test di laboratorio.

Per l'esecuzione dei test sono stati create delle procedure e dei set-up ad hoc, utilizzabili anche in futuro per svolgere test simili. Un importante lavoro è stato fatto per realizzare una configurazione di prova adatta a testate pareti di legno in scala reale controllando al meglio le condizioni al contorni di vincolo e di carico.



# SUMMARY

The research hereinafter describes aims at mechanically characterize the behavior of timber framed buildings, with particular focus on thir behaviour in seismic zones. An extensive experimental campaign divided into three phases has been completed In order to achieve this objective.

In the first phase tests on the connection between sheathing panels and timber studs were conducted. Subsequently the behavior of the connections used for anchoring the walls to the foundation were investigated. In the second stage the full scale timber framed walls were subjected to tests. The walls were realized using different materials and different type of connections to the ground. The results have allowed the study of the individual structural components which constitute an entire building, from the single connector to the entire wall. In order to understand the behavior of the components themselves in a real building, and thus their interaction with each other in case of an earthquake, in the third stepa shaking table test of a three-story building has been done.

All tests have permitted to collect a wealth of data with which to populate a database to be used both to understand in detail the structural behavior of the timber framed building system and in also to validate the proposed formulations.

From the analytical point of view, an equation able to provide the horizontal displacement of a wall subject to a horizontal force has been developed. This equation was then successfully validated by comparing the results with those obtained from laboratory tests.

For the execution of the tests have been created procedures and set-up ad hoc, that could be used also in the future to carry out similar tests. An important work has been done to achieve a configuration suitable to test full scale timber framed walls through well controlling the boundary conditions.





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# Capitolo 1

## Introduzione

### 1.1 Background

Il comportamento strutturale degli edifici in legno a pareti portanti intelaiate è stato, e risulta tuttora, oggetto di numerosi progetti di ricerca. Tali progetti hanno riguardato sia singole componenti strutturali che edifici realizzati in scala reale, sollecitati sia da carichi verticali che da carichi orizzontali derivanti da vento o sisma. E' bene precisare come tuttavia tali studi sono stati condotti quasi esclusivamente nei paesi di maggior diffusione del sistema costruttivo in esame, e dunque in relazione alle tipologie tradizionali locali. Tali studi hanno confermato il buon comportamento strutturale degli edifici in legno, fornendo precise regole ingegneristiche. Per molti anni infatti la costruzione degli edifici in legno era basata, nella maggior parte dei casi, soprattutto su regole legate all'esperienza e alla tradizione del buon costruire. Il sistema costruttivo europeo si differenzia notevolmente da quello tradizionale degli altri paesi (Stati Uniti, Nuova Zelanda, Paesi Scandinavi), essendo caratterizzato da elementi strutturali massicci e da un elevato grado di prefabbricazione. Alcuni studi sono stati condotti su tali sistemi ma ben pochi hanno avuto come oggetto un'analisi dettagliata del comportamento degli stessi in caso di azioni sismiche. I Paesi Scandinavi infatti, così come la Germania, non sono caratterizzati da un elevato rischio sismico, a differenza dell'Italia. Diviene dunque fondamentale comprendere il reale comportamento che gli edifici a pareti portanti intelaiate assumono in presenza di azioni orizzontali derivanti dai terremoti, al fine di poter garantire adeguati livelli di sicurezza e prestazioni ottimali in modo da permettere una piena diffusione all'interno del mercato nazionale. Con

riferimento a tale obiettivo, la tesi di dottorato qui presentata ha come obiettivo quello di investigare il comportamento meccanico degli edifici con comportamento strutturale a pareti intelaiate. Il programma di ricerca ha previsto tre differenti fasi di studio (Figura 1.1), ma concatenate, nell'intento di comprendere sia il comportamento meccanico delle singole componenti strutturali sia la loro interazione all'interno di un edificio reale. La prima fase ha avuto come oggetto lo studio del comportamento meccanico del sistema di connessione pannello-telaio ligneo e dei sistemi di ancoraggio delle pareti alla struttura di fondazione mediante elementi metallici. Risulta evidente l'importanza dei risultati di questa fase, in quanto sono analizzate i sistemi di collegamento fondamentali a garantire la stabilità degli elementi strutturali parete. Nella seconda fase si è proceduto allo studio di pareti a grandezza reale, al fine di stabilire una relazione fra il comportamento delle connessioni e il comportamento d'insieme della parete stessa. Al fine di comprendere la reale interazione degli elementi parete all'interno di un edificio in caso di sisma, nella terza fase è stata eseguita una prova su tavola vibrante su un edificio in scala reale. Tale prova risulta di fondamentale importanza sia per verificare che le vigenti regole progettuali e le tecniche costruttive utilizzate siano tali da garantire un adeguato livello di sicurezza in caso di sisma e sia per individuare le interazioni fra i diversi elementi strutturali in un edificio reale. Il progetto di ricerca svolto ha quindi previsto di studiare la tipologia costruttiva in esame partendo dagli elementi base che la caratterizzano per giungere allo studio di un edificio reale al fine di convalidare i risultati ottenuti in laboratorio sui singoli campioni all'interno di una struttura completa.

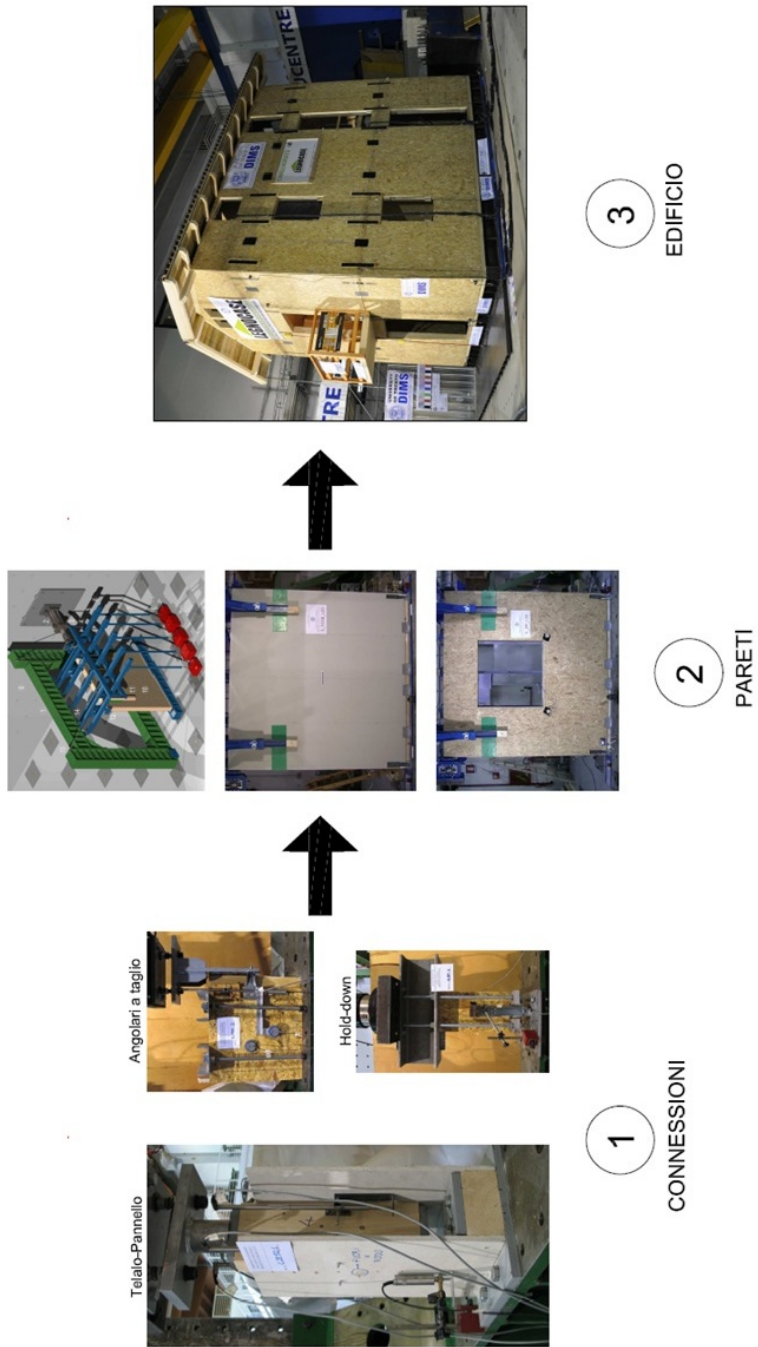


Figura 1.1: *Struttura progetto di ricerca*

## 1.2 Organizzazione della tesi

La tesi è divisa in due parti distinte.

La prima parte, scritta in lingua italiana, introduce il tema degli edifici in legno a parete portante intelaiate. Nel capitolo 2 si spiega come questo sistema sia stato ripreso dai Paesi nordici e dagli Stati Uniti e adattato alle esigenze del mercato italiano. Elevata sismicità e esigenze di prefabbricazione hanno trasformato in molti aspetti il sistema analizzato. Nel capitolo 3 vengono riassunte le caratteristiche che fanno sì che il materiale legno sia molto adatto ad essere utilizzato come materiale da costruzione in zona sismica. Viene poi dettagliatamente spiegato il comportamento degli edifici lignei sollecitati da un evento sismico. Il capitolo 4 infine riassume la campagna sperimentale svolta presso il Laboratorio Prove Materiali e Strutture dell'Università di Trento (test sui singoli componenti e test su pareti in scala reale) e presso il Laboratorio TREE-SLab della Fondazione Eucentre di Pavia (test di un edificio in legno a pareti portante intelaiate in scala reale).

Nella seconda parte, interamente scritta in lingua inglese, vengono riportati gli articoli scientifici pubblicati durante il periodo di svolgimento del dottorato. Tali articoli spiegano in maniera più ampia quanto riassunto nel capitolo 4.

I risultati dettagliati dei test svolti sono stati riassunti in una collana di Report Scientifici del Dipartimento di Ingegneria Meccanica e Strutturale riassunti nella Tabella 1.1. Le ultime due appendici (Appendice F e Appendice G) riportano i risultato di una attività di ricerca parallela svolta anch'essa durante il periodo del dottorato. Tale attività è stata portata avanti in collaborazione con il Professor Roberto Crocetti dell'Università di Lund in Svezia ed ha condotto alla realizzazione ed alla sperimentazione di solai prefabbricati composti legno-calcestruzzo.

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**Tabella 1.1:** *Report di prova redatti*





## Capitolo 2

# Edifici in legno a parete portante intelaiata

Gli edifici multipiano in legno a pareti portanti intelaiate rappresentano una tipologia costruttiva che da moltissimi anni occupa una posizione predominante nel mercato dell'edilizia residenziale di molti paesi, quali gli Stati Uniti, il Canada, la Nuova Zelanda ed i Paesi Scandinavi (Figura 2.1). Anche in Italia tale sistema costruttivo sta vivendo una rapida e costante crescita, proponendosi come valida alternativa a soluzioni tradizionali, realizzate con altri materiali da costruzione. Tale aspetto può essere sicuramente rappresentativo del fatto che le costruzioni civili in legno si stanno riappropriando dell'importanza che rivestivano nel passato. Grazie infatti a caratteristiche di assoluta rilevanza quali rinnovabilità, reperibilità, leggerezza e resistenza, si sta riscoprendo il materiale legno come valido materiale da costruzione, sia in ambito architettonico che ingegneristico.

I principali vantaggi dei sistemi costruttivi in legno possono essere riassunti nei seguenti punti:

- la leggerezza e la facilità di lavorazione rendono possibile trasportare, maneggiare e rifinire i singoli elementi in maniera relativamente semplice e con costi contenuti. Il trasporto e il montaggio degli elementi risulta perciò rapido ed economico;
- il legno e i suoi prodotti derivati sono naturalmente dotati di buone capacità isolanti. La realizzazione di edifici con elevate prestazioni termiche è agevolata dall'utilizzo di tali materiali. La scelta di una corretta stratigrafia di parete



**Figura 2.1:** Esempi realizzativi di edifici a pareti portanti intelaiati

porta ad avere elevati livelli di comfort termico ed igrometrico senza incrementi eccessivi di spessore;

- l'ottimo comportamento in presenza di sollecitazioni sismiche derivante dalla ridotta massa della costruzione e dall'alta capacità di dissipazione energetica garantita dall'elevato grado di ridondanza strutturale dei collegamenti;
- costi confrontabili se non inferiori con le costruzioni realizzate con i materiali tradizionali, anche grazie ai ridotti tempi di realizzazione ed ai minor imprevisti in corso d'opera;
- qualità garantita dei materiali utilizzati tramite la loro certificazione;
- sostenibilità del materiale e del processo produttivo.

Con particolare riferimento al mercato nazionale, il sistema costruttivo a pareti intelaiate è in grado di raggiungere un alto grado di prefabbricazione, che permette di sviluppare, in maniera relativamente semplice e veloce, edifici di assoluta qualità tecnica-architettonica che soddisfano avanzati criteri di sostenibilità. Proprio la prefabbricazione diviene, senza alcun dubbio, l'elemento caratterizzante che il sistema costruttivo in esame assume in Italia (Figura 2.2), distinguendosi dalla concezione classica a telaio leggero (tipica del mercato nordamericano) realizzato completamente in cantiere (Figura 2.3). Gli elementi parete e gli elementi costituenti i solai vengono

realizzati completamente in stabilimento, secondo un processo controllato, ben definito e di elevata precisione. In molti casi anche gli impianti tecnici vengono predisposti all'interno dei vari elementi prefabbricati, rendono necessarie dunque in cantiere poche e semplici lavorazioni che permettono di ridurre costi, tempi e problematiche tipiche di un cantiere edile. Tale aspetto è stato fortemente evidenziato anche in occasione del processo di ricostruzione del patrimonio edilizio in seguito al terremoto che ha colpito la regione Abruzzo il 6 aprile 2009. La velocità di realizzazione, unita alla capacità di sviluppare elevate prestazioni sismiche, hanno fatto in modo che circa il 50% degli edifici inseriti all'interno del progetto di ricostruzione C.A.S.E. fosse caratterizzato da un sistema costruttivo a pareti intelaiate. E' bene sottolineare come la prefabbricazione in realtà sia una peculiarità propria del sistema europeo, caratterizzato da elevata solidità e per il quale si adottano elementi strutturali e finiture tali da garantire i medesimi livelli di comfort ambientale (deformazioni, vibrazioni, isolamento acustico) di costruzioni più massicce, come quelle in muratura o in cemento armato. Al contrario, il modello nordamericano è caratterizzato da elementi strutturali di sezione ridotta (tipicamente 5.08 cm x 10.16 cm) che vengono assemblati direttamente in cantiere per realizzare il telaio ligneo. E' evidente che tale scelta non permette di raggiungere una qualità tipica del sistema prefabbricato e dunque il medesimo comfort.



**Figura 2.2:** *Prefabbricazione in stabilimento e montaggio in cantiere*

L'attenta progettazione consente di ottenere un'elevata durabilità e resistenza in caso di incendio. Spesso infatti vengono presentati esempi negativi non derivanti certamente dalle caratteristiche dei materiali e dei sistemi costruttivi, bensì scaturiti da una completa inadempienza della progettazione della durabilità. Sistemi di protezione e piani di manutenzioni devono essere infatti previsti per qualsiasi tipologia costruttiva e qualsiasi materiale. Un altro aspetto che caratterizza gli edifici in legno a pareti inte-

laiate è la capacità di ottenere ottime prestazioni energetiche derivanti la possibilità di sfruttare lo spazio interno della parete, posizionando del materiale isolante. Tale aspetto, in aggiunta alle intrinseche proprietà del materiale legno e alla possibilità di creare pacchetti di finitura aggiuntivi, permette di ottenere valori di trasmittanza termica ridotti con spessori contenuti. Anche l'inerzia termica, proprietà fondamentale per la climatizzazione estiva volta allo sfasamento del picco dell'onda termica giornaliera, può essere raggiunta tramite un'attenta progettazione della stratigrafia della parete.



**Figura 2.3:** *Costruzione in cantiere di un tipico edificio a pareti intelaiate nordamericano*

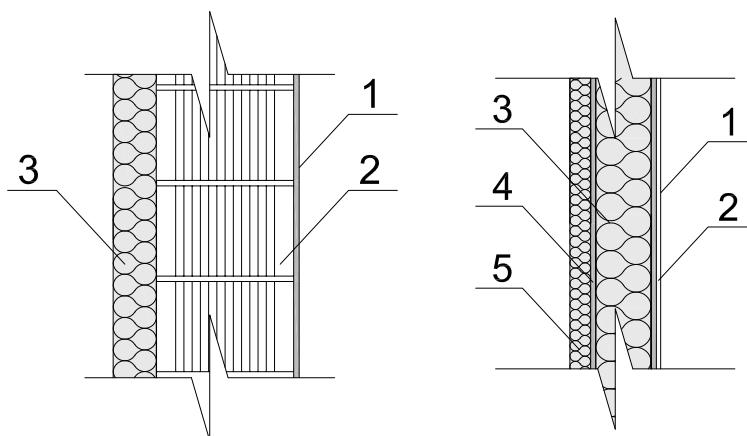
Da un punto di vista termico l'utilizzo di questo tipo di costruzioni segue il percorso che le normative relativamente al risparmio energetico stanno faticosamente tracciando. Le chiusure verticali ed orizzontali a parità di spessore offrono una trasmittanza molto bassa, riducendo le dispersioni di calore dall'involucro e, grazie alle caratteristiche fisiche del legno, mitigano notevolmente i ponti termici. Per meglio comprendere il divario in termini di prestazioni tra le più comuni tipologie edilizie si riporta un semplice confronto (Figura 2.4):

- Stratigrafia A: tipica di edifici con pareti di tamponamento in blocchi laterizi dotati sul lato esterno di uno strato coibente a cappotto;
- Stratigrafia B: tipica in edifici a telaio in legno è caratterizzata da due strati coibenti. Il primo collocato nell'intercapedine interna tra i montanti del telaio, il secondo posto sul lato a contatto con l'esterno come nel caso precedente.

In questo esempio estremamente semplificato si possono cogliere le differenze tra i due sistemi di isolamento termico utili nella riduzione del consumo energetico per

<b>STRATIGRAFIA A</b>		spessore	$\lambda$
Materiale		[mm]	$[\frac{W}{m*K}]$
1	Intonaco di calce e cemento	15	1
2	Mattone forato porizzato leggero	380	0,18
3	Polistirene espanso in lastre 20 $[\frac{Kg}{m^3}]$	120	0,04
<b>Spessore totale 0,52m - trasmittanza 0,189</b>		$[\frac{W}{m^2*K}]$	

<b>STRATIGRAFIA B</b>		spessore	$\lambda$
Materiale		[mm]	$[\frac{W}{m*K}]$
1	Pannello in gesso fibra	12,5	0,352
2	Pannello OSB	15	0,143
3	Lana di roccia (intercapedine)	160	0,04
4	Pannello OSB	15	0,143
5	Pannello isolante in fibra di legno	60	0,04
<b>Spessore totale 0,26m - trasmittanza 0,169</b>		$[\frac{W}{m^2*K}]$	



**Figura 2.4:** Stratigrafia A di un edificio tradizionale (sx) e stratigrafia B (dx) di un edificio in legno.

la climatizzazione invernale. Le prestazioni termiche sono comparabili, tuttavia per le costruzioni tradizionali si deve accettare un incremento di spessore del paramento murario. Per quanto attiene alla climatizzazione estiva in climi caldi, per la seconda tipologia, è necessario apportare modifiche alla stratigrafia per conferire alla struttura una maggiore inerzia termica in modo da sfasare il picco dell'onda termica giornaliera verso le ore notturne.

Infine si deve ricordare che, a differenza di acciaio e calcestruzzo armato, il legno richiede un minore dispendio in termini di energia nella fase di trasformazione e di montaggio. Le foreste se coltivate secondo criteri di gestione oculati rappresentano una risorsa pienamente rinnovabile e ampiamente diffusa. Diviene dunque chiaro come costruire con un materiale del tutto naturale come il legno non solo sia fattibile, ma rappresenti una nuova filosofia e sfida volta alla sostenibilità e alla elevata capacità prestazionale degli edifici.

## 2.1 Sistema costruttivo a pareti portanti intelaiate

Il sistema strutturale degli edifici multipiano in legno a pannelli intelaiati viene definito come un sistema a pareti. A differenza dei sistemi a telaio infatti, gli elementi primari per la resistenza sia ai carichi verticali che orizzontali della struttura, sono costituiti da pareti.



**Figura 2.5:** *Inserimento del materiale isolante all'interno di una parete*

Tali elementi possono essere perimetrali, se posti sui lati esterni dell'edificio, o interni. L'aggettivo intelaiati deriva dalla configurazione interna delle pareti. Quest'ultime sono costituite da un telaio ligneo, composto da montanti ad interasse costante e da due

traversi (superiore ed inferiore). Al fine di garantire la stabilità alle forze orizzontali, si utilizzano uno o due pannelli fissati al telaio ligneo mediante elementi metallici a gambo cilindrico (solitamente chiodi o cambrette). I carichi verticali vengono dunque ripartiti fra un elevato numero di elementi resistenti (montanti) consentendo alle azioni che gravano sulla copertura e sugli orizzontamenti intermedi di distribuirsi secondo aree di influenza ridotte. Grazie a questa caratteristica le sollecitazioni locali non raggiungono livelli elevati e nel complesso viene esaltata la robustezza strutturale. Quest'ultimo aspetto deriva dal fatto che, in caso di crisi di un singolo elemento, i carichi vengono ripartiti su elementi limitrofi evitando, entro limiti ragionevoli, il collasso della struttura.

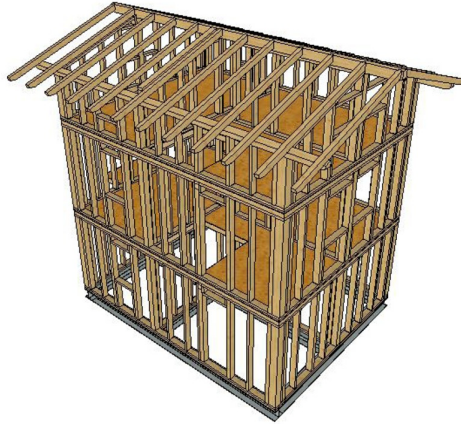


**Figura 2.6:** *Disposizione pareti strutturali*

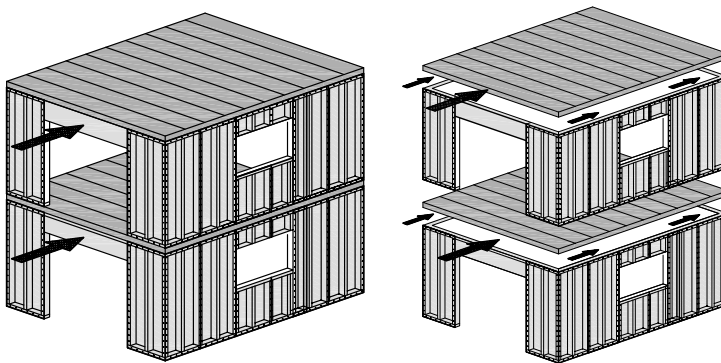
Anche i carichi orizzontali vengono affidati alle varie pareti. Tuttavia, se i carichi verticali sono trasmessi solamente al telaio ligneo costituente ciascuna parete, i carichi orizzontali sono assorbiti dal collegamento pannello-telaio. Senza la controventatura dei pannelli infatti il telaio ligneo costituente la parete risulterebbe labile, non garantendo alcuna rigidezza, resistenza e stabilità della parete strutturale nel proprio piano.

I pannelli di controventamento sono generalmente realizzati mediante pannelli a base di legno truciolari (OSB e pannelli di particelle), pannelli stratificati in legno (pannello compensato) o pannelli in gessofibra. Il collegamento con il telaio ligneo viene realizzato mediante connettori a gambo cilindrico che vengono disposti lungo il bordo del pannello a passo costante. Con il fine di garantire la stabilità del pannello si provvede ad una chiodatura anche sui montanti intermedi. Il comportamento strutturale della parete soggetta ai carichi orizzontali deriva dunque dalla tipologia di connessione utilizzata e dalle relative caratteristiche meccaniche. Gli elementi di solaio vengono solitamente realizzati mediante elementi prefabbricati a sezione scatolare (Figura 2.9)





**Figura 2.7:** *Disposizione dei montanti all'interno dell'edificio*



**Figura 2.8:** *Ripartizione delle forze orizzontali fra le pareti strutturali*

o mediante una semplice orditura di travi in legno. In entrambi i casi si procede ad irrigidire il sistema di solaio mediante un pannello superiore o inferiore opportunamente fissato agli elementi di solaio con il fine di garantire un'adeguata rigidità di piano dell'orizzontamento.

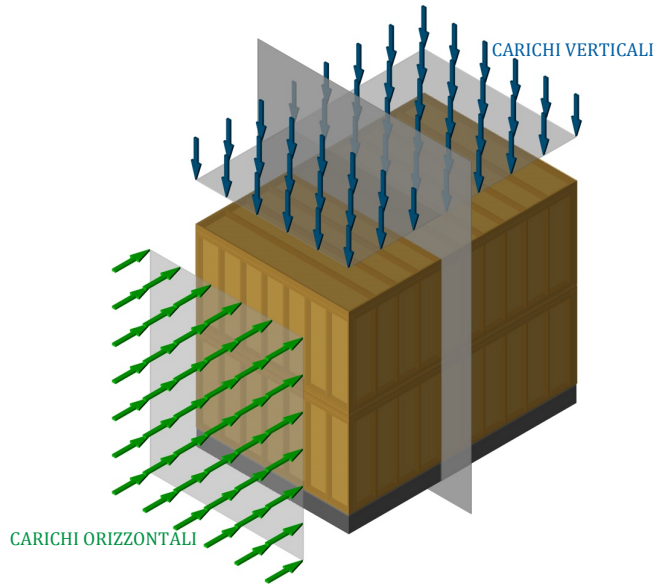


**Figura 2.9:** *Elemento di solaio a sezione scatolare*

Al fine di garantire un'adeguata interazione fra le pareti strutturali, nonché un efficace collegamento degli orizzontamenti alle pareti stesse, vengono utilizzati differenti tipologie di elementi metallici. Tali elementi possono essere costituiti da semplici connettori a gambo cilindrico (quali viti da legno o viti a tutto filetto) o da elementi angolari.

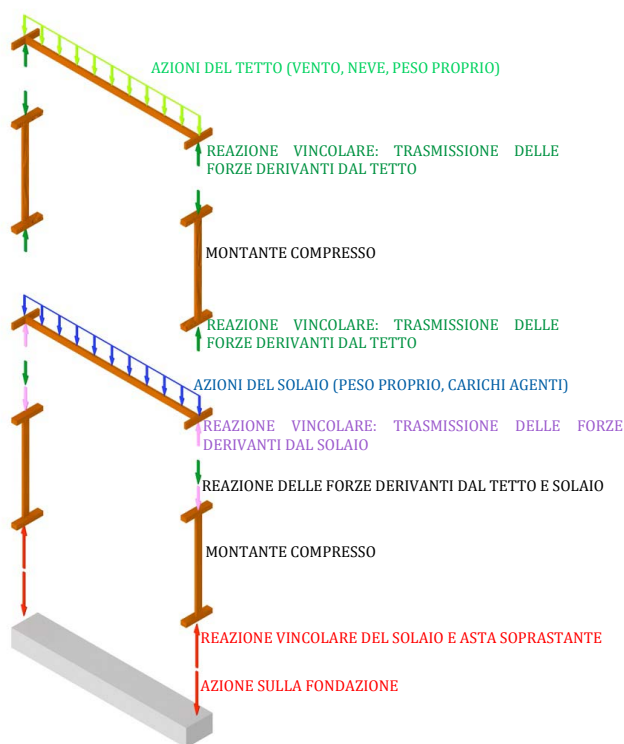
## 2.2 Edifici a pareti portanti intelaiate soggetti a carichi verticali

Un edificio in legno è soggetto sia a forze verticali (peso proprio sovraccarichi, neve) che a forze orizzontali (vento, imperfezioni, sisma), come schematizzato in Figura 2.10. Il comportamento di queste costruzioni sollecitate da carichi verticali è particolarmente vantaggioso perché permette la distribuzione delle forze agenti in maniera uniforme sugli elementi portanti. Facendo riferimento allo schema riportato in Figura 2.11 si può vedere come gli elementi orizzontali (solai di piano, coperture) riportano i carichi ai montanti delle pareti verticali portanti che a loro volta provvedono a scaricarli a terra. Con montanti di sezione limitata si riescono a realizzare costruzioni fino a 3-4 piani. Per costruzioni a più di 4 piani è determinante lo schiacciamento delle fibre degli strati perpendicolari (solai e traversi interposti). I travetti dei solai sono verificati a flessione semplice mentre i montanti delle pareti vengono verificati a compressione semplice e ad instabilità nel piano non controventato dal rivestimento strutturale di parete. Il tra-



**Figura 2.10:** *Edificio a pareti portanti intelaiate soggetto a carichi verticali ed orizzontali*

verso di chiusura della parete deve essere verificato a compressione perpendicolare alle fibre. Per quanto riguarda la trasmissione a terra dei carichi verticali non è necessario alcun particolare accorgimento. Fondamentale è invece prevedere un sistema di separazione efficace nei confronti della risalita dell'umidità fra fondazione in calcestruzzo e parete in legno. Questa risalita solitamente è contrastata mediante la posa in opera di un telo plastico impermeabile sotto il traverso di base della parete. La contemporanea presenza di più elementi delegati a resistere alle azioni verticali permette ai carichi di avere molteplici percorsi per la scesa a terra. Di conseguenza queste costruzioni sono dotate di una robustezza strutturale elevata.



**Figura 2.11:** *Trasmissione dei carichi verticali dalla sommità dell'edificio alle fondazioni*



## Capitolo 3

# Edifici di legno in zona sismica

### 3.1 Legno e sisma

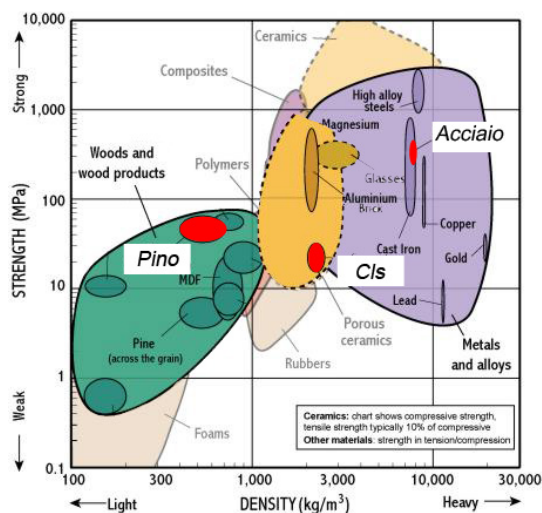
Il materiale legno presenta naturalmente delle caratteristiche utili a renderlo molto adatto ad essere utilizzato per realizzare costruzioni in grado di resistere alle sollecitazioni sismiche. La principale caratteristica che fa sì che il legno sia un materiale vincente dal punto di vista sismico è la sua leggerezza in rapporto alle qualità meccaniche di resistenza, se paragonato agli altri materiali da costruzione come il calcestruzzo o l'acciaio. Facendo riferimento ad esempio ai parametri di peso specifico e di modulo di resistenza a rottura per compressione i valori e i rapporti ottenuti per legno, acciaio e calcestruzzo sono quelli di Tabella 3.1.

<b>Materiale</b>	$\rho[\frac{KN}{m^3}]$	$f_c[MPa]$	$\frac{f_{cd}}{\rho}$
Legno lamellare GL24h	0,38	11,5	30,2
Calcestruzzo	2,4	15,5	6,5
Acciaio Fe430	7,8	250	32,1

**Tabella 3.1:** Resistenza specifica dei materiali comunemente usati in edilizia

Il legno e l'acciaio hanno resistenza specifica molto simile, mentre il calcestruzzo ha un valore circa 5 volte più basso. Questo importante fatto è messo in luce anche in Figura 3.1.

Si vede che a parità di resistenza meccanica ottenibile il calcestruzzo ha una massa di 5 volte superiore. Per quanto riguarda l'acciaio sono invece i problemi di instabilità



**Figura 3.1:** Grafico resistenza-peso specifico dei materiali comunemente usati in edilizia

a far sì che anche per questo materiale le masse in gioco siano superiori a quelle del legno.

Se ora immaginiamo, semplificando di molto il fenomeno, che il sisma provochi sulla struttura una forza proporzionale alle masse in gioco, così come dice la seconda legge di Newton,  $F = m \cdot a$ , su una costruzione di legno le forze generate dal sisma saranno sicuramente inferiori rispetto ad una costruzione realizzata con acciaio o calcestruzzo. Oltre a questi fattori il legno ha anche ottime capacità di resistenza a trazione oltre che a compressione che lo rendono adatto a resistere a forze che invertono il loro senso di applicazione ciclicamente. Tale caratteristica è invece uno dei problemi maggiori per il calcestruzzo che a fronte di un buon comportamento a compressione, mostra una pessima prestazione se sottoposto a sforzi di trazione.

Un'altra importante caratteristica che rende il legno adatto all'uso in zone sismiche è la spiccata dipendenza delle sue caratteristiche meccaniche dalla durata del carico. Infatti per carichi istantanei, come appunto quelli sismici, le proprietà di resistenza hanno un incremento di circa il 10% del valore corrispondente a carichi di durata 5 minuti e di circa il 50% per carichi di lunga durata. Quanto detto vale anche per il modulo elastico che per carichi istantanei ha un incremento di circa il 20% rispetto a carichi di lunga durata.

A fronte degli aspetti positivi sopra elencati va comunque citata una proprietà molto

negativa che caratterizza il materiale legno: la fragilità intrinseca. Tale caratteristica si manifesta soprattutto per sforzi di trazione dove la rottura avviene essenzialmente senza alcun preavviso. La fragilità è dovuta ai difetti naturali, come i nodi o la deviazione di fibratura, inevitabilmente presenti in maniera proporzionale all'aumentare del volume di materiale utilizzato. Questo aspetto renderebbe il legno poco adatto ad essere utilizzato in zona sismica poiché non permette la dissipazione energetica e lo sviluppo della plasticità, proprietà richieste ai materiali nell'approccio moderno all'ingegneria sismica. Nei moderni codici progettuali si distinguono due tipi di sismi: uno associato allo stato limite di danno con probabilità di occorrenza del 50% in 50 anni, che può causare sulla struttura danni di lieve entità, e uno associato allo stato limite ultimo con un probabilità di occorrenza del 10% in 50 anni, che danneggia fortemente la struttura provocando in essa grandi deformazioni senza causarne il crollo. Per resistere a questo ultimo tipo di sisma i materiali devono avere sufficienti capacità plastiche per poter far fronte alle grandi deformazioni richieste. Il materiale legno di per sé non è in grado di adempiere a questa necessità perciò lo studio e la ricerca negli ultimi anni in questo settore si sono sempre più mossi alla ricerca di soluzioni in grado di risolvere questo problema. La soluzione individuata è stata quella di concentrare le risorse plastiche della struttura nelle giunzioni che, per quanto detto, dovranno essere necessariamente duttili.

Il comportamento duttile dei giunti si ottiene attraverso l'utilizzo di elementi meccanici di connessione a gambo cilindrico, come chiodi, viti e bulloni. Tali connettori posseggono uno spiccato comportamento anelastico che si manifesta attraverso la formazione di cerniere plastiche al proprio interno e con plasticizzazioni localizzate nelle fibre legnose. La struttura può quindi contare su inaspettate risorse plastiche solo se la progettazione viene fatta in modo tale da permettere ai nodi di sviluppare tutta la duttilità richiesta prima che le membrature fragili raggiungano il loro punto di rottura. In questo senso la usuale teoria del *Capacity Design* è qui capovolta. Infatti non si dovranno progettare giunti forti e membrature deboli, così come suggerito per calcestruzzo e acciaio, ma bensì travi e colonne sovraresistenti rispetto ai nodi.



## 3.2 Edifici a parete portante intelaiata soggetti a eventi sismici

La tipologia costruttiva in esame risulta particolarmente adatta ad essere utilizzata per la costruzione di edifici in grado di resistere a severi eventi sismici. In essa infatti si sommano le naturali qualità del materiali legno ad essere utilizzato in zona sismica ad una serie di altre caratteristiche in seguito descritte. La leggerezza del legno è ulteriormente sfruttata in maniera da creare una costruzione di ridotto peso, derivante dall'assemblaggio di pareti intelaiate leggere sommate a solai anch'essi di limitata massa. La presenza di un numero elevato di pareti nonché un'ideale disposizione degli elementi di collegamento permette in molti casi di ottenere un notevole comportamento scatolare. Il comportamento scatolare permette un'ideale ripartizione delle forze su tutti gli elementi strutturali presenti in maniera proporzionale alle loro capacità, evitando pericolose concentrazioni di sforzi derivanti da una non ideale configurazione strutturale. Le forze vengono trasmesse dagli orizzontamenti in maniera omogenea alle pareti, le quali, con l'ausilio dei sistemi di collegamento, le trasmettono alla struttura di fondazione e dunque al terreno. Un'elevata capacità di dissipare l'energia durante gli eventi di grande intensità dovuta alla presenza di un elevato numero di elementi metallici di piccole dimensioni in grado, se ben progettati, di esibire buone capacità duttili permettono alla struttura di danneggiarsi e dunque dissipare energia, senza giungere a collasso. Nell'ottica della tradizionale strategia di progettazione sismica di una struttura, i collegamenti metallici vengono assunti come fusibili strutturali o elementi di sacrificio. In altre parole si accetta lo snervamento degli elementi metallici con il fine di preservare gli altri elementi strutturali in legno, la cui rottura potrebbe causare il collasso dell'intera struttura. Il meccanismo di trasferimento delle forze generate dal sisma in fondazione è riassunto in Figura 3.2.

Prima di descrivere sinteticamente i sistemi di connessione che caratterizzano il sistema strutturale in esame è necessario definire le forze che sollecitano le strutture di elevazione, i solai e i relativi meccanismi di collasso. Riferendosi alle sole azioni orizzontali, in quanto, come visto i carichi verticali vengono portati a terra direttamente dallo scheletro della parete, si possono definire quattro meccanismi principali:

- il primo meccanismo è quello che coinvolge la connessione tra pannellatura controventante e telaio ligneo, per un azione di taglio puro (Figura 3.3). Questo

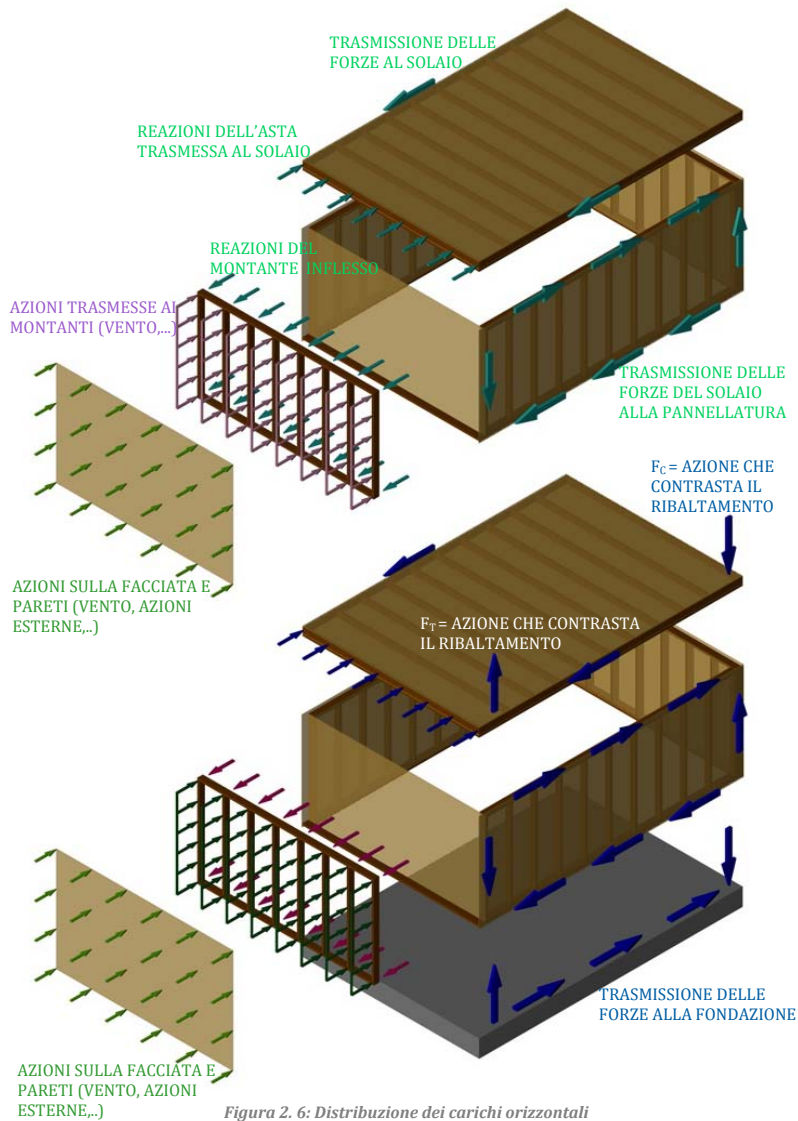
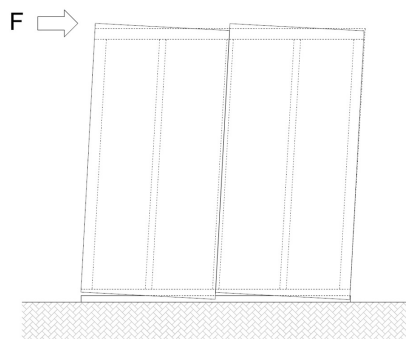


Figura 2. 6: Distribuzione dei carichi orizzontali

**Figura 3.2:** *Trasferimento delle forze orizzontali in fondazione*

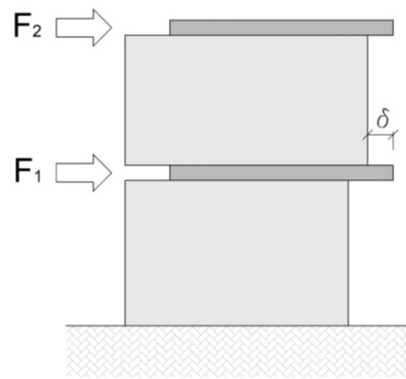
tipo di rottura è caratterizzato da una buona duttilità globale poiché coinvolge un gran numero di elementi metallici distribuiti uniformemente;

- il secondo meccanismo si presenta quando i solai, in corrispondenza dei quali è



**Figura 3.3:** *Rottura della connessione tra pannellatura controventante e telaio ligneo*

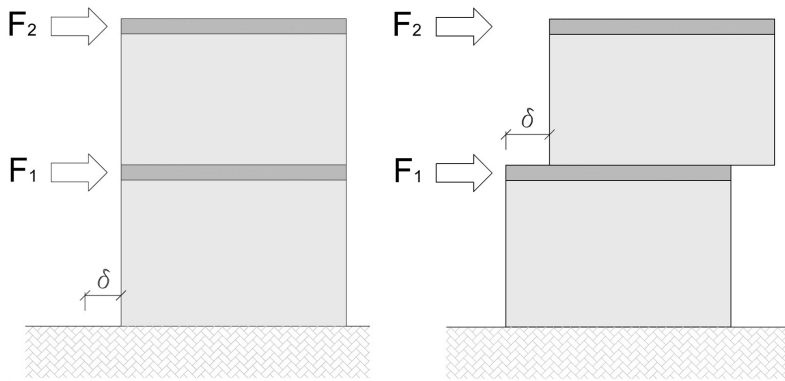
applicata la forza orizzontale di piano, traslano rigidamente rispetto alle pareti che li sostengono (Figura 3.4);



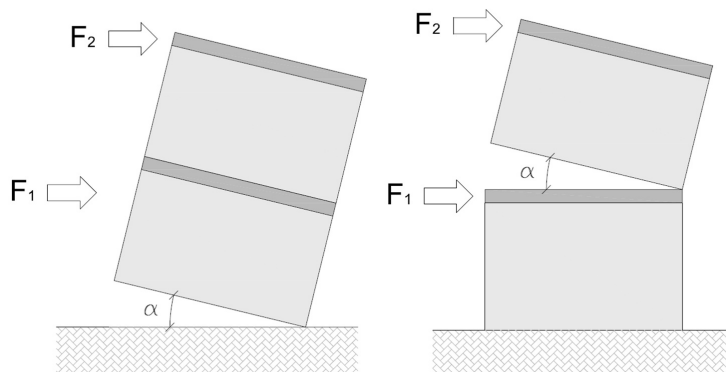
**Figura 3.4:** *Rottura connessione parete-solaio*

- il terzo meccanismo riguarda la traslazione rigida delle pareti, con conseguente scorrimento relativo rispetto alla struttura di fondazione o ai solai di interpiano (Figura 3.5);
- il quarto meccanismo si sviluppa mediante la rotazione rigida in blocco di tutta la costruzione o di una parte disposta al di sopra di una certa quota (Figura 3.6).

Le soluzioni sviluppate per contrastare i meccanismi di collasso sopracitati e garantire una risposta globale della struttura sono molteplici. Sovente vengono adottate delle



**Figura 3.5:** *Rottura per traslazione rigida delle pareti*



**Figura 3.6:** *Rottura per rotazione rigida delle pareti*

soluzioni standardizzate particolarmente apprezzate per l'efficacia meccanica e per la facilità di messa in opera. In quest'ottica, in merito alla possibile traslazione dei solai di interpiano (o di copertura), vengono generalmente disposte una serie di viti da legno che solidarizzano l'estremità dell'elemento orizzontale con quella dell'elemento verticale (Figura 3.7).



**Figura 3.7:** *Collegamento pareti-solaio mediante viti da legno*

Al fine di evitare lo scorrimento delle pareti rispetto agli orizzontamenti sottostanti vengono solitamente utilizzate piastre metalliche forate e chiodate alle due pareti (superiore ed inferiore), assorbendo le forze taglienti che tendono a far scorrere le pareti (Figura 3.8). L'uso di un elemento piatto esterno alla parete velocizza e facilita il montaggio non dovendo realizzare in cantiere intagli passanti attraverso il solaio per l'alloggiamento di piastre o angolari. Allo stesso modo, al fine di assorbire la forza di trazione derivante dal meccanismo di rotazione rigida dei piani superiori dell'edificio, vengono impiegati nastri forati chiodati (tie-down) in corrispondenza dei montanti delle pareti sovrapposte.

Le soluzioni indicate si riferiscono al caso di solaio di interpiano, ma concettualmente non differiscono da quelle adottate per il fissaggio delle pareti del piano terra alla struttura di fondazione. Come nel caso delle pareti ai piani superiori è necessario garantire la stabilità nei confronti della traslazione e della rotazione rigida della parete. A tale scopo si adottano angolari metallici e hold-down chiodati alla parete e tassellati alla struttura di base (Figura 3.9), generalmente in cemento armato. Ai primi è delegata la resistenza nei confronti dell'azione tagliante mentre ai secondi è affidato il compito di trattenere la base della parete a contatto con la fondazione. Una soluzione alternativa per il contrasto alla traslazione orizzontale è ottenuta solidarizzando la parete



**Figura 3.8:** *Elementi angolari per la stabilità a scorrimento e ribaltamento delle pareti superiori*

alla soglia di tracciamento (Figura 3.10). In questo modo impiegando viti inclinate o angolari chiodati ai due elementi in legno si ottiene una valida alternativa.



**Figura 3.9:** *Elementi angolari per la stabilità a scorrimento e ribaltamento delle pareti a piano terra*

Spesso, per garantire una maggiore durabilità e per velocizzare le operazioni di montaggio, le pareti non vengono direttamente fissate a terra ma vengono poste al di sopra di soglie di tracciamento. Queste sono realizzate con profili in legno massiccio posati al di sopra di una guaina impermeabile ed ancorate alla struttura di fondazione mediante tasselli.

E' bene sottolineare come le alternative che possono essere adottate risultano molteplici. Per tale ragione non risulta importante descrivere tutte le possibili soluzioni, bensì evidenziare la loro funzione e lo scopo per cui devono essere concepiti. Da quanto espo-



**Figura 3.10:** *Soglie di tracciamento*

sto dovrebbe risultare chiaro come il comportamento di un edificio in legno a pareti intelaiate dipenda non solo dagli elementi strutturali principali, quali pareti e solai, ma anche dall'interazione fra gli elementi stessi garantita dai sistemi di connessione. La robustezza della costruzione è inoltre ulteriormente garantita dai collegamenti d'angolo fra le pareti ortogonali. Tale vincolo costituisce un presidio per le forze fuori piano dovute all'effetto del vento o alle forze di instabilizzazione laterale. Essi inoltre danno un ulteriore contributo di resistenza nei confronti delle forze ribaltanti generate dalle forze orizzontali applicate in sommità alle pareti.

## Capitolo 4

# Campagna sperimentale

La campagna sperimentale qui presentata ha come obiettivo quello di ricavare dati utili alla caratterizzazione meccanica, con particolare riferimento al comportamento sismico, delle costruzioni in legno a pareti portanti intelaiate. A tale scopo è stata quindi programmato un lavoro di ricerca suddiviso in tre diverse fasi. Nella prima fase si è proceduto allo studio della connessione fra pannelli controventanti a base di legno (OSB, gessofibra) e i montanti costituenti le singole pareti portanti. Successivamente si è investigato il comportamento delle connessioni utilizzate per ancorare le pareti a terra e ai solai d'interpiano. Nella seconda fase sono state analizzate delle pareti a grandezza reale realizzate mediante differenti materiali e diversamente connesse a terra. I risultati emersi hanno permesso lo studio delle singole componenti strutturali che costituiscono un intero edificio, dal singolo connettore fino alla parete vera e propria. Al fine di conoscere il comportamento delle componenti stesse all'interno di un edificio reale, e dunque la loro interazione reciproca in caso di sisma, è stata eseguita una prova su tavola vibrante su un edificio in scala reale. Diviene infatti fondamentale nello studio del comportamento d'insieme di una struttura sottoposta a carichi sismici scorporare le singole componenti strutturali, individuarne il comportamento, e successivamente comprendere la loro interazione all'interno della struttura globale. Le prove effettuate presso il Laboratorio Prove Materiali e Strutture dell'Università di Trento (Figura 4.1) sono riassunte in Tabella 4.1. In totale sono stati svolti 50 test monotoni e 38 ciclici.



**Tabella 4.1:** *Test effettuati*

	Monotone	Cicliche
Angolari a taglio	9	7
Pannello-montante	19	15
Hold-down	11	5
Pareti scala reale	11	11

**Figura 4.1:** *Laboratorio prove materiali e strutture Università di Trento*

## 4.1 Test sui sistemi di collegamento

L'obiettivo principale di questo studio è quello di investigare differenti tipologie di connessione fra le pareti intelaiate in legno e la fondazione in calcestruzzo. Sono state analizzate sia le connessioni utilizzate per il trasferimento delle forze orizzontali di taglio (angolari) che le connessioni atte a contrastare le forze di sollevamento verticali (hold-down). In Figura 4.2 è possibile vedere alcune delle connessioni.



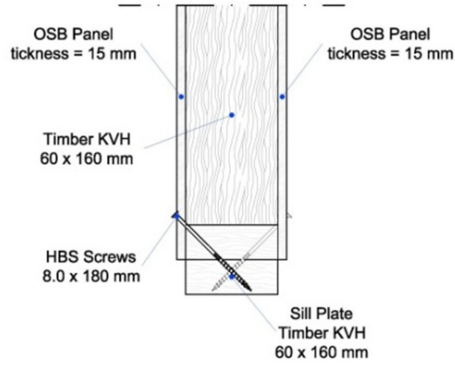
**Figura 4.2:** Angolare a taglio (*sx*) e angolare hold-down (*dx*)

### 4.1.1 Campioni di prova e materiali testati

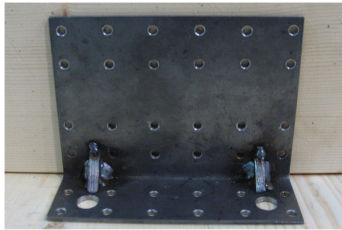
I campioni testati sono stati progettati in modo da simulare le reali condizioni di messa in opera dei vari sistemi. I materiali utilizzati per realizzare i provini sono quelli che usualmente si utilizzano nella realizzazione di questa tipologia di edifici. Le connessioni sottoposte a test sono state quelle disponibili sul mercato. Oltre a queste sono però stati introdotti angolari di nuova concezione, più adatti e più efficaci nel trasferire le forze a terra.

#### Angolari per lo scorrimento

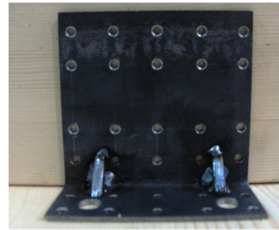
La parete da connettere è stata riprodotta tramite un cassone rigido, formato da un telaio di legno massiccio giuntato di dimensione 60 x 160 [mm] (Figura 4.4) a cui è incollato in maniera pressoché continua sul perimetro di entrambi i lati un pannello di OSB3 dello spessore di 15 mm. L'incollaggio è stato effettuato tramite resina epossidica del tipo XEPOX 70.3.



(a)

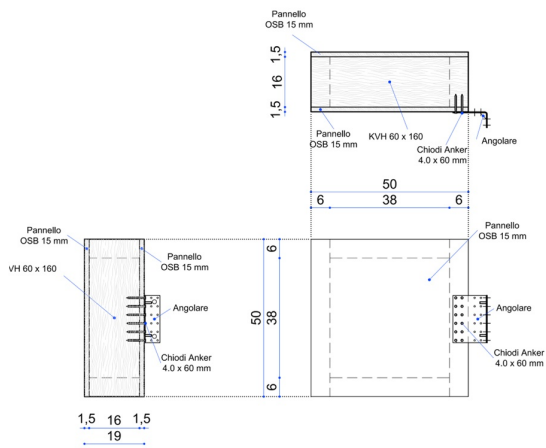


(b)



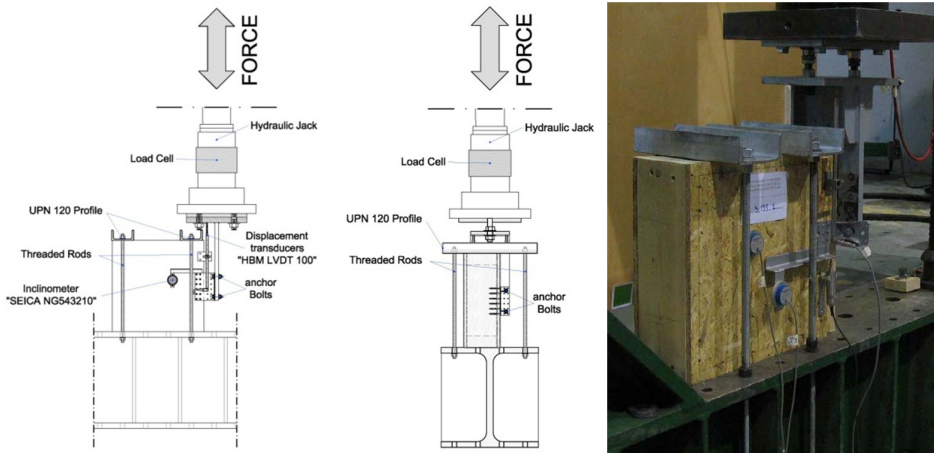
(c)

**Figura 4.3:** Ancoraggio con viti (a) e angolari metallici di nuova concezione (b) e (c).



**Figura 4.4:** Provino per test degli angolari a taglio e particolare di connessione

Solamente nella zona centrale del cassone, dove è stato fissato l'angolare, non è stata messa la colla. Questo per evitare di influenzare il comportamento della connessione. L'angolare testato è poi stato collegato al montante inserito nel cassone nella zona priva di incollaggio tramite chiodi anker 4 x 60 [mm]. Il set-up di prova è stato progettato in modo tale da mettere in evidenza il comportamento della connessione parete-fondazione soggetta alle forze che deve assorbire in fase di utilizzo. La prova è stata condotta in una configurazione nella quale l'attacco parete-fondazione è posto su un piano verticale come visibile in Figura 4.5.



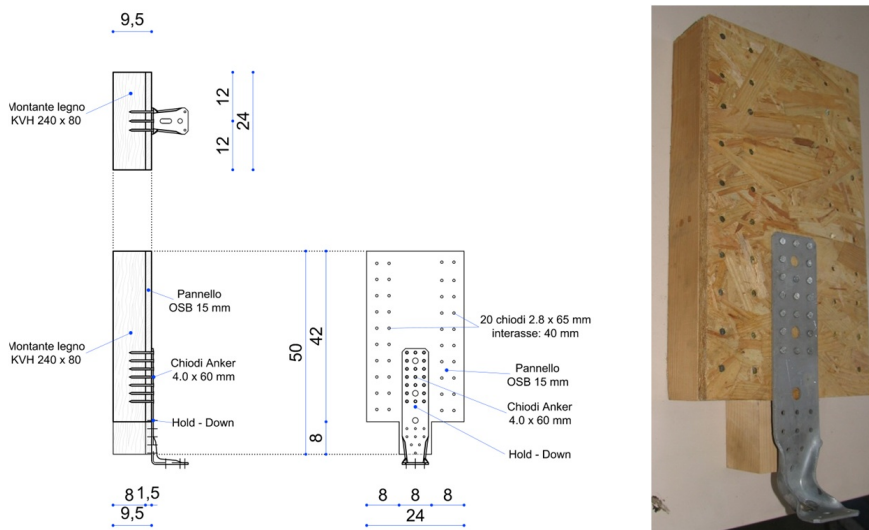
**Figura 4.5:** *Set-up di prova per il test degli angolari a taglio*

Il cassone ligneo, a cui è chiodato l'angolare, è solidarizzato al telaio di prova tramite due profili UPN120 ancorati al telaio con barre filettate M24. La fondazione in calcestruzzo è stata riprodotta tramite una piastra in acciaio dello spessore di 20 mm collegata alla testa del martinetto idraulico tramite bulloni M24. Tra l'elemento a cassone e la piastra, che rispettivamente simulano la parete e la fondazione, è interposto un montante verticale che riproduce la soglia di tracciamento posizionata a terra. Questo accorgimento è stato adottato unicamente nelle prime prove. Si è poi notato che il montante verticale rimaneva fermo senza assorbire alcuna forza. In seguito si è preferito lasciare uno spazio vuoto di 60 mm senza interporre il montante.

### Hold-down

I campioni realizzati permettono la simulazione in laboratorio delle condizioni di normale utilizzo degli angolari, sia dal punto di vista geometrico che tensionale. L'angolare

è solitamente fissato a lato del montante più esterno della parete, al di sopra del foglio strutturale di OSB. Per simulare questo si è utilizzato un montante di legno massiccio giuntato di dimensioni 60 x 240 [mm] a cui è stato chiodato un foglio di OSB dello spessore di 15 mm con chiodi ad aderenza migliorata 2.8 x 65 [mm] come visibile in Figura 4.6.



**Figura 4.6:** *Provino per test degli angolari hold-down e particolare di connessione*

La connessione chiodata è stata inserita per simulare la rigidezza offerta dalla chiodatura fra montante e pannello, che ne impedisce lo slittamento reciproco. Nel montante sono stati realizzati due intagli, per permettere l'aggancio al martinetto e l'applicazione della forza richiesta. Ad alcuni provini non è stato applicato il rivestimento strutturale in OSB, per valutare l'influenza che questo ultimo ha sul comportamento della connessione.

La configurazione dell'apparecchiatura di prova consente l'applicazione di un carico centrato rispetto al montante verticale. Alla testa del martinetto è collegata una trave HEA in acciaio a cui sono fissate tramite bullonatura 4 barre M24. Esse scendono fino alla base del provino, dove vengono fissate a delle piastre di contrasto (Figura 4.7).

L'angolare è poi fissato al provino stesso tramite chiodi anker 4 x 60 [mm] e collegato al telaio di acciaio tramite un bullone di opportuno diametro.

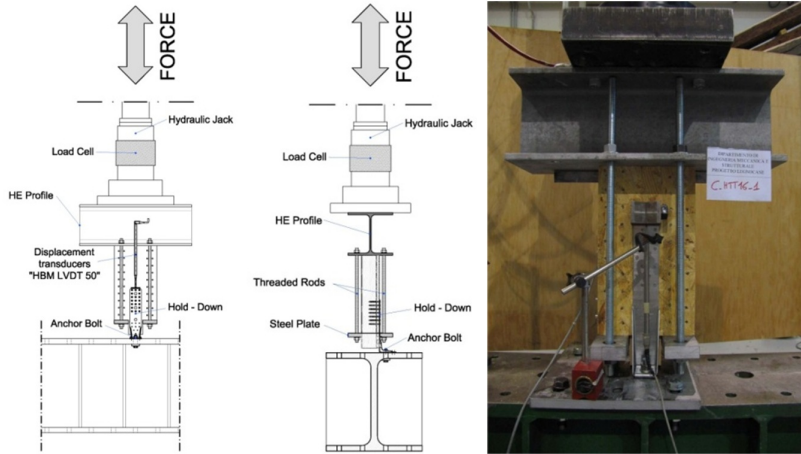


Figura 4.7: Set-up di prova per il test degli angolari hold-down

## 4.2 Risultati dei test

### 4.2.1 Angolari per lo scorrimento

A ogni campione testato è stata dato un nome seguendo il criterio riportato in Tabella 4.2.

Tabella 4.2: Significato sigla provini angolari a taglio

Sigla provino: X _YYYYY _Z		
X	Y	Z
S = static test	135 = 135 Angle Brackets	
C = cyclic test	NEW120 = NEW 120 Angle Brackets	NĂr test
	NEW150 = NEW 150 Angle Brackets	
	SCREW = 4 × HBS 8 × 180 mm Screw	

I risultati ottenuti dai test in termini di forze e rigidezze sono riassunti in Tabella 4.3. Per comprendere in maniera più diretta i risultati e paragonare fra loro i vari sistemi si introducono gli istogrammi seguenti. In Figura 4.8 sono riportate le forze massime raggiunte nei singoli connettori mentre in Figura 4.9 sono riportate le rigidezze dei vari sistemi di unione testati.

Tabella 4.3: Risultati test angolari a taglio

Test	S_135_1	S_135_2	S_NEW120_1	S_NEW120_2	S_NEW150_1	S_NEW150_2	S_NEW150_V_1	S_SCREW_1	S_SCREW_2
Type	135	KR135	NEW120	NEW120	NEW150	NEW150	NEW150	NEW150	NEW150
$t$ [mm]	4	4	3	3	3	3	3	3	3
$b$ [mm]	65	65	120	120	150	150	150	150	150
$H$ [mm]	135	5	109	109	5	5	5	5	5
$v$ [mm]	90	90	50	50	50	50	50	50	50
$\Phi_{holes}$ [mm]	5	5	5	5	5	5	5	5	5
$\Phi_{anchors}$ [mm]	12	12	12	12	12	12	12	12	12
$n_{anchors}$	1	1	2	2	2	2	2	2	2
$r/b$	no	no	yes	yes	yes	yes	yes	yes	yes
Type	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
$\Phi_{connector}$ [mm]	4	4	4	4	4	4	4	4	4
$L_{connector}$ [mm]	60	60	60	60	60	60	60	60	60
$n_{connector}$	8	5	10	10	12	12	12	12	12
$F_{connector}$ [kN]	1,22	1,43	1,59	1,68	1,8	1,9	2,34	2,34	2,34
$k_s$ [N/mm]	440	319	2088	1279	3411	4081	3108	3108	3108
Type	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
$F_{Max}$ [kN]	9,72	7,16	15,85	16,8	21,6	22,84	28,08	12,76	13,88
$v_{Fmax}$ [mm]	28,8	30	25,32	28,81	19,84	17,55	10,66	30	30
$F_u$ [kN]	9,72	7,16	15,85	16,8	21,6	22,84	22,46	12,76	13,88
$v_u$ [mm]	28,8	30	25,32	28,81	19,84	17,55	15,09	30	30
Type	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
$\Phi_{connector}$ [mm]	4	4	4	4	4	4	4	4	4
$L_{connector}$ [mm]	60	60	60	60	60	60	60	60	60
$n_{connector}$	8	5	10	10	12	12	12	12	12
$F_{connector}$ [kN]	1,22	1,43	1,59	1,68	1,8	1,9	2,34	2,34	2,34
$k_s$ [N/mm]	440	319	2088	1279	3411	4081	3108	3108	3108
Type	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
$F_{Max}$ [kN]	9,72	7,16	15,85	16,8	21,6	22,84	28,08	12,76	13,88
$v_{Fmax}$ [mm]	28,8	30	25,32	28,81	19,84	17,55	10,66	30	30
$F_u$ [kN]	9,72	7,16	15,85	16,8	21,6	22,84	22,46	12,76	13,88
$v_u$ [mm]	28,8	30	25,32	28,81	19,84	17,55	15,09	30	30

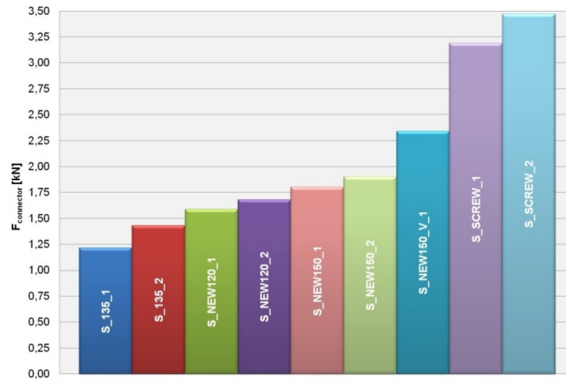


Figura 4.8: Forze massime raggiunte nei singoli connettori

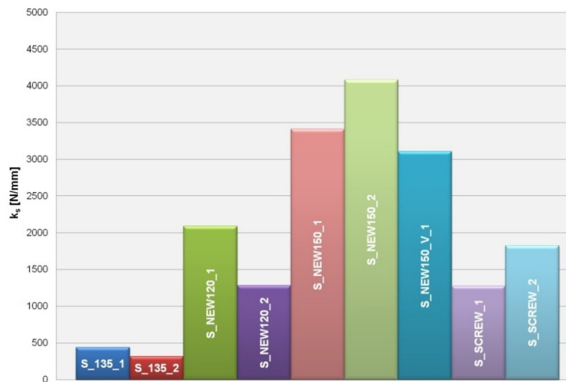


Figura 4.9: Rigidezze dei vari sistemi di unione testati



Per meglio comprendere il comportamento meccanico degli angolari a taglio sono stati sviluppati due modelli di calcolo le cui schematizzazioni sono riportate nel seguito.

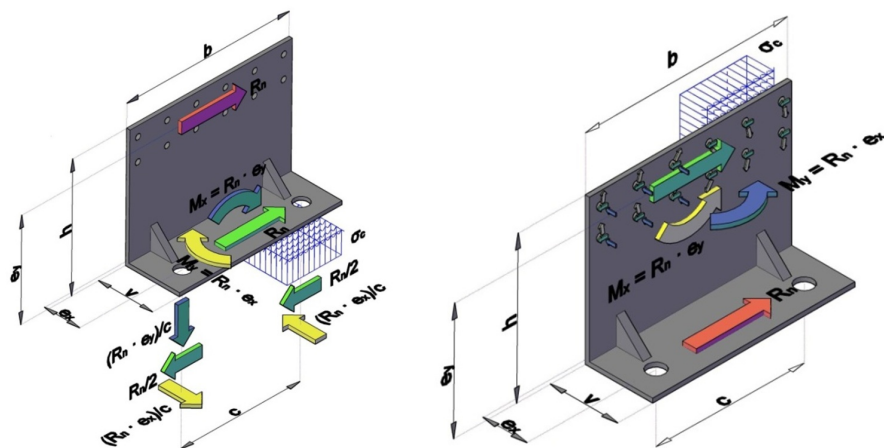


Figura 4.10: Modelli ingegneristici per angolari a taglio

Il modello di sinistra prevede l'applicazione di una forza orizzontale in corrispondenza del baricentro dei chiodi infissi nella parete. Tale schema è il più severo per il calcolo dei tasselli da utilizzare nella fondazione di calcestruzzo. Il modello di destra invece prevede l'applicazione di una forza orizzontale in corrispondenza dei tasselli al piano di fondazione. Con tale schema la chiodatura alla parete risulta più sollecitata. Entrambi questi schemi evidenziano come la geometria della connessione ed in particolare l'ampiezza dei bracci di leva orizzontali  $c$  e verticali  $e_y$ , oltre alla eccentricità  $e_x$  siano fondamentali per il buon funzionamento dell'angolare. Un buon angolare avrà  $c$  molto grande e le eccentricità  $e_x$  ed  $e_y$  ridotte al minimo. Le diverse modalità di rottura che attivano differenti geometrie sono riportate in Figura 4.11 e in Figura 4.12. Un angolare alto e stretto sarà poco rigido e la connessione chiodata si comporterà come una raggiera (Figura 4.11). Un angolare largo e basso invece esibirà un buon comportamento a taglio con elevate rigidzze (Figura 4.12).

## 4.2.2 Hold down

A ogni campione testato è stata dato un nome seguendo il criterio riportato in Tabella 4.4.

I risultati ottenuti dai test in termini di forze e rigidzze sono riassunti in Tabella 4.5.

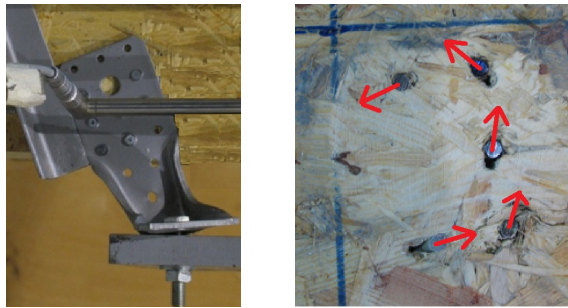


Figura 4.11: *Comportamento connessione a raggiera*



Figura 4.12: *Comportamento connessione a taglio puro*

Tabella 4.4: *Significato sigla provini hold-down*

Sigla provino: X _ YYYYYY _ Z		
X	Y	Z
S = monotonic test	HD16 = Hold-down type 16	NĂr test
C = cyclic test	HD285 = Hold-down type 285	
	HDRP = Hold-down type 340	
	HDRG = Hold-down type 620	

Tabella 4.5: Risultato test hold down

Test	S_HD16_1	S_HD16_2	S_HD16_W_2	S_HD16_W_3	S_HD285_1	S_HD285_2	S_HD285_3	S_HDRP_1	S_HDRP_2	S_HDRG_1	S_HDRG_2
<b>Model</b>	16	16	16	16	285	285	285	RP	RP	RG	RG
$t$ [mm]	2,5	2,5	2,5	2,5	4	4	4	3	3	3	3
$b$ [mm]	65	65	65	65	65	65	65	60	60	80	80
$h$ [mm]	406	406	406	406	285	285	285	340	340	620	620
$v$ [mm]	61	61	61	61	90	90	90	63	63	83	83
$\phi_{holes}$ [mm]	4,7	4,7	4,7	4,7	5	5	5	5	5	5	5
$n_{holes}$	2	2	2	2	3	3	3	3	3	2	2
<i>Thickness</i>	-	-	-	-	-	-	-	56 x 50 x 10	56 x 50 x 10	77 x 70 x 20	77 x 70 x 20
<i>NelArco</i>	139	139	139	139	200	200	200	135	135	210	210
<i>Steel</i>	S275	S275	S275	S275	S235 JR	S235 JR	S235 JR	S355	S355	S355	S355
$f_y$ [MPa]	275	275	275	275	235	235	235	355	355	355	355
$f_u$ [MPa]	430	430	430	430	360	360	360	510	510	510	510
<b>Connector</b>											
<i>Model</i>	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
$\phi_{connector}$ [mm]	4	4	4	4	4	4	4	4	4	4	4
$L_{connector}$ [mm]	60	60	60	60	60	60	60	60	60	60	60
$R_{connector}$	18	18	18	18	19	19	19	20	20	52	52
$F_{connector}$ [kN]	3,04	3,00	2,79	2,75	2,41	2,48	2,36	2,58	2,65	2,04	2,02
<b>Test data</b>											
$F_{tacc}$ [kN]	54,72	55,64	50,28	49,48	45,88	47,04	44,88	51,56	53,08	106,28	105,12
$v/F_{tacc}$ [mm]	24,33	28,92	22,92	23,8	28,76	27,21	28,35	14,96	12,83	18,74	18,96
$F_u$ [kN]	54,72	55,64	50,28	49,48	45,88	47,04	44,88	51,56	53,08	106,28	105,12
$u_h$ [mm]	24,33	28,92	22,92	23,8	28,76	27,21	28,35	14,96	12,83	18,74	18,96
$k_g$ [N/mm]	3886	3041	3155	3041	2223	2942	2837	3424	3424	9276	10045
$\sigma_{acc}$ [MPa]	383,67	400,29	361,73	353,97	229,4	235,2	224,4	381,83	383,19	506,1	500,57
$\frac{\sigma_{acc}}{F_u}$	1,43	1,46	1,32	1,29	0,98	1	0,95	1,08	1,11	1,43	1,41

Negli istogrammi di Figura 4.13 è riportata la forza massima assorbita da ciascun chiodi inserito nell'hold-down mentre negli istogrammi di Figura 4.14 è riportata la rigidezza dell'hold-down. I vari sistemi esibiscono una forza di picco per connettore molto simile così come la rigidezza. Eccezione è fatta dall'angolare HDRG che presenta una resistenza globale più elevata ma una resistenza per connettore inferiore, avvenendo la rottura lato piastra in acciaio. La rigidezza di quest'ultimo hold-down è più che doppia rispetto agli altri testati.

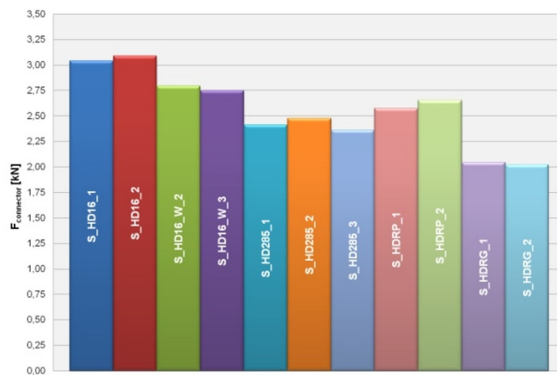


Figura 4.13: Forze massime raggiunte nei singoli connettori

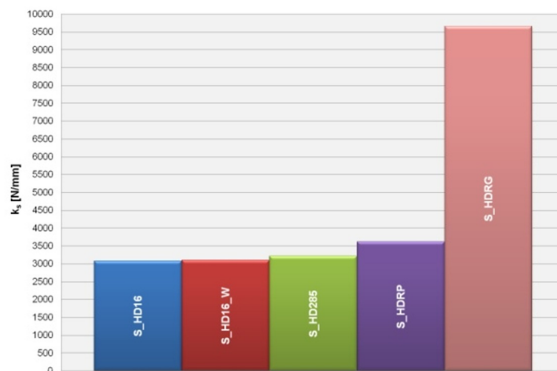


Figura 4.14: Rigidezze dei vari sistemi di unione testati

### 4.3 Prove sui sistemi di collegamento pannello-montante

L'obiettivo principale di questo studio è stato quello di investigare differenti tipologie di connessione fra differenti pannelli strutturali a base di legno e montanti formanti il telaio come visibile in Figura 4.15.

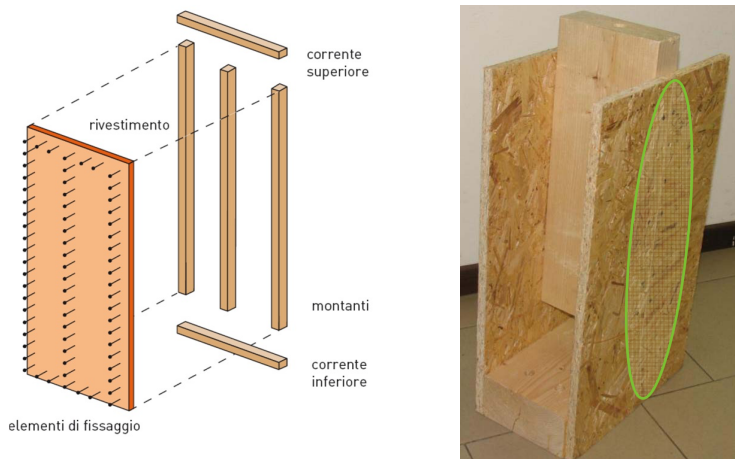


Figura 4.15: Collegamento pannello montante

#### 4.3.1 Campioni di prova e materiali testati

I materiali utilizzati per realizzare i provini sono stati scelti fra quelli che vengono impiegati più frequentemente nei cantieri di case in legno in Italia. Come materiali di rivestimento si è scelto di utilizzare l'OSB e il gessofibra. L'OSB è un pannello a base di legno costituito da diversi strati, a loro volta composti da trucioli di legno prevalentemente lunghi e stretti assemblati mediante un legante. Gli strand degli strati esterni sono paralleli al lato longitudinale o trasversale (Figura 4.16). Un rapporto lunghezza/larghezza degli strand di 10:1 contribuisce a migliorare le proprietà di resistenza a flessione nella direzione dell'orientamento degli strati esterni. Gli strand dello strato interno possono essere orientati in maniera casuale oppure, di norma, in direzione perpendicolare agli strand degli strati esterni.

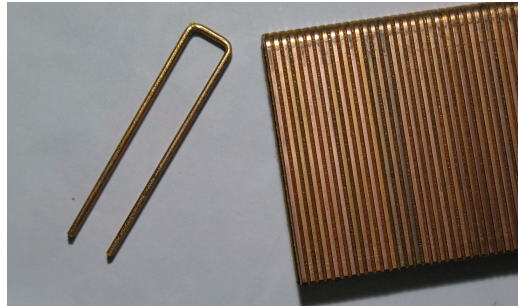
Il pannello in gessofibra è invece formato da una miscela omogenea di gesso e cellulosa ottenuta da carta riciclata selezionata, con aggiunta di acqua (senza altri leganti). L'impasto viene compresso ad alta pressione per formare lastre stabili.

Le tipologie di connettori sono state scelte in base ai pannelli da connettere. La con-



**Figura 4.16:** *Pannelli OSB tipo 3*

nessione dei pannelli di gessofibra al montante viene di norma realizzata con graffe come quelle riportate in Figura 4.17. Nella campagna di prove in oggetto sono state utilizzate graffe  $1,4 \cdot 1,6 \cdot 55$  mm.



**Figura 4.17:** *Cambrette utilizzate per i pannelli in gessofibra*

I chiodi più utilizzati per realizzare la connessione pannello montante con fogli di OSB sono del tipo ad aderenza migliorata (ring) delle dimensioni di  $2,8 - 3,1 \cdot 60$  mm come quelle visibili in Figura 4.18. Tali connettori sono stati infissi nei provini con una chiodatrice a nastro.

Oltre a questa tipologia di connettori sono stati utilizzati anche chiodi lisci standard  $2,5 \cdot 50$  mm e chiodi del tipo a vite  $2,5 \cdot 50$  mm. I campioni testati hanno una struttura costituita da un corrente inferiore e da un montante verticale realizzati in legno massiccio giuntato. Il primo presenta una sezione con dimensioni  $160 \times 80$  mm, mentre il secondo  $160 \times 60$  mm. Il corrente inferiore ed il montante sono collegati tra loro attraverso i pannelli di rivestimento (OSB/gessofibra), ma non sono a diretto contatto uno con l'altro. L'elemento inferiore presenta quattro fori del diametro di 16 mm per l'an-



Figura 4.18: Chiodi Ring

coraggio al telaio di prova. Per attuare l'ancoraggio al martinetto idraulico, l'elemento superiore ha un foro verticale, centrato nella sua sezione, del diametro di 18 mm. La geometria esatta dei provini è riportata in Figura 4.19.

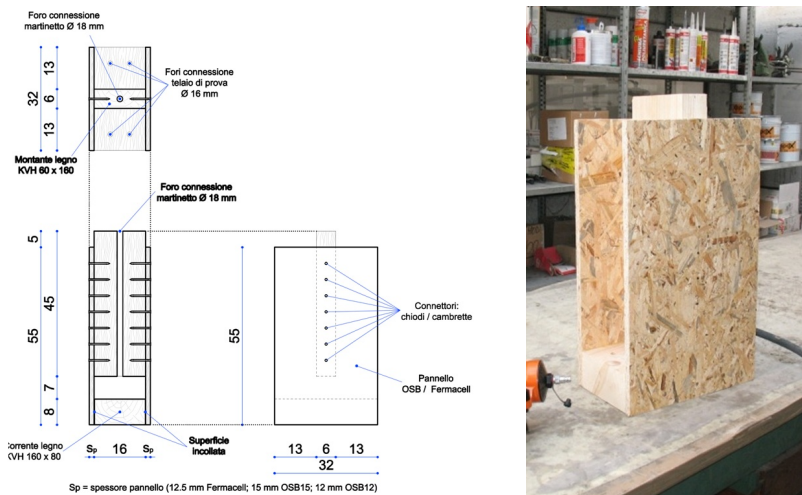


Figura 4.19: Provino per test delle connessioni pannello-montante

Il set-up di prova è stato progettato in modo da mettere in evidenza il comportamento della connessione pannello-montante soggetta a sforzi di taglio puro. Questa condizione di carico è adottata come ipotesi nelle formulazioni proposte dalla maggior parte delle normative per la verifica allo stato limite ultimo. Una rappresentazione schematica della configurazione di prova adottata è riportata in Figura 4.20.

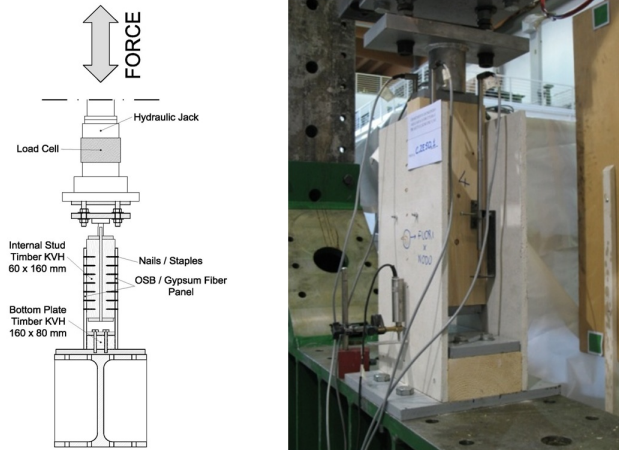


Figura 4.20: Set-up di prova per il test delle connessioni pannello-montante

### 4.3.2 Risultati dei test

A ogni campione testato è stata dato un nome seguendo il criterio riportato nella Tabella 4.6.

I risultati dei test sono riportati nella Tabella 4.7.

Per meglio comprendere quelle che sono le evidenze sperimentali si riportano anche gli istogrammi seguenti in cui è riportata la forza massima applicata ad ogni connettore (Figura 4.21) e la rigidezza di quest'ultimo (Figura 4.22).

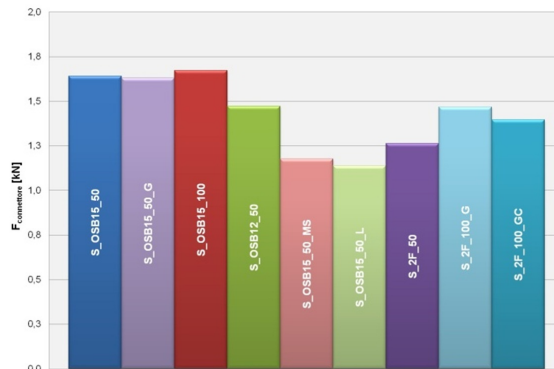


Figura 4.21: Forze massime raggiunte nei singoli connettori

Si può subito notare come i diversi connettori si comportino in maniera sensibilmente diverse. Una prima valutazione può essere effettuata nei confronti dei pannelli



**Tabella 4.6:** *Significato sigla provini connessione pannello-montante*

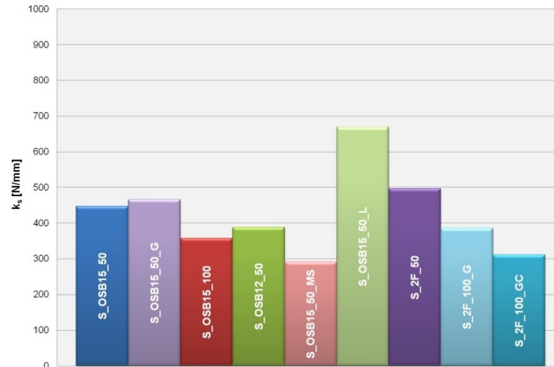
X	YYYY	ZZ	KK
S = monotonic test	OSB15 = OSB3 - 15 mm	50 = spacing mm	MS = screw nails
C = cyclic test	OSB12 = OSB3 - 12 mm	100 = spacing mm	L = smooth shank nails
	2F = gypsum fiber - 12.5 mm		G = joint between panels *
			GC = glued joint between panels**

\* two adjacent sheathing boards connected the same stud

\*\* glued butt joint between two adjacent sheathing boards connected the same stud

**Tabella 4.7:** *Risultati test monotoni*

Test	$F_{max}$	$n_c$	$\frac{F_{max}}{n_c}$	$F_y$	$v_y$	$k_s$
	(kN)	-	(kN)	(kN)	(mm)	(N/mm)
S_OS15_50	22,96	14	1,64	16,65	2,56	447
S_OS15_100	13,38	8	1,67	11,42	4,52	358
S_OS12_50	20,62	14	1,47	13,56	2,16	388
S_FOS15_50	17,34	14	-	13,18	2,07	-
S_2F_50	17,69	14	1,26	15,8	1,69	497
S_OS15_50_MS	16,48	14	1,18	10,66	2,32	292
S_OS15_50_L	15,96	14	1,14	8,27	0,74	669
S_OS15_50_G	45,68	28	1,63	28,36	1,87	466
S_2F_100_G	23,48	16	1,47	18,69	1,9	386
S_2F_100_GC	22,34	16	1,4	20,47	3,88	311



**Figura 4.22:** Rigidezze dei connettori testati

in gessofibra. Nella campagna di prove sperimentali si è visto che il rivestimento in Fermacell, in particolare se sottoposto a trazione, mostra un comportamento spiccatamente fragile. Se si utilizza tale pannello a fini strutturali si ottiene un comportamento buono in termini di resistenza e rigidezza, ma meno duttile rispetto ad altri sistemi. Qualche problema può insorgere nelle zone adiacenti ad aperture o discontinuità, dove il percorso delle forze viene deviato e si ha la possibilità di creare stati di sforzo di trazione. Tali tensioni possono generare fessurazioni premature. Inoltre, il comportamento per forze fuori dal piano è molto scadente. Bassissimi livelli di sollecitazione portano il rivestimento a rottura. Si può inoltre notare che i pannelli di rivestimento in OSB raggiungono il valore di ( $F_{max}$ ) a spostamenti maggiori rispetto ai pannelli di gessofibra (15-16 mm l'OSB contro i 6 mm del gessofibra). Per questo, per pareti intelaiate soggette anche ad azioni orizzontali, computate quindi per resistere ad azioni eccezionali quali il sisma, l'utilizzo di un rivestimento in OSB anziché in gessofibra, garantisce spostamenti maggiori senza rotture precoci del sistema. Nei test di laboratorio si sono sperimentate connessioni con pannelli di rivestimento in OSB da 12 mm e da 15 mm. Le differenze riscontrate hanno fatto emergere un diverso valore di resistenza ultima dei connettori. I test su OSB da 12 mm ( $S\_OSB12\_50$ ) hanno mostrato una resistenza inferiore del 10% per singolo connettore rispetto alle prove su OSB da 15 mm ( $S\_OSB15\_50$ ). In entrambe le tipologie di rivestimento, il valore di  $F_{max}$  si ha per uno spostamento di circa 16 mm.

Durante la preparazione dei campioni testati si è anche notato che l'infissione di chiodi in un rivestimento più sottile quale l'OSB da 12 mm può non dare dei risultati ottimali a causa di un'eccessiva penetrazione della testa del connettore. Questo fenomeno

riduce l'area di contatto tra connettore e OSB.

In seguito a queste considerazioni ed ai risultati ottenuti dai test, si è ritenuto di valutare maggiormente idonei a fini strutturali una parete intelaiata portante realizzata con rivestimento in OSB da 15 mm. Un ultimo aspetto che merita di essere commentato è quello della scelta del connettore da utilizzare per l'ancoraggio del rivestimento al telaio. Non vi è dubbio che le cambrette siano la scelta giusta per collegare i pannelli di gessofibra al telaio: grazie alla loro sezione ridotta riescono a penetrare nel pannello senza generare sforzi ortogonali elevati, tali da poter indurre rotture precoci nel materiale. Allo stesso tempo, le graffe conferiscono duttilità al collegamento. Nella campagna di prove sperimentali si sono testati, oltre ai chiodi ring, chiodi lisci standard e chiodi a vite per l'ancoraggio dei pannelli di OSB al telaio. I connettori lisci, proprio per l'assenza della zigrinatura sul gambo, conferiscono alla connessione una resistenza inferiore del 33% rispetto ai chiodi ring. Una nota particolare caratterizza i connettori a vite: essi presentano una resistenza analoga a quella dei chiodi lisci standard (da essi differiscono del 3%) e un valore di rigidità più basso sia rispetto ai connettori lisci che ai chiodi ring. Dall'analisi della conformazione del chiodo a vite si può desumere che l'idea del produttore fosse quella di creare un chiodo che, grazie al passo elicoidale delle zigrinature sullo stelo, si avvittasse durante l'infissione e conferisse maggior resistenza all'unione. Dai risultati ottenuti si capisce, però, che la lavorazione a vite del gambo del chiodo in realtà non giova, né dal punto di vista della resistenza, né a livello di rigidità.

## 4.4 Prove su pareti in scala reale

L'obiettivo prefissato di questa parte di campagna sperimentale è stato quello di caratterizzare il comportamento meccanico dell'insieme parete-sistema di connessione alla fondazione mediante prove a scala reale. I risultati di questo terzo step sfruttano le sperimentazioni già svolte nei primi due su singoli aspetti della tecnologia a telaio. Il comportamento reale della struttura è condizionato dalla presenza simultanea di molteplici fattori derivanti dalle scelte progettuali e dal contesto in cui è inserita. Per questo motivo il presente studio analizza l'influenza di diverse variabili, che in maniera più o meno significativa determinano i parametri meccanici del sistema, in particolare:

- entità del carico verticale applicato;
- differenti tipologie e spazature per la connessione pannello-telaio ligneo;

- efficacia dei pannelli di rivestimento strutturale esterno (OSB-gessofibra);
- presenza di un foro finestra;
- differenti tipologie di connessione alla fondazione in calcestruzzo;
- presenza del pacchetto di finitura.

I campioni sono stati caricati simulando le reali condizioni di esercizio all'interno di un edificio, ossia tramite una combinazione simultanea di forze verticali e orizzontali. Mediante i diversi tipi di ancoraggio al set-up di prova vengono ricavate una serie di informazioni sull'efficacia dei sistemi di fissaggio. Le prime venti prove presentano la stessa impostazione, essendo tutte vincolate nei confronti dell'azione tagliante e nei confronti dell'azione di ribaltamento. Due prove invece sono state dedicate all'analisi del comportamento della parete in assenza di angolari metallici a contrasto della rotazione rigida della parete (hold down). Il riscontro dei risultati consente l'eventuale validazione del modello senza hold down, nel caso di edifici con numero di piani limitato accettando una maggiore deformazione della parete. Dal punto di vista tecnico, per ridurre l'influenza del disturbo introdotto dagli strumenti necessari all'applicazione dei carichi, si è realizzato un set up di prova con accorgimenti e dispositivi appositamente studiati.

#### **4.4.1 Campioni di prova e materiali testati**

Le dimensioni principali delle pareti sono  $2500 \cdot 2500$  mm, mentre lo spessore varia a seconda del rivestimento strutturale controventante utilizzato. Il telaio è composto da elementi in legno massiccio giuntato. Le sezioni dei montanti esterni e del corrente superiore sono  $100 \times 160$  mm, mentre il corrente inferiore e i montanti intermedi presentano una dimensione standard  $60 \times 160$  mm. I campioni realizzati mostrano tutti il medesimo schema distributivo di montanti intermedi. L'unica tipologia che si discosta è quella con apertura centrale. Gli elementi interni di queste pareti hanno un interasse minore rispetto ai montanti esterni e presentano due elementi aggiuntivi, con sezione  $60 \times 160$  mm, per realizzare le parti orizzontali del foro finestra. Schema distributivo e caratteristiche geometriche sono riportati nella Figura 4.23.

I pannelli di irrigidimento, a seconda dei casi, possono essere in materiale a base di legno (OSB3) con spessore 15 mm o in gessofibra con spessore 12,5 mm. In base ai casi studiati il rivestimento può essere uguale da entrambi i lati o misto. La connessione

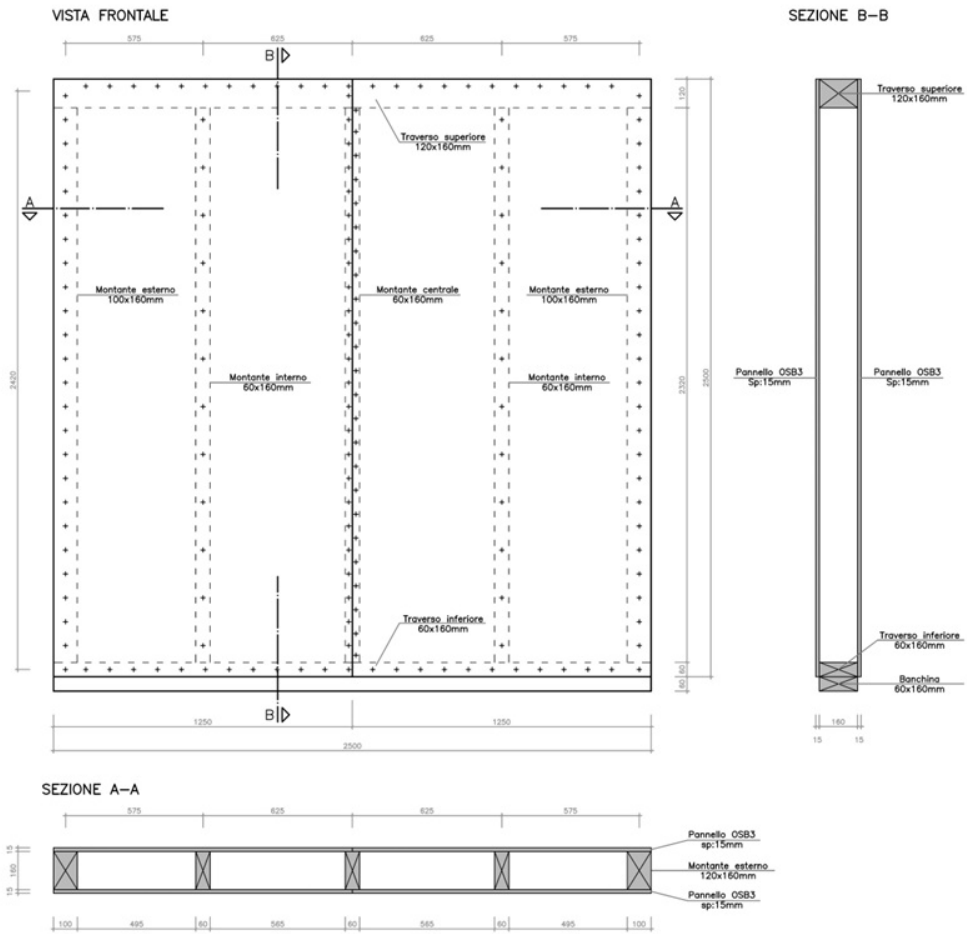


Figura 4.23: Geometria delle pareti testate

tra telaio e pannello nel caso di OSB viene realizzata con chiodi ad aderenza migliorata (ring  $2,8 \cdot 60$  mm), mentre per i pannelli in gessofibra con cambrette  $1,4 \cdot 1,6 \cdot 55$  mm. Il passo della chiodatura rappresenta una delle variabili indagate nei test, poiché costituisce un parametro fondamentale per la resistenza e per la capacità dissipativa della parete.

La chiodatura in ogni caso presenta un passo costante lungo il perimetro esterno dei fogli strutturali (50-100-150[mm]) e una spaziatura doppia sui montanti interni (100-200-300[mm]). La spaziatura in questi ultimi elementi è doppia rispetto a quella adottata sul perimetro, considerato che la loro funzione è unicamente quella di stabilizzare lo sbandamento laterale dei pannelli. Il pacchetto descritto pertanto, rappresenta una componente strutturale che realmente si riscontra nell'edilizia civile sia per dimensioni che materiali (trascurando la mancanza di coibente termico ed eventuali impianti tecnici).

A ogni campione testato è stata dato un nome seguendo il criterio riportato in Tabella 4.8.

Significato sigla provini pareti in scala reale		
XX	YY	ZZ
S = monotonic test	ST = standard wall	L00 = vertical load 00 [kN/m]
C = cyclic test	OP = with opening	50 = nails spacing 50 mm
SC = monotonic test obtained as envelope of cyclic test	2F = gypsum fiber	WHD = without hold down
	FOSB = gypsum fiber and OSB	RIV = wall with finishing

**Tabella 4.8:** *Specimen label significance*

I campioni testati con le relative nomenclature sono riassunti nella Tabella 4.9.

#### 4.4.2 Set-up di prova

La necessità di eseguire test in scala reale, effettivamente aderenti alle condizioni in opera, ha suggerito la realizzazione di un setup di prova appositamente studiato. Lo sviluppo del sistema ha richiesto notevoli sforzi sia dal punto di vista progettuale che

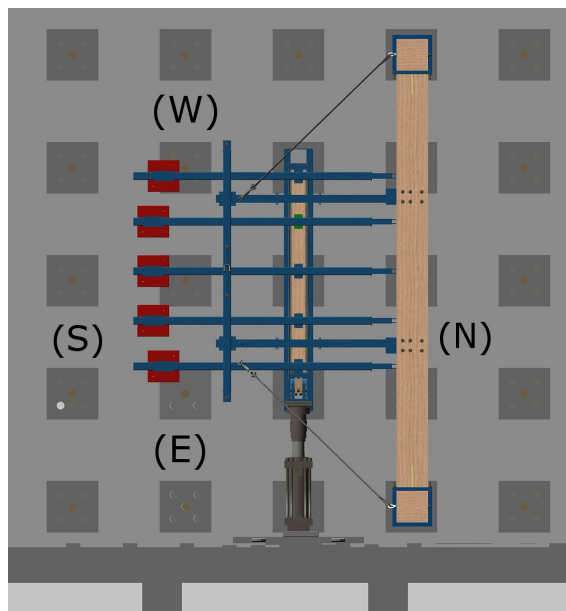
Vertical load	$L : 0[kN/m]$	$L : 10[kN/m]$	$L : 20[kN/m]$
	S_ST_L0	S_ST_L10	S_ST_L20
Nails spacing	Sp:50mm	Sp:100mm	Sp:150mm
	SC_ST_50	S_ST_L20	SC_ST_150
Sheathing panel	OSB	GFB	OSB/GFB
	S_ST_L20	S_2F	S_FOSB
Openings	Standard	window	
	S_ST_L10	S_OP	
Angle brackets		HBS screws	
	New 150		
	S_ST_L20	S_ST_SCREW	
Hold-down	RG	RP	
	S_ST_50_RG	SC_ST_50	
Hold down	H.D. RP	Without hold-down	
	S_ST_L10	S_ST_WHD	
Finishes	Standard	Finishes type 1	Finishes type 2
	S_ST_L20	S_ST_RIV1	S_ST_RIV2

**Tabella 4.9:** *Campioni testati*

costruttivo. Il risultato è un apparecchiatura, adatta alla prova di pareti lignee sia in condizioni monotone che cicliche, in grado di minimizzare gli attriti ed il disturbo dei dispositivi di applicazione dei diversi carichi. La capacità massima di spostamento orizzontale libero consentito dal setup è di 135 mm. Tale valore copre agevolmente il campo di deformazione dei campioni testati considerando i limiti di spostamento massimi a rottura attualmente prescritti dalle normative di prova.

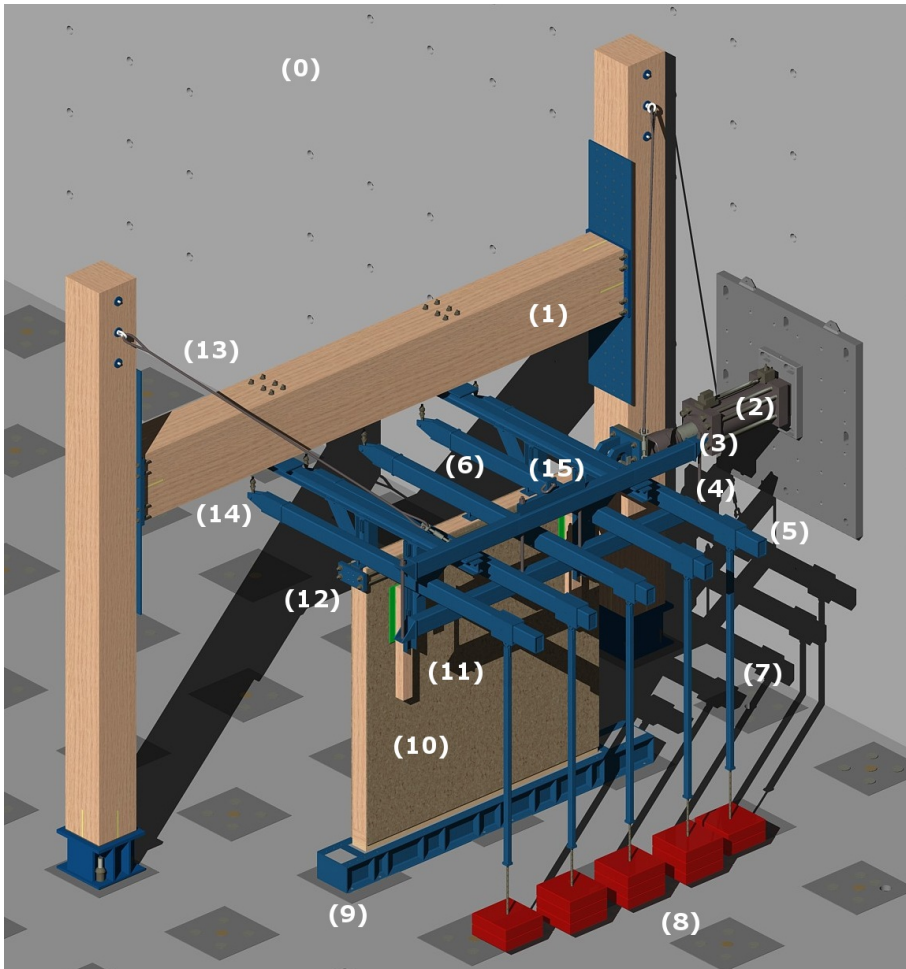
Il setup inoltre è stato progettato per poter testare pareti con differente altezza e con vari spessori (da un minimo di 90 mm ad un massimo di 350 mm).

Di seguito si descrive dettagliatamente il set-up (Figura 4.24 e Figura 4.25) analizzando le differenti componenti.

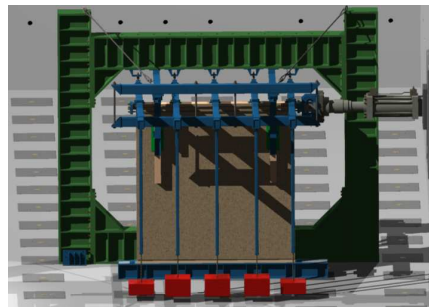
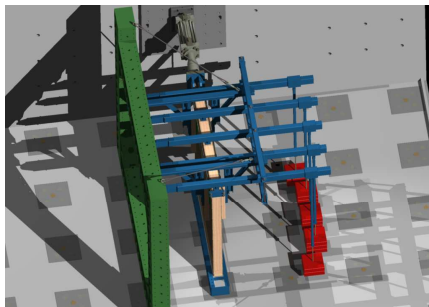


**Figura 4.24:** Vista dall'alto del setup di prova (parete allineata lungo l'asse E-W)





(0)-Muro di contrasto; (1)- Telaio principale; (2)- Martinetto idraulico; (3)- Traversa di sicurezza superiore; (4)- Traversa di sicurezza inferiore; (5)- Manicotti scorrevoli; (6)- Profili tubolari (leve); (7)- Asta di collegamento manicotto-peso; (8)- Piastre di contrappeso; (9)- Base di appoggio della parete; (10)- Parete; (11)- Ritegni torsionali; (12)- Piastra di testa per prove cicliche; (13)- Tiranti dotati di tenditore; (14)- Supporto dotato di cerniera sferica; (15)- Gancio di sollevamento traversa inferiore.



**Figura 4.25:** Modello tridimensionale del set-up di prova



**Figura 4.26:** Immagini del set-up di prova

### Base di appoggio

La funzione di questo componente è simulare la fondazione di calcestruzzo su cui vengono fissate le strutture di elevazione. In laboratorio è stata utilizzata una trave composta da due profili UPN 240 in acciaio tipo Fe430 B distanziati di 20 cm forniti di piatti saldati di chiusura sul fondo e alle due estremità. Le dimensioni complessive della base sono  $3315 \cdot 370 \cdot h : 240$  mm. Tra i due profilati sono saldati 2 supporti di irrigidimento e 8 supporti dotati di una boccola filettata che consentirà il fissaggio della banchina tramite bulloni. Sulle sponde interne inoltre sono state predisposte alcune zanche per migliorare l'aderenza con il getto integrativo che successivamente completerà il blocco di fondazione. Le ali dei profilati nella direzione maggiore sono dotate di una serie di fori destinati all'ancoraggio degli angolari della parete. Sulle ali inferiori invece i fori consentono l'aggancio con barre filettate  $\phi 30$  al solaio attrezzato del laboratorio.

Per rinforzare e rendere maggiormente rigide le ali dei due profilati sono state introdotte su entrambi i lati una serie di costole metalliche. L'adozione di profili metallici consente



**Figura 4.27:** Base cls/acciaio - dettaglio interno boccole

una notevole praticità e versatilità di utilizzo. Gli angolari metallici e gli hold down sono connessi mediante bulloni facilmente rimovibili prova dopo prova senza danneggiare la base. Questo aspetto non influenza in maniera significativa il comportamento della prova rispetto alle strutture reali connesse con bulloni e tasselli per cemento armato (evitando cedimenti lato calcestruzzo).

### Sistemi di stabilizzazione

La parete deve essere stabilizzata per evitare il ribaltamento, assicurando tuttavia una perfetta capacità di scorrimento in direzione della forza orizzontale applicata. La strategia adottata prevede l'utilizzo di ritegni torsionali dotati di "pattini" rivestiti in polizene nella parte di contatto con la parete (Figura 4.28).



**Figura 4.28:** Ritegni torsionali

I ritegni torsionali sono realizzati con profili HE100 saldati secondo un angolo di  $35^\circ$  e dotati all'estremità di flange. Queste consentiranno l'unione con le ali inferiori dei supporti collegati al telaio principale. Per ridurre l'inflessione dei profilati e realizzare

una struttura reticolare spaziale, sono previsti due tiranti in acciaio. Questi sono stati collegati alla traversa del telaio principale ed a ganci orientati predisposti sul lato superiore delle putrelle (Figura 4.29).



**Figura 4.29:** *Fasi di montaggio dei ritegni torsionali*

### **Applicazione del carico orizzontale**

L'applicazione del carico orizzontale rappresenta il punto cruciale della sperimentazione. Il campione verrà sollecitato da una forza orizzontale fornita da un martinetto idraulico fissato alla parete di contrasto del laboratorio. Questo dispositivo oleodinamico presenta dimensioni e peso considerevoli e necessita di idonei sistemi di fissaggio al muro attraverso piastre e ancoraggi presollecati. Durante la prova nel trasferire la pressione di contatto all'elemento testato è necessario considerare lo spostamento relativo introdotto dal movimento del traverso superiore dovuto alla deformazione del telaio. Questa necessità è soddisfatta da un dispositivo che consente notevoli spostamenti verticali. Il nodo centrale è realizzato con un perno  $\phi 55mm$  dotato alle estremità di due cuscinetti a rulli cilindrici che scorrono in una guida laterale. In questo modo il martinetto non viene sollecitato da componenti taglienti potenzialmente dannose e

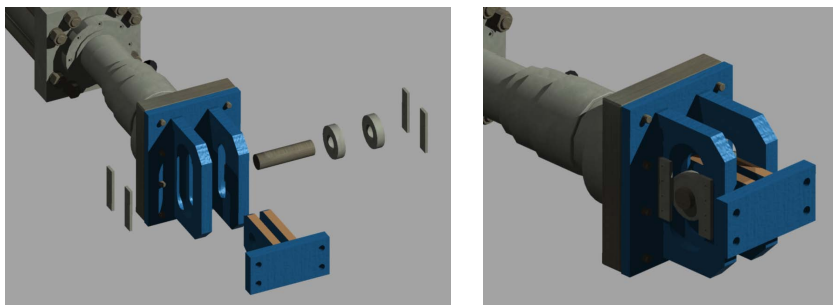
la parete non riceve carichi verticali aggiuntivi. Anche in questo caso non si influenza il comportamento deformativo della parete ne' la modalità di applicazione del carico. Inoltre, per determinare il comportamento in condizioni di utilizzo differenti, il meccanismo consente di applicare sia carichi monotoni che cicli di carico e scarico. Per consentire quest'ultima modalità di prova ed in particolare la fase di scarico, all'estremità opposta della parete è connessa una piastra d'acciaio collegata alla piastra unita alla testa del martinetto tramite quattro barre filettate del diametro di 24 mm.



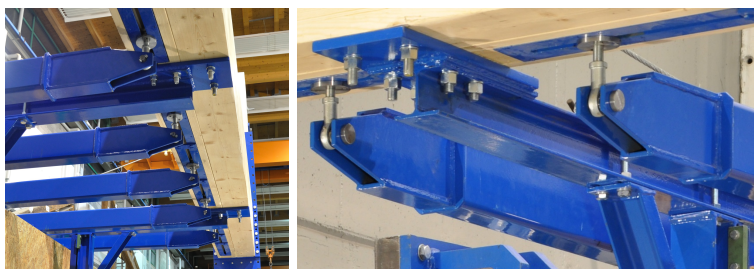
**Figura 4.30:** *Sistema di applicazione carico orizzontale - testa del martinetto*

### Applicazione del carico verticale

L'applicazione del carico verticale è uno degli aspetti che maggiormente contraddistingue questo set-up di prova. La differenza risiede nel sistema di leve che trasmettono la forza al traverso superiore del telaio della parete. Il sistema è composto da profili tubolari dotati alle estremità di un perno per il fissaggio alla traversa orizzontale del telaio principale tramite una cerniera sferica (Figura 4.32).

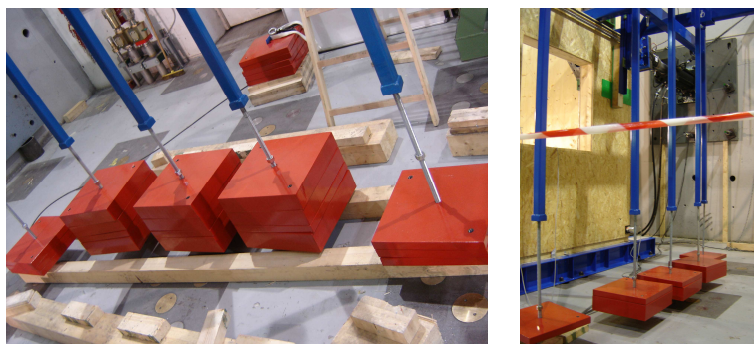


**Figura 4.31:** *Sistema di applicazione carico orizzontale modello tridimensionale*



**Figura 4.32:** *Dettaglio delle cerniere sferiche all'estremità*

Le leve sono abbinare a manicotti scorrevoli di regolazione che consentono un ampio range di variazione del braccio di leva e quindi del carico applicato al livello della parete. I contrappesi sono sospesi ai manicotti tramite aste con lunghezza variabile, che permettono di modificare la distanza da terra. Sfruttando questo sistema si ottiene l'applicazione puntuale del carico in prossimità dell'asse dei montanti della parete senza introdurre attriti significativi durante la prova. L'intensità del carico applicato si mantiene costante durante l'intero svolgimento del test. Posizionando i pesi (Figura 4.33) a pochi centimetri dal pavimento si assicura il massimo livello di sicurezza sia durante lo svolgimento della prova sia nelle fasi di allestimento e smontaggio.

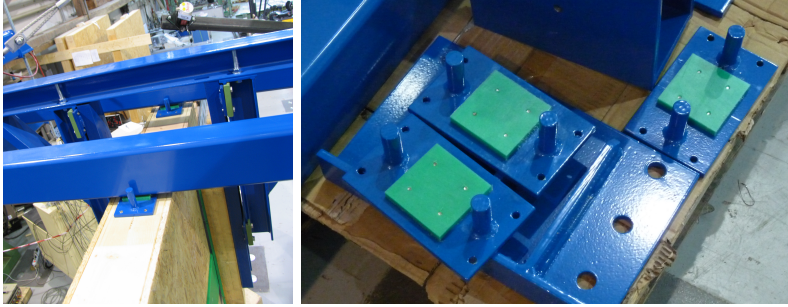


**Figura 4.33:** *Piastre d'acciaio di contrappeso - dettaglio distanza da terra*

In aggiunta, un altro aspetto da non trascurare è la sensibile riduzione della movimentazione delle relative piastre e dei tempi di preparazione della prova.

Per creare una superficie di appoggio uniforme anche con parete in configurazione deformata e per evitare attriti, si interpone tra tubolare e parete una piastra in acciaio con due perni verticali dotata di una superficie di contatto in polizene come visibile in Figura 4.34.

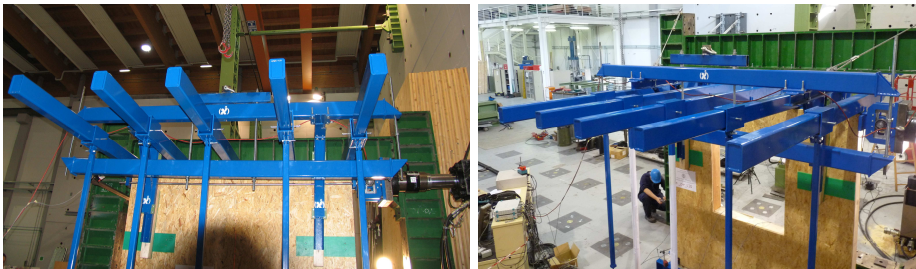
La funzione della piastra è quella di ridistribuire il carico su una superficie prestabilita al cento del traverso superiore della parete, mentre quella dei perni è unicamente di contenere il movimento orizzontale dei tubolari. I limiti sullo spostamento introdotti a causa della complessità del sistema non sono vincolanti in nessun test dato che in fase di progettazione si è assunto un limite massimo di spostamento pari a 100 mm come stabilito dalla normativa.



**Figura 4.34:** *Piastre interposte tra parete e tubolari*

### Traverse di sicurezza

I due profili tubolari che racchiudono i cinque bracci di leva presentano un duplice scopo. Da un lato consentono, tramite aggancio al carroponte, il sollevamento simultaneo di tutti i pesi e dall'altro permettono il fissaggio in sicurezza di tutto il dispositivo di leve al telaio fisso. La traversa superiore è collegata mediante due flange ai profili HE100 che sorreggono il sistema di ritegni torsionali. La traversa inferiore è collegata con quattro barre filettate al profilo superiore. In questo modo durante lo svolgimento della prova questo elemento rimarrà “appeso” senza influenzare lo spostamento verticale ed orizzontale del set-up.



**Figura 4.35:** *Traversa di sicurezza superiore e inferiore*

### 4.4.3 Risultati dei test

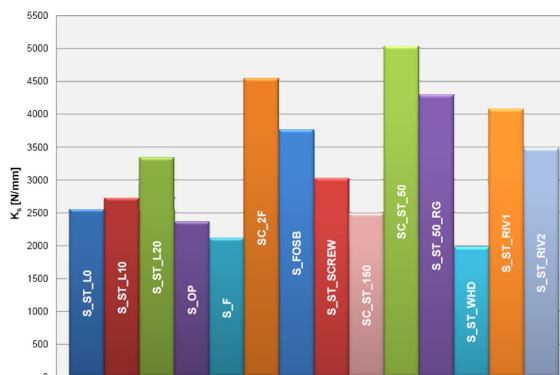
Nella Tabella 4.10 sono riportati i risultati in termini di forze e spostamento raggiunti nei vari test mentre in Tabella 4.11 sono riassunti i valori di rigidezza delle pareti.



	$F_{Max}$	$v_{Fmax}$	$F_u$	$v_u$	$F_y$	$v_y$
$S\_ST\_L0$	60.4	47.2	48.4	69.7	50.7	19.7
$S\_ST\_L10$	70.5	38.8	56.4	68.8	63.1	22.1
$S\_ST\_L20$	77.8	49.7	62.2	86.6	60.2	17.8
$S\_OP$	61.6	71.9	60.6	87.3	51.4	21.4
$S\_F$	22.7	20.5	18.1	44.8	19.1	8.5
$SC\_2F$	48.3	19.8	38.7	27.6	39.6	8.2
$S\_FOSB$	50.7	31.7	40.6	52.8	37.4	9.3
$S\_ST\_SCREW$	69.7	99.7	69.7	99.7	54.8	17.0
$SC\_ST\_150$	62.7	39.7	50.1	59.8	54.6	20.2
$SC\_ST\_50$	84.4	32.1	67.5	55.8	70.3	13.4
$S\_ST\_50\_RG$	121.5	84.4	97.2	88.6	101.9	22.2
$S\_ST\_WHD$	38.9	30.5	31.1	30.6	34.3	16.1
$S\_ST\_RIV1$	82.4	58.2	65.9	66.0	62.5	14.4
$S\_ST\_RIV2$	78.0	54.0	62.4	61.5	63.7	17.7

**Tabella 4.10:** *Tests results*

Per comprendere in maniera più diretta i risultati e paragonare fra loro i vari sistemi si introducono gli istogrammi di Figura 4.36 e Figura 4.37. In Figura 4.36 sono riportate le rigidezze delle pareti testate mentre in Figura 4.37 sono riportate le forze massime raggiunte nei vari test.



**Figura 4.36:** *Rigidezze delle pareti testate*

	$0,1F_{max}$	$0,4F_{Max}$	$v_{0,1}$	$v_{0,4}$	$K_s$
$S\_ST\_L0$	6.0	24.2	2.2	9.3	2550
$S\_ST\_L10$	7.1	28.2	1.6	9.3	2726
$S\_ST\_L20$	7.8	31.1	2.1	9.1	3339
$S\_OP$	6.2	24.7	2.2	10.1	2362
$S\_F$	1.8	7.3	0.5	3.8	2113
$SC\_2F$	4.8	19.3	0.5	3.7	4544
$S\_FOSB$	5.1	20.3	0.8	4.8	3766
$S\_ST\_SCREW$	7.0	27.9	1.2	8.1	3031
$SC\_ST\_150$	6.3	25.1	0.7	8.3	2487
$SC\_ST\_50$	8.4	33.8	1.2	6.2	5034
$S\_ST\_50\_RG$	12.2	48.6	1.3	9.8	4294
$S\_ST\_WHD$	3.9	15.6	0.7	6.6	1980
$S\_ST\_RIV1$	8.2	33.0	1.1	7.2	4079
$S\_ST\_RIV2$	7.8	31.2	1.7	8.4	3480

Tabella 4.11: Tests results

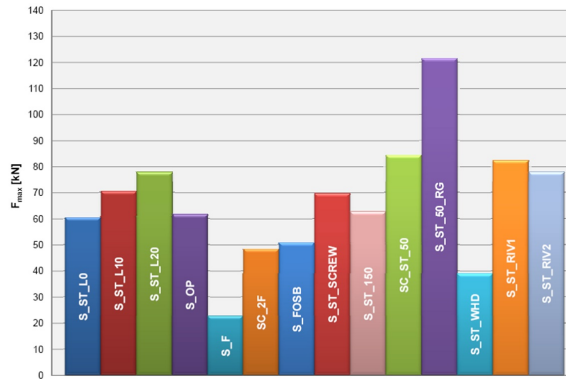


Figura 4.37: Forze massima raggiunte nei test

Nel seguito si vogliono confrontare i risultati ottenuti dalle prove di laboratorio ed estrapolarne alcune considerazioni.

### **Influenza del carico verticale**

Riducendo l'intensità del carico verticale si osserva una progressiva diminuzione del carico orizzontale massimo. La rigidezza decresce al diminuire del carico riportando una diminuzione rispetto al valore massimo. Si intuisce per cui come il carico verticale sia molto importante per definire il comportamento di questa tipologia di costruzioni e come esso influenza in maniera benefica la stabilità globale della parete, riducendo in maniera sensibile il carico gravante sugli hold-down.

### **Influenza della spaziatura delle connessioni**

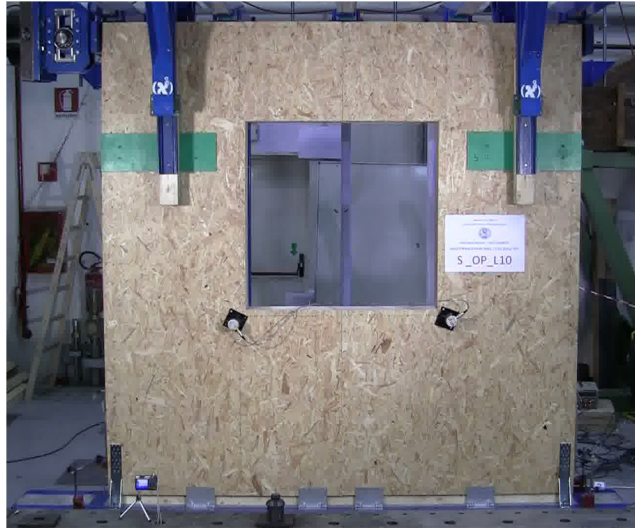
La spaziatura dei connettori lungo il perimetro del rivestimento strutturale influenza in maniera sostanziale il comportamento meccanico e deformativo della parete. Infittendo il passo, è lecito attendere sia una maggiore resistenza, che una maggiore rigidezza. Il test con spaziatura dei connettori minore (50mm) riporta il valore massimo di forza orizzontale. Le curve con spaziatura maggiore presentano un calo in termini di forza del 7,8% (sp: 100mm) e del 25,7% (sp: 150mm). Quindi, come indicato in letteratura, ad un aumento dei connettori corrisponde un aumento della resistenza del sistema e complessivamente dell'efficacia dell'effetto lastra. Si è notato inoltre come, nel caso di spaziatura maggiore, venga portata a rottura la connessione telaio-rivestimento mentre nel caso di spaziatura inferiore è preponderante il contributo del sistema di connessione a terra. La parete a spaziatura 100mm (standard) riesca a sfruttare sia la componente deformativa dovuta al collegamento pannello montante sia quella dovuta alla deformazione dell'hold-down. Nella parete con passo tra i connettori pari a 50mm si porta a rottura prematura l'hold-down mentre in quella con spaziatura 150 è il collegamento parete montante a cedere per primo.

### **Influenza del rivestimento strutturale**

All'interno della campagna sperimentale sono state considerate due tipologie di rivestimento strutturale, ovvero, pannelli OSB e lastre in gessofibra. I campioni sono stati realizzati utilizzando il medesimo materiale su entrambi i lati, oppure adottando sulle due facce opposte materiali differenti. Il valore massimo di carico orizzontale viene raggiunto con l'impiego di OSB su entrambi i lati. Il campione realizzato con lastre di gessofibra, rispetto al precedente, presenta una riduzione del 38%. La parete dotata di rivestimento misto presenta un valore massimo simile a quello ricavato nel caso di pare-

te dotata unicamente di rivestimento in gessofibra. I pannelli in gessofibra conferiscono alla parete una maggiore rigidezza ed un comportamento meno duttile.

### Influenza della presenza di aperture

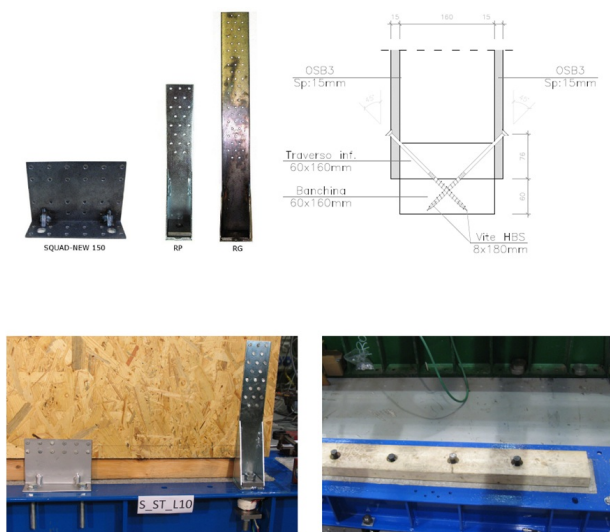


**Figura 4.38:** *Parete con aperture*

La presenza di un'apertura solitamente viene computata all'interno dei calcoli statici come un'interruzione verticale della parete su tutta l'altezza, considerando ai fini strutturali solo le due fasce ai lati dell'apertura. Anche dai test si evince come la presenza della finestra sembrerebbe penalizzare la resistenza complessiva della parete. La presenza del foro riduce la rigidezza di circa il 13% rispetto all'equivalente parete integra. Esaminando il grafico che riporta l'elongazione delle diagonali si nota che la parete con l'apertura presenta allungamenti maggiori evidenziando una maggiore deformabilità. Infine, si nota come la disposizione dei pannelli impiegata influenzi in maniera sostanziale il risultato ottenuto. L'inserimento di un'apertura muta notevolmente il comportamento statico della parete introducendo un indebolimento nei confronti delle azioni orizzontali. Le attuali indicazioni normative a questo proposito trascurano interamente il contributo degli architravi e della pannellatura al di sotto delle aperture, considerando ai fini delle verifiche statiche le sole fasce esterne. Questa ipotesi è più veritiera se l'architrave e la specchiatura al di sotto della finestra sono realizzate con pannelli strutturali non continui rispetto a quelli adiacenti. Trascurare tale apporto è

sicuramente conservativo ed a favore di sicurezza nel caso in cui il materiale costituente il rivestimento strutturale rispetti la continuità in prossimità dei montanti verticali che delimitano l'apertura. Il contributo è significativo quando si dispone di un'adeguata altezza degli elementi e si adottano materiali adeguati. L'impiego di pannelli caratterizzati da elevata resistenza e non soggetti a crisi fragili introdotte da eventuali effetti fuori piano riesce a costituire un'ottima continuità strutturale. A questo proposito, pur non potendo prevedere l'incremento di resistenza dovuto alla continuità, in fase di realizzazione in stabilimento quando possibile è preferibile rispettare la continuità.

### Influenza dei sistemi di connessione a terra



**Figura 4.39:** Sistemi di connessione alla fondazione

Il collegamento della parete alla struttura di fondazione, a contrasto delle forze che inducono la traslazione orizzontale è stata realizzata con due modalità differenti. La prima prevede l'utilizzo di angolari metallici chiodati alla parete e fissati con bulloni alla base di acciaio. La seconda l'inserimento da ambo i lati della parete di viti che solidarizzano la soglia di tracciamento al traverso inferiore del telaio. Analizzando il grafico che riporta lo spostamento orizzontale alla base si nota chela serie corrispondente alla prova con viti, fatta eccezione per un breve tratto iniziale, riporta scorrimenti minori. Il valore di rigidezza più elevato è offerto dalla connessione con angolari mentre con l'impiego di viti si osserva un calo di circa il 9Di norma la pratica costruttiva prevede

l'impiego di appositi angolari per contrastare il sollevamento della parete. Tuttavia, per valutare l'opportunità di ridurre le spese connesse alla messa in opera di hold-down, è stato studiato il comportamento della parete vincolata alla sola traslazione mediante angolari metallici. Questa soluzione risulta accettabile qualora l'intensità delle forze orizzontali che sollecitano la struttura non sia elevata; ad esempio, per strutture ad un solo piano. Il valore massimo di forza orizzontale è stato raggiunto durante la prova con hold-down, mentre in assenza di hold-down si riscontra un calo del 45%. Come si può constatare, la rigidità in assenza di hold down è nettamente minore rispetto alla parete standard. Per valutare l'influenza di differenti sistemi di connessione a contrasto del sollevamento della parete, sono state confrontati i risultati dei test *SC\_ST\_50* ed *S\_ST\_50\_RG*. Il primo prevede l'impiego dell'angolare denominato RP connesso con 20 chiodi anker 4x60mm, mentre il secondo prevede l'impiego del modello RG dotato di 52 connettori dello stesso tipo. Il sistema di connessione con hold-down RG, presenta una resistenza di picco maggiore rispetto al corrispondente test con utilizzo di angolari RP di circa il 44%. L'ultimo confronto è dedicato all'analisi del comportamento della parete strutturale dotata di finiture superficiali. I provini rappresentano due stratigrafie comunemente impiegate nella realizzazione di pareti strutturali interne e pareti perimetrali. Si nota che le curve sono pressoché sovrapposte, indicando che il rivestimento applicato non partecipa in maniera significativa alla resistenza del sistema. Il pacchetto di finitura RIV1 sembra conferire alla parete un piccolo incremento in termini di forza massima assorbibile. Ciò, con buona probabilità è dovuto alla presenza dei pannelli in gessofibra connessi alla parete strutturale mediante cambrette metalliche con passo confrontabile a quello adottato nel caso di rivestimento strutturale. Un'analoga considerazione può essere formulata con riferimento alla rigidità. Infatti anche il parametro  $K_s$  nel caso RIV1 viene in parte incrementato dalla presenza dei fogli di gessofibra impiegati nel rivestimento.

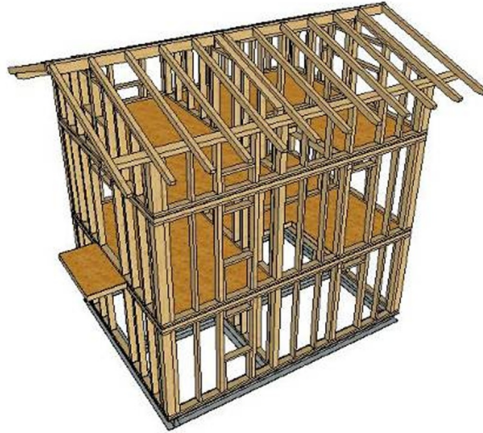
Nell'analisi descritta nei precedenti punti emerge chiara l'interazione tra le performance del sistema e l'efficacia delle due componenti - parete - ancoraggio alla fondazione. La corretta progettazione deve bilanciare la resistenza di entrambe le parti, pena l'incremento di costo ed una condotta poco razionale ai fini statici del componente strutturale. A riguardo si può notare come il comportamento della parete con spaziatura minima, dotata di hold-down RG e maggior numero di squadrette metalliche a taglio, riesca a sviluppare una resistenza ed uno spostamento ultimo maggiori rispetto all'analoga con connessione non adeguata. Un altro aspetto indagato riguarda l'efficienza dei sistemi di vincolo rispetto al meccanismo di rottura per scorrimento

orizzontale. In particolare sono state testate due tipologie particolarmente funzionali: squadrette metalliche chiodate alla parete e tassellate alla fondazione e coppie di viti inclinate che connettono il corrente inferiore della parete alla soglia di tracciamento. Come si è osservato nelle analisi delle curve sperimentali entrambe le soluzioni sono caratterizzate da ottime prestazioni. Tuttavia mentre l'impiego di angolari metallici è largamente adottato dalle aziende specializzate l'utilizzo di connessioni dirette tramite la soglia di tracciamento è meno diffusa. L'inserimento di un elevato numero di viti con passo e inclinazione precisa può portare ad una errata sovrastima dei costi e delle difficoltà tecniche esecutive e di controllo in cantiere. Questi si rivelano infondati se si provvede ad una corretta predisposizione in stabilimento dei preforni lato parete. Operando in questo modo in cantiere vengono posizionate le viti lungo le direzioni già prefissate e dopo un rapido controllo della presenza di tutte le coppie foro-vite si procede all'infissione tramite avvitatore. Dal punto di vista statico osservando le curve carico - scorrimento orizzontale e carico - sollevamento si nota che le viti riportano il minor valore di spostamento orizzontale, mentre gli angolari presentano il minor sollevamento. Tale comportamento è con tutta probabilità imputabile alla capacità delle squadrette metalliche di contrastare efficacemente, dopo la crisi degli hold-down, la rotazione della parete trasferendo notevoli forze di trazione alla fondazione. Le viti pur manifestando un'ottima capacità di contrasto al meccanismo di traslazione non riescono ad esplicare una corrispondente efficacia nei confronti delle forze dovute al sollevamento della parete.

## **4.5 Test sismici su tavola vibrante di un edificio a tre piani in legno a pannelli portanti intelaiati**

I risultati emersi dalla campagna sperimentale descritta nei paragrafi precedenti hanno permesso lo studio delle singole componenti strutturali che costituiscono un intero edificio, dal singolo connettore alla parete vera e propria. Al fine di validare i risultati ottenuti e conoscere il comportamento delle componenti stesse all'interno di un edificio, e dunque la loro interazione in caso di sisma, è stata eseguita una prova su tavola vibrante di un edificio in scala reale. Diviene infatti fondamentale nello studio del comportamento d'insieme di una struttura sottoposta a carichi sismici scorporare le singole componenti strutturali, individuarne il comportamento, e successivamente comprendere la loro interazione all'interno della struttura globale al fine di individuare

i possibili meccanismi di collasso. L'edificio di prova è caratterizzato da una pianta rettangolare 5 m x 7 m e si sviluppa su tre livelli (piano terra, primo piano e sottotetto) per un'altezza massima al colmo di 7.65 m.



**Figura 4.40:** *Vista tridimensionale dell'edificio senza la pannellatura strutturale*

Gli orizzontamenti sono formati da elementi di solaio a sezione scatolare di altezza pari a 140 mm, ai quali sono sovrapposti e chiodati pannelli in OSB dello spessore di 15 mm per garantire il comportamento a diaframma rigido nel piano.

La copertura a due falde è realizzata con travetti in legno massiccio e un tavolato ligneo irrigidito mediante nastri forati metallici. Le pareti strutturali sono del tipo intelaiato con pannelli di rivestimento in OSB di spessore 15 mm, su entrambi i lati fissati mediante chiodi ad aderenza migliorata 2.8 x 60 mm. I montanti e i traversi della parete sono realizzati con elementi in legno massiccio giuntato di sezione 160 x 60 mm e 160 x 100 mm. L'edificio è stato testato senza alcuna finitura e componente non strutturale. Al fine di simulare la massa aggiuntiva relativa ai carichi permanenti portati e ai carichi accidentali previsti in combinazione sismica sono stati posti sui solai dei blocchi in calcestruzzo di peso equivalente (Figura 4.42(a)). Al fine di simulare la massa dell'isolante e dell'intero stratigrafia degli elementi, sono state collocate tegole appesantite in copertura e masse aggiuntive all'interno delle pareti (Figura 4.42(b)).

Al fine di garantire l'interazione fra i vari elementi strutturali nonché il collegamento dell'edificio con la struttura di base sono stati utilizzati differenti dispositivi di connessione. Per quanto riguarda lo scorrimento orizzontale delle pareti strutturali sono state utilizzate piastre chiodate ai piani superiori (Figura 4.43(a)) e coppie di viti da





Figura 4.41: *Edificio di prova*



**Figura 4.42:** *Masse aggiuntive sui solai e all'interno delle pareti strutturali*

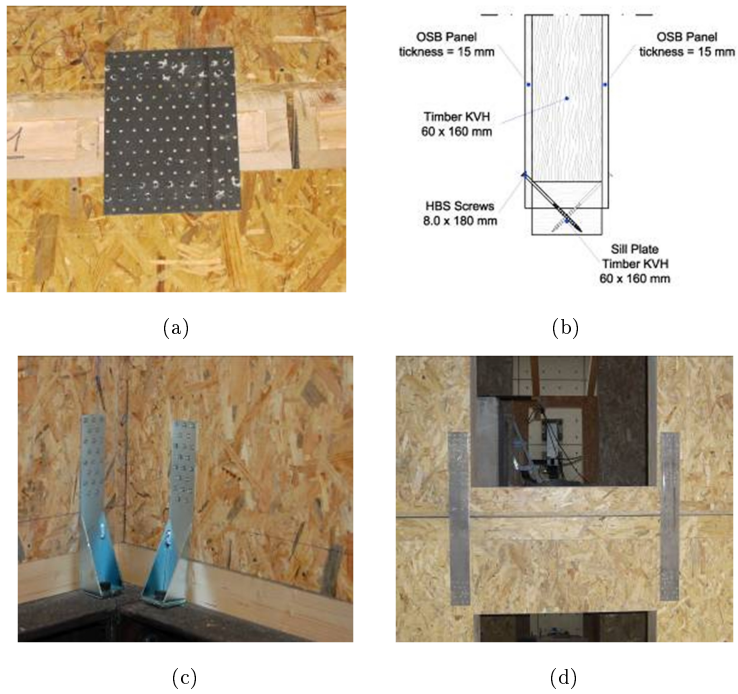
legno 8x180 inclinate a 45 gradi al piano di fondazione (Figura 4.43(b)). Le forze di trazione agli angoli delle varie pareti sono state affidate ad angolari tipo hold-down, al piano di fondazione (Figura 4.43(c)), ed a piastre metalliche chiodate ai piani superiori (Figura 4.43(d)). Il collegamento del solaio con le pareti è stato effettuato mediante viti da legno 8x220.

La progettazione è avvenuta in accordo con la normativa italiana vigente (NTC2008) integrata con le disposizioni previste dall'EC5 e dall'EC8. È stata scelta una destinazione d'uso di civile abitazione assumendo una vita nominale  $V_n$  pari a 50 anni ed una classe d'uso II. Si ipotizza che l'edificio sia ubicato nel sito caratterizzato dal massimo valore del parametro di pericolosità sismica  $a_g$  per un periodo di ritorno pari a 475 anni previsto dalla normativa italiana. La progettazione sismica è stata effettuata mediante analisi statica lineare assumendo un coefficiente di struttura  $q$  pari a 4. Le forze di piano così calcolate sono state ripartite fra le pareti strutturali in proporzione alla loro rigidità, assumendo l'ipotesi di piano rigido del solaio.

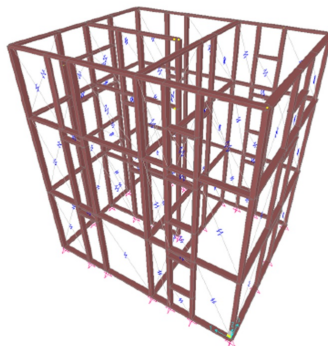
Al fine di stimare i parametri necessari alla progettazione della prova su tavola vibrante sono stati sviluppati alcuni modelli numerici semplificati. In particolare sono stati utilizzati dei modelli lineari al fine di valutare i parametri dinamici dell'edificio utili per una corretta scelta dell'input di prova (Figura 4.44).

Il modello ha restituito i modi di vibrare riportati in (Figura 4.45).

Il montaggio dell'edificio è avvenuto direttamente sulla tavola vibrante. Il collegamento fra l'edificio stesso e la tavola è stato realizzato mediante un graticcio di travi in acciaio disposte in corrispondenza delle pareti strutturali come visibile in Figura 4.46.



**Figura 4.43:** *Dispositivi di collegamento . (a):Piastre chiodate per la traslazione orizzontale delle pareti ai piani superiori -(b):Viti a 45° per la traslazione orizzontale delle pareti al piano fondazione -(c):Hold-down -(d):Tie- down*



**Figura 4.44:** *Modello numerico di analisi*

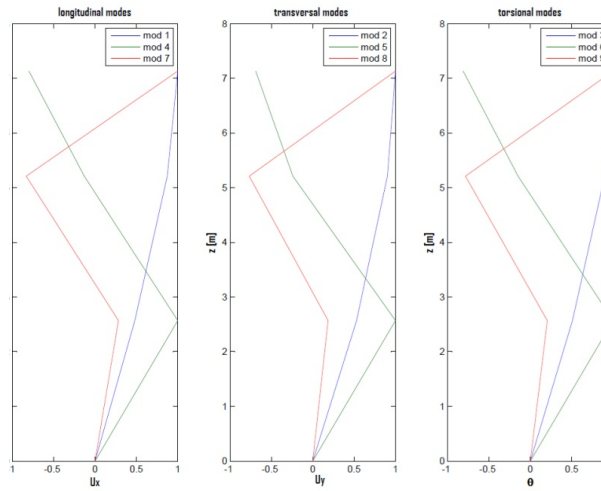


Figura 4.45: *Modi di vibrare edificio di prova*

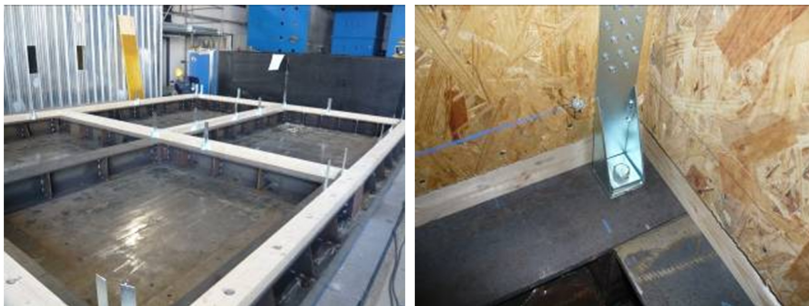


Figura 4.46: *Graticcio di travi in acciaio e collegamento con le pareti strutturali*

La forte prefabbricazione del sistema e l'elevata accuratezza delle lavorazioni in stabilimento hanno permesso di poter portare a termine le fasi di montaggio in 3 giorni lavorativi da parte di 5 operai. In opera, infatti, è stato necessario effettuare solo il posizionamento degli elementi strutturali (pareti, elementi di solaio, pannelli irrigidenti, orditura primaria e secondaria del tetto) collegandoli fra loro. In Figura 4.47 sono visibile le successive fasi di montaggio.

Al fine di conoscere il comportamento della struttura durante i test sismici si è disposto un opportuno set-up di strumenti. Le misure hanno riguardato accelerazioni, spostamenti e deformazioni; i dati sono stati acquisiti con una frequenza di campionamento pari a 1024 Hz. Sono stati posizionati 103 strumenti in aggiunta al sistema di acquisizione ottico che grazie a telecamere a raggi infrarossi è in grado di monitorare in continuo lo spostamento di marker disposti sull'edificio. In Tabella 4.12 è riassunta la strumentazione totale disposta mentre in Figura 4.48 ci sono i particolari di alcuni strumenti.

Tipologia	Numero	Tipo misura
Celle hold-down	12	Forza
Celle tie-down	11	Forza
Potenzimetri a filo	28	Allungamento diagonali parete e solai
Potenzimetri	28	Spostamento verticale e orizzontale pareti
Accelerometri	27	Accelerazioni ai vari piani

**Tabella 4.12:** *Strumentazione disposta*

La struttura è stata sottoposta a 5 differenti intensità di input sismico, scalando opportunamente l'accelerogramma registrato dalla stazione Ulcinj - Hotel Albatros, posta ad una distanza epicentrale di 21 km, durante il terremoto del Montenegro del 15/04/1979 (Mw 6.9).

I valori di accelerazioni massima (PGA) a cui l'edificio è stato sottoposto nelle 5 fasi di prova sono riportate nella Tabella 4.13.

Tali valori derivano da considerazioni effettuate in conseguenza alle analisi di simulazione numerica nonché ai risultati ottenuti dopo le prime fasi di prova. La valutazione del danneggiamento strutturale è stata effettuata inoltre dopo ciascun test mediante un'accurata ispezione visiva. Importante notare come fra una fase di prova e la successiva non è stato eseguito nessun intervento di riparazione sulla struttura. Le ispezioni eseguite sull'edificio a seguito di tutte le prove effettuate non hanno evidenziato al-



(a)



(b)



(c)



(d)



(e)



(f)



(g)



(h)

**Figura 4.47:** *Fasi di montaggio. (a):Trasporto degli elementi strutturali -(b):Montaggio delle pareti al piano terra -(c):Posa degli elementi di solaio -(d):Posa e chiodatura dei pannelli OSB per l'irrigidimento di piano -(e):Montaggio delle pareti al secondo piano -(f):Montaggio delle pareti al terzo piano -(g):Posa dei travetti di copertura -(h):Chiodatura dei nastri forati per l'irrigidimento di piano della copertura*



(a)



(b)



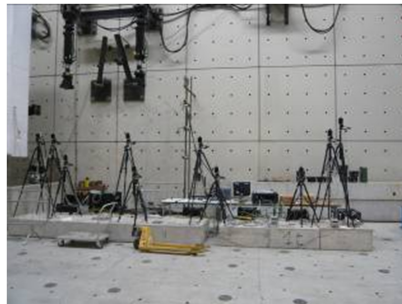
(c)



(d)

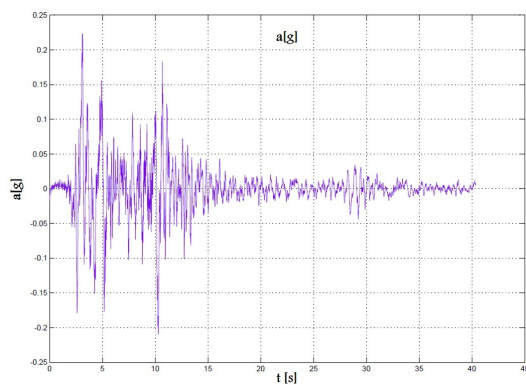


(e)



(f)

**Figura 4.48:** *Strumentazione. (a):Celle di carico per hold-down -(b):Trasduttori di spostamento per lo scorrimento orizzontale ed il sollevamento delle pareti -(c):Trasduttore di spostamento per lo scorrimento orizzontale relativo degli elementi di solaio -(d):Trasduttori di spostamento a filo per la deformazione nel piano degli orizzontamenti -(e):Trasduttore di spostamento per lo spostamento relativo di interpiano in prossimità di una parete interna -(f):Sistema ottico di rilevamento degli spostamenti dei punti di controllo sulla facciata C*



**Figura 4.49:** Accelerogramma di prova. Terremoto del Montenegro - stazione Ulcinj -Hotel Albatros - PGA: 0.224g

Phase	PGA [g]
1	0,07
2	0,28
3	0,5
4	0,7
5	1

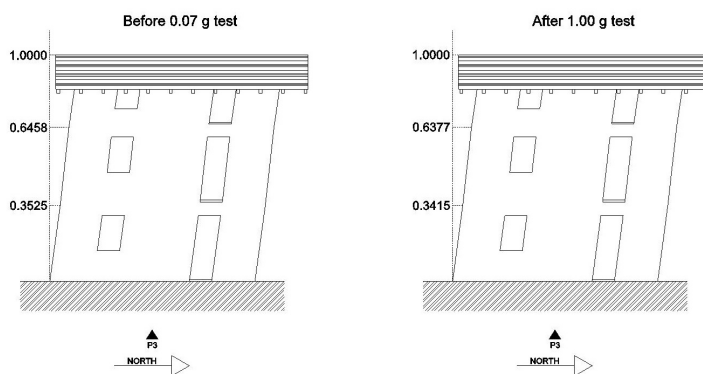
**Tabella 4.13:** Fasi di test e accelerazioni massime



cun danneggiamento visibile agli elementi strutturali, dimostrando dunque l'ottimo comportamento della struttura progettata. Al fine di comprendere il comportamento strutturale dell'edificio durante le differenti fasi dei test sismici sono stati analizzati nel dettaglio i dati forniti dalla strumentazione disposta. In Tabella 4.14 è riportato lo spostamento d'interpiano raggiunto ad ogni fase di test.

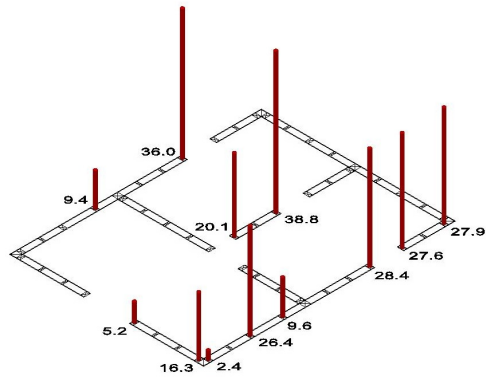
1st floor	Ch 41 (Interior)	Optical S. (External)
Test	[%]	
0.07g	0.03	0.02
0.28g	0.21	0.14
0.50g	0.45	0.59
0.70g	0.79	0.63
1.00g	1.12	1.24

**Tabella 4.14:** Spostamenti d'interpiano misurato tramite trasduttori di spostamento (colonna  $sx$ ) e tramite strumentazione ottica (misura  $dx$ )



**Figura 4.50:** Modi di vibrare ricavati dopo il test a 0.07 g ( $sx$ ) e 1g ( $dx$ )

L'obiettivo finale è stato lo studio delle singole componenti strutturali nonché la comprensione dell'interazione fra le stesse nell'ottica di descrivere il comportamento globale della struttura. Tali analisi hanno fornito informazioni fondamentali sul comportamento dei singoli elementi strutturali all'interno di un edificio reale, sia in campo elastico che in campo non lineare.



**Figura 4.51:** Forze di trazione massime registrate negli hold-down nel test ad 1g

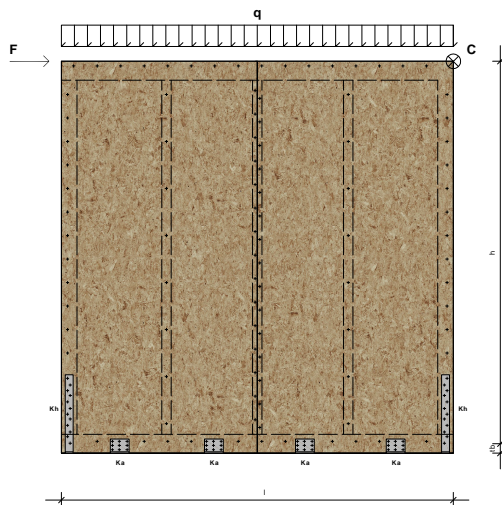


## Capitolo 5

# Conclusioni

Il lavoro di ricerca svolto durante il periodo di dottorato ha avuto due obiettivi primari. Il primo è stato quello di realizzare una campagna sperimentale per creare un ampio database sulle caratteristiche meccaniche degli edifici in legno a parete portante intelaiata a partire dai singoli componenti (connettori pannello-montante, hold-down, angolari a taglio) sino all'intero edificio. Tale database è stato messo a disposizione dell'intero gruppo di ricerca sulle strutture in legno dell'Università di Trento (<http://www.unitn.it/en/dims/16845/timber-structures>) e servirà per validare le formulazioni che sono state e che verranno proposte in futuro. I dati sono stati riassunti all'interno dei sette report scientifici riportati in Tabella 1.1.

Il secondo obiettivo ha riguardato lo sviluppo di una formulazione in grado di prevedere lo spostamento e quindi la rigidezza in campo elastico di una parete intelaiata soggetta a una forza orizzontale. La formulazione, dettagliatamente riportata nell'articolo di Appendice D e qui brevemente riassunta nell'equazione 5.1, è stata validata attraverso le prove a scala reale sulle pareti. Essa permette, partendo dalla conoscenza delle rigidezze delle singole componenti, di risalire con adeguata precisione allo spostamento di una intera parete e, di conseguenza, di poter analizzare rigidezza e spostamento di un intero edificio formato da più pareti. Considerata una parete schematizzabile come in Figura 5.1, analizzando i diversi approcci normativi e le formulazioni disponibili in letteratura lo spostamento orizzontale del punto di controllo C, risulta essere l'unione in serie di tutti i contributi deformativi ed è valutabile secondo l'equazione 5.1.



**Figura 5.1:** Geometria parete e configurazione di carico

$$\Delta = \frac{F \cdot h}{l} \cdot \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{l \cdot n_p \cdot k_c} + \frac{F \cdot i_a}{k_a \cdot l} + \frac{h}{l \cdot k_h} \cdot \left( \frac{F \cdot h}{l} - \frac{q \cdot l}{2} \right) + \frac{h}{l} \cdot \left( \frac{F \cdot h}{l} + \frac{q \cdot l}{2} \right) \cdot \frac{t_b}{E_{90,b} \cdot A_m} \quad (5.1)$$

Nella quale:

- **F**: è la forza orizzontale applicata alla parete;
- **q**: è il carico distribuito applicato al traverso superiore;
- **l**: è la lunghezza totale della parete;
- **h**: è l'altezza della parete;
- **b**: è la base del pannello di rivestimento;
- **G<sub>p</sub>**: è il modulo di taglio G del pannello di rivestimento;
- **n<sub>p</sub>**: è il numero di lati su cui sono posati i pannelli di rivestimento;
- **t<sub>p</sub>**: è lo spessore del pannello di rivestimento;
- **s<sub>c</sub>**: è la spaziatura dei connettori disposti sul contorno di ciascun pannello di rivestimento;
- **k<sub>c</sub>**: è la rigidezza dei connettori che collegano i pannelli di rivestimento al telaio;

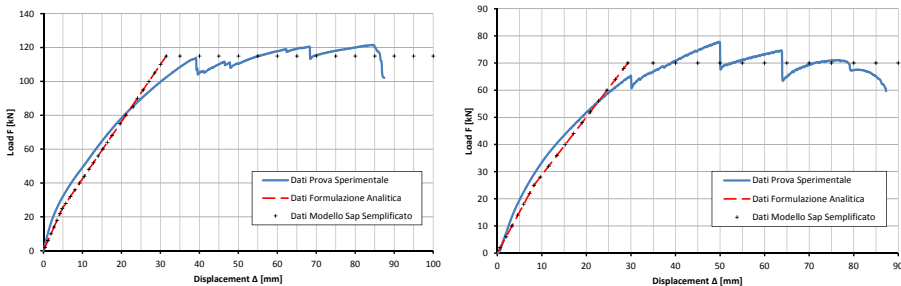
- $\mathbf{k}_a$ : è la rigidezza a scorrimento orizzontale dei collegamenti a taglio;
- $\mathbf{n}_a$ : è il numero dei collegamenti a taglio della parete;
- $\mathbf{k}_h$ : è la rigidezza a sollevamento verticale degli hold-down;
- $\mathbf{t}_b$ : è lo spessore della banchina;
- $\mathbf{A}_m$ : è l'area del montante verticale;
- $\mathbf{E}_{90,b}$ : è il modulo elastico in direzione ortogonale alla fibratura della banchina;

La variabile  $\lambda$  può essere considerata come una funzione di forma essendo essa dipendente del solo rapporto adimensionale  $\alpha = \frac{h}{b}$  del foglio di rivestimento ed assume l'espressione riportata in equazione 5.1.

$$\lambda = \alpha^2 \cdot \left[ \frac{1}{\eta(\alpha)} + \frac{1}{\xi(\alpha)} \right] = \lambda(\alpha) \quad (5.2)$$

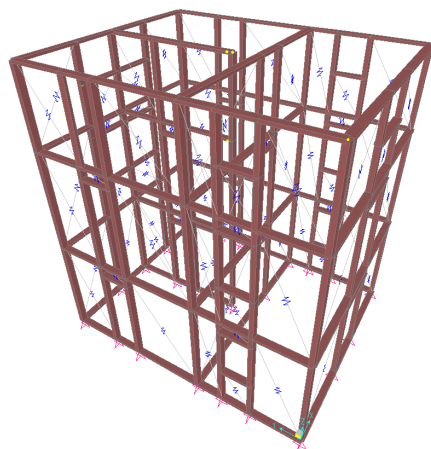
Il parametro  $\lambda$  è solo funzione delle caratteristiche geometriche del pannello utilizzato e della disposizione della chiodatura.

La formulazione riportata è del tutto generica e può essere applicata a qualunque tipo di parete portante intelaiata in legno. Si è poi notato che nelle pareti intelaiate usualmente utilizzate in centro-sud Europa la dimensione dei montanti fa sì che lo schiacciamento della banchina risulta trascurabile rispetto alle altre variabili (la dimensione maggiore fa sì che l'area di impronta sia grande e le deformazioni al di sotto di esso molto basse). Il confronto della formulazione proposta con le prove di laboratorio ha evidenziato l'adeguatezza nelle previsione dello spostamento della formulazione stessa ( Figura 5.2).



**Figura 5.2:** *Paragone risultati test laboratorio - previsione formulazione analitica*

La formulazione ha anche permesso la creazione di un modello ad elementi finiti utilizzato per predire il comportamento della prova su tavola vibrante (Figura 5.3).



**Figura 5.3:** *Modello elementi finiti edificio di prova*

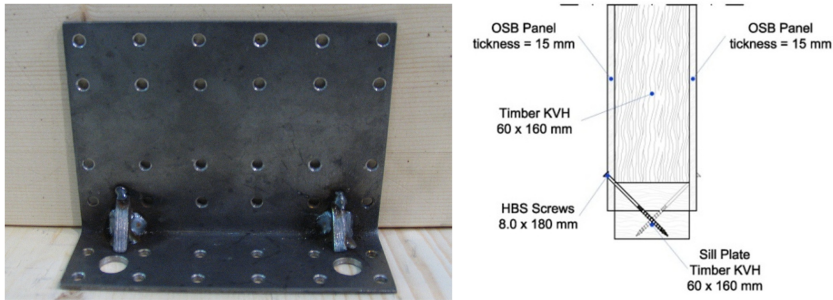
In tale modello le pareti sono state schematizzate come telai labili controventati da una molla obliqua con rigidezza pari a quella risultate dall'applicazione della formulazione di equazione 5.1 alla geometria di ogni parete. Il modello è stato poi implementato utilizzando due differenti ipotesi. Nella prima si è ipotizzato che i dispositivi hold-down restassero inattivi. Con questa ipotesi il modello ha restituito un valore del periodo del primo modo di vibrare di 0.11 s. Nella seconda si è ipotizzato che tutti i dispositivi fossero attivi. Il periodo risultate è stato di 0.32 s. La prova sull'edificio reale, in cui alcuni dispositivi si sono attivati, mentre altri no, ha restituito un periodo di 0.163 s, all'incirca a metà della forbice previsionale.

Si può concludere che sia le prove su pareti che la prova su tavola vibrante hanno validato la formulazione proposta.

Oltre a questi due obiettivi, considerati principali, si sono raggiunti anche altri obiettivi che nel seguito vengono riassunti.

Dopo i primi test svolti sugli angolari a taglio adatti a connettere la parete alla fondazione si è notato che gli angolari attualmente disponibili in commercio sono poco performanti sia per quanto riguarda la rigidezza che la resistenza offerte. Inoltre le forature presenti negli angolari sono posizionate spesso al di fuori dell'area utile per la chiodatura nelle pareti intelaiate. Per questo motivo è stato proposto un angolare di nuova concezione, descritto in dettaglio nell'articolo di Appendice B e riportato in Figura 5.4.

E' stato sviluppato anche un modello ingegneristico adatto alla comprensione del com-



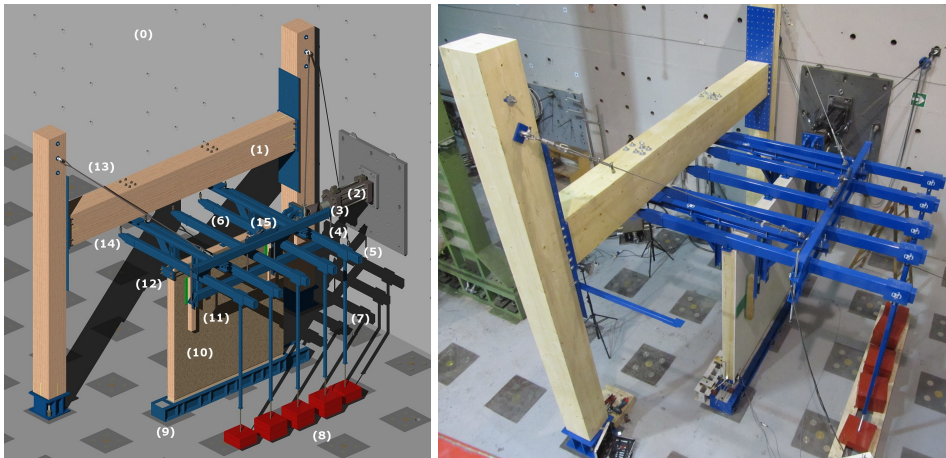
**Figura 5.4:** Collegamento a taglio tramite angolare di nuova concezione ( $sx$ ) e tramite viti per legno ( $dx$ )

portamento meccanico di questo tipo di angolari, modello anch'esso descritto in dettaglio nell'articolo di Appendice B. I risultati dei test hanno mostrato come questi nuovi collegamenti siano molto più efficace di quelli attualmente disponibili in commercio. Inoltre è stato proposto anche un collegamento realizzato tramite viti per legno (Figura 5.4). Anch'esso ha riportato degli ottimi risultati ed è molto interessante soprattutto per la velocità di messa in opera e per l'economicità del sistema stesso.

Prima di eseguire i test sulle pareti a scala reale è stata svolta una ampia ricerca bibliografica riguardo ai set-up e alle procedure di prova utilizzate dai vari centri di ricerca per testare le pareti lignee intelaiate. Tale ricerca bibliografica ha portato ad individuare quelli che sono i punti chiave per avere un set-up adatto a testare tale tipologia di parete. In particolare si è cercato di focalizzare l'attenzione su quelle che sono le condizioni al contorno e come esse vanno ad influire sui risultati di prova. Le modalità di applicazione del carico verticale e del carico orizzontale, nonché l'ancoraggio della parete a terra risultato di fondamentale importanza sull'influenza dei risultati dei test. Per questi motivi è stato progettato e realizzato un set-up ad hoc (Figura 5.5).

Tale set-up di prova applica il carico verticale tramite delle leve e dei pesi in acciaio in modo che il carico stesso sia sempre nello stesso punto della parete e mantenga durante tutta la prova inalterata la propria intensità. Il carico orizzontale è applicato tramite uno speciale giunto realizzato con cuscinetto sferici in maniera da seguire il provino e mantenere il valore del carico orizzontale sempre perpendicolare alla parete. La traversa di base a cui il provino viene fissato è pensata in modo da poter agganciare gli angolari con facilità a dei profili in acciaio e da poter lasciare la parete a contatto con del calcestruzzo in maniera da riprodurre le condizioni di attrito poi realmente presenti in opera.





(0)-Muro di contrasto; (1)- Telaio principale; (2)- Martinetto idraulico; (3)- Traversa di sicurezza superiore ; (4)- Traversa di sicurezza inferiore; (5)- Manicotti scorrevoli; (6)- Profili tubolari (leve); (7)- Asta di collegamento manicotto-peso; (8)- Piastre di contrappeso; (9)- Base di appoggio della parete; (10)- Parete; (11)- Ritegni torsionali; (12)- Piastra di testa per prove cicliche; (13)- Tiranti dotati di tenditore; (14)- Supporto dotato di cerniera sferica; (15)- Gancio di sollevamento traversa inferiore.

**Figura 5.5:** *Set-up per test su pareti intelaiate*

## 5.1 Sviluppi futuri

Questo lavoro di ricerca ha come prosecuzione l'elaborazione e lo sviluppo dei seguenti punti:

- Estensione dell'analisi elastica proposta per una singola parete all'intero edificio.
- Analisi delle pareti portanti intelaiate in campo non lineare, con particolare riguardo alla ricerca dello spostamento ultimo di una parete tramite la conoscenza delle curve sforzo spostamento in campo non lineare dei singoli componenti (chiodi, hold-down, angolari a taglio).
- Sviluppo di dispositivi tipo hold-down in grado di garantire maggiore rigidità e maggiore portata.
- Sviluppo di una metodologia di calcolo per edifici in legno semplici in zona sismica.
- Interpretazione e elaborazione completa dei dati della prova su tavola vibrante.

# Appendice A

## Introduction to scientific papers

In the next appendices seven scientific papers are presented. The first five are the result of the activity carried out at the University of Trento and they resume the results obtained from a wide experimental campaign on timber light frame buildings to assess their behaviour in seismic areas. The articles are presented in order of developed topics. They start with the characterization of connections between timber stud and sheathing panel and the characterization of the connections between timber walls and foundations. Then the result of the tests carried out on full scale timber shear walls are reported. After this topics the analytical and numerical analysis of timber framed shear walls are developed. At the end the shaking table test on full scale building is resumed.

The last two articles are the result of a work carried out in collaboration with Professor Roberto Crocetti at the University of Lund (Sweden). The argument of this last two articles is timber concrete composite structures with prefabricated slab. This topic is also important for the timber buildings in seismic area. The slab acts as a rigid diaphragm in plane and takes the role to distribute the horizontal forces generated by the earthquake. Furthermore the builder would like to prefabricated the building as much as possible inside the factory. The developed technology fulfills this requirement.



# Appendice A

## Paper I

Influence of sheathing to framing connections on  
mechanical properties of wood framed shear walls

*Andrea Conte, Maurizio Piazza, Tiziano Sartori, Roberto Tomasi*

ANIDIS 2011, Bari





# Influence of sheathing to framing connections on mechanical properties of wood framed shear walls

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*Keywords: Wood framed building, shear walls, nails, staples, gypsum-fibre, OSB*

## ABSTRACT

For the horizontal stability of wood framed buildings, diaphragm action in the walls is of crucial importance. The structural behavior and capacity of wall diaphragms are primarily dependent on the sheathing-to-timber joints.

Tests on connections between sheathing panel and wood stud were carried out. Different types of sheathing materials and connectors were combined. OSB with different thickness (12 and 15 mm) and gypsum-fibre panel were tested. Nails and staples were used to connect the panels to the wooden part.

Monotonic and cyclic destructive tests were performed on different connections to determine how the different systems behave. A numerical model was created to simulate the behavior of a wood framed wall subjected to a horizontal force in plane. The data obtained from the tests were used to calibrate a formula suitable to predict the elastic horizontal displacement of the walls.

# 1 INTRODUCTION

Timber frame construction is diffused as structural system for multistory building in North America, New Zealand and in Europe. In timber frame systems floors, walls and roof are created by assembling lumber studs (for wall) or lumber joist (floor) in a framed element, which is sheathed by means of different sheathing materials, in order to act as structural diaphragms and allow the transfer of lateral loads (seismic or wind load) to the foundation.

Usually timber frame building is a prefabricated platform system where construction elements are stacked storey by storey, therefore a lack of structural continuity between vertical elements must be eliminated by means of connectors systems such as angle steel brackets and hold-down (figure 1).

Regarding the behavior of shear walls, for lateral stability timber framework are sheathed on one or both sides, with different type of sheathing materials (e.g. Oriented Strand Boards, Plywood, Fiber board) connected to wooden studs and beams by means of metal fasteners (e.g. nails, screws or staples).

Considering the presence of a lateral load  $F$  at the top of the shear walls, the timber frame must be regarded as unstable, and unable to counteract external force, which is only resisted by the sheathing and metal fasteners. Apart from the possibility to uplift of the shear wall, the total lateral displacement is function of fastener slip and shear distortion of the sheathing (which is usually relatively small and often neglected). Also the global lateral strength capacity of shear diaphragm is often determined by the resistance of the local joint between timber frame and sheathing panel (another possible mechanism is associated to the failure of the lateral steel brackets and hold-down).

For this reason the goal of the research presented in the paper is to characterize experimentally the mechanical behavior of sheathing to framing connections in platform frame shear walls, in terms of strength and stiffness, and to validate a proposed analytical model able to predict the stiffness characteristic of the wall.

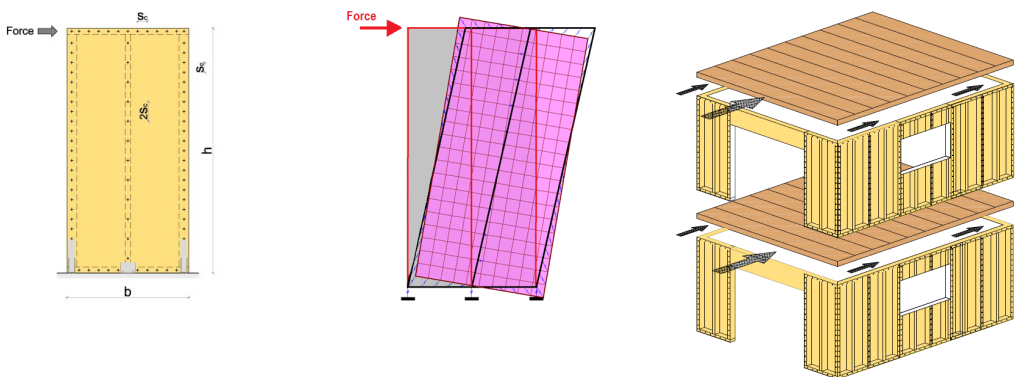


Figure 1. Horizontal force distribution in wood framed building (top) – Single wall geometry and deformation (bottom)

## 2 EXPERIMENTAL INVESTIGATION

### 2.1 Materials and geometry

Since the goal of the experimental phase was to evaluate the mechanical properties of the nailed sheathing to frame member connection, the specimen configuration was the one generally used in push-out tests, i.e. it was based on two side board elements (oriented strand board OSB, or fiber gypsum boards FGB) and a central solid wood element ( $160 \times 60$  mm) held together by fasteners (figure 2). The lateral board elements were glued to a wooden sill beam connected to the steel reaction frame, while the wooden element was connected to the hydraulic jack by means of a steel threaded bar inserted axially in a central hole (figure 2 sx). The instrumentation layout is illustrated in figure 2: the applied force was measured by a load cell placed between the jack actuator and the specimen; the displacements were measured using four L.V.D.T (linear variable differential transformer) transducers placed in pairs on both sides of the specimens, along the shear planes, using specific steel supports for the measuring equipment. In monotonic tests the loading was applied under displacement control up to failure at a constant rate of 0,05 mm/sec, according to the provisions of EN EN 26891.

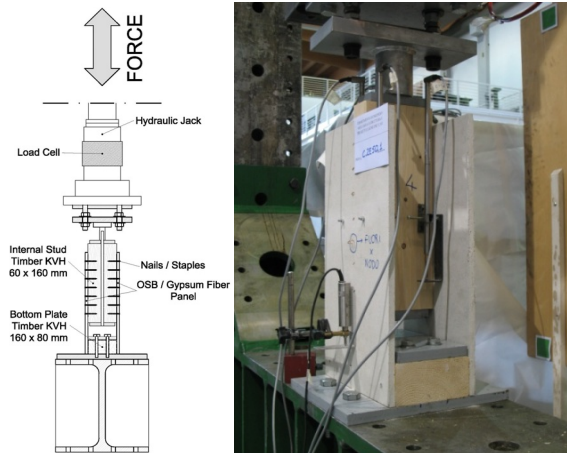


Figure 2. Test Set Up

### 2.2 Test results

Globally 34 specimens were tested (19 monotonic tests, 15 cyclic tests). Different specimens configurations were considered, taking into account the possibility of variable sheathing material (15 mm OSB board, 12 mm OSB board, 12,5 mm FGB board), variable fastener system (deformed nails  $2,8 \times 60$  mm, deformed nails  $2,5 \times 50$  mm, standard smooth nails  $2,5 \times 50$  mm, staples  $1,4 \times 1,6 \times 55$  mm), variable connectors interspace (50 or 100 mm). Moreover some specimens reproduced the possibility to have two adjacent sheathing boards connected to the same stud, in order to evaluate the influence of this structural discontinuity to the mechanical properties. The significance of test label assigned to each specimen is explained in table 1. If not otherwise indicated in the label, the standard fasteners system adopted is  $2.8 \times 60$  mm



annularly threaded nail for OSB sheathing (ring nail), and  $1.4 \times 1.6 \times 60$  mm staples for fiber gypsum boards (FGB).

Table 1. Specimen label significance

Specimen label = X YYYY ZZ KK			
X	YYYY	ZZ	KK
S = monoton ic test	OSB15 = OSB3 - 15 mm	50 = spacing 50 mm 100 = spacing 100 mm	MS = screw nails L = smooth shank nails
C = cyclic test	OSB12 = OSB3 - 12 mm 2F = gypsum fiber - 12.5 mm		G = joint between panels * GC = glued joint between panels**
* two adjacent sheathing boards connected the same stud			
** glued butt joint between two adjacent sheathing boards connected the same stud			

In table 3 are reported, for each specimen configuration, the experimental results in terms of stiffness and strength values, carried out according to the definition reported in European Norm 26891.

### 2.3 Discussion

The experimental data were compared with the values of stiffness and strength carried out from the standard formulas: European (EN 1995-1-1), and national code (Italian national code CNR-DT 206/2007; German national code DIN 1052:2008) has been considered for this purpose.

In table 2 are summarize the different expressions reported in the standards for characteristic embedment strength of a dowel type fastener  $f_{h,k}$ , characteristic yield moment of a dowel type fastener  $M_{y,Rk}$ , mean slip modulus for single shear plane of a fastener  $k_{ser}$ .

Table 4 and table 5 report the comparison between test results and standards values for strength and stiffness respectively. The joint resistance for dowel type fastener according to standard was derived adopting the Johansen formulas in function of  $f_{h,k}$ , and  $M_{y,Rk}$  parameters.

Table 2. Expressions reported in different standards

UNI EN 1995-1-1:2009			
	$f_{h,k}$ (MPa)	$M_{y,Rk}$ (Nmm)	$k_{ser}$ (N/mm)
Staple	$7d^{-0.7}t^{0.9}$	$240d^{2.8}$	$\frac{(\rho_m^{1.5}d^{0.8})}{30}$
Nail	$0.082\rho_k d^{0.3}$	$0.30f_u d^{2.6}$	$\frac{(\rho_m^{1.5}d^{0.8})}{30}$
CNR-DT 206/2007			
	$f_{h,k}$ (MPa)	$M_{y,Rk}$ (Nmm)	$k_{ser}$ (N/mm)
Staple	$7d^{-0.7}t^{0.9}$	$0.2f_{u,k}d^3$	$\frac{(\rho_m^{1.5}d^{0.8})}{60}$
Nail	$0.082\rho_k d^{0.3}$	$\frac{1.8}{d0.4} f_{u,k} \frac{d^3}{6}$	$\frac{(\rho_m^{1.5}d^{0.8})}{25}$
DIN 1052:2008			
	$f_{h,k}$ (MPa)	$M_{y,Rk}$ (Nmm)	$k_{ser}$ (N/mm)
Staple	$7d^{-0.7}t^{0.9}$	$0.30f_u d^{2.6}$	$0.6 \frac{(\rho_m^{1.5}d^{0.8})}{60}$
Nail	$0.082\rho_k d^{0.3}$	$0.45f_u d^{2.6}$	$\frac{(\rho_m^{1.5}d^{0.8})}{25}$

Some experimental results are also reported in figure 3 in terms of force - displacement relationship, where again the y-axis refers to the force transmitted to each specimen fastener (ration F/n<sub>connectors</sub>). The straight lines reproduce the values provided from EN 1995 for strength and stiffness of the single fastener.

According to Table 2, European and Italian code provide the same relation for nail yielding moment  $M_{y,Rk}$ , while in DIN 1052 is reported a 50% higher value; on other hand, differently to what reported in EN 1995 and CNR-DT-206, the German Standard doesn't take into account the additional resistance contribution indicated as "rope effect" for nail and staple connection.

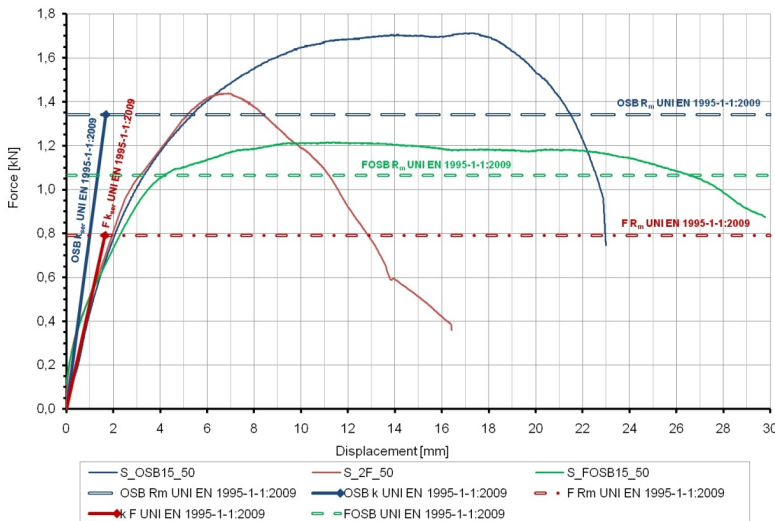


Figure 3. Force vs. displacement graph: comparison between test results and UNI EN 1995. The results refer to a single fastener.

Regarding the value for staple yielding moment  $M_{y,Rk}$ , the expression in DIN 1052 and EN 1995 is the same for  $f_{u,k} = 800$  MPa, while the equation reported in Italian standard CNR-DT-206 provides slightly lower values. It is worth noting that for embedment strength  $f_{h,k}$ , the same expressions are reported for all the three Standards. For all the specimen manufactured with staples and fiber gypsum boards, the experimental results showed about 50% higher values compared to the Standards equations. On the contrary the values carried out from Code expressions are more similar (about 15% lower) to the experimental results for OSB boards nailed to the timber frame. During the experimental campaign three kind of shank have been adopted in OSB to wood nailed connection: smooth wire nail, annularly threaded nail (ring nail), and helically treaded nail (screw nail). Only in the case of ring nails the deformed shank seems to provide a remarkable improvement in term of resistance capacity, while the results are very similar in the case of screw nails and smooth nails. From the experimental data reported in Table 4 it is also possible to deduce: the variation of interspace between connectors (from 50 mm to 100 mm) and the presence of two adjacent sheathing boards connected the same stud (with or without a glued butt joint between boards) doesn't seem to affect the global resistance of the joints; the variation of OSB board thickness (from 12 to 15 mm), according to Johansen model, influences the experimental results.

Regarding the values for slip modulus of nailed connection, all three Standard taken into account presented similar expression (slightly lower values for EN 1995). However it is possible to notice from table 5 that these equations seem to overestimate the real stiffness of nailed connections measured during the experimental campaign. To Author's opinions the formula reported in standard could have been validated for higher diameter nails, and therefore could not be suitable for the small size nails used for board to timber connection in sheathed diaphragms. On the contrary the Standard formulas seem to work quite better in the case of staple connections, even though the differences in the expressions reported in table 2: in particular the German Code DIN 1052 prescribes a 40% reduction of stiffness in the case of fiber gypsum board (FGB).

The analysis of experimental results in term of strength and stiffness of the sheathing to framing connections (table 4 and in table 5) let to evaluate the different structural products at the present adopted in timber frame panels. As far the use of fiber gypsum boards (FGB) is concerned, it is worth underlining that their extreme brittle behavior could affect the global response of the diaphragm, especially under cyclic loads.

Even though the experimental results showed good results in term of strength and stiffness of stapled FGB to timber connection, the ductility properties of this connection seems, as illustrated in figure 3, very poor: after reaching the force peak value at a displacement level of 6 – 7 mm, a sudden impairment of strength is observed, whereas the OSB boards maintained the level of resistance up to 20-24 mm.

Table 3. Results from monotonic push-out test

Test	$F_{max}$	$n_{connectors}$	$F_{max}/n_{connectors}$	$F_v$	$v_v$	$k_s$
	[kN]	-	[kN]	[kN]	[mm]	[N/mm]
S_OSB15_50	22,96	14	1,64	16,65	2,56	447
S_OSB15_100	13,38	8	1,67	11,42	4,52	358
S_OSB12_50	20,62	14	1,47	13,56	2,16	388
S_FOSB15_50	17,34	14	-	13,18	2,07	-
S_2F_50	17,69	14	1,26	15,80	1,69	497
S_OSB15_50_MS	16,48	14	1,18	10,66	2,32	292
S_OSB15_50_L	15,96	14	1,14	8,27	0,74	669
S_OSB15_50_G	45,68	28	1,63	28,36	1,87	466
S_2F_100_G	23,48	16	1,47	18,69	1,90	386
S_2F_100_GC	22,34	16	1,40	20,47	3,88	311

Table 4. Strength: test results vs standard. The results refer to a single fastener

Test	$R_{m,test}$	$R_{m,UNI EN}$	$\Delta_{UNI EN}$	$R_{m,D1}$	$\Delta_{DIN}$	$R_m$	$\Delta_{CNR}$
		1995-1-1	1995-1-1	N1052	1052	CNR-DT 206	DT206/
	[kN]	[kN]	[%]	[kN]	[%]	[kN]	[%]
S_OSB15_50	1,64	1,34	-18,3	1,05	-35,8	1,34	-18,3
S_OSB15_100	1,67	1,34	-19,9	1,05	-37,1	1,34	-19,9
S_OSB12_50	1,47	1,26	-14,1	0,96	-35,0	1,26	-14,7
S_2F_50	1,26	0,79	-37,4	0,69	-45,1	0,71	-44,0
S_OSB15_50_MS	1,18	0,89	-24,4	0,78	-33,8	0,89	-24,4
S_OSB15_50_L	1,14	0,88	-23,0	0,80	-30,0	0,88	-23,0
S_OSB15_50_G	1,63	1,34	-17,8	1,05	-35,5	1,34	-17,8
S_2F_100_G	1,47	0,79	-46,1	0,69	-52,7	0,71	-51,8
S_2F_100_GC	1,40	0,79	-43,4	0,69	-50,3	0,71	-49,4

Table 5. Stiffness: test result vs standard. The results refer to a single fastener.

Test	d	TEST	UNI EN 1995-1-1: 1:2009	DIN 1052:2008	CNR-DT 206/2007
		$k_{s,test}$	$k_{ser}$	$k_{ser}$	$k_{ser}$
	[mm]	[N/mm]	[N/mm]	[N/mm]	[N/mm]
S_OSB15_50	2,8	447	800	860	860
S_OSB15_100	2,8	358	800	860	860
S_OSB12_50	2,8	388	800	860	860
S_2F_50	1,5	497	483	337	561
S_OSB15_50_MS	2,5	292	731	786	786
S_OSB15_50_L	2,5	669	731	786	786
S_OSB15_50_G	2,8	466	800	860	860
S_2F_100_G	1,5	386	483	337	561
S_2F_100_GC	1,5	311	483	337	561

The displacement demand is a fundamental request in the seismic analysis, therefore the capacity to attain large deformation of the structural components has a great importance.

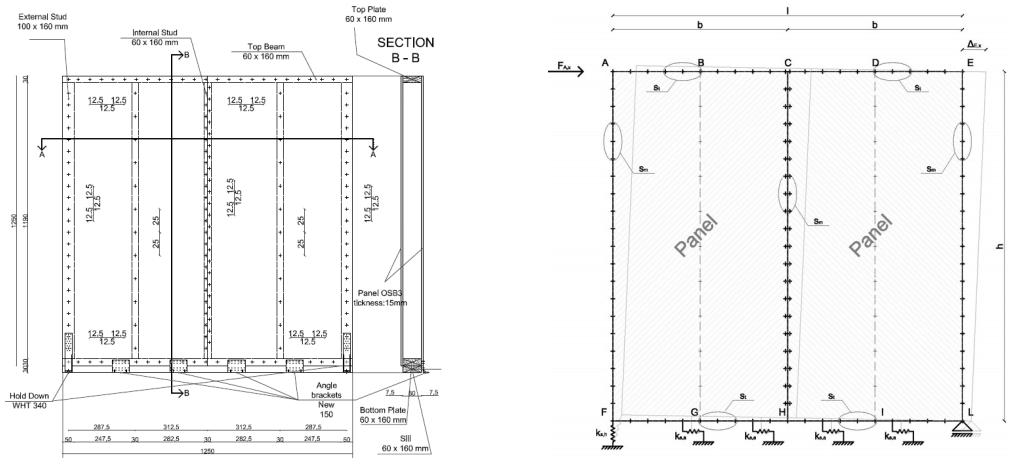


Figure 4. Wall geometry (above) and numerical model (bottom)

As observed by Kozem Šilih and Premrov [5], because the tensile strength of the sheathing is smaller than the strength of timber frame, the boards represent the weaker part of the entire system, where every presence of lack of structural continuity (e.g. an opening or a gap), could lead to brittle local failure of the walls. In the case of frame walls sheathed with FGB on one side and OSB on the other (FOSB15\_50), the strength capacity more close to the case of FGB on both side, but showing a more ductile behavior (the displacement at failure is similar to the case of OSB on both side). In the case of OSB sheathed connections has been remarked the rule of the board thickness (15 mm and 12 mm) on the connection resistance capacity.

During the specimen manufacturing process has been observed that, especially for the OSB sheet with a thickness of 12 mm, the fasteners could be easily overdriven and then bias the final capacity of the connection. An ad hoc experimental investigation has been performed, to this purpose, by Jones et.al.

From the discussion of these experimental results the Authors adopted, as standard sheathing to framing connection system for the next experimental and numerical investigation, the OSB boards fastened with  $2.8 \times 60$  mm annularly threaded nails.

### 3 HORIZONTAL DISPLACEMENT IN WOOD FRAMED SHEAR WALLS

A correct evaluation of horizontal displacements in wood framed shear walls is crucial in order to perform damage and serviceability assessment in multistorey buildings subjected to horizontal loads.

To this purpose the Author are proposing an analytical expression for the elastic displacement of timber frame walls subjected to in plane horizontal forces. This expression has been tested and validate through an ad hoc numerical model which

simulates the behavior of a standard wall configuration hereinafter described: timber frame with internal studs ( $160 \times 60$  mm) and external studs ( $160 \times 100$  mm) and bottom beams ( $160 \times 60$  mm) made of structural solid timber; 15 mm OSB sheathing boards fastened on both sides through annularly threaded nails  $2.8 \times 60$  mm; hold-down system to transmit the overturning vertical force to the foundation floor; L-shaped steel bracket to transmit the lateral force to the foundation floor.

The mechanical behavior of the connection systems adopted (hold-down and L-shaped), have been carried out in a previous experimental campaign described by Conte et. al.

In figure 4 is depicted the standard diaphragm wall and a scheme of the numerical model. The horizontal displacement produced in the point E as result of a force applied to the point A, is given by elastic contributions of the following structural components: (1) sheathing boards; (2) timber frame; (3) connection system between sheathing boards and timber frame; (4) connection system for lateral forces between wall diaphragm and foundation; (5) hold-down system for vertical tension forces between wall diaphragm and foundation; (6) contact between timber beams (stud and bottom beam) for vertical compression forces between wall diaphragm and foundation.

### *3.1 Numerical analyses*

In order to investigate the role of the different structural components and validate the propose analytical expression, the numerical model of the “standard” shear wall (figure 4) is here discussed. A finite element model has been generated utilizing a general purpose FEM code, assigning the elements parameters according to table 6.

Pinned frame elements have been utilized to model the timber frame, which should therefore be regarded as a mechanism: the sheathing boards, modeled through shell elements, guarantee the stability of the wall. The frame elements are connected to the shell points through linear elastic springs (Joint link) simulating the behavior of nailed connections. The bottom beams is then connected to the foundation by means of Joint linear link endowed with parameters validated through a previous experimental campaign (Conte et. al. 2011).

The evaluation of the influence of each structural component on the lateral flexibility of the wall is computed through the numerical model with the following procedure; the displacement in the E point of the shear wall is computed considering in the model the influence of all the structural components; the displacement in the E point of the shear wall is computed assigning in the model an infinitive stiffness to the structural component analyzed; the difference of the two calculated displacements represents the contribution on the global horizontal displacement of the analyzed structural component.

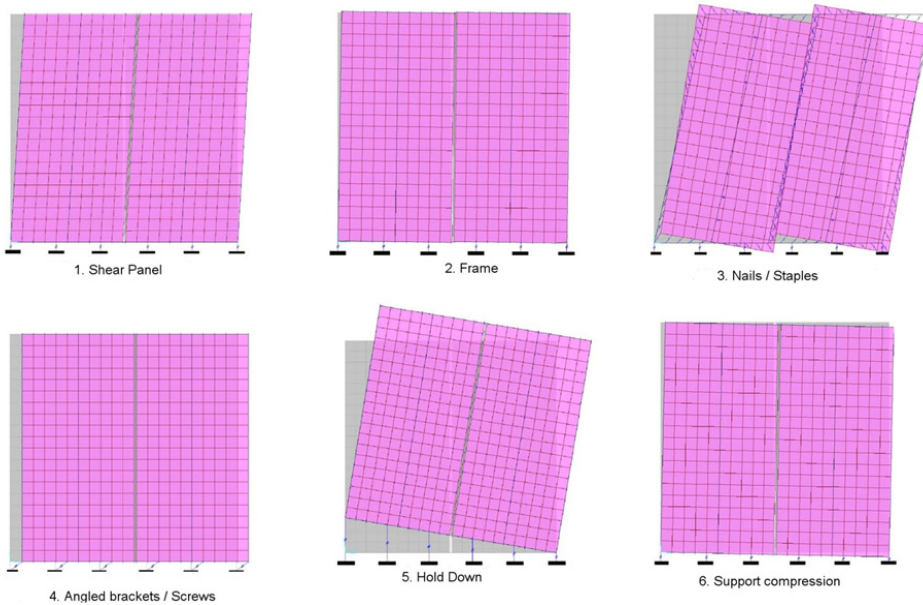


Figure 5. Horizontal components displacement in wood framed shear walls

This method has been adopted for all the components apart from the nailed connections, whose contribution on the global deformation is strictly related to the flexibility of framing members, see Källsner and Girhammar. In this case a better approximation of the contribution of the structural component is obtained from the model if all the other components are infinitely stiffened.

Figure 5 illustrates graphically the effect of the flexibility of the different structural components analyzed on the horizontal displacement.

Table 6. Finite element model parameters

Component	Element	Modeling (SAP 2000)	Parameters
1	Sheathing Panel	Shell	OSB (orthotropic – t = 15 mm) Fermacell (orthotropic – t = 12.5 mm)
2	Timber Frame	Frame	Timber - C24
3	Nails / Staples	2 Joint Link (elastic)	Nail Ring (k = 403 N/mm) Staple (k = 411 N/mm)

4	Angle brackets / Screws	2 Joint Link (elastic)	New 150 (k = 3746 N/mm) HBS Screw 8.0 × 180 mm (k = 387 N/mm) KR 135 (k = 440 N/mm)
5	Hold-downs	2 Joint Link (elastic)	WHT 340 (k = 3620 N/mm) WHT 620 (k = 9656 N/mm)
6	Compression at bottom beam	2 Joint Link (Elastic)	k = 98600 N/mm

### 3.2 Comparison between different approaches

In Standard and scientific literature different formulation has been proposed to evaluate the displacement values to be assigned to the single elastic components of a diaphragms shear walls under later load (without the presence of vertical load). Here will be discussed the approaches proposed in the Commentary (Erläuterung) of DIN 1052, in New Zealand Standard NZS 3603:1993, in Canadian Standard CSA 086-01:2005, and by Källsner and Girhammar in their elastic analytical model for fully anchored light-frame timber shear walls. These formulation has been compared in figure 6 with the results carried out from the analytical model, in order to verify the reliability of different approaches.

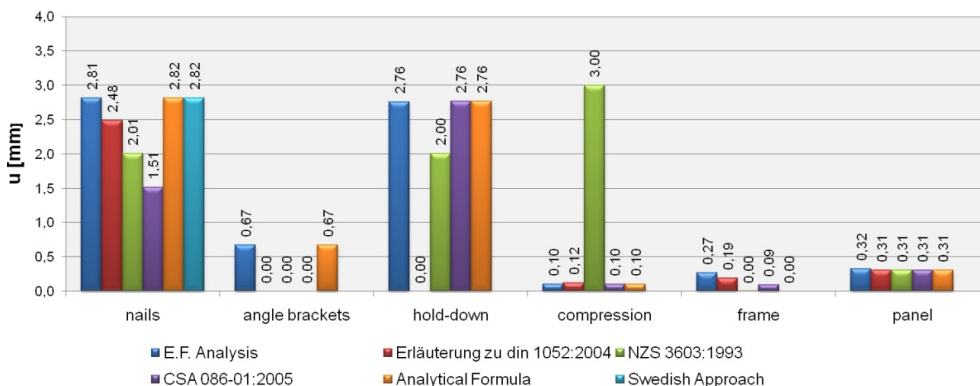


Figure 6. Comparison between the numerical model and different analytical approaches

For sheathing deformation the expressions analyzed are very similar one to the other. The New Zealand formula seems to overestimate the deformation of the bottom plate compressed locally by the vertical stud (maybe taking into account the smaller



section usually adopted for timber frame members in New Zealand, which could produce a higher local deformation). Källsner and Girhammar proposed a formally correct analytical model able to accurately predict the nails deformation. Most of the approaches taken into account seems to neglect some elastic component (deformation of timber frame, deformation of angle brackets, compression deformation), and to give more importance to other (nails deformation, hold-down deformation).

### 3.3 Formula suitable to predict the horizontal displacement of wooden shear walls

Taking into account the different approaches previously discussed the Authors proposed the following analytical expression for the global displacement of a diaphragms shear walls under later load (without the presence of vertical load).

$$\Delta_{E,x} = \frac{Fh}{l} \cdot \frac{1}{G_p n_p t_p} + \frac{F}{ln_p} \cdot 4.54 \cdot \frac{s_c}{k_c} + \frac{Fh}{lk_H} \cdot \frac{h}{l} + \frac{Fht_b}{lE_{90b}A_m} \cdot \frac{h}{l} \quad (1)$$

The single components of the equation are: (1)  $\Delta_p$  sheathing panels – elastic theory (adopted in all the approaches); (2)  $\Delta_c$  connectors between frame and sheathing (according to Källsner and Girhammar 2009); (3)  $\Delta_h$  connections against the horizontal slip – elastic theory; (4)  $\Delta_v$  connection against vertical uplift – elastic theory; (5)  $\Delta_b$  bottom beam compression – elastic theory.

The parameters indicated in eqn. (1) are: F is the horizontal force applied in A [N]; h is the panels height [mm]; l is the total wall length [mm];  $G_p$  is the G modulus of the sheathing panel [MPa];  $n_p$  is the number of sides on which are placed the sheathing panel (1 if only one side; 2 if both sides);  $t_p$  is the thickness of the sheathing panel [mm];  $s_c$  is the spacing of the connectors placed on the edge of each panel [mm];  $k_c$  is the stiffness of the connectors that connect the panel to the wood frame [N/mm];  $k_a$  is the horizontal stiffness of the angle brackets [N/mm], to be determined experimentally or from product technical data;  $n_a$  is the number of the angle brackets connected to the wall;  $k_H$  is the vertical stiffness of the hold-down [N/mm], to be determined experimentally or from product technical data;  $t_b$  is the thickness of the bottom beam [mm];  $A_m$  is the transversal area of the final vertical stud of the wall [mm<sup>2</sup>];  $E_{90,b}$  is the elastic modulus of the bottom beam [MPa].

According to figure 6, the proposed equation is able to predict more accurately the importance of the single elastic components with respect to the existing formulations. Is worth noting that the frame deformation contribution has not be taken into account by the Author's proposal.

## 4 CONCLUSION AND FUTURE WORK

The present work represent the first phase of a wider research project aimed to investigate all the structural components of a multistorey platform frame building. The future work will be dedicated to experimental analysis of diaphragms shear wall under monotonic and cyclic load, in order to provide an effective validations of the displacement expression proposed in the present paper.

Other problems that will be taken into account in this further analysis are the action of lateral and vertical load on the shear wall, the presence of window or door opening, the post elastic behaviour of the single structural component. This experimental investigation is indispensable in order to deepen the knowledge of the response of multistorey platform frame building under lateral load, especially during seismic event, when post elastic behavior of the structural element is involved.

## 5 ACKNOWLEDGMENT

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## REFERENCES

- CNR-DT 206:2007 – Istruzioni per la progettazione, l'esecuzione ed il controllo delle strutture di legno.
- Conte A., Piazza M., Tomasi R., Sartori T., Experimental investigation on connections between wood framed shear walls and foundations, Structural Engineering World Congress, Villa Erba, Como, Italy, 2011
- CSA Standard 086-01:2005. Engineering design in wood
- DIN 1052:2008 Deutsches Institut für Normung e.V. – Entwurf, Berechnung und Bemessung von Holzbauwerken. Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau
- DIN Deutsches Institut für normung e.V. Erläuterung zu DIN 1052:2004-08.
- EN 1995-1-1:2009 – Design of timber structures – Common rules and rules for buildings
- EN 26891:1991 – Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics
- Jones S. N., Fonseca F., Strength of OSB Shear Walls with Over-Driven Sheathing Nails, World Conference on Timber Engineering 2000, Canada
- Källsner B., Girhammar U.A., Analysis of fully anchored light-frame timber shear walls-elastic model, Materials and Structures (31), p. 2171-2181, 2009
- Kozem S. E., Premrov M., Analysis of timber-framed wall elements with openings, Construction and Building Materials, 2010
- NZS 3603:1993 – Timber structures standard



# Appendice B

## Paper II

Experimental investigation on connections between  
wood framed shear walls and foundations

*Andrea Conte, Maurizio Piazza, Tiziano Sartori, Roberto Tomasi*

SEWC 2011, Structural Engineer World Congress, Como



# Experimental investigation on connections between wood framed shear walls and foundations

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## Summary

Light timber structures have always been considered performing well during earthquakes, because of the high ratio between strength and mass. In platform frame constructions the task of resisting to the horizontal forces (wind, earthquake) is entirely left to the shear walls. The mechanical characterization of connections between the wall panels and the foundation and between the wall panels at different storeys, is extremely important in order to investigate the global response of multi-storey building to lateral loads.

To this purpose different types of commercial or ad hoc designed connections system were experimental investigated by means of monotonic and cyclic tests, in order to derive the stiffness and strength parameters. The experimental results confirmed the good mechanical behaviour of the proposed systems compared to traditional existing system.

A simple engineering model is proposed to describe the force transmission mechanism between the connections elements.

## Keywords

*Wood framed constructions, shear walls, hold-down, angle brackets*

## Theme

*Structural design – earthquake/wind – wood*

## 1. Introduction

In modern timber frame construction system for multi-storey building floors and walls are created by assembling lumber studs (for wall) or lumber joist (for floor) in a framed element, which is sheathed by means different sheathing materials, in order to act as structural diaphragms and

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allow the transfer of lateral loads (seismic or wind load) to the foundation. Modern platform system is a prefabricated system where construction elements are stacked storey by storey. As the consequence there is a lack of structural continuity between vertical elements, that must be eliminated by means of connectors systems able to transmit vertical and horizontal forces. In particular it is possible to distinguish two different types of connections according to the force transmitted to the ground: hold-down has to resist to the traction force due to the overturning moment; angle brackets transfer the horizontal shear to the ground. The scheme of force transmission is depicted in figure 1.

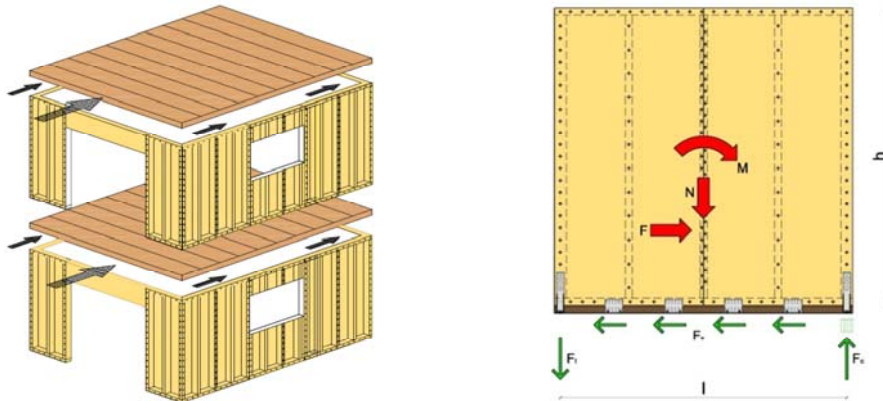
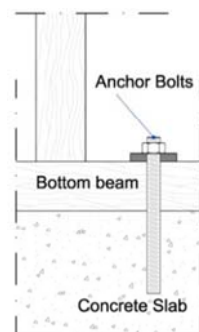


Figure 1- Role of the connection system between vertical panels and horizontal diaphragm

## 2. Angle brackets

### 2.1 Connection system

To prevent sliding, anchor bolts, inserted in a central hole drilled in the bottom beam of the shear wall, are usually adopted to connect the base of a shear wall to the foundation or at the floor level. However in shear walls factory-prefabricated as a “closed panel” (which includes insulation and may include internal and external lining) standard joining solutions are not possible and therefore alternative methods for in site assembling are needed. At the present a common practice solution is the use of steel angle bracket externally nailed to the timber wall as depicted in figure 1: they are manufactured from perforated mild steel sheets, cold bended at 90°, usually reinforced with ribs in order to add extra rigidity. The joint configuration is unsymmetrical: the angel brackets are inevitably fixed only to one side of the external walls, and often also in the internal. The geometry and shape of angle bracket may vary depending on the producers, but most of them are not specifically designed to transfer



shear forces in platform frame buildings system. As depicted in figure 2 there is only a narrow band in the bottom beam of the shear wall where is possible to fix the nails, fulfilling the standard requirement about the minimum edge distances: none of the commercial type of angle bracket consider in this analysis, seems to be optimized for being fully nailed to the base of the wall (for most of the products the holes are situated outside the useful band of the bottom beam or in the empty part of the panel).

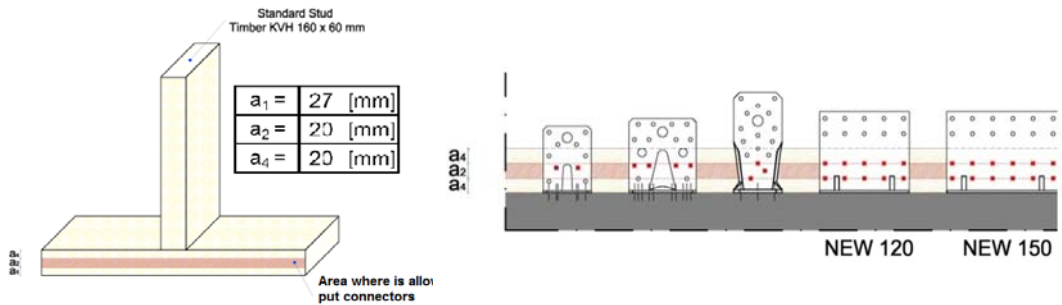


Figure 2- Useful band of the bottom beam where is possible to insert nails

Taking into account a standard geometry of the shear walls described by the Authors [1], where the bottom beams adopted were 60 × 160 mm timber elements, two type of angle brackets has been ad hoc designed: the shape and the geometry proposed wanted to optimized the maximum number of fastener nailed to the bottom beam of the shear walls, taking into account also the possibility of the presence of a further sill beam between the panel and the foundation floor. The vertical flange of the new angle bracket was endowed by four holes rows: the two rows in the bottom part of the vertical flange are used in the case of the walls panel fixed directly to the foundation, the two rows in the upper part of the vertical flange are used in the case of the presence of a interposed 60 × 160 mm sill beam. The new angle bracket NEW120 and NEW150 differ only for the length of the holes row and for the number of possible fastener for each row (figure 3).



Figure 3- Connection with screws, angle brackets NEW150 and NEW120

Another type of connection system was proposed during the experimental campaign, where self-



tapping screws connect the bottom beam to the lower wooden sill beam (see figure 3). The screws are inserted with an inclination of  $45^\circ$  with respect to the horizontal plane, and could be easily fixed in site on factory-prefabricated as a “closed panel”. The lower sill plate could be previously fixed to the foundations floor with anchors bolts inserted in a central position. Therefore the joint configuration could be considered perfectly symmetrical with respect to the central axis of the shear wall.

## 2.2 Force transmission model

Preliminary finite element analysis showed that the behaviour of angle brackets is strongly influenced by the connector system geometry. In order to demonstrate this result let us consider different internal force distributions in a typical angle bracket as depicted in figure 4 (forces in green and yellow colours): in the left scheme the external force is located in the centroid of connectors group of the angle bracket vertical flange, in the right scheme in the centroid of connectors group of the angle bracket horizontal flange. It is worth noting that the force transmission schemes of figure 4 are statically admissible and therefore, if they are safe, the lower bound plastic theorem could be applied. In the force distribution scheme on the left of figure 4, moment  $M_y$  and  $M_x$  and a shear force  $R_n$  are transmitted to the angle bracket horizontal flange. Moment  $M_x = R_n \times e_y$  is acting on a vertical plane and causes compression  $\sigma_n$  on the horizontal flange and tension  $R_n \times e_y / c$  on the anchor bolt, moment  $M_y = R_n \times e_x$  is acting on an horizontal plane and causes lateral forces  $R_n \times e_x / c$  on the anchor bolts. A more efficient angle bracket geometry should limit the torsional effect on the steel flange and on the anchor bolts, and therefore minimize the eccentricity  $e_x$  and  $e_y$ , and maximize the distance  $c$  between anchors (i.e. the width  $b$  of the angle bracket). Similar considerations could be led for the force distribution scheme on the right of figure 4, where moment  $M_y$  and  $M_x$  and shear force  $R_n$  are transmitted to the angle bracket vertical flange. Also in this case the efficiency of the connections is favoured by limiting the eccentricity  $e_x$  and  $e_y$ , whereas a larger polar moment of inertia  $J$  of the nails fastener group (sum of squares of distances of fasteners in group from the connection centroid) is to be preferred (the polar moment of inertia  $J$  is proportional to the width  $b$  of the angle bracket). This simple model demonstrates that stiffness and strength of angle brackets are strictly related to the eccentricity of the connection system geometry, and that a higher width of angle bracket favours the mechanical behaviour of the connectors.

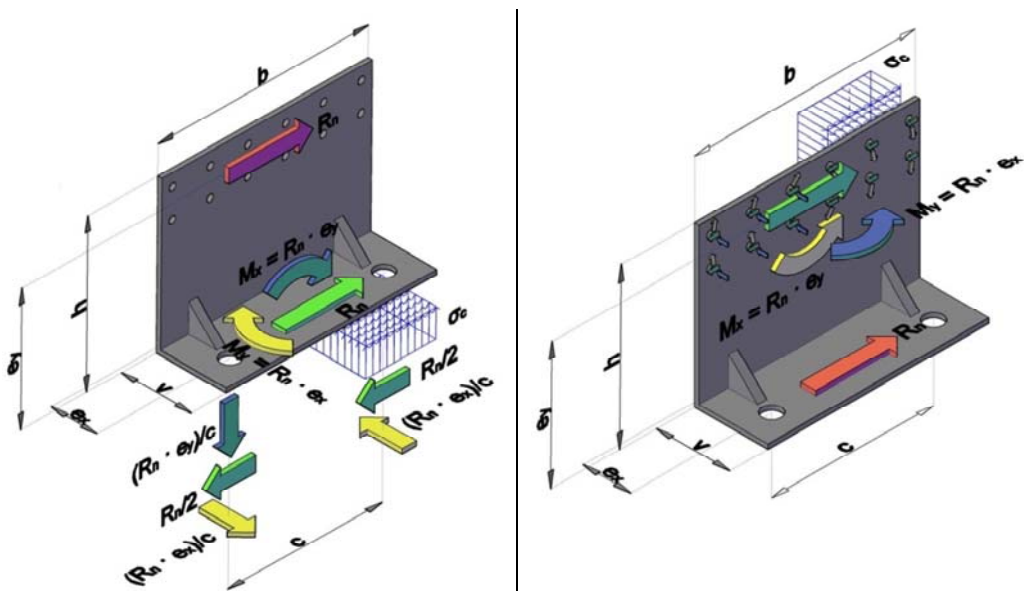


Figure 4- Influence of the angle bracket geometry on force patterns on anchor bolts (left) and on nails fastener group (right)

## 2.3 Experimental campaign

### 2.3.1 Materials and geometry

The specimen and set-up geometry was designed in order to most accurately reproduce the actual shear force pattern between wall and foundation. The specimen is a rigid box manufactured with a timber frame sheathed with 15 mm OSB boards. The board were glued with epoxy resin to the box perimeter, beside the edge where the angle bracket was fixed with annularly threaded nail 4 × 60 mm (anker nail). As shown in figure 5, the wooden box simulating the timber frame wall is fixed to a steel base reaction frame by 2 UPN 120 profiles counteracting the vertical displacements and to the hydraulic jack by mean of a rigid vertical steel plate which simulates the foundation slab. The connection system joints the wooden box to the rigid vertical steel plate: the non-symmetric set-up configuration reproduces the actual in site geometry (figure 1). The applied force was measured by a load cell placed between the jack actuator and the specimen; the relative and global displacements along the shear planes were measured using 3 L.V.D.T (linear variable differential transducers) placed on both sides of the specimens; the rotation referred to the vertical nailed plate of the angle bracket was measured by means of an inclinometer. The tests were implemented with a displacement control method; in monotonic tests the loading was applied under displacement control up to failure at a constant rate of 0,05 mm/sec, according to the provisions of EN 26891 [2]

### 2.3.2 Test results

In the experimental campaign on lateral shear connections system globally 16 specimens were tested (9 monotonic test an 7 cyclic test). The meaning of test label assigned to each specimen is illustrated in table 1. 135 stands for a commercial type angle bracket with a height of vertical flange of 135 mm, NEW 120 and NEW 150 stand for newly designed angle bracket with a lateral width of 120 mm and 150 mm respectively, SCREW stands for self-tapping screws inserted with an inclination of 45° with respect to the horizontal plane connecting the bottom beam to the sill beam.

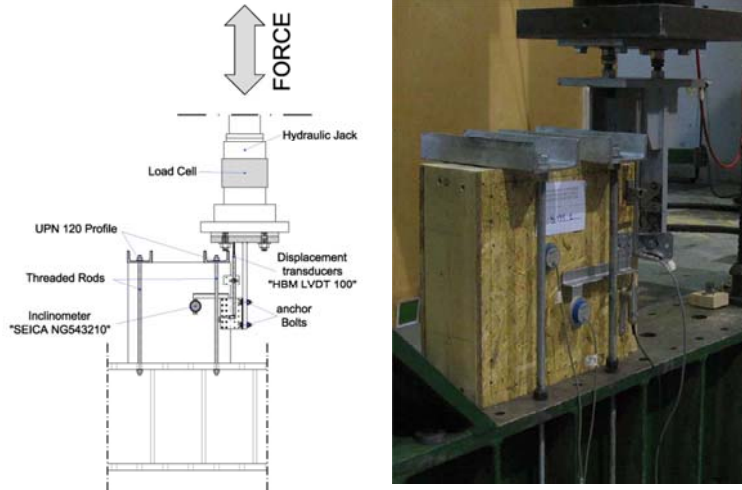


Figure 5 - Test set up for angle brackets and screws connection

Table 1 – Specimen label significance

Sigla provino: X_YYYYY_Z		
X	Y	Z
<p><b>S</b> = static test  <b>C</b> = cyclic test</p>	<p><b>135</b> = 135 Angle Brackets  <b>NEW120</b> = NEW 120 Angle Brackets  <b>NEW150</b> = NEW 150 Angle Brackets  <b>SCREW</b> = 4 × HBS 8 × 180 mm Screw</p>	<p>N° test</p>

When in the label for the specimen NEW 150 the letter “V” is present, in the specimen configuration the bottom beam of the shear wall is adjacent to the foundation slab. In other cases there was a sill beam between shear wall and the foundation slab.

In table 2 are reported, for each specimen configuration, the experimental results in terms of stiffness and strength values, carried out according to the definition reported in European Norm 26891. Some experimental results are also reported in figure 6 in terms of force - displacement relationship, where the straight lines reproduce the values provided from EN 1995 [3] for strength and stiffness of the single nail multiplied by the total number of nails.

Table 2- Test Results

Test		S_135_1	S_135_2	S_NEW120_1	S_NEW120_2	S_NEW150_1	S_NEW150_2	S_NEW150_V1	S_SCREW_1	S_SCREW_2
Angle Brackets	Type	135	KR135	NEW120	NEW120	NEW150	NEW150	NEW150	-	-
	t [mm]	4	4	3	3	3	3	3	-	-
	b [mm]	65	65	120	120	150	150	150	-	-
	H [mm]	135	5	109	109	5	5	5	-	-
	v [mm]	90	90	50	50	50	50	50	-	-
	$\Phi_{holes}$ [mm]	5	5	5	5	5	5	5	-	-
	$\Phi_{anchor}$ [mm]	12	12	12	12	12	12	12	-	-
	nanchors	-	1	1	2	2	2	2	-	-
rib	no	no	yes	yes	yes	yes	yes	-	-	
Connector	Type	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	8 x 180	8x180
	$\Phi_{connector}$ [mm]	4	4	4	4	4	4	4	8	8
	L <sub>connector</sub> [mm]	60	60	60	60	60	60	60	180	180
	n <sub>connector</sub>	-	8	5	10	10	12	12	4	4
	F <sub>connector</sub> [kN]	1,22	1,43	1,59	1,68	1,80	1,90	2,34	3,19	3,47
	k <sub>s</sub> [N/mm]	440	319	2088	1279	3411	4081	3108	1275	1825
Test Data	F <sub>Max</sub> [kN]	9,72	7,16	15,85	16,80	21,60	22,84	28,08	12,76	13,88
	v <sub>Fmax</sub> [mm]	28,80	30,00	25,32	28,81	19,84	17,55	10,66	30,00	30,00
	F <sub>u</sub> [kN]	9,72	7,16	15,85	16,80	21,60	22,84	22,46	12,76	13,88
	v <sub>u</sub> [mm]	28,80	30,00	25,32	28,81	19,84	17,55	15,09	30,00	30,00

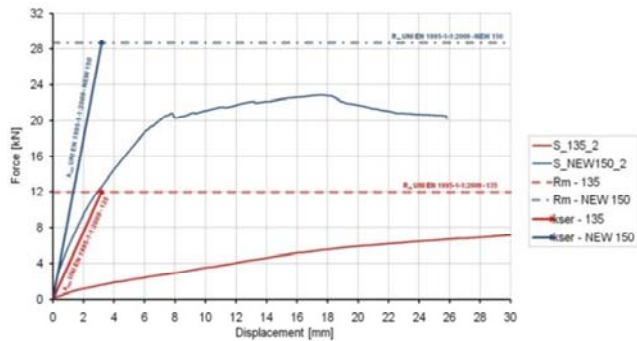


Figure 6 – Force vs. displacement graph: comparison between test results and EN 1995 for angle bracket NEW150 and 135

### 2.3.3 Discussion

To better comment the test results, histograms in Figure 7 have been reported. On the left side is plotted the ratio between maximum force achieved in the test and the number of fasteners adopted in the connection system. On the right side is reported the global stiffness of the connections.

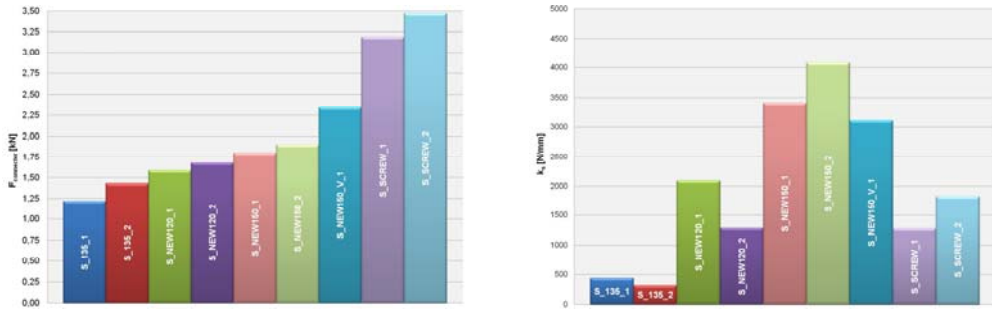


Figure 7 - Ratio between maximum force and number of connectors (left) and stiffness of the connection (right)

According to the charts in Figure 7, angle bracket 135 performs badly both in stiffness and strength. The stretched geometry of this angle bracket, according to the model explained in chapter 2.2, makes it very deformable. In addition, the presence of only one anchor bolt and the possibility to insert just few nails in the wooden wall, increase the rotation and consequently the deformation of this angle bracket (see Figure 8).

Both angle brackets NEW150 and NEW120 have proved to provide high values of strength and stiffness. The connection to the foundations made by two anchors bolts is very stiff. Furthermore the geometry is suitable to provide a larger polar moment of inertia of the fastener group, that prevents the rotation of the vertical flange. Test results show that the difference in strength between NEW120 and NEW150 is proportional to the number of nails. On the contrary stiffness values increases more than proportional way from NEW120 to NEW150 (55% higher). The connection made by HBS screws points out a good behaviour for both strength and stiffness. In the test, as described above, screws 8x180 were used. Different screws can be used depending on the bottom and sill beam thickness. This solution is particularly advantageous because its easiness of installation and its symmetry in force pattern, and it can be an interesting alternative to the use of angle brackets. The idea is to pre-drill the wall in the factory and then insert the screw at the building site in the pre-drilled holes. On the contrary stiffness values increases more than proportional way from NEW120 to NEW150 (55% higher). The connection made by HBS screws points out a good behaviour for both strength and stiffness. In the test, as described above, screws 8x180 were used. Different screws can be used depending on the bottom and sill beam thickness. This solution is particularly advantageous because its easiness of installation and its symmetry in force pattern, and it can be an interesting alternative to the use of angle brackets. The idea is to pre-drill the wall in the factory and then insert the screw at the building site in the pre-drilled holes.

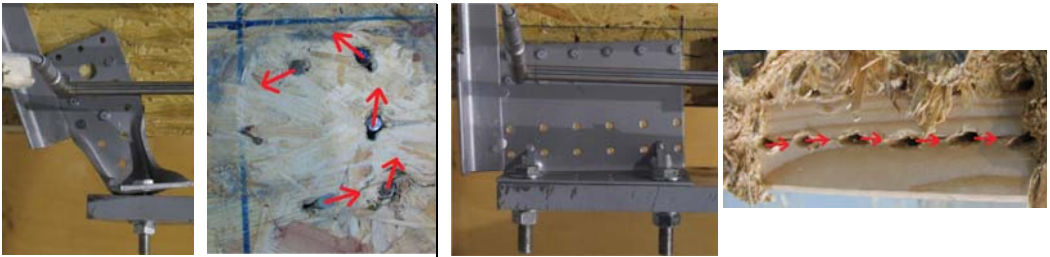


Figure 8 – Angle bracket 135 deformation (left) and angle bracket NEW150 deformation (right)

### 3. Hold-down

#### 3.1 Connection system

The vertical forces at the lower corner of shear walls (figure 1) necessitate metal hold-down to connect the shear wall chord to the foundation or to lower shear walls. Hold-down elements prevent the uplift and the rotation of the panel, so they are designed with a particular geometry to resist to vertical forces (see figure near table 3): they are usually L-shape metal plate endowed with ribs in order to transfer the vertical force (aligned with the vertical flange symmetry axis) to the anchor bolt placed centrally to the base flange. In some hold-down system, in addition to standard anchor washer, a further thick washer plate was adopted as to prevent negative effects caused by local buckling mechanisms and to distribute the compression stress over a larger surface. In traditional timber frame technology hold-down are placed inside the thickness of the wall panel: vertical flange is nailed to the external stud of a timber shear wall and the anchor bolts is connected the foundation or to the lower wall through the sill beam. On the contrary when shear walls are prefabricated “closed panel”, the only method for assembling in site is to place the hold-down only to one side of the shear walls (inevitably in the external and usually also in the internal). The joint configuration is therefore, as in the case of angle bracket unsymmetrical.

#### 3.3 Experimental campaign

##### 3.3.1 Materials and geometry

The specimen and set-up geometry was designed in order to most accurately reproduce the actual vertical force pattern between wall and foundation. Hold-downs are usually aligned with the axis of external stud of timber shear walls, an nailed over the OSB boards: therefore the specimen was manufactured nailing a 15 mm OSB sheet (annularly threaded nail 2.8 x 60 mm) to a timber beam with section 80 x 240 mm. In some specimen the OSB sheet was omitted for comparison. This composed element was clamped to a steel HEA profile by mean of four

threaded bars and two steel plate as depicted in figure 9; the HEA steel profile is jointed to the hydraulic jack head, whereas the hold-down joints the specimen with the steel reaction frame: the non-symmetric set-up configuration reproduces the actual in site geometry (figure 9). The applied force was measured by a load cell placed between the jack actuator and the specimen; the relative and global displacements along the shear plane were measured using 3 L.V.D.T placed on both sides of the specimens; the rotation referred to the vertical nailed plate was measured by means of an inclinometer.

The tests were implemented with a displacement control method; in monotonic tests the loading was applied under displacement control up to failure at a constant rate of 0,05 mm/sec, according to EN 26891 [1].

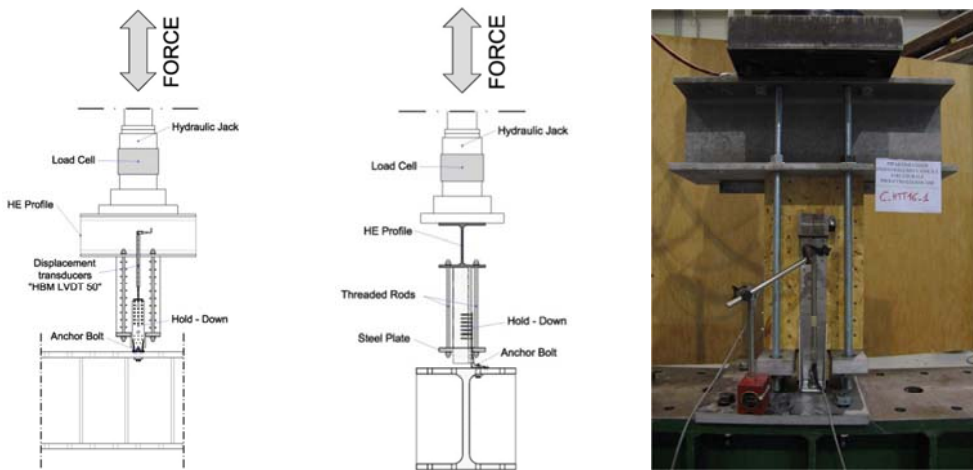


Figure 9 - Hold-down test set up

### 3.3.2 Test results

In the experimental campaign on lateral shear connections system globally 16 specimens were tested (11 monotonic test and 5 cyclic test). The meaning of test label assigned to each specimen is illustrated in table 3.

Table 3 – Specimen label significance

Sigla provino: X YYYYYY Z		
X	Y	Z
S = monotonic test C = cyclic test	HD16 = Hold-down type 16 HD285 = Hold-down type 285 HDRP = Hold-down type 340 HDRG = Hold-down type 620	N° test

When in the label the letter “W” is present, in the specimen configuration the OSB sheet is omitted. In table 4 are reported, for each specimen configuration, the geometry and the

experimental results in terms of stiffness and strength values, carried out according to the definition reported in European Norm 26891.

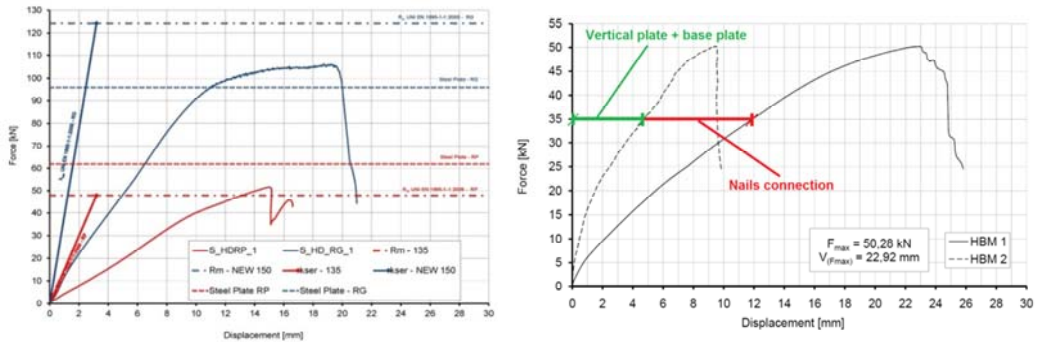


Figure 10 – Force vs. displacement graph: comparison between test results and EN 1995 for hold-down (left); different deformation contributions (right)

Table 4- Test results

Test		S_HD16_1	S_HD16_2	S_HD16_W_2	S_HD16_W_3	S_HD285_1	S_HD285_2	S_HD285_3	S_HDRP_1	S_HDRP_2	S_HDRG_1	S_HDRG_2
Hold - Down	Model	16	16	16	16	285	285	285	RP	RP	RG	RG
	t [mm]	2,5	2,5	2,5	2,5	4	4	4	3	3	3	3
	b [mm]	65	65	65	65	65	65	65	60	60	80	80
	h [mm]	406	406	406	406	285	285	285	340	340	620	620
	v [mm]	61	61	61	61	90	90	90	63	63	83	83
	Φholes [mm]	4,7	4,7	4,7	4,7	5	5	5	5	5	5	5
	nholes	-	2	2	2	3	3	3	3	3	2	2
	Thick washer [mm]	-	-	-	-	-	-	-	56 x 50 x 10	56 x 50 x 10	77 x 70 x 20	77 x 70 x 20
	Net Area [mm²]	139	139	139	139	200	200	200	135	135	210	210
	Steel	S275	S275	S275	S275	S235	S235	S235	S355	S355	S355	S355
f <sub>y</sub> [MPa]	275	275	275	275	235	235	235	355	355	355	355	
f <sub>u</sub> [MPa]	430	430	430	430	360	360	360	510	510	510	510	
Connector	Model	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60	4 x 60
	Φ <sub>connector</sub> [mm]	4	4	4	4	4	4	4	4	4	4	4
	L <sub>connector</sub> [mm]	60	60	60	60	60	60	60	60	60	60	60
	n <sub>connector</sub>	18	18	18	18	19	19	19	20	20	52	52
F <sub>connector</sub> [kN]	3,04	3,09	2,79	2,75	2,41	2,48	2,36	2,58	2,65	2,04	2,02	
Test data	F <sub>Max</sub> [kN]	54,72	55,64	50,28	49,48	45,88	47,04	44,88	51,56	53,08	106,28	105,12
	V <sub>Fmax</sub> [mm]	24,33	28,92	22,92	23,80	28,76	27,21	28,25	14,96	12,83	18,74	18,96
	F <sub>u</sub> [kN]	54,72	55,64	50,28	49,48	45,88	47,04	44,88	51,56	53,08	106,28	105,12
	v <sub>u</sub> [mm]	24,33	28,92	22,92	23,80	28,76	27,21	28,25	14,96	12,83	18,74	18,96
	k <sub>s</sub> [N/mm]	3386	2774	3155	3041	2223	2942	4472	3817	3424	9276	10045
	σ <sub>acc</sub> [MPa]	393,6	400,29	361,73	355,97	229,40	235,20	224,40	381,93	393,19	506,10	500,57
	σ <sub>acc</sub> /f <sub>y</sub>	-	1,43	1,46	1,32	1,29	0,98	1,00	0,95	1,08	1,11	1,43

### 3.3.3 Discussion

Charts in Figure 12 have been reported, in order to better comment the test results. On the left side is plotted the maximum force divided by the number of nails while on the right is highlighted



the global stiffness of the connections. The comparing of the tests where a OSB panel is interposed between timber and hold-down with the ones where hold-down was connected directly to timber stud, showed that the interposition of the OSB doesn't affect the behaviour of the connections (only a slight increase of connection strength is observed with OSB boards). As a consequence, the bearing capacity of an hold-down can be calculated neglecting the presence of the OSB layer interposed with an acceptable accuracy. From the stiffness point of view all type of hold-down show similar values. The use of hold-down HD285, HD16 or HDRP does not induce substantial differences in both stiffness and carrying capacity of the timber framed wall. Only RG showed significantly higher values in term of strength (two times higher) and stiffness (three times higher) if compared with the others hold-down: this could be a valid solution when greater strength and stiffness are requested.

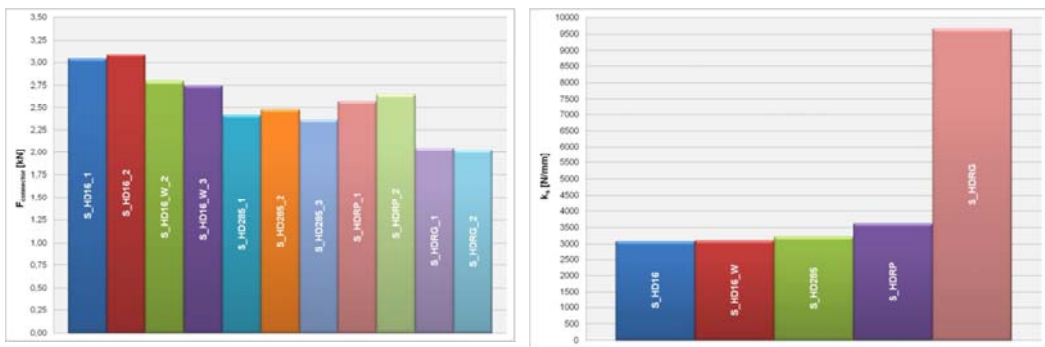


Figure 11 - Ratio between maximum force and number of connectors (left); stiffness of the connection (right)

A formula suitable to calculate both strength and stiffness of the hold-down is not included in EN 1995. The results of the test confirm that the calculation of the bearing capacity obtained by the sum of the single connectors capacity with the Johansen theory is a good approximation (figure 10). This is true if the vertical steel flange is subjected to a strength compatible with the failure criterion. Otherwise the carrying capacity of the hold-down is given by the tension resistance of the vertical nailed flange (this happens for HDRG, see figure 12). Regarding the stiffness, a similar calculation is not from the safe side, because it leads to overestimate five times the values obtained in the tests. As a matter of fact, in addition to the deformation due to the nailed connection, there are other two contributions that should be taken into account. The total deformation is given by three principal components (Figure 12). The first contribution is due to local deformation of the base steel plate. An additional thick washer can improve this behaviour, as happens with the HD340. The second contribution is the result of the deformation of the nailed connections (this part is the only one that is implemented in EN 1995). The last contribution is given by the elongation of the vertical steel flange. In Figure 10 right are highlighted the deformation contribution given by different components. From these considerations it is clear that a formula suitable to well predict the stiffness of this system is needed.



Figure 12 - Steel base deformation, nailed connection deformation and deformation of the vertical flange

#### 4. Conclusion and future Work

The commercial product usually adopted for factory-prefabricated shear wall seem not to completely fulfil the requirements for platform frame system. Regarding the shear connections different solutions (NEW120, NEW150, SCREW) have been proposed by the Authors, which should be surely further investigated both with numerical and experimental analysis. Future work will be address to propose reliable design model for practitioners and researcher in order to correctly predict the mechanical behaviour of the joint, and to broaden the experimental data with a campaign on real scale timber frame walls subjected to lateral monotonic and cyclic load.

#### Acknowledgment

The authors gratefully acknowledge the CHI-Quadrato Consortium for partly financing the study, within the research program supported by Autonomous Province of Trento, and the company Rothoblaas s.r.l. for supplying the connections system used for experimentation. The research was also partially financed within the framework of the RELUIS Project (Italian Emergency Management Agency).

#### References

- [1] Conte A., Piazza M., Tomasi R., Sartori T., Influence of sheathing to framing connections on mechanical properties of wood framed shear walls, Structural Engineering World Congress, Villa Erba, Como, Italy, 2011
- [2] EN 26891:1991 – Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics
- [3] UNI EN 1995-1-1:2009 – Design of timber structures – Common rules and rules for buildings



# Appendice C

## Paper III

Characterization of the mechanical behavior of light  
frame timber shear walls through full scale tests

*Tiziano Sartori, Maurizio Piazza, Roberto Tomasi, Paolo Grossi*

WCTE 2012, Auckland



# CHARACTERIZATION OF THE MECHANICAL BEHAVIOUR OF LIGHT-FRAME TIMBER SHEAR WALLS THROUGH FULL-SCALE TESTS

Tiziano Sartori<sup>1</sup>, Maurizio Piazza<sup>2</sup>, Roberto Tomasi<sup>3</sup>, Paolo Grossi<sup>4</sup>

**ABSTRACT:** For the horizontal stability of wood framed buildings, diaphragm action in the walls is of crucial importance. For this reason the mechanical behaviour of these walls subjected to in-plane forces have been investigated. 11 monotonic and 11 cyclic destructive tests were performed on full scale walls (2.5 m x 2.5 m). In order to recreate the real walls boundary condition a specific test set-up was built. Different parameters were changed during the tests. Different vertical loads were applied, different angle brackets and hold-downs were used and different sheathing panels were connected to the wood frame. Also the presence of openings were investigated. Furthermore two walls with non-structural finishing were tested to correlate the inter storey drift with the level of damage. Data obtained from the experimental tests highlighted the role of vertical load in the mechanical behaviour in term of resistance and stiffness of the walls, and in terms of the type of failure, which can occur in the holddown elements or in the connection between stud and sheathing panel. The data collected from these tests were then used within a frame of a research project aimed to characterize the behaviour of multi-storey timber buildings taking into account the properties of the structural components; the final step of the research is the validation of the behaviour of a three storey wood frame building subjected to a seismic event in a shaking table test.

**KEYWORDS:** Wood framed shear walls, hold down, angle bracket

## 1 INTRODUCTION

Timber framed buildings represent one of the most used constructions system for

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residential houses around the world. In the U.S.A., in Canada and in the Nordic Country most of the residential buildings are made by this method. Also in Italy, in the last few years, a growing interest in this type of constructions was born. Because of its great widespread a lot of researches were carried out in order to evaluate the mechanical behaviour of shear walls. Nevertheless some aspects need to be still investigated. In particular is of great importance to do research in order to enlighten the peculiarity of the Italian system. In Italy, as usually in some other European countries, this type of construction is prefabricated in the factory. The tendency is therefore to prefabricate as much as possible in factory, in order to reduce the time spent in the construction site.

## 2 EXPERIMENTAL CAMPAIGN

### 2.1 SCOPE OF THE TESTS

The goal of the performed tests was the evaluation of the influence of some variables on the behavior of shear wall. The importance of vertical load, the influence of nail spacing on the system behavior in relation to failure mode and ductility of the wall have been analyzed. The behavior of the wall using different sheathing panel (OSB Oriented Strand Board-GFB Gypsum Fiber) and the presence of a window in the wall that interrupts partially the continuity of the sheathing panels were investigated. Different solutions to avoid the sliding and the uplift of the wall have been tested (angle brackets "new 150"/screws – hold down “RG”/“RP” according to Figure 2). Also to estimate the importance of elements against the uplift of the wall two tests without hold-down were performed. Finally, in order to compare the level of interstorey drift displacement with the damage of the non- structural components two walls were tested with internal and external finishes. In the table 1 the explanation of the label specimens are reported.

**Table 1: Specimen label significance**

Specimen label = XX_YY_ZZ		
XX	YY	ZZ
S = monotonic test	ST = standard wall	L00 = vertical load 00 [kNm]
C = cyclic test	OP = with opening	50 = nails spacing 50 mm
SC = monotonic test obtained as envelope of cyclic test	2F = gypsum fiber	WHD = without hold down
	FOSB = gypsum fiber and OSB	RIV = wall with finishing

In table 2 is reported the name of the specimens related to the parameters investigated.

**Table 2: Comparisons between different monotonic tests depending on the parameters investigated**

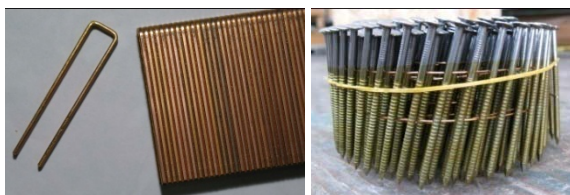
Vertical load	L:0kN/m S_ST_L0	L:10kN/m S_ST_L10	L:20kN/m S_ST_L20
Nails spacing	Sp:50mm SC_ST_50	Sp:100mm S_ST_L20	Sp:150mm SC_ST_150
Sheathing panel	OSB S_ST_L20	GFB S_2F	OSB/GFB S_FOSB

Openings	Standard S_ST_L10	Window S_OP	
Angle brackets	"New 150"	HBS screws	
	S_ST_L20	S_ST_SC REW	
Hold-down	RG S_ST_50_R G	RP SC_ST_50	
	H.D. RP	Without hold-down	
Hold down	S_ST_L10	S_ST_WH D	
Finishes	Standard	Finishes type 1	Finishes type 2
	S_ST_L20	S_ST_RIV1	S_ST_RIV2

## 2.2 MATERIALS AND GEOMETRY

The materials used for the realization of the samples are the commonly used in the Italian building tradition.

For this reason the horizontal load carrying capacity was provided by panel made of oriented strand boards (OSB) or gypsum fiber boards (FGB). The frame was made of solid wood elements (KVH). The connection between studs and panels was realized through ring nails or metal staples.



**Figure 1:** metal staples (sx) and ring nails (dx)

The dimensions of the elements used in the construction of the walls are shown in table 2.

**Table 3:** materials used in the production of samples.

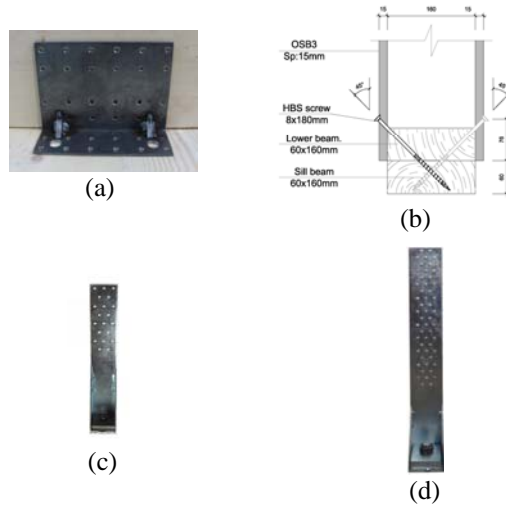
Frame	External stud: 100x160mm	Internal stud: 60x160mm
	Upper beam: 100x160mm	Lower beam: 60x160mm
Sheathing panel	OSB Th: 15mm	FGB Th: 12.5mm
	fastener system	Ring nails 2.8x60mm

The connection to the foundation was made with different systems. In particular to prevent horizontal sliding of the wall two solutions were adopted. The first one involves the use of steel brackets nailed to the lower beam of the frame (Figure 2a). The second one is done by the direct connection with wood screws between the wall and the lower wooden sill beam, which is bolted to the concrete foundation (Figure 2b). To prevent the uplift and the rotation of the shear wall hold down elements were inserted. Two models of hold-down featured by different strength were used (Figure 2c – 2d). Angle brackets and hold-downs were both nailed with anchor nails 4x60mm. The anchoring systems used are listed in table 3 and reported in Figure 2.

**Table 4:** connection systems between wall and foundation.

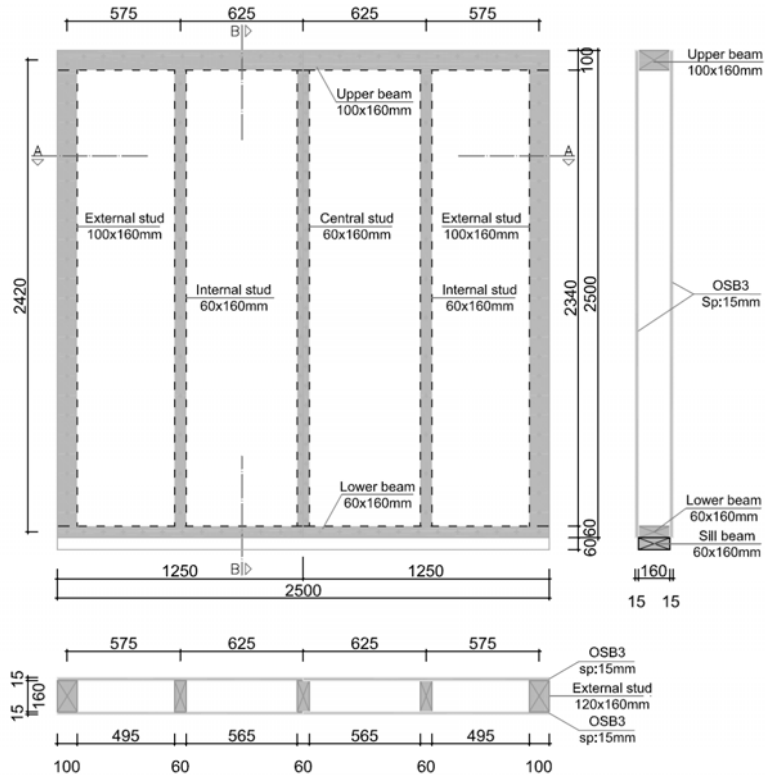


		Connection to the wall	Connection to the foundation
Angle bracket “new 150”		12 anker nails 4x60mm	2 bolt diam 12mm
HBS screws		25 screws inserted at 45°	
Hold down RP	down	20 anker nails 4x60mm	1 bolt diam 16mm
Hold down RG	down	52 anker nails 4x60mm	1 bolt diam 20mm



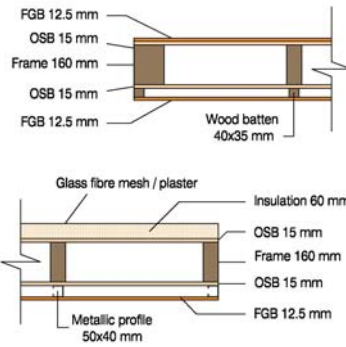
**Figure 2:** (a) angle bracket model “new 150”- (b) connection with wooden screws – (c) hold down “RP” – (d) hold down RG

The samples tested had square shape with side of 2.5m and a thickness ranging between 185 mm and 190 mm depending on the material used for sheathing panel (Figure 3).



**Figure 3:** geometry of samples

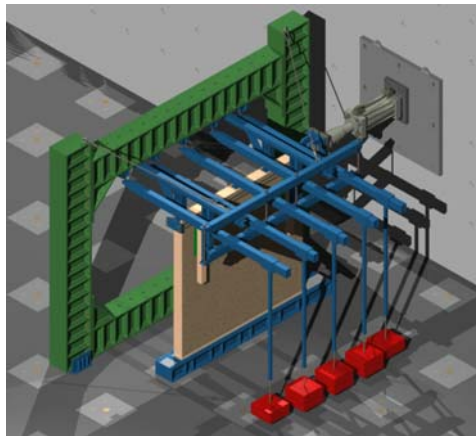
The internal frame was made of solid wood elements. For the outer stud and top beam were used 100x160mm sections while the interior stud and the bottom beam were used 60x160 mm sections. The spacing between stud amounts to 625 mm. Regarding the structural coating material different configurations were tested. In addition to the standard configuration equipped with OSB and the solution with GFB on both sides of the frame was tested also the mixed configuration. In addition different geometries have been tested. In particular walls with a central opening and walls with non-structural surface finishes were studied. The wall with opening contained a few additional items inside the frame to allow the nailing on the perimeter of the opening (Figure 21). The walls with non-structural finishing have the classical stratification used in the construction of perimeter structural walls and internal shear walls. Both walls tested had a counter wall for the installations.



**Figure 4:** composition of walls equipped with finishing (S\_ST\_RIV1 above – S\_ST\_RIV2 below)

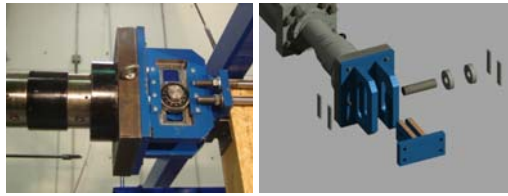
### 2.3 TEST SET-UP

The setup used during the experimental campaign allows the simultaneous application of vertical load and horizontal displacement. The application of the vertical loads was obtained by counterweights suspended to horizontal lever hinged on a fixed frame. To allow the three-dimensional independent free displacement of the beams spherical hinges were used.



**Figure 5:** test set-up

To apply different load levels, the position of the point of suspension of counterweights on the levers can be translated. The horizontal displacement at the top of the wall is provided by a hydraulic actuator with a special device. This allows relative rotation and the vertical scroll between the plate against the wall and actuator fixed to the contrast wall. In order to enable cyclic tests, on the opposite side of the wall, was fixed a plate attached with four threaded bars to the head of the actuator.



**Figure 6:** device designed to apply the horizontal displacement.

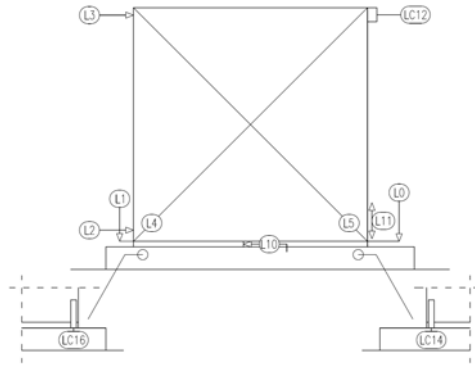
To prevent overturning of the wall in the direction orthogonal to the test, vertical pads connected to the fixed frame were placed. To minimize the friction forces between the wall and set up the pads were covered with sheets of plastic material with a low friction surface (polyzene). Angle bracket and hold-down that constrain the sample were fixed to a base made of steel and filled with concrete. This structure has been realized by two UPN profiles spaced with steel traverses to whose inside concrete has been cast. On the traverses were welded threaded bushes to allow screwing bolts of the sill beam, while on the wings of UPN profiles were prepared holes for the insertion of the angle brackets and hold down's bolts (Figure 7). Using these components it was possible to reproduce the real fixing between the sill beams and the foundation and it was also possible to replace quickly the sample without making every time a new hole in the concrete base.



**Figure 7:** anchor the sill beam and the wall fixings

## 2.4 TEST PROCEDURE

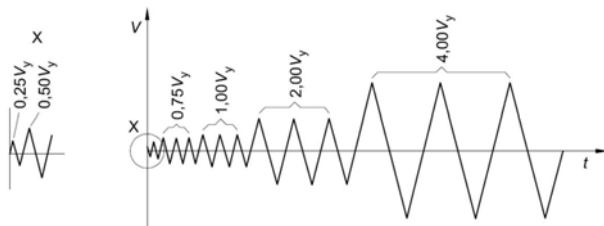
The specimens were installed in the test apparatus inserting laterally the walls until the exact position under the beams for the vertical load was reached. Metal connection systems were then nailed to the panel on the vertical flange and bolted to the upper flange of the base beam on the horizontal flange. Then vertical load was applied moving down the levers beams until the contact with the upper surface of the wall. A total amount of 6 linear variable displacement transducer (LVDTs) were positioned on the wall as shown in Figure 8. All the transducers were connected to a datum remote controller.



**Figure 8:** instruments position

The LVDT L0 measured the uplift of the wall when the load is applied in monotonic tests, while the LVDT L1 measured the same displacement only for the cyclic tests. L2 measured the horizontal relative displacement between the ground and the bottom surface of the panel. L3 measured the horizontal displacement at the upper top plate. L4 and L5 were wire displacement transducer placed to measure the shear deformation in the plane of the panel. LC12 is the load cell of the hydraulic jack and it measured the force applied at the top; another transducer incorporated in the actuator measured the displacement at the same point. LC14 and LC16 were load cells incorporated in the hold-down bolts to measure the vertical reactions under the wall. The load was applied approaching the actuator head until the contact with the head of the panel was reached. In the monotonic tests the load was applied only by contact without fixing the head of the actuator to the specimen.

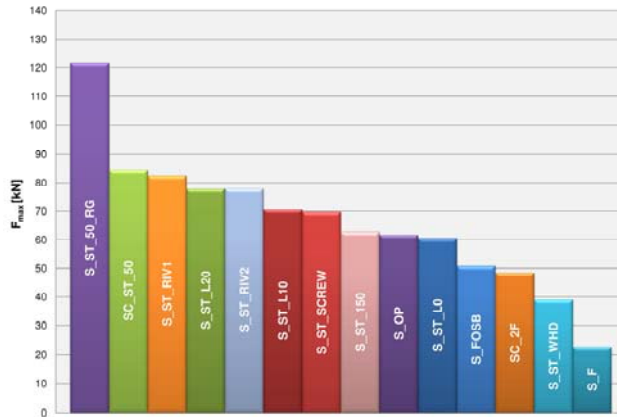
In the monotonic test the load was applied approaching the actuator head until the contact with the head of the panel was reached. The horizontal load was applied by monotonically advancing the actuator until the specimen failure. The rate of loading was selected equal to 0.05 mm/s. The cyclic test procedure was formulated in accordance with UNI EN 12512 [1]. In particular the amplitude of the cycles is expressed as a function of the yield slip ( $V_y$ ). A single conventional value on which to calibrate the amplitude of cycles equal to 10mm was adopted. The sequence of the cycles and the number of repetitions for each level of imposed displacement is reported in Figure 9. The rate of loading was selected equal to 0.05 mm / s for the first five cycles ( $1 \times 0.25 - 1 \times 0.5 - 3 \times 0.75 V_y$ ), then 0.1 for the cycles corresponding to the displacement  $V_y$  and 0.2 for the cycles up to the end of the test. All load cell and LVDTs were connected to a main data acquisition system with a recording frequency equal to 2 Hz and were zeroed just before starting the test.



**Figure 9:** cyclic procedure according to EN12512

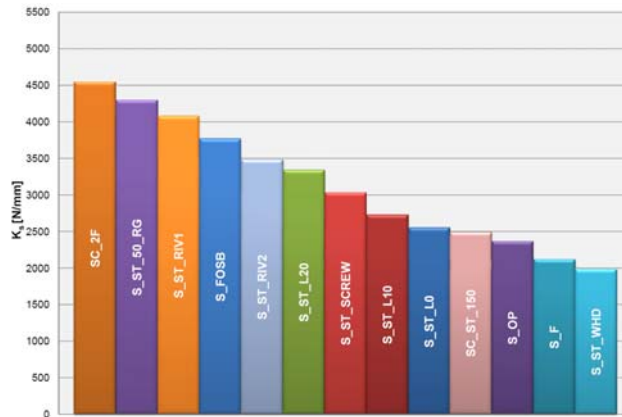
### 3 TEST RESULTS

11 monotonic (S) and 11 cyclic (C) tests were done. Some cyclic tests are then enveloped to obtain monotonic curves (SC). The results in term of maximum load achieved are resumed in the chart of Figure 10. As expected the maximum load was reached by the wall with OSB panel nailed with spacing of 50 mm equipped with the stronger hold-down (RG) and the maximum vertical load applied (20 kN/m). The same configuration of wall nailed with spacing of 150 mm, equipped with weakest hold down and with the same vertical load reach half value of horizontal carrying capacity. The lowest load was obtained from the wall sheathed only on one side by gypsum fiber with 20 kN/m of vertical dead load.



**Figure 10:** Maximum force achieved in each test

The stiffness of each tested wall are reported in the chart of Figure 11.



**Figure 11:** Stiffness of tested walls

The stiffness of the walls varies between 4500 N/mm and 2000 N/mm. The maximum stiffness was achieved by the wall sheathed with gypsum fiber. Similar stiffness is showed by the wall sheathed with OSB nailed with spacing of 50 mm and equipped by stronger hold-down. The lower limit of stiffness is reached by the wall without hold downs.

### 3.1 INFLUENCE OF WALL COMPONENTS

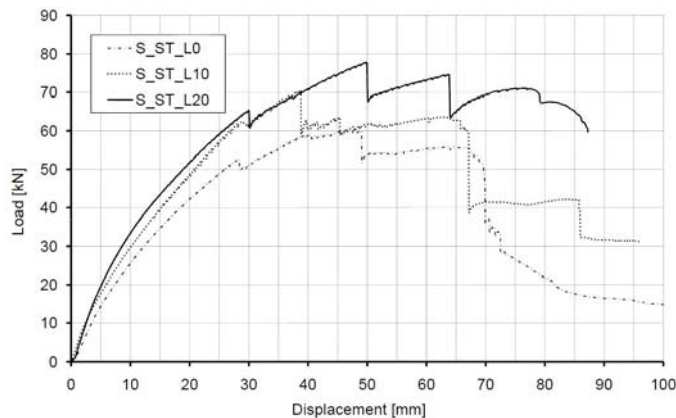
In the next paragraphs the influence of each investigated parameters have been analyzed and discussed. In each paragraph has been examined a single variable without changing the other boundary conditions.

#### 3.1.1 Influence of vertical load

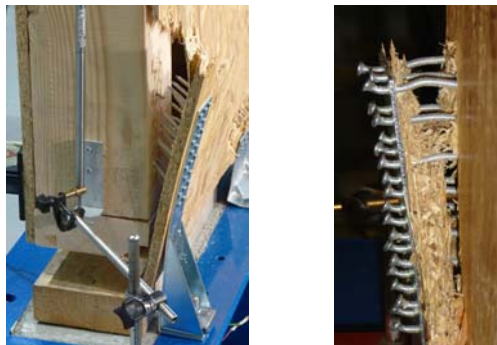
The influence of vertical load is particularly important for the rocking effect. The vertical force in fact reduces the traction force to be absorbed by the hold-down and consequently the rocking movement of the wall.

Analyzing the three levels of load proposed (0-10-20 kN/m) it's clear that to an increase in vertical load corresponding increase also in the maximum horizontal load carrying capacity (L10: 20% - L20: 28% in comparison to L0). Furthermore to high level of vertical load is associated major horizontal stiffness (L10: 11.6% - L20: 31% in comparison to L0).

The effects abovementioned are clearly visible looking at the experimental curves reported in the Figure 12 where the failure was due to the braking of hold-down (Figure 13).



**Figure 12:** Impose displacement vs horizontal load carrying capacity of the wall (different magnitude of vertical load)



**Figure 13:** Breaking of the hold down

The stiffness of the wall loaded with 10 kN/m is 12 % higher that the wall without vertical load and the wall with 20 kN/m is 31 % higher that the unloaded wall.

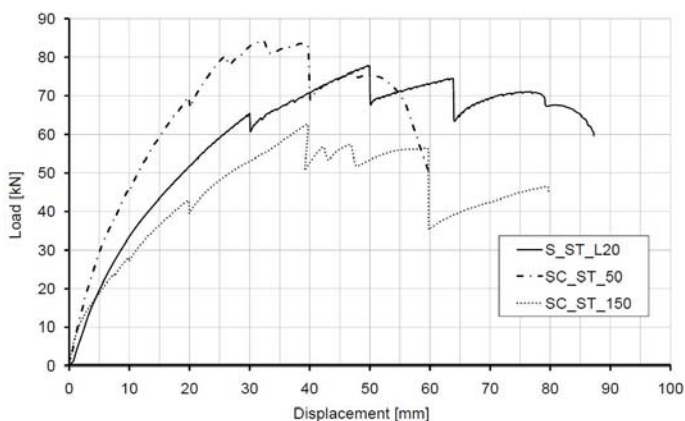
**Table 5:** data obtained from the test with different magnitude of vertical load

Test		S_ST_L0	S_ST_L10	S_ST_L20	
Dati Test	$F_{Max}$	[kN]	60,4	72,5	77,8
	$v_{Fmax}$	[mm]	47,2	40,4	49,7
	$F_U$	[kN]	48,4	58,0	62,2
	$v_U$	[mm]	69,7	68,8	86,6
	$F_y$	[kN]	50,7	64,2	60,2
	$v_y$	[mm]	19,7	22,8	17,8
	$K_s$	[N/mm]	2550,2	2846,1	3339,1

### 3.1.2 Influence of nail spacing

According to the formulas introduced in the technical literature to a lesser spacing of the connectors corresponds a greater racking load carrying capacity of the wall. This is true only if the failure occurs due to the breaking of the connection between sheeting panel and studs. The spacing of the connectors in the tested walls was chosen to cover the entire range of possible variation. The spacing of 50 mm is a lower limit constructively acceptable, while the 150 mm is the maximum spacing allowed by current regulations. The tests reported in Figure 13 have been two different type of failure. Specimen S\_ST\_L20 and specimen SC\_ST\_150 failed because of the breaking of hold down. In the specimen SC\_ST\_150 failed the connection between studs and sheeting panel. For this reason only comparison between stiffness can be done. It's also important to underline that hold downs, angle brackets and magnitude of vertical load are equal for each analyzed walls.

The stiffness obtained from wall with 150 mm nails spacing is 2487 N/mm. For the wall with 100 mm nail spacing is 34.2% higher and for the wall with 50 mm nail spacing 102%.



**Figure 14:** Impose displacement vs horizontal load carrying capacity of the wall (different spacing of nails)

If we compare the test S\_ST\_50\_RG (balanced failure between hold-down and panel-stud connection) with the test SC\_ST\_150 (failure at panel-stud connection) the load carrying capacity is 94% higher.

**Table 6:** data obtained from the test with different nails spacing



Test		S_ST_L20	SC_ST_50	SC_ST_150	
Dati Test	$F_{Max}$	[kN]	77,8	84,4	62,7
	$V_{Fmax}$	[mm]	49,7	32,1	39,7
	$F_u$	[kN]	62,2	67,5	50,1
	$V_u$	[mm]	86,6	56,0	59,8
	$F_y$	[kN]	60,2	69,7	54,6
	$V_y$	[mm]	17,8	13,3	20,2
	$K_s$	[N/mm]	3339,1	5033,8	2487,3

### 3.1.3 Influence of sheathing panel

The mechanical behavior of the wall is closely related to the characteristics of the panels nailed to the frame. Three different configurations were tested in which were used two type of panel with the same nail spacing. The first wall was sheathed with OSB on both sides (S\_ST\_L20), the second one with Gypsum Fiber panel on both sides (SC\_2F) and the third with OSB on one side and Gypsum Fiber on the other side (S\_FOSB).

The walls tested in all the three configurations have shown similar stiffness. The maximum load carrying capacity for the wall sheathed with GFB or with the mixed configuration is 35% lower than the wall sheathed with OSB. The wall with GFB failed due to the out of plane brittle rupture of the panel (Figure 15). This is the reason why the entire behavior of the wall is also brittle. The walls with mixed configuration is less brittle because the OSB panel take the out of plane force after the brittle failure of the GFB panel.



Figure 15: Out of plane failure of the GFB panel

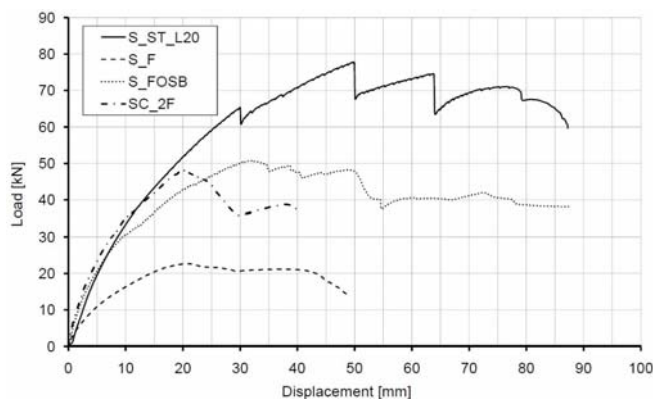


Figure 16: Impose displacement vs horizontal load carrying capacity of the wall (different sheathing panel)

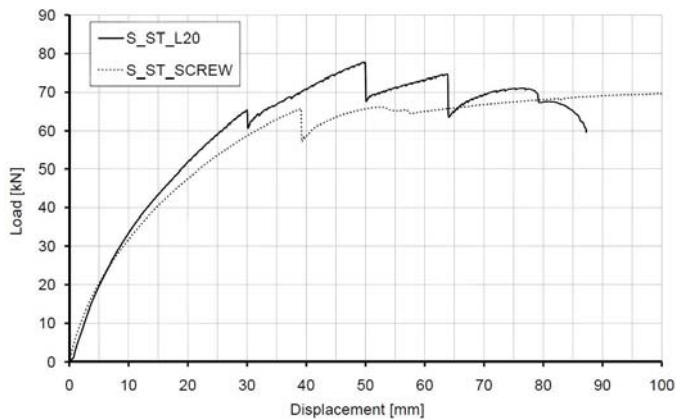
The maximum horizontal force measured for the test S\_F sheathed by GFB on one side was 53 % lower than test S\_2F, as expected. This means that the resistant of a wall with asymmetrical configuration is about half part of the value obtained from the case with symmetrical configuration.

**Table 7:** data obtained from the test with different sheeting panel

Test		S_ST_L20	S_F	SC_2F	S_FOSB	
Dati Test	F <sub>Max</sub>	[kN]	77,8	22,7	48,3	50,7
	V <sub>Fmax</sub>	[mm]	49,7	20,5	19,8	31,7
	F <sub>u</sub>	[kN]	62,2	18,1	38,7	40,6
	V <sub>u</sub>	[mm]	86,6	44,8	27,6	52,8
	F <sub>y</sub>	[kN]	60,2	19,1	39,6	37,4
	V <sub>y</sub>	[mm]	17,8	8,5	8,2	9,3
	K <sub>s</sub>	[N/mm]	3339,1	2113,0	4544,2	3766,3

### 3.1.4 Influence of different horizontal anchorages

Tests on two different types of shear anchorage were performed. Connection with special angle brackets and connection with wooden screws (derived from a previous experimental campaign [2,3]). Both solutions were designed to achieve the same resistant against horizontal force. The tests confirm the efficiency of these systems that have shown similar behavior. The only difference was that the angular absorb a part of vertical traction force that slightly increases the stiffness of the wall.



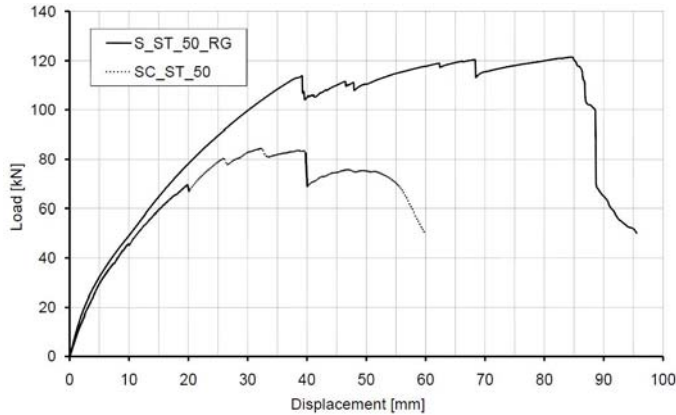
**Figure 17:** Impose displacement vs horizontal load carrying capacity of the wall (different horizontal anchorages)

**Table 8:** data obtained from the test with different horizontal anchorages

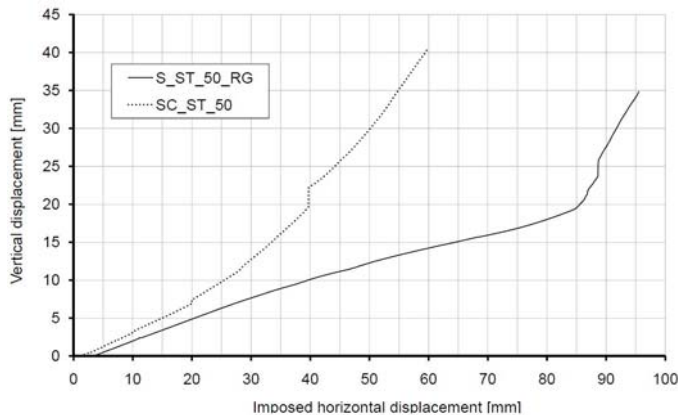
Test		S_ST_L20	S_ST_SCREW	
Dati Test	F <sub>Max</sub>	[kN]	77,8	69,7
	V <sub>Fmax</sub>	[mm]	49,7	99,7
	F <sub>u</sub>	[kN]	62,2	69,7
	V <sub>u</sub>	[mm]	86,6	99,7
	F <sub>y</sub>	[kN]	60,2	54,8
	V <sub>y</sub>	[mm]	17,8	17,0
	K <sub>s</sub>	[N/mm]	3339,1	3031,3

### 3.1.5 Influence of hold down

The hold down influence the horizontal load carrying capacity of the wall if the failure on the wall is due to the failure of the hold down. As shown in Figure 18 the maximum load achieved by the wall equipped with stronger hold down (S\_ST\_50\_RG) is 121,5 kN. The maximum load for the wall with weaker hold down (SC\_ST\_50) is 30 % lower. It is important to underline that the magnitude of vertical applied load is the same.



**Figure 18:** Impose displacement vs horizontal load carrying capacity of the wall (different hold downs)



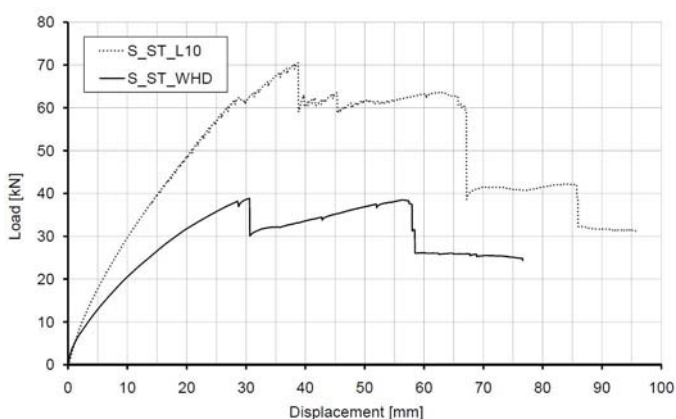
**Figure 19:** Impose displacement vs uplift of the corner wall

As long as the hold down remains inactive (i.e. the uplift of the wall is zero) the stiffness of wall is not effected by the presence of the hold down. When the wall starts to uplift the stiffness of the hold down became important (this point is located at 25 kN in the y-axis of Figure 18).

**Table 9:** data obtained from the test with different hold downs

Test		SC_ST_50	S_ST_50_RG	
Dati Test	$F_{Max}$	[kN]	84,4	121,5
	$v_{Fmax}$	[mm]	32,1	84,4
	$F_u$	[kN]	67,5	97,2
	$v_u$	[mm]	55,8	88,6
	$F_y$	[kN]	70,3	101,9
	$v_y$	[mm]	13,4	22,2
	$K_s$	[N/mm]	5033,8	4294,0

In addition wall without hold-down (S\_ST\_WHD) were tested with a vertical dead load of 10 kN/m. This vertical load was chosen to simulate a wall inserted in a single storey building where the use of hold down is often avoided by the designer. The reduction of resistance is 46.3%, while the stiffness decreases about 30.4%.



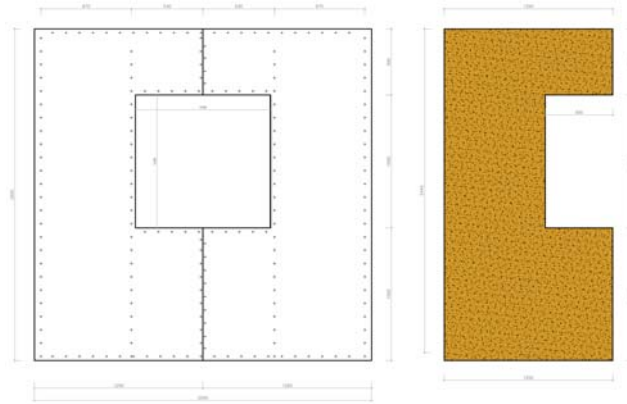
**Figure 20:** Impose displacement vs horizontal load carrying capacity of the wall (presence of hold down)

**Table 10:** data obtained from the test (presence of hold down)

Test		S_ST_WHD	S_ST_L10	
Dati Test	$F_{Max}$	[kN]	38,9	72,5
	$v_{Fmax}$	[mm]	30,5	40,4
	$F_u$	[kN]	31,1	58,0
	$v_u$	[mm]	30,6	68,8
	$F_y$	[kN]	34,3	64,2
	$v_y$	[mm]	16,1	22,8
	$K_s$	[N/mm]	1980,3	2846,1

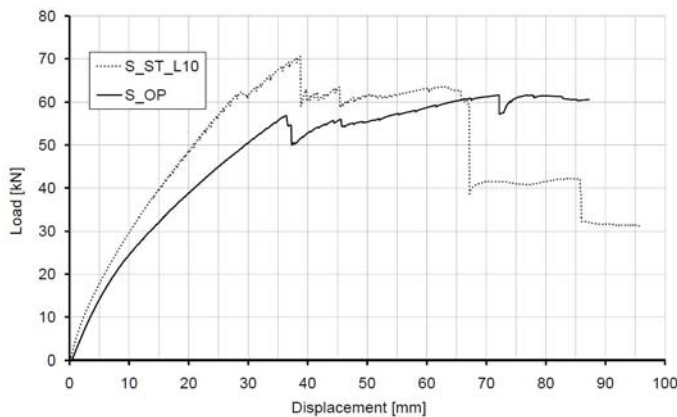
### 3.2 INFLUENCE OF OPENINGS

The openings in the walls, like windows or door, involve inevitably a decrease of strength and stiffness in comparison with wall without openings. In the test presented a particular geometry was chosen (Figure 20). From the rectangular OSB panels the shape of the opening (1 x 1 m) has been cut, without interrupt the continuity of the panel itself above the window. The vertical wooden studs are placed to guarantee the continuity of the shear flow around the panels.



**Figure 21:** Panels disposition in the wall with opening

For the geometry tested (S\_OP) the decrease of strength was 15% and the decrease of stiffness was 17%.



**Figure 22:** Impose displacement vs horizontal load carrying capacity of the wall (presence of openings)

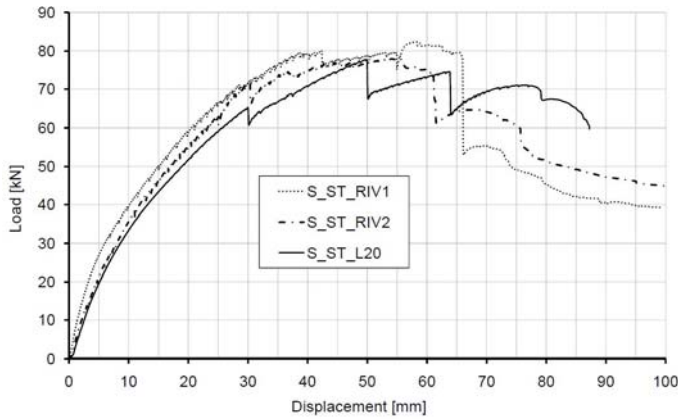
The results shows that if the sheathing panels are not interrupted above the windows and the windows is not too big, the behaviour of the wall is not so influenced by the presence of the opening. Completely different behaviour should be assume if the panels is completely cut above the windows [4]. The type of sheeting panel is important, in the case of out of plane force

**Table 11:** data obtained from the test (presence of openings)

Test		S_OP	S_ST_L10	
Dati Test	F <sub>Max</sub>	[kN]	61,6	72,5
	v <sub>Fmax</sub>	[mm]	71,9	40,4
	F <sub>u</sub>	[kN]	60,6	58,0
	v <sub>u</sub>	[mm]	87,3	68,8
	F <sub>y</sub>	[kN]	51,4	64,2
	v <sub>y</sub>	[mm]	21,4	22,8
	K <sub>s</sub>	[N/mm]	2361,7	2846,1

### 3.3 INFLUENCE OF NON-STRUCTURAL FINISHING AND CORRELATION BETWEEN INTERSTOREY DRIFT AND DAMAGE

The influence of non-structural finishing (e.g. plaster board, insulation, counter wall) on the mechanical behavior of the timber wall has been investigated. The tested stratigraphy are the one presented in Figure 4 (S\_ST\_RIV1 – S\_ST\_RIV2). The wall S\_ST\_RIV2 shown the same behavior as the wall without finishing. Instead the wall S\_ST\_RIV1 shown 6% higher maximum horizontal load carrying capacity and 22% higher stiffness that the wall without finishing.



**Figure 23:** Impose displacement vs horizontal load carrying capacity of the wall (non-structural finishing)

It was observed that the GFB panels stapled on the OSB structural panel produce an slightly increment in strength and stiffness of the wall S\_ST\_RIV1. In fact the staples were 55 mm long and the insertion in the wooden stud was 27.5 mm .

**Table 12:** data obtained from the test (non-structural finishing)

Test		S_ST_L20	S_ST_RIV1	S_ST_RIV2	
Dati Test	$F_{Max}$	[kN]	77,8	82,4	78,0
	$v_{Fmax}$	[mm]	49,7	58,2	54,0
	$F_u$	[kN]	62,2	65,9	62,4
	$v_u$	[mm]	86,6	66,0	61,5
	$F_u$	[kN]	60,2	62,5	63,7
	$v_u$	[mm]	17,8	14,4	17,7
$K_s$	[N/mm]	3339,1	4079,2	3480,4	

No damages of the finishing were observed in the wall S\_ST\_RIV2 until the imposed horizontal displacement of 60 mm. In fact, the counter wall and the external insulation were very flexible and the movement of the structural part didn't affect the finishing.

Some damages were observed in the wall S\_ST\_RIV1 on the side where GFB panels were fixed. In Table 13 the sequence of damage related to horizontal drift are reported.

**Table 13:** correlation between interstorey drift and damage for wall S\_ST\_RIV1

$\Delta_h$ [mm]	Drift [%]	Damage
31.5	1.26	Crack in the GFB panel close to hold down
39	1.56	Crack in the GFB panel close to angle brackets
62	2.48	Cracks in the left bottom corner of the GFB panel



**Figure 24:** Some damages observed in the test S\_ST\_RIV1

The low level of damages observed was also related to the boundary condition of the finishing. Both to the bottom and top of the wall they were free to move and rotate.

## 4 CONCLUSIONS

Data obtained from the experimental tests show the influence of different parameters in the mechanical behaviour of wall panels. The influence of vertical load is fundamental for the rocking effect, taking advantages both in term of resistance and stiffness of the walls. The influence of nail spacing is great, linearly correlated to the resistance of the walls. This statement is true only if the hold downs are stronger than the connection between stud and panel. The angle bracket act as contrast the horizontal load transmitted to the foundation, whereas seems not to affect the rocking behaviour of the wall. The type of sheathing panel are important: all of them have to be suitable for structural use and if they are able to take load out of plane, the wall behaves better. The roles of the opening change according to the way that they are cut into the wall. The non-structural finishing tested in the experimental campaign has a lower influence in the behaviour of the wall.

All the collect data will be use in the future work to validate numerical and analytical model for the prediction of the behaviour of timber frame building.

## ACKNOWLEDGEMENT

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## REFERENCES

- [1] UNI EN 12512:2003 Timber structures - Test methods - Cyclic testing of joints made with mechanical fasteners. 2003.
- [2] Andrea Conte; Maurizio Piazza ; Tiziano Sartori; Roberto Tomasi. Experimental investigation on connections between wood framed shear walls and foundations. SEWC,Como, 2011.
- [3] Andrea Conte; Maurizio Piazza ; Tiziano Sartori ; Roberto Tomasi. Influence of sheathing to framing connections on mechanical properties of wood framed shear walls. ANIDIS, Bari, 2011.
- [4] UNI EN 1995-1-1:2009 Eurocode 5 - Design of timber structures - Part 1-1: General - Common rules and rules for buildings. 2009.





# Appendice D

## Paper IV

Analytical and numerical analysis of timber framed  
shear walls

*Daniele Casagrande, Simone Rossi, Tiziano Sartori, Roberto Tomasi*

WCTE 2012, Auckland



## **ANALYTICAL AND NUMERICAL ANALYSIS OF TIMBER FRAMED SHEAR WALLS**

**Daniele Casagrande<sup>1</sup>, Simone Rossi<sup>2</sup>, Tiziano Sartori<sup>3</sup>, Roberto Tomasi<sup>4</sup>**

**ABSTRACT:** One of the prominently important performances required for shear walls of wood-framed construction system should be to resist against the lateral forces generated by seismic events. This paper deals with numerical and analytical elastic models for sheathed shear walls. The models are based on the assumption of elastic load-slip relations for both sheathing-to-framing joints and hold-downs or angle brackets, considering only static loads. Both elastic model and analytical formula have been previously experimentally verified through full scale tests. A model suitable to predict the behavior of the wall with different geometry was developed by means of parametric studies. The influence of vertical loads are included in the models. In addition, the contributions of different components are estimated and a simplified formula is presented. The analytical model was amplified in order to deduce the stiffness matrix of a timber walls structure and to understand the real distribution of horizontal loads between the walls: nowadays the wall stiffness in fact is assumed linear proportional to wall length. A simplified wall numerical model was used in order to run elastic modal analysis of buildings.

**KEYWORDS:** Wood framed shear walls, seismic performance, wall displacement, elastic model

### **1 INTRODUCTION**

The analysis of a timber framed shear wall subjected to a horizontal load is fundamental in order to understand the behaviour of a building under seismic events. Especially study the interaction among walls becomes necessary to evaluate the seismic forces distribution in an elastic design way of buildings. Nowadays in fact most designs approaches assume that the horizontal force distribution depends linearly on the walls length: sheet shear and sheet nail-slip are considered the uppermost components of wall deformation. The present work is focused on the analytical and numerical linear elastic analysis of a timber framed shear wall under a horizontal load,

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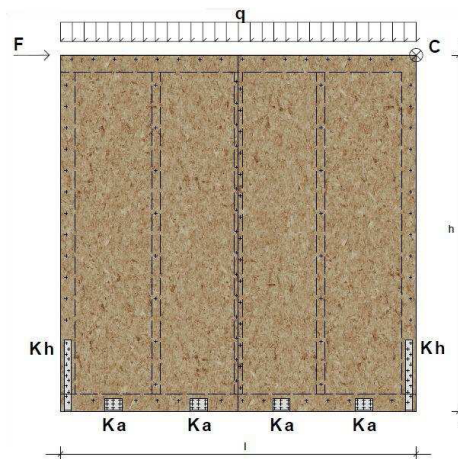
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characterized by different configurations. Firstly, an analytical formula to evaluate the horizontal displacement is presented, taking into account of all deformation contributions and different configurations (i.e. geometry, type of connections, vertical load). Then a parametric study of wall stiffness is reported: it is demonstrated that a priori linearly proportional to the wall length can not be assumed. Secondly, a complete linear model of wall is described in order to validate the proposed equation. A simplified numerical model, calibrated on analytical formula, is also reported. Lastly all results are compared to experimental data obtained by full scale timber frame wall tests performed in the mechanical and structure laboratory of the University of Trento [1].

## 2 ELASTIC HORIZONTAL DISPLACEMENT THEORETICAL STUDY

The elastic horizontal displacement (Point C, Figure 1) of a timber framed shear wall can be evaluated by adding the different contributions of deformation: sheathing boards, connection system between sheathing boards and timber frame, connection system for lateral forces and hold-down system. The study is conducted considering the presence of an uniformly distributed vertical load  $q$  and a concentrated horizontal force  $F$ ; the wall dimensions are  $l \times h$ .

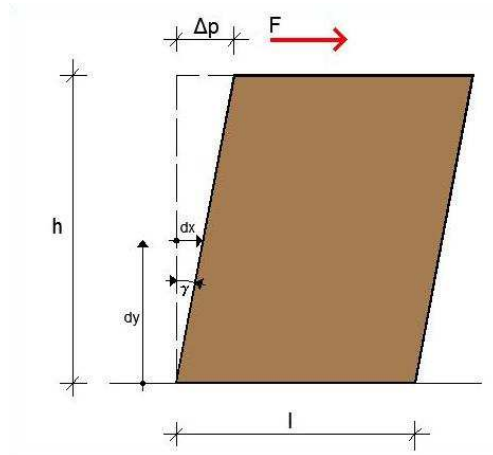


**Figure 1:** Load configuration and geometry wall

### 2.1 SHEETING BOARD DEFORMATION

The shear deformation component  $\rho$  of sheathing boards is independent on vertical load and can be evaluated in the following way. Referring to Figure 2 the shear deformation is evaluated as:

$$\rho = \frac{F}{K_s} \quad (1)$$



**Figure 2:** sheeting board deformation

The displacement  $\Delta_p$  can be evaluated considering  $X=1$ :

$$\Delta_p = \gamma \cdot h = \frac{F \cdot h}{G_p \cdot n_p \cdot t_p \cdot l} \quad (2)$$

where  $n_p$  is the number of sheeting boards (1 or 2) and  $t_p$  is the sheeting board thickness.

## 2.2 SHEET NAIL-SLIP

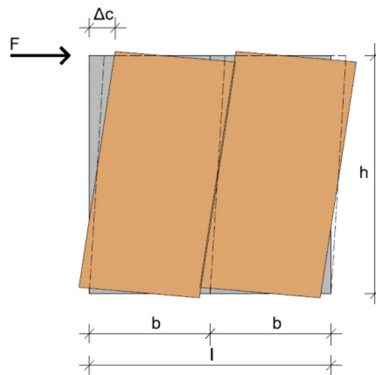
The sheeting boards are connected to the timber frame by means of a widespread nailing (or stapling). The distance between fasteners along the perimeter studs  $s_p$ , the top and bottom rails  $s_{ml}$  is assumed constant and equal to  $s$ . Along the intermediate stud the distance is assumed  $s/2$ .

The horizontal displacement caused by the sheet nail-slip  $\Delta_c$  for a single board wall of length  $b$  is given by the following equation [2,3]:

$$\Delta_c = \frac{F \cdot h^2}{k} \cdot \left[ \frac{1}{\sum_{i=1}^n x_i^2} + \frac{1}{\sum_{i=1}^n y_i^2} \right] \quad (3)$$

where  $n$  is the number of fasteners,  $k$  is the stiffness of each fastener, and  $x_i$  and  $y_i$  are the coordinates of each fastener from the centre of the panel.

The deformation due to the nailing is the following:



**Figure 3:** sheet nail-slip deformation

Defining the parameter  $\alpha$  as ratio between the height  $h$  and the length  $b$  of the board, we can write:

$$\sum_{i=1}^n x_i^2 \cong \frac{1}{6} \cdot \left(1 + 3 \cdot \frac{h}{b}\right) \cdot \frac{b}{s} \cdot b^2 = \frac{1}{6} \cdot (1 + 3 \cdot \alpha) \cdot \frac{b^3}{s} \quad (4)$$

$$\begin{aligned} \sum_{i=1}^n y_i^2 &\cong \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \frac{h}{b}\right) \cdot \frac{b}{s} \cdot h^2 = \\ &= \frac{1}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha\right) \cdot \frac{\alpha^2 \cdot b^3}{s} \end{aligned} \quad (5)$$

Defining:

$$\frac{1}{6} \cdot (1 + 3 \cdot \alpha) = \eta \quad (6)$$

$$\frac{\alpha^2}{12} \cdot \left(6 + \frac{5}{2} \cdot \alpha\right) = \xi \quad (7)$$

and

$$\sum_{i=1}^n x_i^2 \cong \eta \cdot \frac{b^3}{s} \quad (8)$$

$$\sum_{i=1}^n y_i^2 \cong \xi \cdot \frac{b^3}{s} \quad (9)$$

the displacement  $\Delta_c$  is given by

$$\Delta_c = \frac{F \cdot b^2}{k} \cdot \alpha^2 \cdot \left[\frac{1}{\eta} + \frac{1}{\xi}\right] \cdot \frac{s}{b^3} \quad (10)$$

Or, defining

$$\lambda = \alpha^2 \cdot \left[\frac{1}{\eta} + \frac{1}{\xi}\right] \quad (11)$$

by:

$$\Delta_c = \frac{F \cdot b^2}{k} \cdot \lambda(\alpha) \cdot \frac{s}{b^3} \quad (12)$$

Extending the result to a  $l$  long wall (sheeted with more  $b$  long panels) then we have:

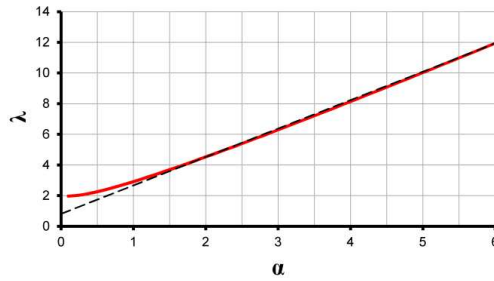
$$\Delta_c = \frac{F}{k} \cdot \lambda(\alpha) \cdot \frac{s}{l} \quad (13)$$

The parameter  $\alpha$  assumes the role of a board shape coefficient. For  $\alpha > 1$  the Equation 11 can be simplified and rewritten as:

$$\lambda = 0,81 + 1,855 \cdot \alpha \quad (14)$$

For the typical value  $\alpha=2$ ,  $\lambda$  is 4.52, as proposed by Källsner et al.

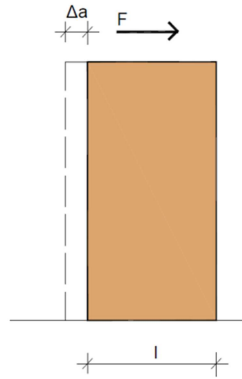
The  $\lambda$  vs  $\alpha$  curve is reported:



**Figure 4:**  $\lambda$  vs  $\alpha$  curve

### 2.3 RIGID BODY TRANSLATION

The wall is connected to the foundation by means a connection system (angle brackets, screws) in order to prevent a rigid body translation.



**Figure 5:** rigid body translation

As in the previous cases the vertical load is not influent and, neglecting the friction phenomena, the rigid body translation component  $\Delta_a$  is given by:

$$\Delta_a = \frac{F}{k_a \cdot n_a} \quad (15)$$

With  $k_a$  stiffness of the each connection and  $n_a$  number of connections.

Defining  $i_a$  as the distance between angle brackets (which can be expressed as the ratio between wall length and the number of connections) we obtain:

$$\Delta_a = \frac{F \cdot i_a}{k_a \cdot l} \quad (16)$$

### 2.4 RIGID BODY ROCKING

The hold-down system is used to prevent the rotation of the wall caused by the overturning moment of the horizontal force ( $F \cdot h$ ). The hold-down, placed on the in-tension angle, is loaded by a force equal to  $F \cdot h/l$ .

The hold-down elongation can be evaluated by:



$$v = \frac{F \cdot h}{l} \cdot \frac{1}{k_h} \quad (17)$$

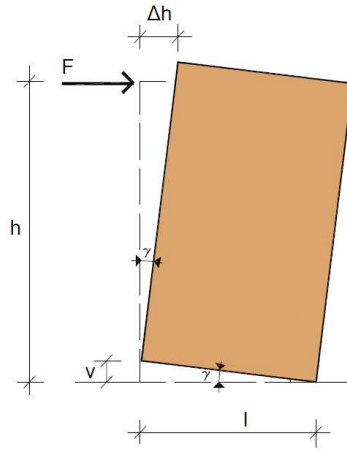
Where  $k_h$  is the stiffness of the hold-downs

Considering the rigid rotation of the wall, as reported in Figure 6, the rotation angle can be determined using:

$$\gamma = \frac{v}{l} = \frac{F \cdot h}{l^2} \cdot \frac{1}{k_h} \quad (18)$$

therefore the rocking component  $\Delta_h$  is:

$$\Delta_h = \gamma \cdot h = \frac{F \cdot h^2}{l^2} \cdot \frac{1}{k_h} \quad (19)$$



**Figure 6:** rigid rotation caused by hold-down elongation

Differently from the previous cases, the hold-down deformation component is influenced by the vertical load, which produces a stabilizing moment:

$$M = q \cdot \frac{l^2}{2} \quad (20)$$

Two different situations are possible:

- The wall rotates because the overturning moment is greater than the stabilizing moment;
- The wall does not rotate because the stabilizing moment is greater than the overturning moment;

Follows that the rocking component  $\Delta_h$  can be evaluated as :

$$\Delta_h = \begin{cases} \frac{h}{l \cdot k_h} \left( \frac{F \cdot h}{l} - \frac{q \cdot l}{2} \right); & \text{for } F \cdot h > \frac{q \cdot l^2}{2} \\ 0; & \text{for } F \cdot h \leq \frac{q \cdot l^2}{2} \end{cases} \quad (21)$$

The first equation refers to the active hold-down and the second one refers to not-active hold-down.

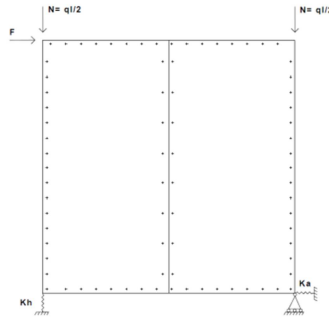
## 2.5 TOTAL HORIZONTAL DISPLACEMENT

To sum up, considering all the deformation contributions and rewriting the vertical distributed load as a concentrated force like you can see on the Figure 7, when the hold-down is active, the displacement of a timber framed shear wall can be evaluated using:

$$\Delta = \frac{F \cdot h}{l} \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{l \cdot n_p \cdot k_c} + \frac{F \cdot i_a}{k_a \cdot l_a} + \frac{F \cdot h^2}{l^2 \cdot k_h} - \frac{N \cdot h}{l \cdot k_h} \quad (22)$$

In the same way, when the hold-down is not active, the displacement can be evaluated using:

$$\Delta = \frac{F \cdot h}{l} \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{l \cdot n_p \cdot k_c} + \frac{F \cdot i_a}{k_a \cdot l_a} \quad (23)$$



**Figure 7:** wall with concentrated load scheme

The above equations can be rewritten respectively as follows:

$$\Delta = \frac{F}{K_P} + \frac{F}{K_C} + \frac{F}{K_A} + \frac{F}{K_H} - \frac{N \cdot h}{l \cdot k_h} \quad (24)$$

$$\Delta = \frac{F}{K_P} + \frac{F}{K_C} + \frac{F}{K_A} \quad (25)$$

Where  $K_P$  is the panel stiffness, where  $K_C$  is widespread nailing stiffness, where  $K_A$  is the shear connectors stiffness, where  $K_H$  is the hold-down stiffness. In addition,  $K_{TOT}$  is the wall stiffness and it can be estimated with:

$$\frac{1}{K_{TOT}} = \frac{1}{K_P} + \frac{1}{K_C} + \frac{1}{K_A} + \frac{1}{K_H} \quad (26)$$

if the hold-down is not active, the wall stiffness is:

$$\frac{1}{K_{TOT}} = \frac{1}{K_P} + \frac{1}{K_C} + \frac{1}{K_A} \quad (27)$$

Considering the wall stiffness, when the hold-down is active, we can write:

$$\Delta = \frac{F}{K_{TOT}} - \frac{N \cdot h}{l \cdot k_h} \quad (28)$$

defining:

$$\Delta_N = \frac{N \cdot h}{l \cdot k_h} \quad (29)$$

so we can write:

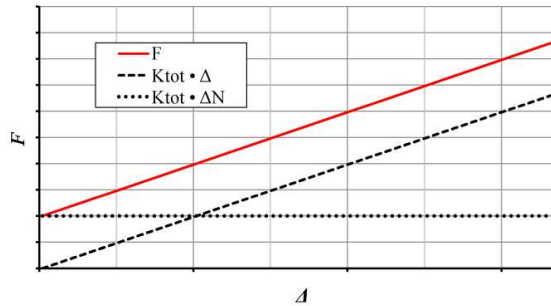
$$\Delta = \frac{F}{K_{TOT}} - \Delta_N \quad (30)$$

Therefore we obtain:

$$\begin{cases} \Delta = \frac{F}{K_{tot}} - \Delta_N \\ F = K_{tot} \cdot \Delta + K_{tot} \cdot \Delta_N = K_{tot} \cdot (\Delta + \Delta_N) \end{cases} \quad (31)$$

- These equations allow us to make two important consideration: the displacement  $\Delta$  is limited from the vertical load, indeed the  $F/K_{tot}$  part is decreased by vertical load part  $\Delta_N$ .
- the force  $F$  required to move the wall of  $\Delta$  is the sum of the elastic force  $K_{tot} \cdot \Delta$  and of the vertical load part  $K_{tot} \cdot \Delta_N$ .

This behavior is reported in Figure 8:



**Figure 8:** Force  $F$  vs displacement  $\Delta$  linear relationship

In the same way, when the hold-down is not active, the wall displacement is:

$$\Delta = \frac{F}{K_{TOT}} \quad (32)$$

and the vertical load is irrelevant.

### 3 $K_{tot}$ PARAMETRIC STUDY

From (31) it follows that the force  $F$  required depends on the wall stiffness  $K_{tot}$ . Most design approaches assume  $K_{tot}$  linearly proportional to the wall length, without considering its dependence on wall geometry, connection system or vertical loads. At this purpose it is presented a parametric study of  $K_{tot}$ , evaluating how it is influenced by the wall length  $l$ .

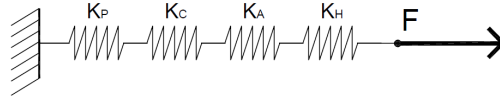
If the Hold-down is active  $K_{tot}$  can be obtained by

$$\frac{1}{K_{tot}} = \frac{1}{K_P} + \frac{1}{K_C} + \frac{1}{K_A} + \frac{1}{K_H} \quad (33)$$

Explicit

$$\frac{1}{K_{tot}} = \frac{h}{l} \cdot \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot s_c}{l \cdot n_p \cdot k_c} + \frac{i_a}{l \cdot k_a} + \frac{h^2}{l^2 \cdot k_h} \quad (34)$$

Therefore the wall can be schematized by 4 springs in series as shown in the Figure 9.



**Figure 9:** wall rheological model

In order to underline the dependence on the wall length the following quantities are defined:

$$\frac{1}{\vartheta} = \frac{k}{G_p \cdot n_p \cdot t_p} = \frac{l}{K_P} \quad (35)$$

$$\frac{1}{\beta} = \frac{\lambda \cdot s_c}{n_p \cdot k_c} = \frac{l}{K_C} \quad (36)$$

$$\frac{1}{\varphi} = \frac{i_a}{k_a} = \frac{l}{K_A} \quad (37)$$

$$\frac{1}{\delta} = \frac{h^2}{k_h} = \frac{l^2}{K_H} \quad (38)$$

The Equation 34 can be rewritten as

$$\frac{1}{K_{tot}} = \frac{1}{\vartheta \cdot l} + \frac{1}{\beta \cdot l} + \frac{1}{\varphi \cdot l} + \frac{1}{\delta \cdot l^2} \quad (39)$$

Therefore the wall stiffness is given by

$$K_{tot} = \frac{\vartheta \cdot \beta \cdot \varphi \cdot \delta \cdot l^2}{(\vartheta \cdot \varphi \cdot \delta + \beta \cdot \varphi \cdot \delta + \vartheta \cdot \beta \cdot \delta) \cdot l + \vartheta \cdot \beta \cdot \varphi} \quad (40)$$

Equation 40 shows as the stiffness wall is not linearly proportional to the wall length  $l$ . This is true only when the hold-down are not active. In fact when  $\delta$  goes to infinity we obtain:

$$\lim_{\delta \rightarrow \infty} K_{tot} = \frac{\vartheta \cdot \beta \cdot \varphi \cdot l}{\vartheta \cdot \beta + \vartheta \cdot \varphi + \varphi \cdot \beta} = \omega \cdot l \quad (41)$$

According to the previous equation, the non linear proportionality is given by the presence of the active hold-down; the stiffness  $K_H$  is in fact proportional to the squared wall length  $l^2$ . In order to understand the influence of the contribution of hold-down a parametric study of wall stiffness is here presented.

Defining  $K_{tot,b}$  as the stiffness of a wall long as the board length  $b$ , the relationship  $K_{tot,l}/K_{tot,b}$  vs  $l/b$  are shown in Figures 10-13. The dashed line stands for the linear proportional wall stiffness dependence on wall length.

In order to underline the influence of Hold-down contribution two non-dimensional parameters were are defined:

$$\bar{h} = \frac{h}{b} \quad (42)$$

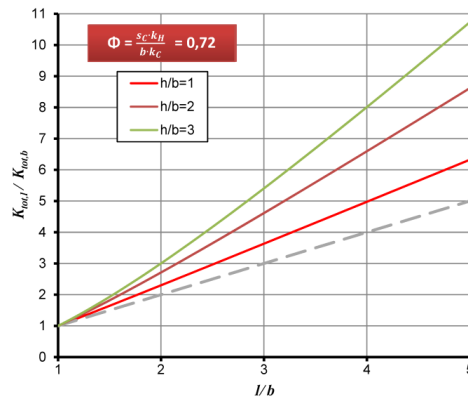
$$\Phi = \frac{s_c \cdot k_h}{b \cdot k_c} \quad (43)$$

The study has been developed for  $\bar{h}=1, 2, 3$  and for values of  $\Phi$  from 1.44 to 2.87.

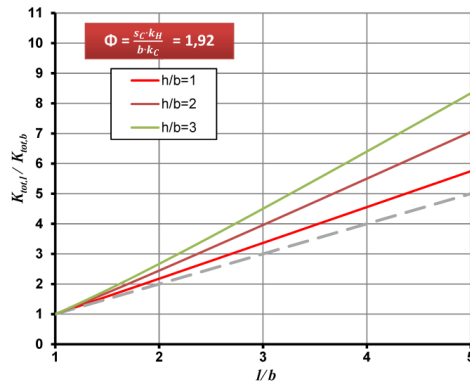
The first parameter  $h$  shows the influence of hold down contribution by the height of the wall; the second one is referred to the stiffness of the hold down connection in relation to the sheet nail-slip stiffness. The influence of the hold down component becomes large in fact when the wall is height and the hold-down connection is weak.

The other contributions  $K_p, K_p, K_c$ , was evaluated considering the following properties:

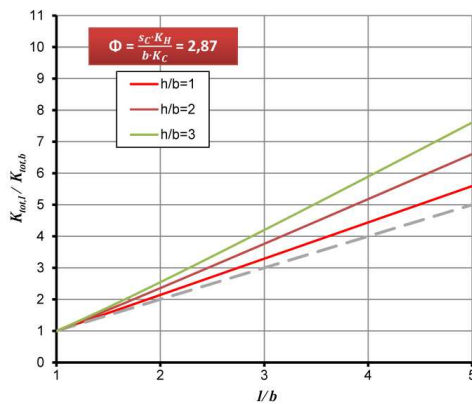
- 2 OSB/3 15 mm sheathing boards
- 2,8 x 60 mm ring nails
- angle brackets ( $k_a = 3342\text{N/mm}$ ) [4]
- Angle brackets distances  $i$  equal to 625 mm



**Figure 10:** behavior dimensionless for  $\Phi=0,72$



**Figure 11:** behavior dimensionless for  $\Phi=1,92$



**Figure 12:** behavior dimensionless for  $\Phi=2,87$

According to figures 10-12, when the wall height increases and the parameter  $\Phi$  decreases the continuous lines comes off the dashed line.

As example, assuming  $h=2$  and  $\Phi= 1,44$  and for the above assigned valued for  $K_p$ ,  $K_p$ ,  $K_c$ , a wall long 4 times  $b$  is 6 times stiffer than a wall with a length equal to  $b$ , and not 4 times.

Therefore the wall stiffness cannot be considered a priori linear proportional to the wall length.

## 4 WALL NUMERICA ANALYSES

In an effort to validate the results of analytical and theoretical studies a numerical model was developed using program SAP 2000.

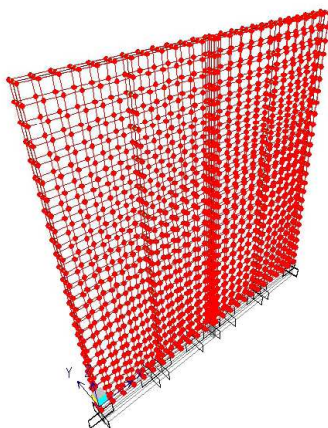
The analysis was performed by means of a wall complete model, where all sheet nail-slips are modelled by means of a linear link.

After checking the agreement between the numerical and theoretical results a simplified model is proposed.

#### 4.1 COMPLETE NUMERICAL MODEL

The elastic behaviour of a framed timber wall loaded by a horizontal load was investigated using a numerical model using software SAP 2000. This model is called “complete” unlike the “simplified” one, developed in order to perform numerical analyses of buildings.

Pinned frame elements have been utilized to model the timber frame, which should therefore be regarded as a mechanism: the sheathing boards, modeled through shell elements, guarantee the stability of the wall. The frame elements are connected to the shell points through linear elastic springs (Joint link) simulating the behavior of nailed connections. The bottom beams is then connected to the foundation by means of Joint linear link in order to model Hold-Down and angle brackets and endowed with parameters validated through a previous experimental campaign



**Figure 13:** complete numerical model

From performed analyses, the proposed equation is able to predict more accurately the importance of the single elastic components with respect to the existing formulations [5].

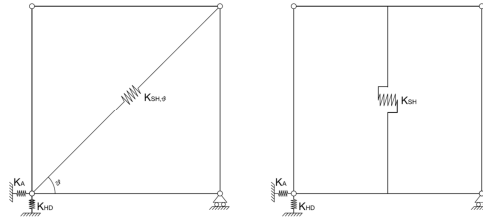
#### 4.2 SIMPLIFIED NUMERICAL MODEL

After validating the proposed equation to predict the horizontal displacement of a framed timber wall by means of a complete numerical model, a simplified model is proposed in order to perform elastic analysis on timber framed buildings. This model allows an easier and faster analysis of a building without losing accuracy compared to the complete one.

The shear deformation components (sheet nail-slip and sheathing board deformation) are modelled by means of an equivalent linear link which stiffness,  $K_{sh,eq}$  is valuated using the follow analytical equation:

$$K_{sh} = \frac{K_C \cdot K_P}{K_C + K_P} \quad (44)$$

Two different simplified models are proposed, where the equivalent linear link has two different orientation. In the first one a diagonal spring is used and in the second a vertical link whose lateral stiffness is defined.



**Figure 14:** simplified numerical models

It is important to underline that in the second model the stiffness of timber framed has to be considered infinity to prevent the bending deformation of the beams.

In the first one model the stiffness of the diagonal spring can be obtained as:

$$K_{sh,\theta} = \frac{K_{sh}}{\cos^2(\theta)} \quad (45)$$

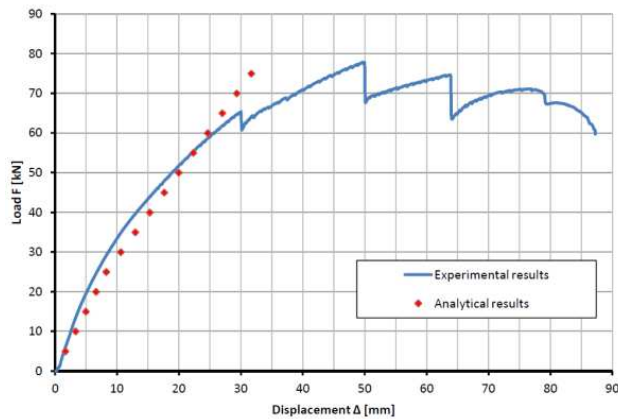
Both models have a vertical spring for Hold-down contribute ( $K_H$ ) and a horizontal one for translation contribute ( $K_A$ ). Vertical loads can be designed as spread or concentrated loads on the top beam.

## 5 VALIDATION OF RESULTS

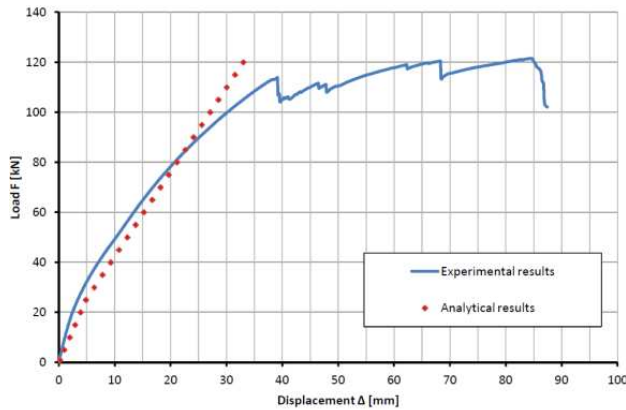
After checking the theoretical equation by means of a complete numerical model, a comparison with experimental tests on full scale walls results was developed.

Displacements under different intensities of the horizontal load were predicted using the analytical formula and compared with displacement of load vs displacement curve of tests.

In particular the results of two types of walls are reported.







**Figure 15:** load vs displacement

The results show a good correlation between analytical prediction and experimental results. It is important underline that the analytical formula is not created to fit the horizontal force vs displacement curve but to predict the elastic displacement under a specified load.

## 6 CONCLUSIONS

A theoretical formula to predict the horizontal displacement of a timber framed wall is reported. This can be considered very useful in order to predict displacements under a horizontal force and to calibrate a simplified model of the wall for elastic analysis (i.e modal analysis).

The parametric study of the lateral wall stiffness shows that there is not a linear proportionality with the wall length cause the hold down deformation component.

It is also very important take account of vertical loads since they reduce the horizontal displacements and at the same time increase the lateral force requested to get a specified displacement.

Analytical formula was validated with experimental results obtained by full scale wall tests.

## ACKNOWLEDGEMENT

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## REFERENCES

- [1] Sartori T., Grossi P., Piazza M., Tomasi R., Characterization of the mechanical behaviour of light-frame timber shear walls through full-scale tests, Word Congress of Timber Engineering 2012, Auckland, New Zealand
- [2] Girhammar U.A, Kallsner B., Plastic models for analysis of fully anchored light-frame timber shear walls. *Engineering Structures*, (31):2171–2181, 2009.
- [3] Girhammar U.A, Kallsner B., Analysis of fully anchored light-frame timber shear walls-elastic model. *Materials and Structures*, (42):301–320, 2009.
- [4] Conte A., Piazza M., Sartori T., Tomasi R. Experimental investigation on connections between wood framed shear walls and foundations. SEWC, Como, 2011.
- [5] Conte A., Piazza M., Sartori T., Tomasi R. Influence of sheathing to framing connections on mechanical properties of wood framed shear walls. ANIDIS, Bari, 2011.

# Appendice E

## Paper V

Shake table test on 3-storey light-frame timber  
building

*Tiziano Sartori, Daniele Casagrande, Roberto Tomasi, Maurizio Piazza*

WCTE 2012, Auckland



## SHAKE TABLE TEST ON 3-STOREY LIGHT-FRAME TIMBER BUILDING

**Tiziano Sartori<sup>1</sup>, Daniele Casagrande<sup>2</sup>, Roberto Tomasi<sup>3</sup>, Maurizio Piazza<sup>4</sup>**

**ABSTRACT:** Shake table tests were carried out on a 7m x 5m three stories timber light frame building (7.5 m high) at the TreesLab laboratory (Eucentre) in Pavia. The goal of the research project was to evaluate the seismic behavior of a typical Italian prefabricated timber building and to study the interaction between the singular structural components tested in quasi-static way. The 1979 Montenegro Earthquake ground motion, recorded at Ulcinj-Hotel Albatros, was selected as the ground motion for seismic tests. The maximum peak ground acceleration was scaled to 0.07g, 0.27g, 0.5g, 0.7g and 1g in order to evaluate the performance of the building at different levels of seismic inputs. A frequency and damping evaluation tests were used before and after each seismic test to identify natural frequencies, modal shapes and equivalent viscous damping ratio, exciting the structure with a low amplitude white random noise. After each seismic excitations the specimen was inspected for evidences of damage. The building designed for a 0,28g PGA representing a hazard level of 10% probability of exceedance in 50 years or a return period of 475 years, showed no visual damages at all the stage of the tests.

**KEYWORDS:** wood-frame multi-storey buildings, seismic performance, shake table test

### 1 INTRODUCTION

#### 1.1 TIMBER-FRAMED MULTI-STOREY BUILDINGS

The timber-frame multi-storey buildings are gaining increasing importance, presenting a sustainable alternative to solutions made with other building materials, as happened during the reconstruction following the earthquake in Abruzzo in 2009 (CASE project). In fact, as demonstrated by the behaviour of wooden buildings in areas of high seismic risk, the low density of the material has as an advantage the minor stresses induced by a seismic event. On the other hand, some systems such as wood-framed buildings (the most popular for civil construction are in the United States, Scandinavia, Canada and New Zealand) are

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characterized by great structural redundancy of connections with good mechanical capacity of ductility and energy dissipation, for which then, in accordance with the recent regulations (Eurocode 8, NTC 2008), is assigned one of the highest q-factor.

The frame construction system is emerging in our country, thanks to the high level of prefabrication, but with some peculiarities, both in terms of geometry and materials used, compared to the lightweight solutions typically adopted in the North American and North European market: the vertical stud and transoms have a major sections and also the cladding panels are preferably thicker and heavier.

## 1.2 INDUSTRIAL RESEARCH PROJECT CHI QUADRATO

For these previous mentioned reasons it has been considered necessary as part of an industrial research project dedicated to sustainable wooden buildings, carry an extensive experimental campaign having as its goal the mechanical characterization in terms of stiffness, strength and ductility of components and structural links of a wooden framed construction system when the geometric parameters and materials used, vary. As part of this research angular connection between the wall frame and horizontal partitions, connections between the top plates and wooden covering sheets and horizontal wall framed have been subjected to static tests in accordance with the protocols of the rules of monotonic and cyclic type [1,2].



**Figure 1:** Test Set-up of the tests on the connections between vertical stud and sheathing panel and the connections between wall and foundation



**Figure 2:** Test Set-up of the tests on framed walls subject to horizontal action (monotonic and cyclic) and different magnitude of vertical load

The results of this experimental campaign have allowed the optimization of the manufacturing technology and the creation of an experimental database which has been used in the analysis of the response of multi-storey buildings in seismic areas, both through linear (modal response spectrum analysis) and non-linear methods (analysis of push-over, time-history analysis).

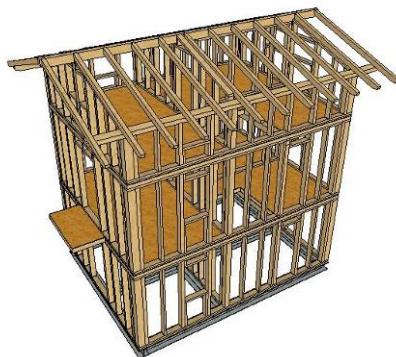
### **1.3 SCOPE OF TESTS**

In order to validate the algorithms based on the knowledge of basic structural components, it has been designed and built a multi-storey wooden building in full scale to be tested on a shake table. Important it is the comparison between the current calculation methods and assumptions implemented in the Regulations and the results of experiments. It therefore seems essential to determine which is the behavior of individual structural components within the building in full scale subjected to seismic action, how they interact and what are the possible failure mechanisms.

## **2 DESIGN AND DESCRIPTION OF THE BUILDING**

### **2.1 GEOMETRY OF THE BUILDING AND THE STRUCTURAL ELEMENTS**

The test building has a rectangular plan, 5m x 7m and it is on three levels (ground floor, first floor and attic) to a maximum height to the peak of 7.65 m.



**Figure 3:** *Three-dimensional view of the building without structural panels*

The horizontal elements are formed by box section elements 140 mm height to which are superimposed and nailed with OSB panels with 15 mm thickness in order to ensure the behavior at rigid in-plane diaphragm.



**Figure 4:** Testing building

The roof with two pitches has been made by solid wood beams and a wooden plank with stiffened perforated metal strips. The structural walls are framed with 15 mm thickness OSB panels, fixed both sides by ring nails 2.8 x 60 mm.

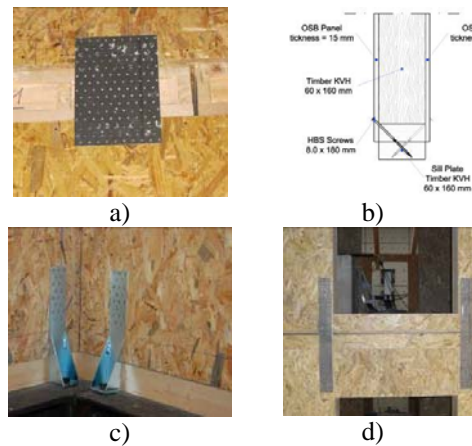
Uprights and transoms of the wall has been made with solid wood, section 160 x 60 and 160 x 100 mm. The building has been tested without finish and non-structural components. In order to simulate the additional mass relative to the permanent loads and accidental loads provided in case of seismic event, they have been placed on the floors some concrete blocks of equivalent weight. In order to simulate the mass of the insulation and the entire stratigraphy of the roof, they have been placed on the roof itself the tiles weighed and additional materials within the walls.



**Figure 5:** Additional masses on the floors and inside the structural wall

## 2.2 GEOMETRY AND ORGANIZATION OF THE CONNECTION SYSTEMS

In order to ensure the interaction between the various structural elements and the connection of the building with the foundation, they have been used different connecting devices. For the horizontal sliding of the structural walls, they have been used plates nailed to the upper floors (Figure 6a) and pairs of wood screws 8x180 inclined at 45 ° to the plane of the foundation (Figure 6b). The tensile forces in the corners of the walls have been assigned to hold-down, to the foundation plan (Figure 6c) and to metal plates nailed to the upper floors (Figure 6d). The connection of the floor with the walls has been carried out by wood screws 8x220.



**Figure 6:** a: Nail plates used to prevent horizontal translation of floors - b: Screws inserted at 45° used to prevent the horizontal translation of walls at foundation level - c: Hold-down - d: Tie-down

### 2.3 STRUCTURAL PROJECT

The design has been made in accordance with Italian Standard (NTC2008) integrated with the European provisions Eurocode 5 and Eurocode 8. The building is assumed to be a residential house located in site characterized by the maximum value of seismic hazard with a return period of 475 years, as prescribed by Italian law.

The seismic design has been carried out by static analysis assuming a behaviour factor  $q$  equal to 4. The forces of the project so calculated have been divided between the walls in proportion to their structural rigidity, assuming rigid floors.

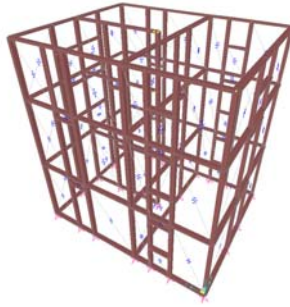
**Table 1:** Seismic hazard parameters in relation to the different limit condition

LIMIT CONDITION	$T_R$	$a_g$	$F_0$	$T_c$
SLO	30	0,049	2,418	0,249
SLD	50	0,068	2,518	0,268
SLV	475	0,277	2,280	0,424
SLC	975	0,402	2,328	0,476

### 2.4 ESTIMATION OF PARAMETERS OF THE TEST BY PRELIMINARY NUMERICAL ANALYSIS

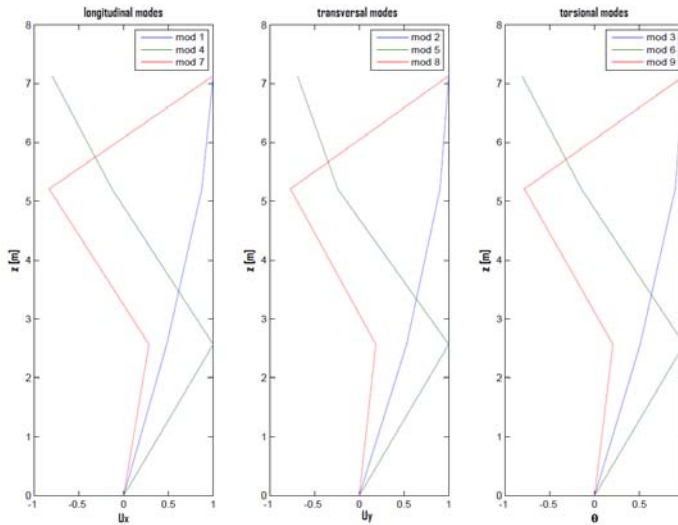
In order to estimate the parameters necessary for the design of the shake table test, some numerical simplified models have been developed. In particular, they have been used some linear models in order to evaluate the dynamic parameters of the building, useful for a correct choice of the input parameters.





**Figure 7:** Numerical model

Each wall was modelled by an equivalent diagonal spring, calibrated in two different ways. In the first one (mod. A) the OSB shear deformation contribution was implemented; in the second one (mod. B) the sheeting nail-slip stiffness contribution was added the OSB shear deformation. The hold-down and angle brackets contribution was neglected since the frequency evaluation tests would have been under a low intensity exciting. The predicted longitudinal fundamental period was equal to 0.11 s for mod. A and to 0.30 s for mod. B.



**Figure 8:** Elastic mode shapes

A non-linear model has been implemented for the estimation of the base shear capacity and of the over turning moment (OTM).

These analyses have also made it possible to design the test phases at different intensities of seismic input, in relation to the expected limit states.

### 3 SEISMIC TESTS ON SHAKE TABLE

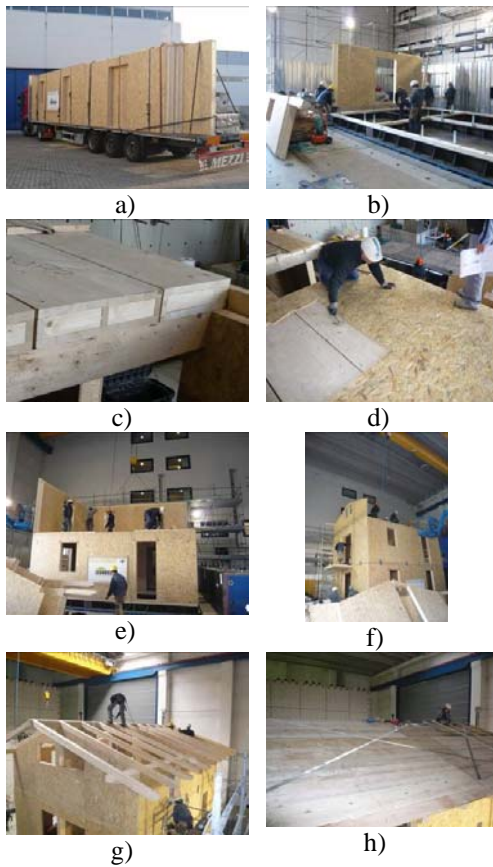
### 3.1 ASSEMBLY

The assembly of the building has been done directly on the vibrating bench. The connection between the building and the table has been realized by a grid of steel beams arranged in correspondence of the structural walls.



**Figure 9:** Grid of steel beams arranged in correspondence of the structural walls

The significant prefabrication of the system and the high accuracy of the machining in factory permitted to complete the phases of assembly in 3 days by 5 workers. During the assembly, in fact, it was necessary to perform only the placement of structural elements (walls, floor panels, stiffened panels, primary and secondary warping of the roof) connecting them with the systems described in paragraph 2.2. In Figure 10, they are visible the different phases of assembly.



**Figure 10:** Assembly phases. a: Structural elements carriage - b: First floor walls placement – c: 1st Floor elements placement – d: OSB boards nailing and placement – e: Second floor walls placement – f: Third floor walls placement o – g: Roof beams placement – h: Roof Perforated metal strips

### 3.2 SET-UP OF THE MEASUREMENTS

The measures concerned accelerations, displacements and deformations. Data were acquired with a sampling frequency of 1024 Hz: 103 instruments have been positioned in addition to the optical acquisition system which can monitor in continuous the displacement of the markers placed on the building, thanks to the various infrared cameras (sampling frequency: 60 Hz).

The positioning of the measurement’s instruments has taken place taking into account the mono directionality of motion of the bench, therefore on the walls parallel to the seismic and structural elements most stressed.

The following measurement’s instruments have been installed for all 12 monitored walls (6 on the first floor and 6 on the second) :

- displacement transducer capable of measuring horizontal scrolling, the vertical displacement and the shear deformation of the walls;
- strain capable of measuring the deformation of the metal elements of connection (hold-down and tie-down).

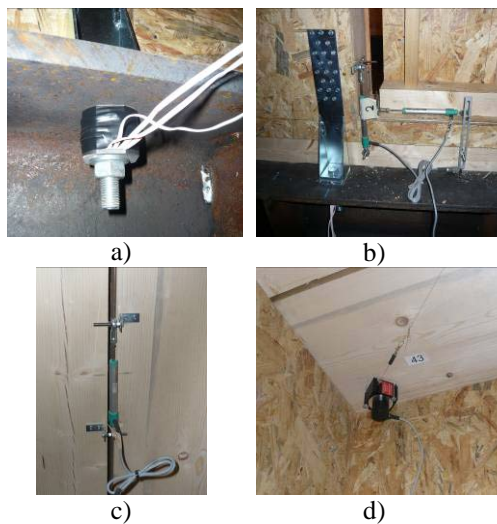
The following Table 2 recaps the measurement’s instruments installed.

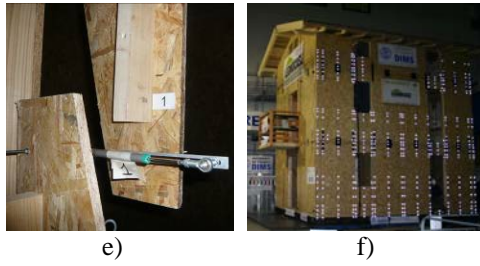
**Table 2:** Measurement’s instruments

Typology	Qty	Type of measure
Cells hold-down	12	Force
Cells tie-down	11	Force
Wire Potentiometers	28	Wall shear deformation
Potentiometers	28	Wall upflit and slippage
Accelerometers	27	Accelerations

It was decided to not monitor the walls of the attic as not subject to significant shifts.

The use of an optical acquisition system has allowed also to monitor both the absolute displacements of the test building and the relative displacements of the control points of the walls of the front C (Figure 11).

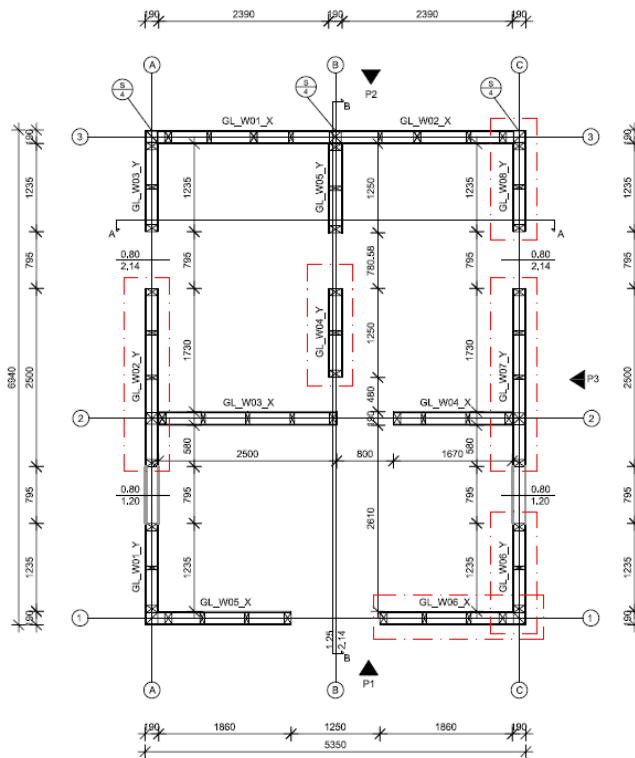




**Figure 11: Instrumentation.** a: Hold down load cells - b: Wall uplift and slippage potentiometers – c: Displacement transducer for horizontal slip between floor elements– d: wire displacement transducer for in plane deformation of horizontal diaphragms– e: Interior wall inter-storey drift potentiometer – f: Optical acquisition system markers

The acceleration measurements have been done with a pairs of unidirectional accelerometers disposed on each of the 4 corners of the two intermediate floors. 8 accelerometers on the eaves and 2 on the peak have been adopted on the roof. The acceleration measurements have been done in the horizontal directions, only.

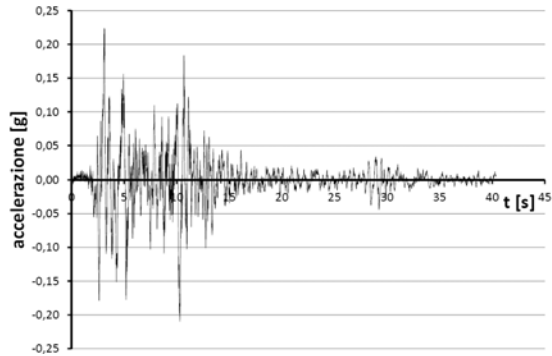
Some displacement transducers have been used to monitor the possible deformations of the floors in its plane and the relative sliding of the base docks compared to the steel base.



**Figure 12: Walls monitored – First floor**

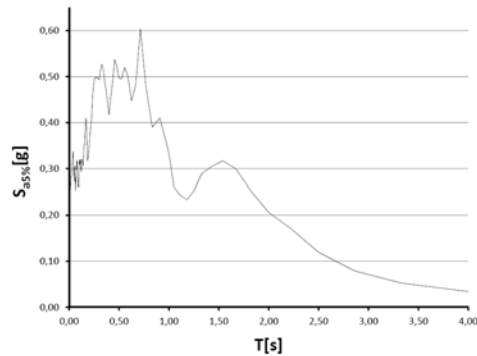
### 3.3 TESTING SEQUENCE

The test building has been subjected to 5 different intensity of seismic input, scaling appropriately the accelerogram recorded by the station "Ulcinj - Hotel Albatros", located at an epicentral distance of 21 km, during the Montenegro earthquake of 15/04/1979 (Mw 6.9).



**Figure 13:** Accelerogram. Montenegro earthquake – “Ulcinj -Hotel Albatros” station– PGA: 0.224g

The choice of this accelerogram occurred in relation to the pseudo-acceleration response spectrum, particularly significant in the range of the theoretical fundamental frequency of the building.



**Figure 14:** Pseudo acceleration response spectrum ( $\xi=5\%$ )

The values of maximum acceleration (PGA) at which the building has been tested in the 5 stages of test, are shown in the table below. These values have been derived considering the analysis of numerical simulation as well as to the results obtained after the first test phase.

**Table 3:** Test phases and maximum acceleration

PHASE 1	0,07g
PHASE 2	0,28g
PHASE 3	0,50g
PHASE 4	0,70g
PHASE 5	1,00g

Each of the seismic testing have been preceded by a phase of "tuning", which was necessary to calibrate the control parameters of the shake table in order to obtain a good correspondence between the feedback and the reference. This procedure has been done submitting the building to seismic motion of a low intensity flat white noise.

During this phase, we proceeded to the acquisition of the signals of the different accelerometers in order to be able to conduct after the identification procedure of the dynamic structure. Indeed, it is essential to estimate with good accuracy the dynamic properties (fundamental frequencies, vibration modes, damping equivalent) of the test building in order to quantify the damage to the building after each seismic test.

The evaluation of the structural damage has also been done after each test by careful visual inspection.

It is important to underline that between one phase and following one, any kind of repair on the structure hasn't been done.

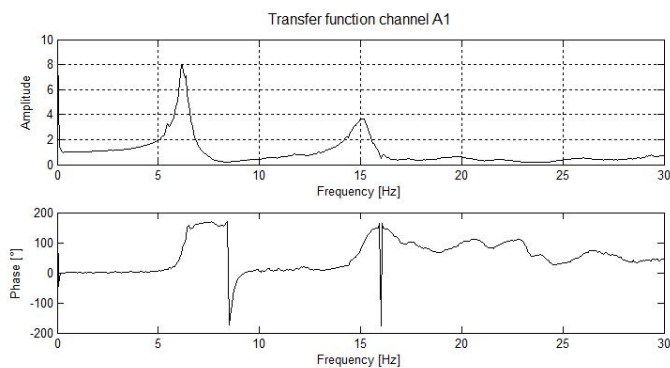
## 4 RESULTS

### 4.1 INSPECTION RESULTS

The inspections performed on the building after all the tests has shown no visible damages to structural elements.

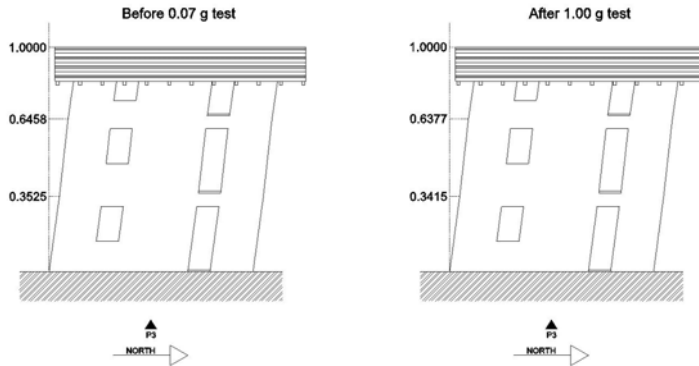
### 4.2 VARIATION OF FUNDAMENTAL PERIOD

The purpose of the frequency evaluation tests was to identify the fundamental period (or frequency) and mode shapes of the test structure after each seismic test. Ten accelerometers time-histories through the structure and one accelerometer on the shake table were analysed to evaluate the transfer functions TFs as ratio, in the frequency domain, between the story acceleration responses and the base motion. The peak amplitude method was used to estimate the fundamental frequency of the longitudinal direction. The data were pre-processed by means of an anti-aliasing filter (frequency of Nyquist equal to 128 Hz), a linear trend removing and the PSD Welch's method computation (using a Hanning window). The sampling frequency was equal to 256 Hz and the frequency resolution to 0.0625 Hz.



**Figure 15:** TF Amplitude and Phase between channel A1 (1<sup>st</sup> floor) and base motion after 0.28g test

The fundamental period was unchanged until 0.70 g and equal to 0.163 s (6.125 Hz). After the phase 4 and the phase 5 the period increased respectively to 0.165 s (6.060 Hz) and to 0.172 s (5.814 Hz). The fundamental period variation suggests that some damages in the structure happened only for the higher acceleration seismic tests. No significant 1<sup>st</sup> mode shape variation was evaluated.



**Figure 16:** Mode shapes before 0.07g test and after 1.00g

### 4.3 INTER STOREY DRIFT MEASUREMENT

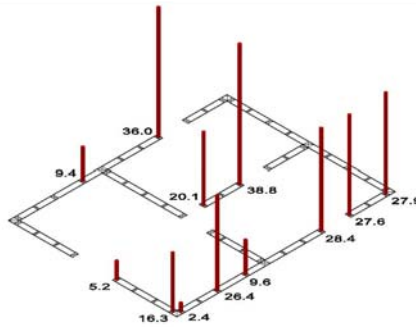
One of the most important parameters to relate the input seismic action to the performances of the structure is the inter storey drift measurement, which is can associated a damage level to. The measurements were performed by means of two potentiometers inside the building and processing the data of optical acquisition system outside. The potentiometers data were filtered with a lowpass filter using a passband frequency of 25 Hz and a stopband frequency for 50 Hz with a 40 db attenuation. The optical acquisition system were not filtered. The maximum inter storey drifts for 1<sup>st</sup> floor are reported in the following table:

**Table 4:** 1<sup>st</sup> floor maximum inter storey drifts

<i>1<sup>st</sup> floor</i>	Ch 41 (Interior)	Optical S. (External)
<i>Test</i>	[%]	
<b>0.07g</b>	0.03	0.02
<b>0.28g</b>	0.21	0.14
<b>0.50g</b>	0.45	0.59
<b>0.70g</b>	0.79	0.63
<b>1.00g</b>	1.12	1.24

### 4.4 HOLD DOWN FORCE MEASUREMENTS

The peak tensile hold down forces during each seismic test are reported. No significant forces were registered until the 0.50 g test. The maximum force was measured in the 1.25 m wall hold down an it was equal to 38.8 kN. It's important to underlined that the collapse of hold down was evaluated equal to 50 kN in the monotonic quasi static hold down test.



**Figure 17:** Peak Tensile Hold down forces for 1.00 g [kN]

## 5 CONCLUSIONS

A three-storey light framed timber building shake table tests has been presented. Five seismic tests were performed in order to investigate different structure limit states.

The building, designed according to the Italian and European standards, showed high seismic performances: no visible damages were noticed for all tests.

However the increasing of longitudinal fundamental period has highlighted a non-elastic behaviour of the tested structure after 0.70g test. The magnitude of measured 1<sup>st</sup> floor inter storey drifts (1.12% for 1.00g test) confirmed this statement according to the results reported in [3] and [4].

The peak tensile hold down measured forces suggest that until 0.50 g test that the role of hold downs could be neglected. For higher input test they become necessary to avoid rocking of the wall. Probably the box structural building behaviour is predominant at lower input tests.

## 6 FUTURE DEVELOPMENTS

The analyses of all instruments are going on in order to investigate the seismic behaviour of the tested structure. Acceleration, Optical system, Tie-Down measures will be examined. Capacity spectra, global hysteretic response and energy response will be also performed. The main goal is to understand the interaction between structural elements under seismic loads.

## ACKNOWLEDGEMENT

The authors gratefully acknowledge the CHI-Quadrato Consortium for partly financing the study, within the research program supported by Autonomous Province of Trento and Legnoscasse factory to provide the material for the specimens.

## REFERENCES

- [1] Conte A., Piazza M., Sartori T., Tomasi R. (2011), Experimental investigation on connections between wood framed shear walls and foundations, Proceedings of the Structural Engineering World Conference, Como Cernobbio, Italy 2011.
- [2] Conte A., Piazza M., Sartori T., Tomasi R. (2011), Influence of sheathing to framing connections on mechanical properties of wood framed shear walls, Congresso Ingegneria Sismica Anidris 2011, Bari
- [3] CUREE project: <http://www.curee.org/projects/woodframe/>
- [4] NEESWood project: [http://jvw.eng.ua.edu/neeswood\\_reports.html](http://jvw.eng.ua.edu/neeswood_reports.html)
- [5] UNI-EN 1998-1 Eurocode 8 Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for building





# Appendice F

## Paper VI

Timber-concrete composite structures with  
prefabricated FRC slab

*Roberto Crocetti, Tiziano Sartori, Mathias Flansbjer*

WCTE 2010, Riva del Garda



# TIMBER-CONCRETE COMPOSITE STRUCTURES WITH PREFABRICATED FRC SLAB

Roberto Crocetti<sup>1</sup>, Tiziano Sartori<sup>2</sup>, Mathias Flansbjerg<sup>3</sup>

**ABSTRACT:** This study relates to the development of innovative composite structures, comprising timber beams and prefabricated concrete slabs, with high prefabrication level, high performance and good durability. For such a purpose completely dry shear connection systems were investigated. Moreover, innovative and very efficient materials, such as fibre reinforced concrete and modified wood were used for the manufacture of the specimens. The investigated shear connectors were; *i*) shear anchor-key of wood and *ii*) special inclined steel tubes. In both cases, the connectors are incorporated in the prefabricated concrete slab, which then easily is connected to the timber sub-structure only by means self-tapping screws. Both types of connectors exhibited a great performance, both in terms of strength, stiffness and ductility. The results from this project will allow the use of timber-concrete composite beams more effectively in large-span structures and also contribute to both simple and rapid manufacture and erection.

**KEYWORDS:** Shear connection, timber-concrete structure, dry connection, modified wood, FRC concrete

## 1 INTRODUCTION

Satisfactory strength and stiffness properties are among the most important prerequisites for a timber-concrete composite structure. However, only these properties are in general not enough to guarantee the success of the structure. In fact, besides strength and stiffness, also other aspects must be considered in the design of timber-concrete composite structures, such as *i*) ductility and

*ii*) simplicity and rapidity of manufacture and erection.

A valid technique - for achieving both high strength and stiffness properties, which at the same time allows for a simple and rapid manufacture and erection - is the use of prefabricated concrete slabs. Only a few investigations have been performed on timber-concrete beams with prefabricated slab. These studies entail the use of concrete slabs with pockets (or holes) for the shear connectors. The empty pockets need then to be filled by adequate material, e.g. non-shrinking concrete. Therefore, the method may be quite time consuming due to the fact that the filling material needs time, both in order to be poured in the pockets, but also time for curing, before a paving layer/water-

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proof insulation can be applied on the top of it.

The investigation presented herein focus on the use of composite structure with high prefabrication level, high performance and good durability. For such a purpose completely dry connection systems are investigated, i.e. systems where the prefabricated concrete slab is connected to the timber sub-structure only by means of self-tapping screws. Moreover, the use of very efficient materials – such as modified wood (furfurylated) and fibre reinforced concrete (FRC) – is investigated.

## 1.1 BACKGROUND

The behaviour of composite timber-concrete beam is bounded by two extreme stiffness limits (see [1]), namely:

*i)* A lower limit, where no horizontal force transfer occurs between the two layers, i.e. “fully non-composite action”. In this case the two layers of the cross section, i.e. the concrete layer and the timber layer, move independently one to the other. The cross section will then have two individual neutral axes and discontinuous bending strain at the interface.

*ii)* An upper limit where the composite cross section has a single neutral axis and the bending strain at the interface between layers are identical, i.e. “fully composite action”.

During bending of a real composite timber concrete beam, horizontal relative displacement (slip) occurs at the interface between the two materials. Slip reduces the efficiency of the composite structure with regard to both stiffness and strength. The structural behaviour of such a beam is something in between a fully non-composite situation and a fully composite situation. Such behaviour is referred to as “partial composite action”.

## 1.2 AIM SCOPE AND LIMITATIONS

The principal aim of this study is to develop a new innovative timber-concrete composite structure with prefabricated concrete slabs that has a high prefabrication level, high performance and good durability. The scope

is to perform both numerical and experimental investigation on the load-slip behaviour of innovative shear connectors between timber and fibre reinforced concrete (FRC) [2].

The investigation presented herein is limited to the study of load-slip behaviour of different shear anchor-keys and shear-loaded timber-concrete specimens.

## 2 STATE OF THE ART

The very first applications of timber-concrete composite concerned the refurbishment of existing timber floors. Several examples of this kind of intervention are reported in the literature. In [3] and in [4], for example, different types of shear connectors for refurbishment of ancient timber structures are shown, along with the achieved improvement in structural efficiency.

During the last decades, several research studies have been conducted, with focus on design methods of new timber-concrete composite floor and bridges. These research works have provided a base of engineering data needed for practical applications. The competitive merit of such composite structures is borne out by several examples of successful projects decks both in Europe but also overseas.

In literature there are several studies that primarily focus on load-slip behaviour of different types of shear connector ([5] and [6]). However, the major part of these studies is based on the use of cast-in site concrete slab. A very few researchers have investigated on the possibility to using prefabricated concrete slabs connected to timber beams. On the other hand, this technique has been used both in USA and Europe in steel-concrete composite structures, since the beginning of the 1980's. Experience has shown that steel-concrete composite structures with prefabricated decks offer several advantages, such as faster erection time, higher quality, better working environment and a dry bridge deck surface [7], [15]. When it comes to timber-concrete composite bridges, large amount of work has

been conducted in Finland which included development of various tests methods for shear connectors. Recently some investigations [5], [12] have been performed at the technical University of Luleå, Sweden, focusing on the development of connector technologies for timber-concrete composite structures that are suitable only to prefabricated slabs. This research shows that the improvement perspectives of such a technique are enormous, but new possible solutions need to be tested. According to [5] and [12], the use of a prefabricated concrete slab implies several advantages such as a better material control, time saving and also, consequently money saving, etc.

### 3 PRELIMINARY INVESTIGATIONS

Before starting the investigations on timber-concrete systems, some preliminary tests were performed on “pure” shear anchor-keys in order to study the suitability of such shear connectors. Finite element analyses were carried out, before testing, in order to obtain a qualitative estimation concerning the behaviour of the connections.

#### 3.1 INVESTIGATIONS ON WOODEN SHEAR ANCHOR-KEYS

A total of six specimens, each consisting of a piece of wood (shear anchor-key) connected to a timber member were tested, see Figure 1. For two specimens (G45\_1, G45\_2) the shear anchor-key was connected to the timber member by means of screws and glue together, whilst for the remaining four specimens (W45\_A, F45\_1, F45\_2, F45\_3) only screws were used.

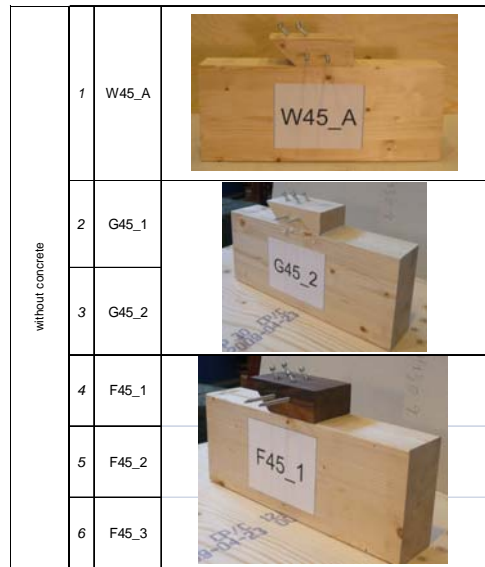


Figure 1: The shear anchor-key specimens

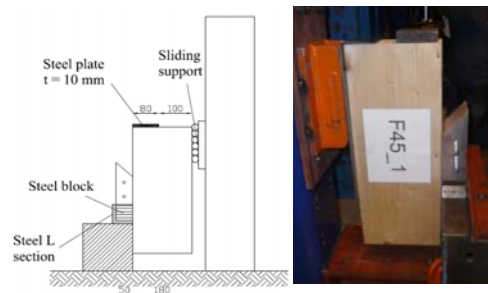


Figure 2: Pictures of the test set-up for shear anchor-key tests

Two types of materials were used for the shear anchor-keys, namely C24-graded spruce and furfurylated beech.

In the wooden anchor-key made of furfurylated beech the holes where the screws were driven into were pre-drilled. The test set-up for the shear anchor-key tests is shown in Figure 2.

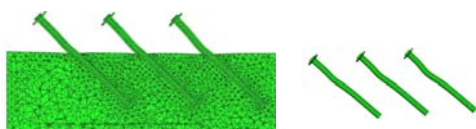
#### 3.2 QUALITATIVE NUMERICAL ANALYSES

Three types of shear connections were investigated, namely:

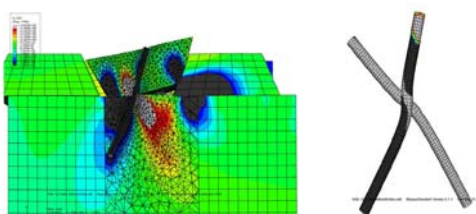
- Connections with steel tubes, “T-system”, see Figure 3.

- Connections with wooden anchor-keys and "X-placed" self-tapping screws, "W30-system", see Figure 4.
- Connections with wooden anchor-keys and self-tapping screws placed with an angle of 45°, "W45-system", see Figure 5.

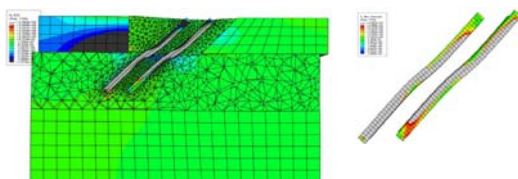
The Finite element analyses were performed with ABAQUS/Standard. For each type of connection a three dimensional model was created and every single part was modelled separately by solid elements. The model of the simulated systems are shown in Figures 3, 4 and 5, respectively.



**Figure 3:** FE Model of the T-system



**Figure 4:** FE Model of the W30 system



**Figure 5:** FE Model of the W45 system

The FE models reproduce the geometry of the specimens that were later tested in the laboratory. The external constraints and the loads were applied according to the laboratory test set-up. The wood material was modelled as an orthotropic material with the mechanical properties shown in Table 1.

**Table 1:** Assumed mechanical properties of the wood

Property	[MPa]	Property	[-]
$E_1$	12600	$\nu_{12}$	0.37
$E_2$	200	$\nu_{13}$	0.42
$E_3$	200	$\nu_{23}$	0.47
$G_{12}$	630	$\nu_{21}$	0.04
$G_{13}$	592	$\nu_{31}$	0.03
$G_{32}$	100	$\nu_{32}$	0.35

The steel was modelled as an isotropic-elastic material with modulus of elasticity  $E_s=210$  GPa and a Poisson's ratio  $\nu_s=0.3$ . The modulus of elasticity for the concrete slab was assumed  $E_c=30$  GPa and the Poisson's ratio  $\nu_c=0.2$ .

The withdrawal behaviour of the screw was simulated by means of tie constraints placed between the outer surface of the screw and the inner surface of the hole in the timber.





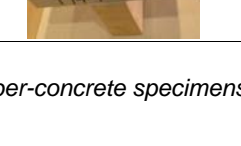


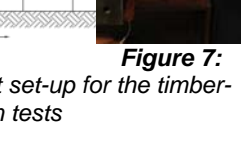
The main findings obtained from the FE-analyses are:

- The connections type T with steel tubes showed less slip at the interface between timber and concrete than the other models. From the model it was also clear that a critical part can be the screw head where there is a stress concentration.
- The connection type W30 with "X-placed" screws indicated a significant rotation of the wooden shear anchor-key due to the inclination of the edge that is in contact with concrete.
- The connection type W45 with parallel screws with an angle of 45° showed a satisfactory behaviour and no significant local deformation in the wooden part.

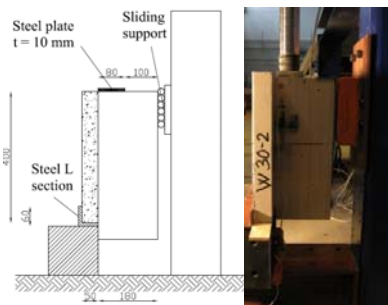
## 4 LABORATORY TESTS ON TIMBER-CONCRETE SPECIMENS

Shear tests were performed on four series of different types of connectors, see Figure 6. A total of eight specimens were tested in order to investigate the load-slip behaviour of the

connectors. The test set-up is shown in Figure 7.

with concrete slab	7	W45_1	
	8	W45_2	
	9	W30_1	
	10	W30_2	
	11	T12_1	
	12	T12_2	
	13	T14_1	
	14	T14_2	

**Figure 6:** The timber-concrete specimens



**Figure 7:**

Pictures of the test set-up for the timber-concrete specimen tests

As it can be seen in Figure 7 an asymmetric test set-up was used for this study. Such a set-up is simpler to construct than a symmetric test set-up, in which two concrete slabs and a central timber element are used. On the other hand, a shortcoming of the asymmetric test set-up configuration is that it leads to an overestimation of approximately 10% both in terms of shear strength and of

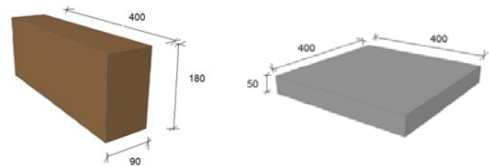
slip modulus, if compared to a symmetric test set-up [12].

The load was applied to the wood beam via a 10 mm thick steel plate with an area of  $80 \times 80 \text{ mm}^2$ . The edge of the concrete slab was placed on an L shaped support. A thin strip of fibreboard was placed between the support and the concrete surface in order to evenly distribute the contact stresses. A low friction sliding support was used to minimize the vertical friction force at the upper horizontal support. The relative displacement between the beam and the slab was measured at mid-height of the specimen, at both side of the timber member. The load and the displacements were recorded continuously during the test with a frequency of 10 Hz.

The loading procedure was carried out according to EN 26891:1991 [10]. The first load cycle, up to  $0.4F_{\text{est}}$  and back to  $0.1F_{\text{est}}$ , was carried out in load control and the subsequent loading to failure in displacement control. The load was maintained constant for approximately 30 s between loading and unloading phases.

#### 4.1 MATERIALS AND GEOMETRIES

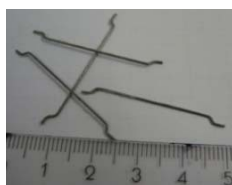
The specimens were designed taking into account the findings obtained in the preliminary study. The geometric properties of the specimens are resumed in Figure 8.



**Figure 8:** Geometry of the timber-concrete specimens

The amount of steel fibres used for the manufacturing of FRC slabs was  $45 \text{ kg/m}^3$ . The aspect ratio of the fibres was  $l/d=6.3$ , where  $l$  is the length and  $d$  is the diameter. The shape of the fibres is shown in Figure 9.





**Figure 9:** Steel fibres

Six standard cube tests with dimensions  $150 \times 150 \times 150 \text{ mm}^3$  were performed in order to measure the compressive strength of the steel fibre reinforced concrete. The results are resumed in Table 2. The first three specimens were tested 28 days after manufacturing of the cubes, whilst the last three specimens were tested later, i.e. at the same day when tests on the timber-concrete specimens were performed.

**Table 2:** Density and compressive strength of the fibre reinforced concrete

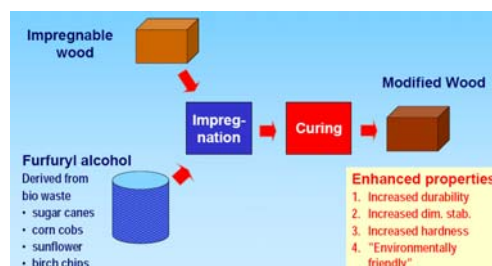
Specimen	Density [g/cm <sup>3</sup> ]	Strength $f_c$ [MPa]
1	2.38	57.8
2	2.36	56.1
3	2.35	57.9
4	2.34	58.1
5	2.34	57.6
6	2.35	57.8
<i>Mean</i>	2.35	57.6
<i>Std. dev.</i>	0.02	0.73
<i>CoV</i>	0.006	0.013

The timber used for all specimens was glulam L40 (similar to the European class GL32), which consists of laminations of Norway spruce with characteristic tensile strength parallel to the grain,  $f_{t,0,k} \geq 22 \text{ MPa}$  and bending strength of the finger joints  $f_{m,j,k} \geq 39 \text{ MPa}$ . The mean density of the specimens was  $\rho_m = 490 \text{ kg/m}^3$  and it was obtained at current moisture content  $MC = 13\%$ .

The material used to manufacture the shear anchor- keys for specimens type W45, W30 and G45 was spruce with strength class C24. The moisture content was 14%.

The material used for manufacturing the shear anchor-key of specimens type F45 was furfurylated beech.

Furfurylation is a wood modification process, using furfuryl alcohol, obtained from renewable resources of corn cobs or sugar cane residuals. Due to its polarity, furfuryl alcohol can penetrate into the cell wall, where it polymerizes. Furfurylation of wood provides a high protection level against bio-degradation. Beside the bioresistance, wood properties like dimensional stability and hardness are significantly improved by the furfurylation of wood. These wood properties depend on the amount of furfuryl alcohol that is brought into the cell wall. Figure 10 summarizes the furfurylation process [9] and [13].



**Figure 10:** Product process of furfurylated wood

The mean density of the furfurylated wooden shear anchor-keys was  $885 \text{ kg/m}^3$ .

The material used for the manufacturing of the steel tubes for specimens type T12 and T14 was ordinary steel S355.

Full threaded screws with different dimensions were used in order to connect the concrete slab with the timber element, namely:

- Screws 11x250 for connections type T (i.e. connections with tubes).
- Screws 7x180 for connections type W, G and F (i.e. connections with wooden shear anchor-key).

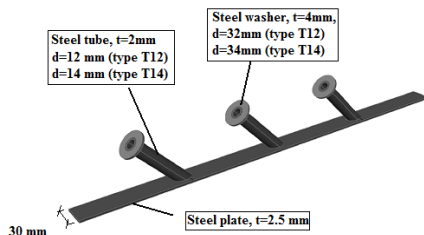
The screws used in this study were laboratory tested at University of Trento, Italy, in March 2008 [14]. The mean value for the ultimate tensile strength was  $f_u = 1256 \text{ MPa}$ .

## 4.2 CONNECTIONS TYPE T: SHEAR CONNECTORS WITH STEEL TUBES

For connections type T (see Figure 11), special steel tubes were applied in the formwork before the prefabricated concrete slab was cast. In order to achieve a better stiffness, the tubes had an inclination of 45° to the longitudinal axis of the beam. The principal dimensions of the steel parts of connections type T are shown in Figure 12.



**Figure 11:** Steel tubes in the formwork before concrete casting



**Figure 12:** Steel tubes geometry

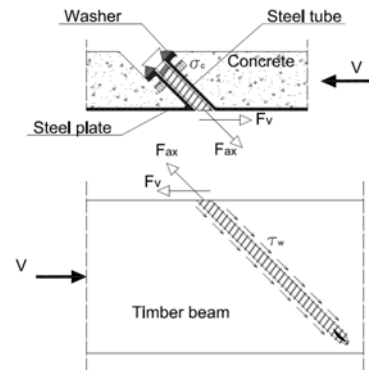
Approximately one month after the concrete was cast, the prefabricated slab was placed on the top of timber members. Successively, self-tapping screws with dimensions  $d=11\text{ mm}$  and  $l=250\text{ mm}$  were driven in the steel tubes and tightened to the timber sub-structure using a torque wrench. The screws were pre-tensioned with a torque moment of 160 Nm.

Two types of configurations were adopted for the connections type T, namely:

- Connection type T12, with tube diameter of 12°mm.

- Connection type T14, with tube diameter of 14°mm.

Nominally identical full-threaded screws were used for both types of T-connections. However, fast setting cement (Quik-Rok® Fast Setting Cement) was poured into the tube for the case of connections type T14, immediately before the screw was tightened. This was done in order to fill the gap between the screw and the tube. On the contrary, no fast setting cement was used for connections type T12. The static principle of connections “type T” is shown in Figure 13.



**Figure 13:** Illustration on the load path in the connection type T

The shear force  $V$ , is transferred from the concrete slab to the timber beam both by shear action  $F_v$  – in the direction of the force  $V$  – and by tension action  $F_{ax}$  – in the direction of the screw axis. In the wooden part, shear is resisted by embedment capacity of the wood, while tension is resisted by withdrawal capacity. In the concrete slab, shear is resisted by contact pressure between the screw and the internal part of the tube, while tension is resisted by axial pressure of the screw head on the top of the tube, via a steel washer. Due to the inclination of the screws, compression stresses will develop at the interlayer between timber and concrete. Such compression stresses generate friction between the two materials, which also contributes to increase the stiffness and the strength of the timber-concrete connection.

### 4.3 CONNECTIONS TYPE W: SHEAR CONNECTORS MADE OF WOODEN ANCHOR KEYS

For connection “type W”, see Figure 14, wooden shear anchor-keys were applied in the prefabricated slab before concrete was cast. Self-tapping double threaded screws with dimension 6.5x220 were driven in the anchor-keys, perpendicularly to the direction of the applied load, before concrete casting. Such screws have two main functions, namely:

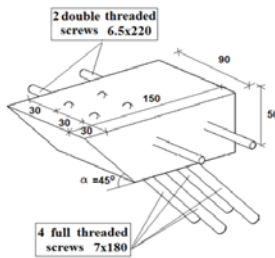
- Proper anchorage of the shear anchor-key to the concrete slab.
- Reduce the risk for splitting of the anchor-key during loading of the specimen.

Approximately one month after the concrete was cast, the prefabricated slab was placed on the top of the timber members. Successively, self-tapping screws with dimensions  $d = 7 \text{ mm}$  and  $l = 180 \text{ mm}$  were driven through the shear anchor-key to the timber sub-structure.



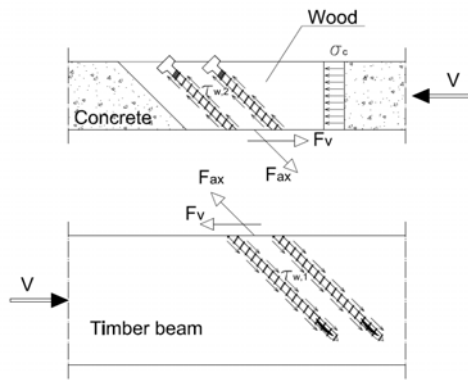
**Figure 14:** Wooden shear anchor-key before concrete casting. Left: W45-type. Right: W30-type.

The geometry of connection type W45 along with the used screws, are shown in Figure 15.



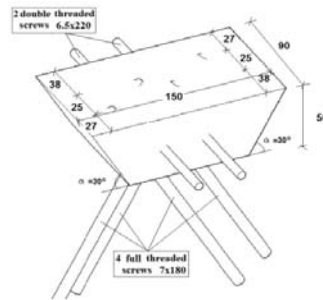
**Figure 15:** Illustration of the W45 system

The static principle of connection type W45 is shown in Figure 16. The shear force  $V$ , is transferred from the concrete slab to the timber beam both by shear action  $F_v$  – in the direction of the force  $V$  - and by tension action  $F_{ax}$  – in the direction of the screw axis. Both in the wooden shear anchor-key and in the timber member, shear is resisted by embedment capacity of the wood, while tension is resisted by withdrawal capacity. Due to the inclination of the screw, compression stresses will develop at the interlayer between timber and concrete. Such compression stresses generate friction between the slab and the timber member, which also contributes to increase the stiffness and the strength of the timber-concrete connection.



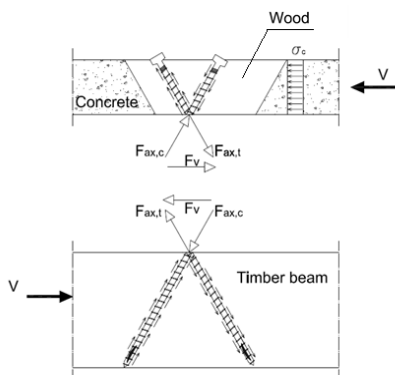
**Figure 16:** Illustration on the load path in the W45 system

The geometry of connection type W30 along with the used screws, are shown in Figure 17.



**Figure 17:** Illustration of the W30 system

The static principle of connection type W30 is shown in Figure 18 and it is rather similar to the static system of connection type W45. The main difference is that, during loading, in the connection type W30 two screws act in tension while the other two screws act in compression, see Figure 18. On the other hand – connection type W45 – all screws act in tension during loading.



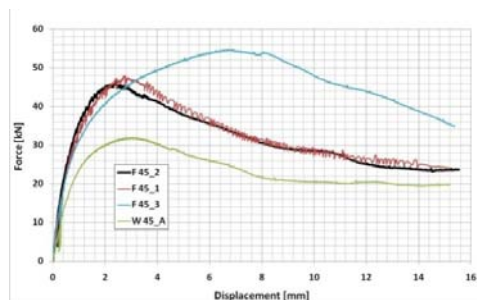
**Figure 18:** Illustration of the load path in the W30 system

## 5 TEST RESULTS

The tests were carried out at SP Technical Research of Sweden, division Building and Mechanics during September 2009. The load-slip curves for the tested specimens are shown in Figures 19, 20, 21 and 22.

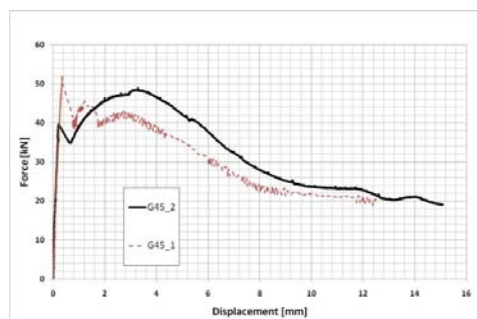
### 5.1 SPECIMENS WITHOUT CONCRETE SLAB

As it can be seen in Figure 19, the shear anchor keys made of furfurylated wood (specimens F45) exhibited higher stiffness and higher load capacity compared to similar shear anchor-keys made of spruce. An examination of the specimens after the tests revealed that – for the case of furfurylated shear anchor-keys – failure occurred due to reached withdrawal capacity of the screws at the timber member. Whereas, for the case of shear anchor-keys made of spruce, failure occurred due to reached withdrawal capacity at the wooden anchor-keys.



**Figure 19:** Load slip curve for specimens type F45 and W45\_A

For the two specimens where the shear anchor-key was connected to the timber member by means of screws and glue together (specimens G45) the initial stiffness was very high, as it was expected, see Figure 20.

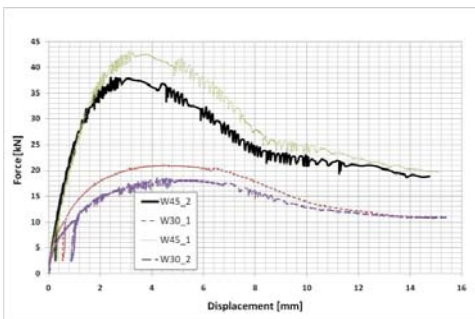


**Figure 20:** Load slip curve for specimens type G45

Both failure at the glue line and withdrawal failure of the screws, occurred at similar load level. Due to very high stiffness of the glue, the first failure occurred at the glue line. Successively, the load transmission from the anchor-key to the timber member was shifted from the glue line to the screws. From this point, the load-slip behaviour of these specimens was very similar to the behaviour of similar specimens where no glue was used (i.e. W45\_A). The final failure, also for this case, occurred due to reached withdrawal capacity of the screws at the shear anchor-key.

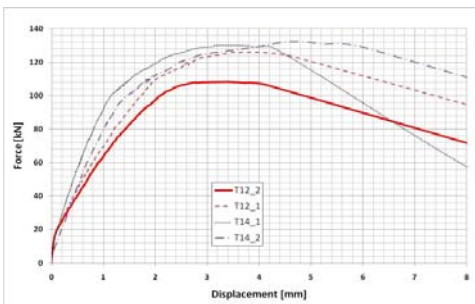
## 5.2 SPECIMENS WITH CONCRETE SLAB

The load-slip behaviour of specimens type W45 was nearly linear up to approximately 30 kN, see Figure 21. On the other hand, the specimens type W30 exhibited almost immediately a nonlinear load-slip response. The discrepancy in behaviour between of these two types of specimens is mainly due to the rotation of the shear anchor-key in the slab, which takes place for the case of specimens type W30, as also explained in section 3.2. Specimens type W45 showed a significantly higher load-carrying capacity and a higher stiffness than specimens type W30. The final failure for specimens of type W45 occurred due to reached withdrawal capacity of the screws at the shear anchor-key. On the other hand, the final failure for specimens type W30 occurred mainly due to large rotation of the shear anchor-key.



**Figure 21:** Load slip curve for specimens type W30 and W45

Both highest failure load and highest stiffness of all tested specimens was obtained for the T-specimens, see Figure 22.

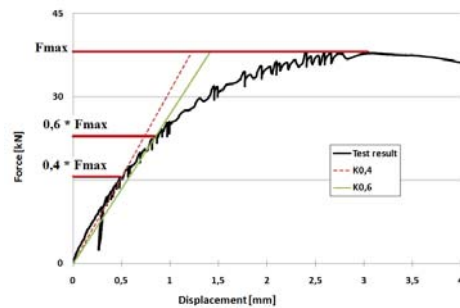


**Figure 22:** Load slip curve for specimens type T12 and T14

The specimens in which the tubes were filled with fast setting cement (i.e. specimens type T14) showed a slightly larger stiffness than those without fast setting cement (i.e. specimens type T12). Failure of the T-type specimens occurred after rather large deformations, either *i*) due to splitting of the timber member into two pieces, along the direction were the screws were driven or *ii*) due to tensile failure of the screws.

## 5.3 STIFFNESS AND STRENGTH FOR THE DIFFERENT TESTED CONNECTIONS

During loading of the specimens, both stiffness and failure loads were recorded for each tested specimens. The procedure to obtain stiffness and failure load from the load-slip curves is indicated in Figure 23.



**Figure 23:** Procedure to obtain  $K_{0,4}$ ,  $K_{0,6}$  and  $F_{max}$  from a load-slip curve

In Table 3, stiffness and failure load for a single connector are shown. For the case of G-type, F-type and W-type specimens, the values reported in the table are those for strength and stiffness of the entire system (i.e. anchor-key plus four screws). For the case of T-specimens, the values reported in the table are those for strength and stiffness of only a single screw.

**Table 3:** Stiffness and failure load for a single connector.

Specimen	$K_{0,4}$ [kN/mm]	$K_{0,6}$ [kN/mm]	$F_{max}$ [kN]
F45_1	48	36	48
F45_2	57	43	46
F45_3	42	29	55
<i>Mean F45</i>	<i>49</i>	<i>36</i>	<i>50</i>
G45_1	217	180	52
G45_2	228	195	49
<i>Mean G45</i>	<i>222</i>	<i>188</i>	<i>51</i>
W30_1	20	14	21
W30_2	16	10	19
<i>Mean W30</i>	<i>18</i>	<i>12</i>	<i>20</i>
W45_A	32	28	32
W45_1	27	24	43
W45_2	31	27	38
<i>Mean W45</i>	<i>29</i>	<i>25</i>	<i>41</i>
T12_1	42	29	42
T12_2	45	29	36
<i>Mean T12</i>	<i>43</i>	<i>29</i>	<i>39</i>
T14_1	49	38	43
T14_2	41	30	44
<i>Mean T14</i>	<i>45</i>	<i>34</i>	<i>44</i>

Table 4 gives an indication of stiffness and failure load that can be achieved for a meter of a hypothetical timber-concrete composite beam, if the tested shear connections were to be used. The following assumptions were made:

- Four shear anchor-key per meter, for F45, G45, W45 and W30 connections.
- Ten screws per meter, for T-connections.

Such assumptions were made on the basis of practical issues, when considering a possible application of the studied shear connectors in real floor structures.

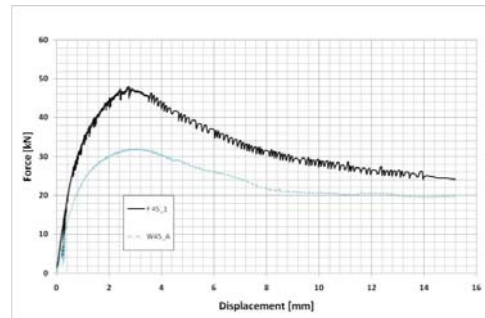
**Table 4:** Stiffness and failure load for a meter of a hypothetical timber-concrete composite beam, using the tested shear connections

System	Anchor-key or screw/m	$K_{0,4}$ [kN/mm/m]	$K_{0,6}$ [kN/mm/m]	$F_{max}$ [kN/m]
F45	4	196	145	199
G45	4	888	751	202
W45	4	115	101	162

W30	4	71	48	80
T12	10	435	292	391
T14	10	450	342	438

## 6 DISCUSSION

The shear tests performed on specimens without concrete slab showed that shear anchor-key made of furfurylated wood behaved considerably better - both in terms of strength and stiffness - than anchor-keys made of spruce, see Figure 24.



**Figure 24:** Comparison between test on the specimen made of spruce (W45\_A) and specimens made of furfurylated wood (F45\_1)

The main reason for this discrepancy in behaviour between the two materials is the fact that density of furfurylated wood is approximately twice the density of spruce. Consequently, the withdrawal capacity is larger for screws that are driven in furfurylated wood than for screws driven in spruce. In the case of spruce, the screws were gradually pushed-in through the shear anchor-key during loading. Eventually, the specimen failed due to complete push-in (read: reached withdrawal capacity) of the screw through the shear anchor-key. In the case of furfurylated wood, on the contrary, no visible push-in of the screw through the shear anchor-key could be observed. Eventually, failure occurred in the timber member, due to reached withdrawal capacity of the screw. Furfurylated wood has a rather brittle behaviour. However, no tendency to split was observed during testing. In the experiments, double threaded screws were

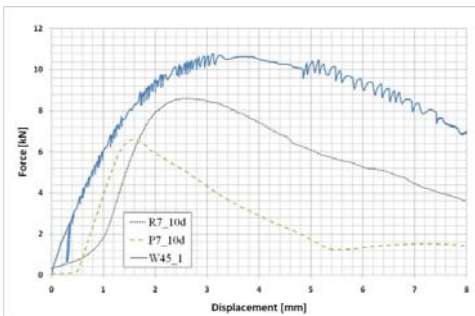
driven in advance in the furfurylated wood – in predrilled holes – perpendicularly to the direction of the applied load. This precaution turned out to be very effective for preventing premature splitting.

Pure withdrawal tests on screws of the same type as the screws adopted for this study were performed at the University of Trento [14]. The screws tested at the University of Trento were driven in the wood both parallel and perpendicularly to the grain. It should be noted here that the load-testing machine used in Trento had some problems with the grip. In fact, a full action of this grip on the screw head was only achieved after an initial deformation in the order of magnitude 0.5-1 mm. This is the reason why the curves for the pure withdrawal tests have a lower gradient during the initial phase of loading.

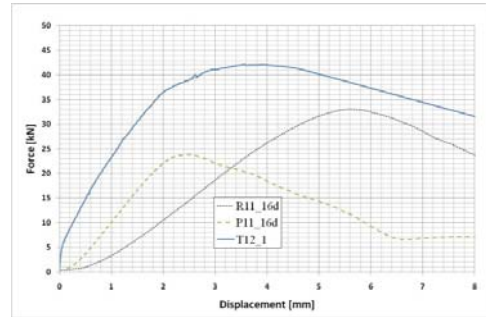
Figure 25 and Figure 26 show the load-slips curves for:

- i) Laboratory withdrawal tests on single screws inserted perpendicularly and parallel to the grain. (These tests were performed at the University of Trento).
- ii) For one screw of a W-type specimen (Figure 25) and a T-type specimen (Figure 26)

The screws in Figure 25 had a diameter of 7 mm and a penetration depth of 70 mm. On the other hand, the screws in Figure 26 had a diameter of 11 mm and a penetration depth of 176 mm.



**Figure 25:** Load-slips curves for i) pure withdrawal tests with different inclination of the screw to the grain (R7\_10d,  $\alpha=90^\circ$  and P7\_10d,  $\alpha=0^\circ$ ) and ii) one screw of the specimen W45\_1,  $\alpha=45^\circ$ .



**Figure 26:** Load-slips curves for i) pure withdrawal tests with different inclination of the screw to the grain (R11\_16d,  $\alpha=90^\circ$  and P11\_16d,  $\alpha=0^\circ$ ) and ii) one screw of the specimen T12\_1,  $\alpha=45^\circ$ .

Disregarding the initial phase of the load-slip curves for the “pure” withdrawal tests due to the abovementioned problems related to the grip of the testing machine, the following observation can be made:

- The stiffness of a single screw of W45-type specimens is similar to the stiffness of a similar screw that is tested for withdrawal capacity.
- The strength of a single screw of W45-type specimen is larger than the strength of a similar screw tested for withdrawal capacity.
- A single screw of T-type specimen has both larger stiffness and larger strength than that of a similar screw tested for withdrawal capacity.

The reasons for the abovementioned discrepancies can be attributed to the following reasons:

- During loading, the screws of either W45-type specimens or T-type specimen, are taking the applied load not only to pure axial stress, but also by shear, which increase both strength and stiffness of the connection.
- In both W45-type specimens and T-type specimens, compression stresses will develop at the interlayer between timber and concrete during loading. Such

compression stresses generate friction between the slab and the timber member, which also contributes to increase the stiffness and the strength of the connection (see section 4.2 and 4.3).

The very high stiffness observed at the initial stage of loading in T-type specimens can be attributed to the precompression generated by the applied torque moment on the screws. Such a precompression generates a large friction between the timber member and the concrete slab, which contributes to increase the stiffness.

## 7 CONCLUSIONS

In this study an analysis of properties of an innovative prefabricated timber-concrete composite system, with different types of shear connectors was presented. Preliminary shear tests were performed on specimens without concrete slab, i.e. consisting of solely *i*) a timber member and *ii*) a wooden shear anchor-key made either of spruce or furfurylated beech. Successively, shear tests on timber concrete specimens with different shear connectors were performed. The investigated shear connectors in this case were:

- i*) Shear anchor-key of spruce with different geometries.
- ii*) Special steel tube (with and without fast setting cement filling).

On the basis of the obtained resultants, the following general conclusions can be drawn:

- Furfurylated wood has both mechanical and physical properties that make it suitable for applications for shear anchor-key connections.
- In all tests where shear anchor-keys of soft wood were used, failure occurred due to reached withdrawal capacity at the anchor-key. On the contrary, withdrawal failure occurred at the timber member, when furfurylated wood was used.
- Type-W30 connections are not completely suitable as shear

connectors for timber-concrete structures, mainly due to the large rotations that occur at the anchor-key during loading.

- Type-T connections have revealed to be very suitable for applications in prefabricated timber-concrete composite structure, due to their extremely high strength and stiffness.
- In type-T connections, the use of tubes with larger dimension and then filling the gap between the tube and the screw with fast setting cement only slightly increases strength and stiffness of the connection.

It is worth to point out that all tested specimens are very easy to manufacture. It is believed, therefore, that the proposed shear connections are very suitable for applications in timber-concrete composite structures with prefabricated slab.

## ACKNOWLEDGEMENT

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## REFERENCES

- [1] A. Ceccotti. Timber concrete composite structures STEP lecture E13
- [2] S. Shah, B. Rangan. Fiber Reinforced Concrete Properties. *Journal Proceeding of American Concrete Institute*, Volume 68, Issues 2, p. 126-137
- [3] A. Parisi, M. Piazza. Restoration and Strengthening of Timber Structures: Principles, Criteria, and Examples. *Structural Design and Construction*, Volume 12, Issues 4, November 2007



- [4] L. Bathon, O. Bletz, R. Bahmer. Concrete bearings a new design approach in wood-concrete-composite applications. In: WCTE, 2006
- [5] E. Lukaszewska, H. Johnsson, L. Sthen. Connections for Prefabricated Timber-Concrete Composite Systems. In: WCTE, 2006
- [6] R. Gutkowski, J. Balogh, J. Natterer, K. Brown. Laboratory tests of composite wood-concrete beam and floor specimens. *Proceedings of the World Conference on Timber Engineering 2000*
- [7] N. Shunichi, C. Peter, Steel concrete composite structures: introduction, *Structural Engineering International*, Volume 19, Number 4, pp. 395-395, November 2009
- [8] Peter Harryson. Industrial Bridge Engineering -Structural developments for more efficient bridge construction. PhD thesis, Department of Civil and Environmental Engineering Structural Engineering, Concrete Structures, Chalmers University of Technology, Goteborg, Sweden 2008
- [9] Stig Lande. Furfurylation of wood – wood modification by the use of furfuryl alcohol. PhD thesis, Department of Ecology and Natural Resource Management, Norwegian University of Life Science, Norway 2008
- [10] European Standard EN 26891. Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristic. February 1991
- [11] H. J. Blass, I. Bejtka. Screws with continuous threads in timber connections. *International RILEM Symposium on Joints in Timber Structures* p. 193 -201
- [12] Elzbieta Lukaszewska. Development of Prefabricated Timber-Concrete Composite Floors. PhD thesis, Department of Civil, Mining and Environmental Engineering Division of Structural Engineering, Luleå 2009
- [13] A. Treu, A. Pilgård, S. Puttmann, A. Krause and M. Westin, Material properties of furfurylated wood for window production. The international research group on wood protection, Section 4 Process and properties
- [14] Certificato di prova n° 12299/237, Materials and structures laboratory University of Trento, 19 March 2008
- [15] P. Collin, A. Stoltz, M. Möller. Innovative Prefabricated Composite Bridges. IABSE SYMPOSIUM, Melbourne, Australia, 2002

# Appendice G

## Paper VII

Timber-Concrete composite floors with  
prefabricated Fibre reinforced Concrete (FRC) deck

*Tiziano Sartori, Luca Costa and Roberto Crocetti*

Early stage researcher, COST Action FP1004, Zagreb 2012



# **Timber-Concrete composite floors with prefabricated Fibre reinforced Concrete (FRC) deck**

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## **Summary**

The presented investigation concerns the study of novel types of timber-concrete composite floors, manufactured with so called “dry connections”. Two full-scale floor elements, each with a different shear connection system, were experimentally investigated. The two shear connector systems used in the investigation were: i) shear anchor-key of furfurylated wood and ii) inclined steel tubes. In both cases, the connectors were incorporated in the prefabricated concrete slab, before casting of concrete. The slab was then connected to the underlying timber beams by means of self-tapping screws. The top slab for both specimens was made of fibre reinforced concrete (FRC). Both dynamic and quasi-static bending tests were performed. The spans of the two floors were different, which resulted in different depths of the timber beam. It was found that both the proposed connection systems performed more than satisfactory, showing a very high degree of composite action even at load levels close to the failure load. Moreover, it appeared that the assembling of the prefabricated fibre reinforced concrete slab with the underlying beams occurred in an extremely easy fashion.

## **1. Introduction**

Timber-concrete composite structural systems have been extensively used during several decades both in floors and in bridges. Most of the studies performed to date have focused on composite systems where “wet” ordinary concrete was cast on top of timber beams with preassembled different shear connectors. Even though such systems have proven to perform very well from the point of view of statics and dynamics, in-situ concrete casting has some clear disadvantages, e.g:

- the introduction of a “wet” component (concrete) on a typically “dry” structure (timber)
- the time needed for the concrete to cure, which adds to the time required on-site before the next scheduled action can be taken
- low stiffness and high creep while the concrete cures
- the high cost of cast-in-situ concrete slabs, mainly due to the cost of transporting fresh concrete, the use of props, the use of formworks and the labor.

The investigation presented herein focus on the use of composite structure with high prefabrication level and high performance. For such a purpose, completely dry connection systems were investigated, i.e. systems where the prefabricated concrete slab is connected to the timber sub-structure only by means of self-tapping screws. Moreover, the use of very efficient materials – such as modified wood (furfurylated) and fibre reinforced concrete (FRC) – is investigated.

## 2. Materials and Method

In order to investigate the behaviour of the prefabricated composite system, two full-scale tests were carried out at the laboratory of Structural Engineering, Lund University. The spans were different for the two tested floors, namely 6 m and 8 m respectively, which resulted in two different cross sections for the timber beams, namely 90x360 mm<sup>2</sup> and 90x450 mm<sup>2</sup>. The timber used for the manufacturing of the floors was spruce glulam (GL30c). The moisture content of the beams was approximately 12%. For the production of 1m<sup>3</sup> of FRC, 45 kg of steel fibres and 375 kg of cement, which gave a mean value of compression strength  $f_c \approx 55$  MPa for the concrete. In the following, the two tested systems will be referred to: *F45* and *T12*. The shape of the specimens is shown in Fig.1.

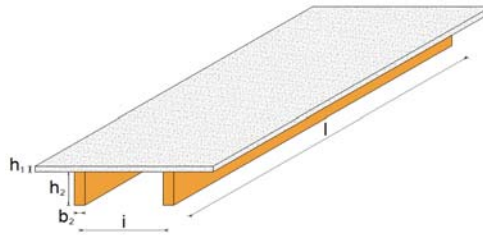


Fig. 1: Geometry of the tested specimens

During the bending tests, the load was applied by an actuator in a displacement controlled manner. The load was distributed on four lines in order to induce stresses and deformations in the floor similar to those induced by a uniformly distributed load. The total load applied to the specimen, the mid-span deflection, and the relative slip between slab and beam at the supports was continuously measured during testing.

### 2.1 Manufacturing of F45 floor system

The main dimensions of F45 floor system are reported in Table 1.

Table 1: Geometry of the F 45 composite system dimensions in [mm]. See also Fig 1.

Span ( $l$ )	Slab width	Slab thickness( $h_1$ )	Beam width ( $b_2$ )	Beam depth ( $h_2$ )	Beam spacing ( $i$ )
6000	1600	50	90	360	800

For the manufacturing of the F45 floor system, wooden shear anchor-keys were applied in the prefabricated slab before concrete was cast. Self-tapping

double threaded screws with dimension 6.5x220 were driven in the anchor-keys, perpendicularly to the direction of the applied load, before concrete casting, see Fig.2. The main functions of such screws are to allow for a proper anchorage of the shear anchor-key to the concrete slab and also to reduce the risk for premature splitting of the anchor-key. The anchorage between concrete slab and underlying timber beams was made by self-tapping screws with dimensions 7x180 driven through the shear anchor-key to the timber sub-structure with an inclination of 45°, see Fig.2. The ultimate tensile strength of the screws was approximately  $f_u = 1250$  MPa.

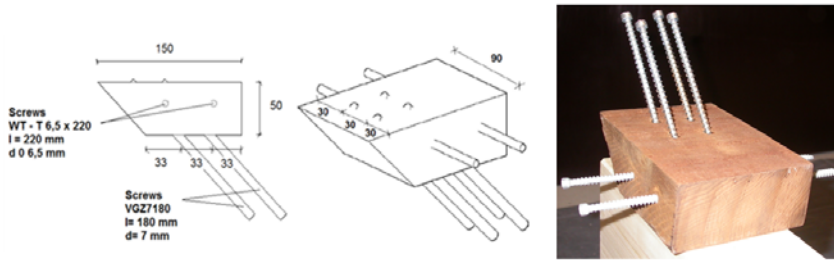


Fig. 2: Shear connectors in F45 specimens

The wood used for the anchor-key was maple impregnated with furfuryl alcohol. Furfurylation improves shape stability and compression strength and make it more suitable to be used with the concrete. The spacing between the wood anchor-keys was 250 mm, for a total of 24 anchor-keys on each beam.

## 2.2 Manufacturing of T12 floor system

The main dimensions of T12 floor system are reported in Table 2.

Table 2: Geometry of the T12 composite system dimensions in [mm]. See also Fig 1.

Span ( $l$ )	Slab width	Slab thickness( $h_1$ )	Beam width ( $b_2$ )	Beam depth ( $h_2$ )	Beam spacing ( $i$ )
8000	1600	50	90	450	800

For the manufacturing of the T12 floor system, special steel tubes were applied in the formwork before the prefabricated concrete slab was cast.



Fig. 3: Shear connectors in T12 specimens (note the white plastic plugs inserted on the top of the tubes to prevent concrete from entering in the tubes during casting)

In order to achieve a better stiffness, the tubes had an inclination of  $45^\circ$  to the longitudinal axis of the beam. Approximately one month after the concrete was cast, the prefabricated slab was placed on the top of timber members. Successively, self-tapping screws with dimensions  $d = 11\text{mm}$  and  $l = 250\text{mm}$  were driven in the steel tubes and tightened to the timber sub-structure. The spacing between the steel tubes was 100 mm.

### 3. Results and Analysis

Deformations of the shear connectors generate horizontal movement, i.e. slip at the interface between concrete and timber. Such a behavior is referred to as “partial composite action” and, as the slip increases it reduces the efficiency of the cross section. The efficiency of a shear connection for a composite beam can be estimated using the following equation, see [1]:

$$\eta = \frac{EJ_{real} - EJ_0}{EJ_\infty - EJ_0} \quad (1)$$

where  $\eta$  is the efficiency,  $EJ_\infty$  is the bending stiffness of the beam with full composite action,  $EJ_0$  is the bending stiffness of the beam with no composite action and  $EJ_{real}$  is the actual bending stiffness of the beam.

#### 3.1 Behavior of the F 45 floor system

The main results concerning F45 floor system are shown in Fig. 4. The curve on the left shows the relationship between the “equivalent uniformly distributed load”  $q$  (i.e. the total load applied divided by the slab area) and



the deflection  $f$  at mid-span. As it is observed, the behaviour was linear up to a load level of approximately  $35 \text{ kN/m}^2$ , which is well above the design load for common floor structures. The decrease in stiffening observed after this load level was reached, may be the result of a severe crushing which occurred at one of the beam support, due to large compression perpendicular to the grain. The curve on the right shows the variation of the efficiency  $\eta$  with increasing load levels. At load levels comparable to those at the serviceability limit state (i.e.  $1\text{-}3 \text{ kN/m}^2$ ) the efficiency is

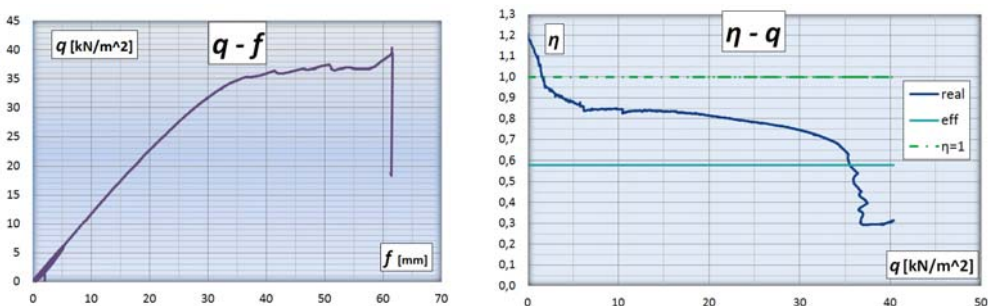


Fig. 4: Left: Load-deflection behavior and Right: efficiency-load curve

high, close to 1.0. The “real efficiency” (dark blue) was always higher than the “effective efficiency” (light blue) as evaluated according to the  $\gamma$ -model, proposed in EC 5 (Annex B). The collapse of the system occurred at  $q \approx 42 \text{ kN/m}^2$ , with the propagation in one of the two beams of two large cracks in the direction parallel to the grain. The lower crack was located at the interface between the second and the third lamella in the bottom part of the timber beam; the upper crack along a line running through the points of the screws used in the shear connections. For the  $F45$  system, the first natural frequency determined experimentally was  $f \approx 14 \text{ Hz}$ , which is in agreement with the value of the first eigenfrequency determined analytically.

### 3.2 Behavior of the T 12 floor system

The main results concerning T12 floor system are shown in Fig. 5.

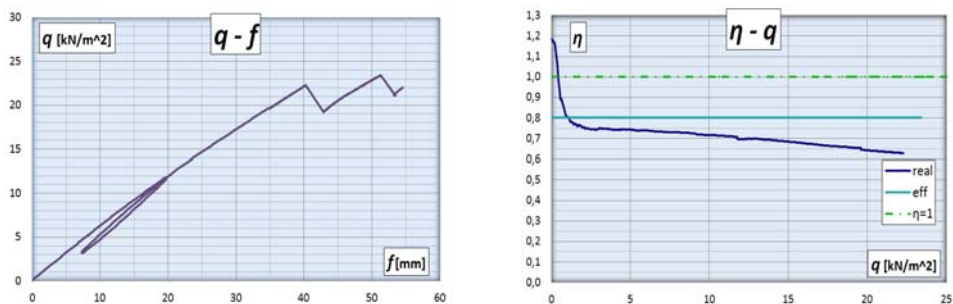


Fig. 5: Left: Load-deflection behavior and Right: efficiency-load curve

The curve on the left shows the relationship between the “equivalent uniformly distributed load”  $q$  (i.e. the total load applied divided by the slab area) and the deflection  $f$  at mid-span. As it can be observed, the behaviour was linear up to a load level of approximately 22 kN/m<sup>2</sup>, which is well above the design load for common floor structures. The curve on the right shows the variation of the efficiency  $\eta$  with increasing load levels. At load levels comparable to those at the serviceability limit state (i.e. 1-3 kN/m<sup>2</sup>) the efficiency is high, close to 1.0. The collapse of the system at  $q \approx 24$  kN/m<sup>2</sup> occurred due to bending failure at a finger joint in one beam and at a knot in the other beam. Both the finger joint and the knot were located close to mid-span. For the *T12* system, the first natural frequency determined experimentally was  $f \approx 11$  Hz, which also is in agreement with the value of the first eigenfrequency determined analytically.

## 5. References

- [1] R. Crocetti, T. Sartori, M. Flansbjerg., *Timber-concrete composite structures with prefabricated FRC slab*. Riva del Garda, WCTE 2010.
- [2] Lukaszewska E. Johnsson H. Fragiacomio M., *Performance of connections for prefabricated timber-concrete composite floors*. Materials and Structures Volume 41, Number 9, 1533-1550.
- [3] Lukaszewska E. *Development of Prefabricated Timber- Concrete Composite Floors*. PhD thesis (2009), Luleå University of Technology.



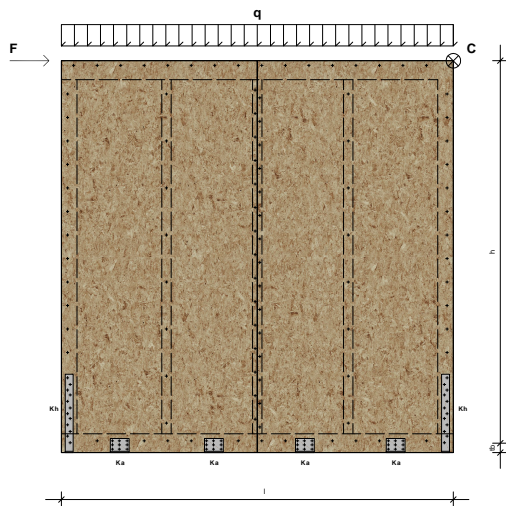
# Appendice H

## Conclusions and future developments

The research carried out during the PhD had two primary objectives. The first was to set up an experimental campaign to create a large database focusing on the mechanical properties of timber framed buildings starting from the singular components (panel-stud connectors, hold-down, angle brackets) to analyze the whole building. This database has been made available to the entire research group on timber structures of the University of Trento (<http://www.unitn.it/en/dims/16845/timber-structures>) and it will be used both to understand in detail the structural behavior of the timber framed building system and to validate the proposed formulations. The data were summarized in the seven scientific reports listed in Table 1.1.

The second objective was the development of a formulation able to predict the displacement and hence the stiffness in the elastic range of a timber framed wall subject to a horizontal force. The formulation, reported in detail in the article of Appendix D and briefly summarized in the equation H.1, has been validated through tests on full scale walls. Starting from the knowledge of the stiffnesses of the individual components, the formula allows to find the displacement of an entire wall with adequate precision and to be able to analyze stiffness and displacement of an entire building, composed of multiple walls. Given a wall schematized as reported in Figure H.1, after analyzing the different standard approaches and formulations available in literature the horizontal

displacement of the control point C could be evaluated according to the equation H.1.



**Figura H.1:** Wall geometry

$$\Delta = \frac{F \cdot h}{l} \cdot \frac{1}{G_p \cdot n_p \cdot t_p} + \frac{\lambda \cdot F \cdot s_c}{l \cdot n_p \cdot k_c} + \frac{F \cdot i_a}{k_a \cdot l} + \frac{h}{l \cdot k_h} \cdot \left( \frac{F \cdot h}{l} - \frac{q \cdot l}{2} \right) + \frac{h}{l} \cdot \left( \frac{F \cdot h}{l} + \frac{q \cdot l}{2} \right) \cdot \frac{t_b}{E_{90,b} \cdot A_m} \quad (\text{H.1})$$

Where:

- **F**: is the applied horizontal force;
- **q**: is the uniformly distributed vertical load applied on the wall;
- **l**: is the total length of the wall;
- **h**: is the height of the wall;
- **b**: is the base of the sheathing panels;
- **G<sub>p</sub>**: is the G modulus of the sheathing panel;
- **n<sub>p</sub>**: is the number of sides where the sheathing panel is placed;
- **t<sub>p</sub>**: is the thickness of the sheathing panel;
- **s<sub>c</sub>**: is the connectors spacing along the boundary of the sheathing panel;

- $\mathbf{k}_c$ : is the stiffness of the connectors between sheathing panels and timber stud;
- $\mathbf{k}_a$ : is the horizontal stiffness of the angle brackets;
- $\mathbf{n}_a$ : is the number of the angle brackets applied on the wall;
- $\mathbf{k}_h$ : is the stiffness of hold-downs;
- $\mathbf{t}_b$ : is the thickness of the sill beam;
- $\mathbf{A}_m$ : is the area of the external vertical stud;
- $\mathbf{E}_{90,b}$ : is the E modulus perpendicular to the graining of the sill beam;

The  $\lambda$  variable can be considered as a form function since it only depends on the dimensionless ratio  $\alpha = \frac{h}{b}$  of the sheathing panel and it corresponds to the expression shown in equation H.2.

$$\lambda = \alpha^2 \cdot \left[ \frac{1}{\eta(\alpha)} + \frac{1}{\xi(\alpha)} \right] = \lambda(\alpha) \quad (\text{H.2})$$

The parameter  $\lambda$  is only a function of the geometric characteristics of the used sheathing panel and of the layout of the studs.

The formulation reported is general and can be applied to any type of load-bearing timber framed walls. In the framed walls usually used in central-south Europe, the size of the vertical studs permits to neglect the displacement due to the crushing of the sill beam, compared to the other variables (the largest dimension causes the foot print area to be large and the deformation it is very low).

The comparison of the proposed formulation with laboratory tests showed the good accuracy of the formula (Figure H.2).

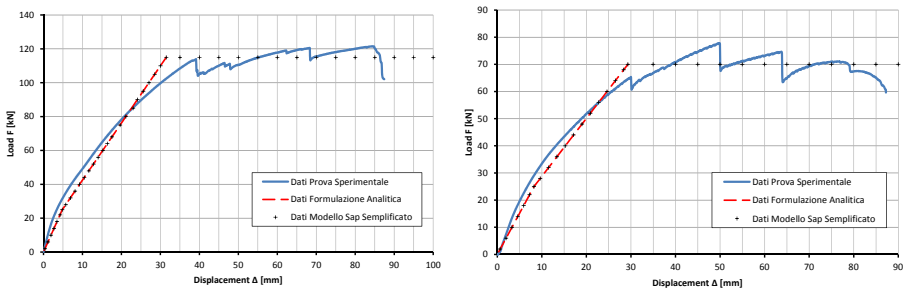
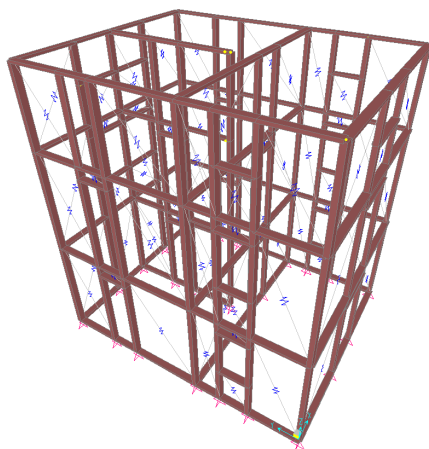


Figura H.2: Comparison between tests and analytical formula

The formulation has also enabled the creation of a finite element model used to predict the behavior of a shaking table test (Figure H.3).



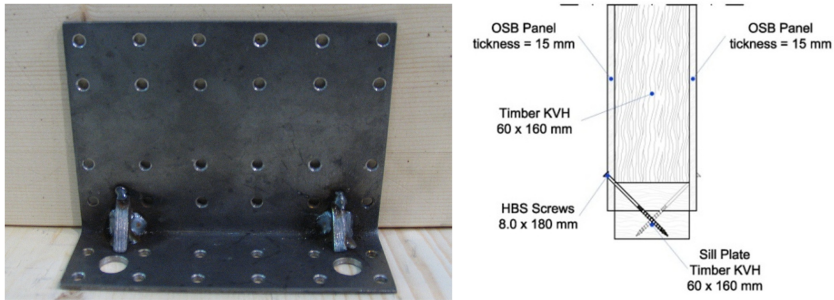
**Figura H.3:** *FE model of timber building*

In this model, the walls have been schematized as unstable frames braced by an oblique spring with stiffness equal to that resulting from the formulation of equation H.1 applied to the geometry of each wall. The model has been implemented by using two different hypotheses. In the first one it was assumed that only OSB panels were deformable. With this assumption the model returned a value of the period of the first vibration mode of 0.11 s. In the second one it was assumed that also the connections between panels and studs were active. The resulting period was of 0.32 s. The real test on the building, in which some devices are turned on, while others are not, gave a period of 0.163 s, approximately as average of the one predicted by the formula. It can be concluded that both tests on the walls and the shaking table test have validated the proposed formula.

Other goals, which are summarized below, were achieved in addition to these two objectives, which are considered of major importance.

After the first tests carried out on the angle brackets suitable to connect the wall to the foundation it has been noted that the devices commercially available are ineffective both as regards the stiffness and the resistance offered. Furthermore, the holes in the angle brackets are often positioned outside the area useful to insert the nail in the framed walls. For this reason it has been proposed a new device described in detail in the Appendix B and reported in Figure H.4.

An engineering model suitable to understand the mechanical behavior of angle brackets was also developed. The model is described in detail in the article of Appendix B.



**Figura H.4:** *Proposed new angle brackets (sx) and shear connection with screws (dx)*

The test results showed that the new connections are much more effective than those commercially available. In addition, a connection made of wood screws (Figure H.4) was also proposed. Moreover this connection has reported good results and it is very interesting especially for the speed of insertion and for the cost-effectiveness of the system itself.

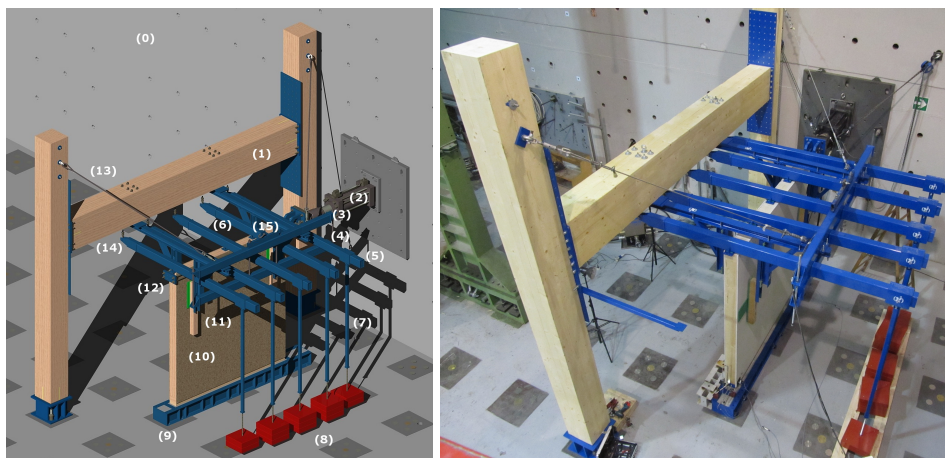
Before performing tests on full scale walls, a wide literature search was done concerning the set-up and test procedures used by the various research centers to test the timber framed walls. This literature review has led to identify the points in order to have a set-up suitable to test this type of wall. In particular the attention was focused on what the boundary conditions are and how they should influence the test results. The way of application of both vertical and horizontal load, as well as the anchorage of the wall to the ground, is of fundamental importance on the influence of the test results. For these reasons, it was designed and created an ad hoc set-up (Figure H.5).

Such a set-up applies the vertical load through steel weights and levers. The load has always the same magnitude and it is applied at the same point on the wall. The horizontal load is applied via a special joint made of spherical hinge in order to follow the sample and to maintain the value of the horizontal load always perpendicular to the wall. The base to which the specimen is fixed is designed to be able to easily attach the angle brackets to the steel profiles and leave the wall in contact with the concrete in order to reproduce the real conditions of friction.

## H.1 Future developments

The key point for the future development of this research work are:





(0)-Muro di contrasto; (1)- Telaio principale; (2)- Martinetto idraulico; (3)- Traversa di sicurezza superiore ; (4)- Traversa di sicurezza inferiore; (5)- Manicotti scorrevoli; (6)- Profili tubolari (leve); (7)- Asta di collegamento manicotto-peso; (8)- Piastre di contrappeso; (9)- Base di appoggio della parete; (10)- Parete; (11)- Ritegni torsionali; (12)- Piastra di testa per prove cicliche; (13)- Tiranti dotati di tenditore; (14)- Supporto dotato di cerniera sferica; (15)- Gancio di sollevamento traversa inferiore.

**Figura H.5:** *Set-up suitable to test timber walls*

- Extension of the elastic analysis proposed for a single wall, to the whole building.
- Analysis of the timber framed walls in non-linear range, with particular regard to the development of a formula suitable to predict the ultimate displacement of a wall through the knowledge of the properties of the individual components (nails, hold-down, angle brackets).
- Development of hold-down devices able to ensure greater stiffness and greater resistance.
- Development of a methodology for calculating simple timber buildings in seismic areas.
- Complete interpretation of the datas got from shaking table test.

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# Bibliografia

- [1] A. Conte, M. Piazza, R. Tomasi, and T. Sartori. Indagine sperimentale su collegamenti tra fondazione in calcestruzzo e pareti intelaiate lignee. Technical report, Dipartimento di Ingegneria Meccanica e Strutturale - Università di Trento, 2010.
- [2] E. Acler, M. Piazza, R. Tomasi, and M. Webber. Experimental investigation of the behaviour of different types of connections between the xlam panels and the concrete slab. In *Structural Engineering World Congress, Villa Erba, Como, Italy*, 2011.
- [3] M. Andreolli, M. Piazza, R. Tomasi, and R. Zandonini. Ductile moment-resistant steel-timber connections. *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, 164(2):65–78, 2011. Andreolli, Mauro Piazza, Maurizio Tomasi, Roberto Zandonini, Riccardo.
- [4] Alessandro Armanini. Prova su tavola vibrante di un edificio in legno a pareti portanti intelaiate: progetto ed esecuzione. Master’s thesis, Dipartimento di ingegneria meccanica e strutturale, Università degli studi di Trento, 2011.
- [5] CNR-DT-206:2007. Istruzioni per la progettazione, l’esecuzione ed il controllo delle strutture di legno, 2007. CNR, Consiglio Nazionale delle Ricerche, Roma, Italia.
- [6] M. Collins, B. Kasal, P. Paevere, and G. C. Foliente. Three-dimensional model of light frame wood buildings. i: Model description. *Journal of Structural Engineering*, 131(4):676–683, 2005.
- [7] A. Conte. Indagine sperimentale su pareti intelaiate in legno: influenza dei sistemi di collegamento sulla resistenza e rigidezza globale. Master’s thesis, Dipartimento di ingegneria meccanica e strutturale, Università degli studi di Trento, 2010.

- [8] A. Conte, M. Piazza, T. Sartori, and R. Tomasi. Experimental investigation on connections between wood framed shear walls and foundations. *SEWC, Como*, 2011.
- [9] A. Conte, M. Piazza, T. Sartori, and R. Tomasi. Influence of sheathing to framing connections on mechanical properties of wood framed shear walls. *ANIDIS, Bari*, 2011.
- [10] A. Conte, M. Piazza, R. Tomasi, and T. Sartori. Experimental investigation on connections between wood framed shear walls and foundations,. In *Structural Engineering World Congress, Villa Erba, Como, Italy*, Cernobbio, Como, Italy, 2011.
- [11] A. Conte, M. Piazza, R. Tomasi, and T. Sartori. Indagine sperimentale su collegamenti tra rivestimento strutturale e telaio ligneo nei sistemi a parete portante. Technical report, Dipartimento di Ingegneria Meccanica e Strutturale - Università di Trento, 2010.
- [12] CSA 086-01:2005. Engineering design in wood, 2005. CSA, Canadian Standard Association, Toronto, Canada.
- [13] DIN 1052: 2008. Design of timber structures. General rules and rules for buildings, 2008. DIN Deutsches Institut für Normung, Berlin, Germany.
- [14] J. D. Dolan and R. O. Foschi. Structural analysis model for static loads on timber shear walls. *Journal of Structural Engineering*, 117(3):851–861, 1991.
- [15] B. Dujic, S.Klobacr, and R. Zarnic. Shear capacity of cross-laminated wooden walls. In *Proceedings of World conference timber engineering*, 2008.
- [16] J. T. Easley, M. Foomani, and R. H. Dodd. Formulas for wood shear walls. *Journal of the Structural Division*, 108:2460–2478, 1982.
- [17] J. Ehlbeck, H. Kreuzinger, G. Steck, and H. J. Blaß. *DIN 1052 Erläuterungen*. Bruderverlag, 4 2005.
- [18] EN12512:2001/A1:2005. Timber Structures - Test methods - Cyclic testing of joints made with mechanical fasteners, 2005. CEN, European Committee for Standardization, Brussel, Belgium.

- [19] EN 12512:2001. Timber structures - Test methods - Cyclic testing of joints made with mechanical fasteners, 2001. CEN, European Committee for Standardization, Brussel, Belgium.
- [20] EN1995-1-1:2009. Design of timber structures Part 1-1: General-common rules and rules for building, 2009. CEN, European Committee for Standardization, Brussel, Belgium.
- [21] R. H. Falk and R. Y. Itani. Finite element modeling of wood diaphragms. *Journal of Structural Engineering*, 115(3):543–559, 1989.
- [22] A. Filiatrault. Static and dynamic analysis of timber shear walls. *Canadian Journal of Civil Engineering*, 17(4):643–651, 1990.
- [23] A. Filiatrault, H. Isoda, and B. Folz. Hysteretic damping of wood framed buildings. *Engineering Structures*, (25):461–471, 2003.
- [24] A. Filiatrault, H. Isoda, and B. Folz. Hysteretic damping of wood framed buildings. *Engineering Structures*, 25(4):461 – 471, 2003.
- [25] R. Finn. Scientific report on dynamic tests of timber frame structures. Technical report, Otto Graf Institute, 2006.
- [26] R. O. Foschi. Analysis of wood diaphragms and trusses. part i: Diaphragms. *Canadian Journal of Civil Engineering*, 4(3):345–352, 1977.
- [27] I. Gavric, A. Ceccotti, and M. Fragiaco. Experimental cyclic tests on cross-laminated timber panels and typical connections. In *Proceedings of Anidis conference, Bari, Italy*, 2011.
- [28] U. A. Girhammar and B. Källsner. Elasto-plastic model for analysis of influence of imperfections on stiffness of fully anchored light-frame timber shear walls. *Engineering Structures*, 31(9):2182 – 2193, 2009.
- [29] U. A. Girhammar and B. Källsner. Analysis of influence of imperfections on stiffness of fully anchored light-frame timber shear walls – Elastic model. *Materials and Structures*, 42:321–337, 2009.
- [30] Paolo Grossi. Pareti intelaiate in legno: indagine sul comportamento meccanico tramite prove sperimentali a scala reale. Master’s thesis, Dipartimento di ingegneria meccanica e strutturale, Università degli studi di Trento, 2011.

- [31] A. K. Gupta and G. P. Kuo. Wood-framed shear walls with uplifting. *Journal of Structural Engineering*, 113(2):241–259, 1987.
- [32] A. K. Gupta and G. P. Kuo. Modeling of a wood-framed house. *Journal of Structural Engineering*, 113(2):260–278, 1987.
- [33] A. K. Gupta and G. P. Kuo. Behavior of wood-framed shear walls. *Journal of Structural Engineering*, 111(8):1722–1733, 1985.
- [34] R. Y. Itani and C. K. Cheung. Nonlinear analysis of sheathed wood diaphragms. *Journal of Structural Engineering*, 110(9):2137–2147, 1984.
- [35] F. Fonseca J. Judd. Analytical model for sheathing-to-framing connections in wood shear walls and diaphragms. *Journal of structural engineering*, pages 345–352, 2005.
- [36] S. N. Jones and F. Fonseca. Strength of osb shear walls with over-driven sheathing nails. In *Proceedings of World conference timber engineering*, 2000.
- [37] J. Judd and F. Fonseca. Analytical model for sheathing-to-framing connections in wood shear walls and diaphragms. *Journal of Structural Engineering*, 131(2):345–352, 2005.
- [38] B. Källsner and U. A. Girhammar. Analysis of fully anchored light-frame timber shear walls-elastic model. *Materials and Structures*, 42:301–320, 2009.
- [39] Bo Källsner and Ulf Arne Girhammar. Plastic models for analysis of fully anchored light-frame timber shear walls. *Engineering Structures*, 31(9):2171 – 2181, 2009.
- [40] B. Kallsner and U. A. Girhammar. Plastic models for analysis of fully anchored light-frame timber shear walls. *Engineering Structures*, (31):2171–2181, 2009.
- [41] B. Kallsner and U. Ar. Girhammar. Analysis of fully anchored light-frame timber shear walls-elastic model. *Materials and Structures*, (42):301–320, 2009.
- [42] T. zur Kammer. *Zum räumlichen Tragverhalten mehrgeschossiger Gebäude in Holztafelbauart*. PhD thesis, Institut für Baukonstruktion und Holzbau der Technischen Universität Carolo, 2006.
- [43] B. Kasal and R. J. Leichti. Nonlinear finite-element model for light-frame stud walls. *Journal of Structural Engineering*, 118(11):3122–3135, 1992.

- [44] Bohumil Kasal and Robert J. Leichti. Incorporating load sharing in shear wall design of light-frame structures. *Journal of Structural Engineering*, 118(12):3350–3361, 1992.
- [45] M. H. Kessel. Tafeln - eine linear elastische beschreibung. *Holzbau Kalender, Bruderverlag Karlsruhe*, pages 599 – 632, 2003.
- [46] M. H. Kessel and T. Zur Kammer. Three-dimensional load-bearing behaviour of multistorey timber frame buildings. In *Proceedings of World conference timber engineering*, Lahti, Finland, 2004.
- [47] M. H. Kessel and M. Meyer. Wood panel diaphragms with free sheathing joints. In *Proceedings of World conference timber engineering*, Portland, OR, USA, 2006.
- [48] T. Leung, A. Asiz, Y. Hei Chui, L. Hu, and M. Mohammad. Predicting lateral deflection and fundamental natural period of multi-storey wood frame buildings. In *Proceedings of World conference timber engineering*, Riva del Garda, Italy, 2010.
- [49] J. van de Lindt. Evolution of wood shear wall testing, modeling, and reliability analysis: Bibliography. *Practice Periodical on Structural Design and Construction*, 9(1):44–53, 2004.
- [50] J. W. van de Lindt, S. Pei, W. Pang, and S. M. H. Shirazi. Collapse testing and analysis of a light-frame wood garage wall. *Journal of Structural Engineering*, 138(4):492–501, 2012.
- [51] C. Loss, D. Zonta, and M. Piazza. Analytical model to evaluate the equivalent viscous damping of timber structures with dowel-type fastener connections. In *Proceedings of World conference timber engineering*, Auckland, New Zealand, 2012.
- [52] M. Piazza A. Polastri T. Sartori R. Tomasi M. Andreolli, D. Casagrande. Test sismici su tavola vibrante di un edificio a tre piani in legno a pannelli portanti intelaiati. *ANIDIS, Bari*, 2011.
- [53] A. Asiz M. Noory, I. Smith. Tests and numerical models for shear-walls with various layer. *WCTE conference*, 2010.
- [54] A. Crosatti M. Piazza, R. Tomasi. Indagine sperimentale su sistemi di connessione legno-legno con elementi a gambo cilindrico ed elementi di superficie. Technical



- report, Dipartimento di Ingegneria Meccanica e Strutturale Università di Trento, 2009.
- [55] Roberto Modena Maurizio Piazza, Roberto Tomasi. *Strutture in legno*. Hoepli, 2006.
- [56] K. McMullin and D. Merrick. Seismic performance of gypsum walls - experimental test program. Technical report, San Jose State University, 2001.
- [57] C. Ni and E. Karacabeyli. Effect of overturning restraint on performance of shear walls. In *Proceedings of World conference timber engineering*, Whistler, Canada, 2000.
- [58] NZS 3603. Timber Structures Standard, 1993. NZS, Wellington, New Zealand.
- [59] Alexander J. Salenikovich. *The racking performance of lightframe shear walls*. PhD thesis, Wood Science and Forest Products, Virginia Polytechnic Institute and State University, VA, 2000.
- [60] T. Sartori, R. Tomasi, A. Francescotti, and G. Scandolari. Edifici in legno realizzati in Abruzzo con il sistema a telaio prefabbricato. *Atti convegno Strategie di sviluppo sostenibile (Master MICA)*, 2010.
- [61] E. Kozem Silih and M. Premrov. Analysis of timber-framed wall elements with openings. *Construction and Building Materials*, 24(9):1656 – 1663, 2010.
- [62] R. L. Tuomi and W. J. McCutcheon. Racking strength of light-frame nailed walls. *Journal of the Structural Division*, 104:1131–1140, 1978.
- [63] UNI EN 26891:1991. Timber Structures - Joints Made With Mechanical Fasteners: General Principles for the Determination of Strength & Deformation, 1991. CEN, European Committee for Standardization, Brussel, Belgium.
- [64] Sara Zanolli. Analisi del sistema costruttivo a pareti lignee intelaiate: dettagli costruttivi, aspetti strutturali ed energetici. Master's thesis, Dipartimento di ingegneria meccanica e strutturale, Università degli studi di Trento, 2011.
- [65] *ETA-09/0323 GH Angle brachet 55-65-90 with and without rib*. 2009.
- [66] *ETA-09/0324 Angle Bracket 110095, Angle Bracket 110135 - Angle Bracket 110285 with and without elongated holes and GH brackets type KR-Holz*. 2009.

- 
- [67] *ETA-09/0322 GH Angle Brackets Type AB Top 80 - AB Top 120 - AB Top 80 Vario - AB Top 120 Vario - AB 1867 - AB 2197 - AB 110/170S - AB 50/595 - AB 80/598 and AB 110/5911.* 2009.
- [68] *UNI EN 12512:2003 Strutture di legno - Metodi di prova - Prove cicliche di giunti realizzati con elementi meccanici di collegamento.* 2003.
- [69] *UNI EN 26891:1991 Strutture di legno. Assemblaggi realizzati tramite elementi meccanici di collegamento. Principi generali per la determinazione delle caratteristiche di resistenza e deformabilità.* 1991.

