OPTIMISATION OF TIMBER FRAME CLOSED PANEL SYSTEMS FOR LOW ENERGY BUILDINGS

by

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To Nicolás and Sofía.

DECLARATION

This thesis is the result of my own work and includes nothing, which is the outcome of work done in collaboration except where specifically indicated in the text. It has not been previously submitted, in part or whole, to any university of institution for any degree, diploma, or other qualification.

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SYNOPSIS

Optimisation of Timber Frame Closed Panel Systems for Low Energy Buildings

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The United Kingdom published a legally binding document to reduce national greenhouse gas emissions by year 2020 up to 34% against the 1990 levels. This target also fulfils the Europe 2020 strategy of 20% carbon emission reductions by year 2020 (EC, 2010). Emissions due to space heating count for around 60% of the total domestic emissions (DCLG, 2012). The report "Rethinking Construction" published in 1998 emphasised the opportunities to improve the quality and efficiency of the UK construction sector (Egan, 1998). More recently, a framework has been published by the Government to tackle fuel poverty by building more energy efficient homes (DECC, 2015). In terms of energy performance, Passivhaus is recognised as one of the most energy efficient and researched construction standards which requires an exceptionally high-level of insulation and airtightness.

Closed-panel timber frames are a relatively new system in UK with an opportunity for growth. These advanced panels are pre-fitted in the factory, reducing the on-site work. However, closed-panel systems present a more complex sole plate fixing detail which can have an undesirable long-term impact on the structural and thermal performance of the building. The work presented in this thesis investigates the structural considerations, racking performance, of timber frame closed panel systems for future building regulations. The thesis underlines the significance of structural stability, serviceability and detailing in relationship with long-term thermal efficiency and airtightness, according to Passivhaus standard.

An experimental study was carried out to investigate the structural racking performance of advanced closed panel systems. A comparison was made between the behaviour of the timber frame panels and the analytical PD 6693-1. A set of different wall panel built-ups is presented for optimised Passivhaus design, including thermal bridge-free sole plate details. A timber frame racking software application was developed to optimise the structural design of shear walls. A parametric study was carried out with this tool to generate efficient timber frame wall design tables for different applied racking loads and U-values. The software application also allows for direct specification of robust sole plate base fixings and thermal bridge free details.

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SOFTWARE

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CONTENTS

List of Tables

LIST OF FIGURES

LIST OF ABBREVIATIONS AND ACRONYMS

AEC	Architecture, Engineering and Construction
BIM	Building Information Modelling
BRE	British Research Establishment
CAM	Computer Aided Manufacturing
CITB	Construction Industry Training Board
COCIS	Centre for Offsite Construction and Innovative Structures
CSA	Canadian Standard for Wood Design
CSH	Code for Sustainable Homes (Derogated)
DPC	Damp Proof Course
DER	Dwelling Emission Rate
EPDM	Ethylene Propylene Diene Monomer
EPS	Expanded Polystyrene
EPSRC	Engineering and Physical Science Research Council
ERDF	European Regional Development Fund
ETICS	External Thermal Insulation Composite System
EWP	Engineered Wood Products
FEA	Finite Element Analysis
FEM	Finite Element Modelling
FRP	Fibre Reinforced Polymer
GUI	Graphical User Interface
HAM	Heat, Air and Moisture
IFC	Industry Foundation Class
K2	Dual-frame closed timber frame panel
KTP	Knowledge Transfer Partnership
LCA	Life Cycle Assessment
LIM	Lowest Isopleth for Mould
LVL	Laminated Veneer Lumber
MC	Moisture Content

MIS	Minimum Income Standard
MMC	Modern Methods of Construction
MVHR	Mechanical Ventilation with Heat Recovery
MW	Mineral Wool
NCCI	Non-contradictory Complimentary Information Document
OSB	Orientated Strand Board
RAE	Royal Academy of Engineering
RIBA	Royal Institute of British Architects
RTC	I-Joist closed timber frame panel
SAP	Standard Assessment Procedure
SIPs	Structural Insulated Panels
SLS	Serviceability Limit State
TER	Target Emission Rate
TFCP	Timber Frame Closed Panel
TFS	Timber Frame Systems
TRADA	Timber Research and Development Association
UDL	Uniform Distributed Load
ULS	Ultimate Limit State
UK	United Kingdom
UKAS	United Kingdom Accreditation Service
VBA	Visual Basic for Applications
VCL	Vapour Control Layer
VMI	Viitanen Mould Index
WBP	Weather and Boil Proof
WF	Flexible Wood Fibre
WHE	Whole House Engineering
X-LAM	Cross Laminated Timber
XPS	Extruded Polystyrene

LIST OF APPENDICES

1 INTRODUCTION

In the United Kingdom, the whole housing sector accounts for around 30% of the total CO_2 national emissions (AEA, 2008) and space heating represents more than half of those housing emissions (DCLG, 2012). Recently, procedures and regulations have been released to set a global standard for low-energy construction. Passivhaus is one of these low-energy schemes which has been in the public domain for over twenty-five years. On average, a certified Passivhaus dwelling build in the UK saves almost 90% of the space heating demand if compared with a house built to 1990s building regulations (Schnieders & Hermelink, 2006).

This chapter introduces the concepts of closed timber frame panels, low energy building and affordable housing. This provides essential background to demonstrate the opportunity for off-site close timber frame panel as a preferred construction system for the residential market and for future building regulations.

The chapter also defines the scope of the thesis within this broad field of study. The research aims and objectives that were investigated throughout the total period of study are underlined. Lastly, at the end of the chapter, an outline of the content and structure of the thesis is described.

1.1 Background to the project

The United Kingdom, as member of the European Union which ratified in 2002 the Kyoto protocol (UNFCCC, 1997), committed to reduce by 12.5% the greenhouse gas levels of year 1990 by year 2012. Greenhouse gases are mostly carbon dioxide. Indeed, the actual global reduction of Kyoto greenhouse gases for the period 1990-2012 was of almost 25% but emissions from buildings increased in the same period (DECC, 2014). The UK carbon emissions reduction targets for the year 2020 and 2050 are 34% and 80% respectively. These reduction figures also fulfil the EU target of 20% carbon emissions reduction by year 2020 (EC, 2010).

Timber presents ideal properties to be manufactured offsite under lean manufacturing principles and with different levels of very high finishing detailing. In addition, timber panelised systems also benefit from excellent carbon footprint, low thermal conductivity, high strength-to-weight ratio and ease of construction.

As a result, closed timber frame panels, as a new modern form of engineering and construction, need to be considered in order to exploit commercially available forest resources in higher value-added end products for the construction industry and, in particular, for low-energy building technologies. Also, the prescription of bioconstruction materials can reduce the embodied carbon footprint associated with the building (Kemp, 2010).

1.1.1 Closed Timber Frame Panels

Timber Frame Closed Panel (TFCP) systems, when carefully detailed, are an example of Modern Methods of Construction (MMC) which can easily accommodate satisfactory levels of insulation between the studs without compromising weight, structural stability and cost.

Furthermore, the economic, industrial and social housing transformation occurred in the last decade, and triggered by the global UK housing demand, caused the development of new MMC. As a result, timber frame open panels are being replaced by TFCP systems. In addition, optimisation on labour cost, construction time and quality assurance is achieved by manufacturing these panels off-site under indoor controlled conditions.

Higher levels of quality and accuracy are required in the production of closed panel systems which must be supported by an internal cultural change of quality fabrication and reduced tolerances at all levels.

1.1.2 Market review and affordable housing

Recent previsions in housing requirements made by the Government Coalition only for England were around 232,000 homes per annum, including social, self-building and private market sectors (Holmans, 2013). This information was slightly over the forecast published by MTW Research (MTW, 2012). The figures in this report and the performance of the new UK housing market since 2006 are shown in Figure 1-1.

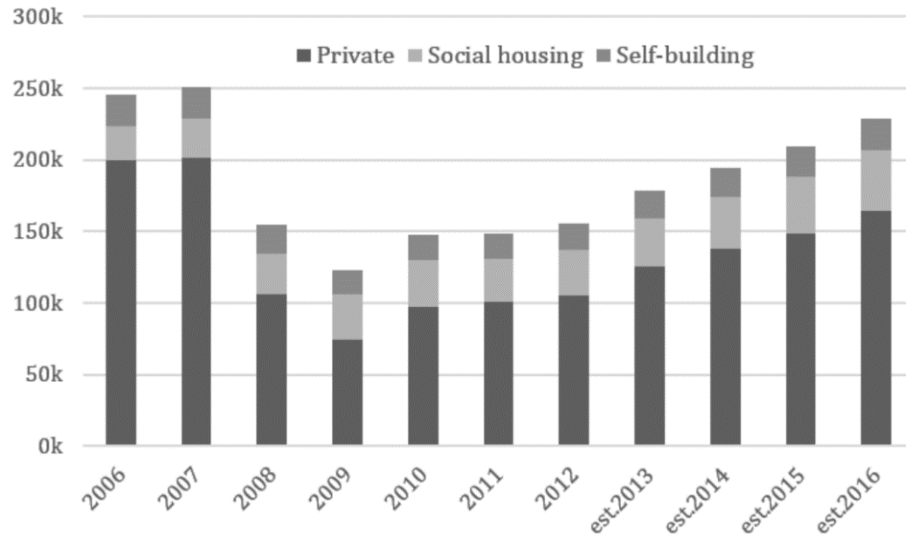


Figure 1-1 Number of dwellings built in UK since 2006

Currently, the existing maximum capacity within the industry is estimated to be around 150,000 homes. Furthermore, traditional building materials such as bricks and blocks and skilled labour are receding resulting in a short-term imbalance where supply does not meet housing demand.

These foreseen events will cause a change in the industry towards more productive manufacturing processes. Modern methods of construction and particularly off-site lean manufacturing processes have been highlighted as a viable solution to provide affordable housing (Hairstans, 2010).

The timber frame construction market includes a range of structural systems manufactured for both the residential and the commercial sector (Figure 1-2):

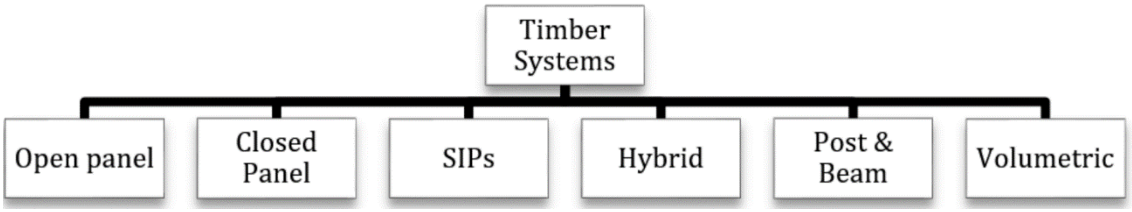


Figure 1-2 Structural timber frame systems

According to the MTW report (MTW, 2012), around 37,000 timber frame dwelling units were built in year 2012 for a total market value of just under £380 million. This represented a market share of about 25%. The value share within the different timber systems is shown in Table 1-1.

Table 1-1 Value share by timber system in 2006, 2012 and est. 2016 adapted from MTW (2012)

Timber system (%)	2006	2012	2016 (predicted)
Open panel	74	83	68
Closed panel	10	7	12
SIPs	8	3	12
Hybrid	3	5	2
Post & Beam	3	1	4
Volumetric	2	1	2
TOTAL sales (million £)	420	450	750

The drop and increase in hybrid and closed panel systems in recent years (2012-2016) may be explained by the difficult market conditions that have led to all sectors of the panellised systems including several business failures. However, the growth of panelised systems for the UK timber frame sector disagree with previous estimates (Vailikangas, 2002). In that research, the author predicted a market share for open panel, closed panel and SIPs of 65%, 25% and 7% respectively.

From the above information, it is concluded that market share of open panel systems will be decreased in the period 2016-2020 benefiting closed and SIPs panels. The likely reasons of this growth in closed and SIPs panels may be quicker erection schedules and better overall construction quality. Nevertheless, this market assessment does not include other potential offsite timber systems such as cross laminated timber (X-LAM) or Brettstapel (Dowel-lam, Nailed-lam).

1.2 Problem Overview

The dissemination of new structural design codes in Europe, including a set of ten Eurocode documents, has made the design process more transparent but also more onerous. As a result, it is frequent within the Architecture, Engineering and Construction (AEC) sector to prescribe overestimated structural elements for both the design and the materials utilised. This common practise is seconded by the lack of specialised affordable software for the highly disaggregated timber industry (Hairstans, 2010).

Apart from the recent upgrading on building regulations, the United Kingdom timber frame construction sector is also facing severe challenges due to the arrival of new Engineered Wood Products (EWP), the implementation of lean manufacturing techniques and the recent substitution of the British Standards BS 5268 in benefit of the Eurocode 5. Therefore, there is a clear need for a design software platform to produce transparent Eurocode 5 compliant and replicable reports which also enables for the inclusion of product specific mechanical properties.

A racking design software application, able to carry out accurate and quick structural calculations for timber frame walls, has been developed. This tool facilitates the optimisation of the structure by effectively examining the materials and the design employed. The application Tedds® and Tedds for Word (Tedds) from CSC (UK) Ltd. was selected as a programming platform due to its compatibility with Microsoft® Office, wide UK commercialisation and contrasted experience in structural design.

Existing information and literature on closed timber frame panels, even for those systems Passivhaus certified, does not include combined structural and thermal properties. Furthermore, there is a lack of information about the impact of inappropriate sole plate details for closed panel systems.

This research provides a series of structural-efficient closed timber frame wall panel designs for low energy buildings including various critical sole plate connection details and a comprehensive hygrothermal analyses of the solutions for future reference.

1.3 Scope of Thesis

In order to successfully propose a series of closed timber frame wall panel systems for low energy buildings, limitations were put on the scope of the research. Although various thermal analyses were carried out for several European climates, this research focuses on the UK context only.

The Passivhaus standard (Feist, 1993) was set as the benchmark for low energy building consideration. Passivhaus is a proven construction methodology in compliance with the European Directive 2010/31/EU, Energy Performance of Buildings (EPBD), where all new buildings are required to be Nearly Zero Energy (NZEB) by the end of 2020 (EC, 2010).

The thermal performance investigations were conducted for two proposed closed timber frame panel configurations and for the sole plate base connections between the timber frame and the foundation. Furthermore, two common foundation types were considered: slab on grade and suspended timber floor system. No further research was undertaken for other timber frame details.

The research addresses low-rise and low-energy building typologies only where lateral stability governs the structural design. This is found in energy efficient timber frame buildings where the frame dimensions are considerable wider to accommodate thicker insulation batts or rolls and therefore no axial or bending failure occur. On the contrary, large timber frame imperforations due to large windows, especially in south orientations, can frequently cause racking and other instability issues.

Platform frame systems enable for a higher degree of prefabrication and standardisation which is in agreement with the scope of the research. Hence, timber frame racking performance was the only parameter investigated for structural optimisation purposes and under static lateral loading only. However, this parameter also indirectly included provisions for robust closed panel sole plate fixing specification.

Research efforts were not given to other important aspects of building performance such as acoustic or fire performance nor other environmental aspects like the building life cycle assessment (LCA) or the green credentials of the insulation materials.

1.4 Aims and Objectives

The main aim of this thesis was to develop two optimised closed timber frame wall panels for high thermal performance. A review of the different timber frame systems provided a gap knowledge refer to the relationship between thermal and structural performance for closed panel systems.

The methodology used to achieve this aim was to analytically determine adequate insulated timber frame wall build-ups for Passivhaus certification with satisfactory levels of structural performance. A limitation on wall panel deformation of 10 mm was also considered in order to provide long-lasting airtight construction details. Within this aim, a particular importance was given to the relationship between structural and thermal performance of the closed panels and with special focus on the sole plate base fixing detail.

A secondary aim was the development of a racking software application to provide structural engineers with a platform for flexible design and closed timber frame optimisation. A direct outcome of the development of the software application was the optimisation of timber frame wall designs due to the parametric analyses undertaken directly by the tool. This also facilitated the delivery of a set of robust details for low energy buildings and Passivhaus design.

These two aims were achieved by completing the following core research objectives as follows:

i. To carry out a data gathering of timber frame shear walls and sole plate connection tests from open timber frame panels.

ii. To propose two different closed panel timber frame configurations suitable for low energy building design.

iii. To investigate the hygro-thermal performance of these closed panel systems based on different materials and sole plate details.

iv. To carry out 2-D thermal Finite Element Analysis (FEA) of different sole plate fixing details for thermal optimisation and future reference.

v. To investigate the impact of current timber fraction calculations on the overall thermal performance of Passivhaus timber frame buildings.

vi. To develop a simplified theory for the analytical optimisation of closed panel timber frame sole plate details.

vii. To compare the analytical and experimental racking results of the two proposed closed panel systems under partially and fully restrained sole plate base fixing conditions.

viii. To develop a Eurocode-compliant software application to optimise the structural performance of timber frame shear walls by enabling flexible design and by integrating specific material data from test results.

ix. To validate this software application by comparing the analytical output obtained with the results achieved from other calculation tools under the same analytical methodology.

x. To perform a parametric analysis for shear wall optimisation.

xi. To integrate the output of the optimised racking walls with the results from the thermal analyses and providing technical data-sheets for direct Passivhaus timber frame wall specification.

1.5 Thesis Outline

This thesis is comprised of six chapters and ten appendices. A high-level literature review including housing construction industry in UK, Modern Methods of Construction, closed panel timber frame walls, and thermal and structural requirements for affordable low-energy buildings is presented in Chapter 2.

A study of the thermal performance for the proposed closed panel timber frame wall systems is discussed in Chapter 3. This investigation includes a specific literature review on the subject, the complete description of the two timber frame configurations, an investigation on FEA software for linear and point thermal bridge simulation and a study on the hygrothermal performance of the wall assemblies on different climates.

The structural performance optimisation of the closed timber frame panels is detailed in Chapter 4. The research presents a literature review on timber frame shear walls, investigations on different timber frame materials, an explanation of the research methodology containing analytical and experimental research and the comparison between the tests undertaken with the analytical method provided by PD 6693-1.

Chapter 5 describes the methodology and development of the Trimble Tekla Tedds Timber Frame Racking Design Application. This validated software provides to structural engineers with a transparent and flexible timber frame racking design being the calculation run more than 2,000 times a year.

The outcomes and conclusions related to the research carried out in the previous chapters are summarised. These conclusions set the basis for a simplified model containing relevant structural and thermal information. Lastly, two simplified models containing optimised details for the proposed closed timber frame panel systems are provided. Potential areas for future work and further research are also reported in this chapter.

2 REQUIREMENTS FOR FUTURE AFFORDABLE HOUSING

This chapter is a summary and a critical discussion of the research and background information related to low-energy timber buildings. The first three sections set the framework to the research, present a historical review of different building regulations and introduces modern methods of timber construction (MMC). This first part justifies the research work carried out on advanced timber frame closed panel as a potential mainstream construction system for low-energy dwellings.

The last three sections of the chapter provide a high-level literature review as an outline to the three different main fields of work considered in this research: the thermal performance of energy efficient timber frame buildings, the racking optimisation of shear walls and the use of integrated software for timber design.

An extensive literature review of these key research areas is later included into each of the relevant chapters. The organisation of this chapter is shown in Figure 2-1.

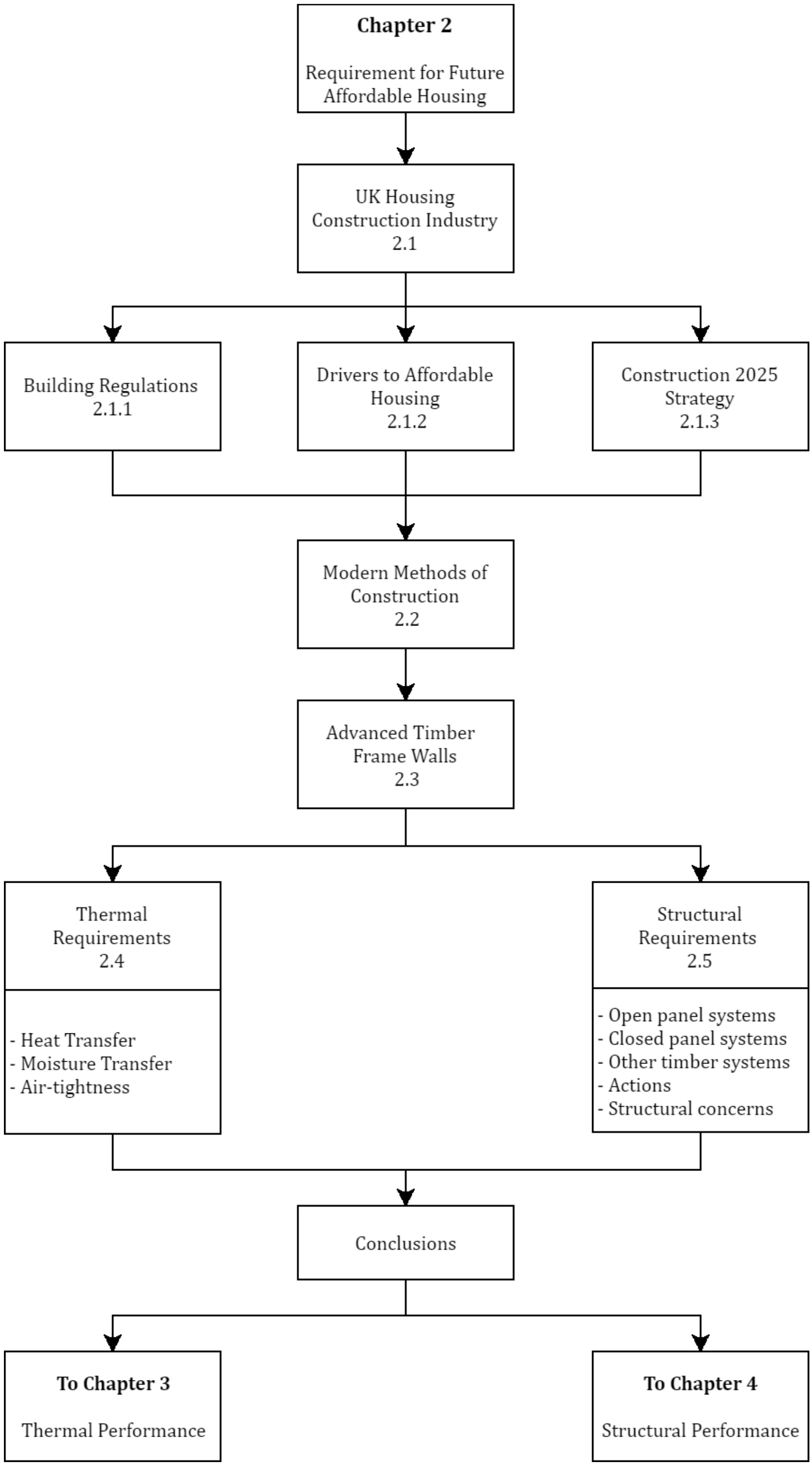


Figure 2-1 Organisation of Chapter 2

2.1 The Housing Construction Industry in UK

A background to the UK construction industry relevant to the housing sector is included in this section. The review contains past, present and future UK building regulations, other voluntary construction standards, the drivers for affordable housing and a summary of the Construction 2025 strategy published by the UK Government.

2.1.1 Building regulations

In Great Britain, the first attempt to establish a unified building regulations was in 1936, when an optional guidance on the control of construction and building conditions was introduced (Britain, 1936). About thirty years later, the first mandatory Building Standards (1963) for England and Wales were published.

Building regulations have been amended to progressively minimise primary energy consumption and hence, to mitigate carbon dioxide emissions. Table 2-1 summarises the changes in the thermal properties of the building fabric for new homes over the last 50 years according to the Scottish Building Standards and the England and Wales Building Regulations.

Table 2-1 Historical review of building fabric requirements for Scotland and England & Wales Building Regulations

		Scotland				England and Wales			
		1965	**1981**	**1995**	**2010**	**1963**	**1983**	**1997**	**2010**
U-value	wall	1.70	0.70	0.47	0.30	1.70	0.60	0.45	0.25
(W/m^2K)	roof	1.50	0.40	0.25	0.20	1.50	0.40	0.35	0.18
	floor	-	-	0.45	0.25	-	-	0.45	0.20
	windows	4.80	4.80	3.30	2.00	4.80	4.80	3.10	1.80
Airtightness	q_{50}	-	-	10	10	-	-	10	10
Glazing area	%	12	12	22.5	25	12	15	25	25
		wall	wall	floor	floor	wall	wall	floor	floor

In addition, energy performance has increasingly been emphasised in recent revisions to the Building Regulations. Minimum thermal properties for the building fabric are clearly defined in both documents. The following parameters have been repeatedly amended since then:

- Thermal transmittance U_{value} in W/m^2K
- Airtightness Q_{50} in $m^3\,m^{-2}\,h^{-1}$

Concurrently, allowance for minimum glazing area has also been modified. This percentage of glazing area is now related to the total floor area of the building instead of the total wall area as stated in earlier regulations. A historical review of minimum requirements is also described in Table 2-1.

Several contributors have been targeted for the implementation of governmental policies and measures: energy supply, land and waste management, industrial processes, transport system and housing. The UK residential sector counts for around 15% of the total national carbon emissions. Various energy-related schemes were published across the UK with more or less success, such as the Code for Sustainable Homes (DCLG, 2010). This voluntary environmental assessment method, revised on May 2014, aimed for progressive reductions in the Dwellings Emission Rate (DER) over the Target Emission Rate (TER) as shown in Table 2-2. However, this code was finally withdrawn on March 2015.

Table 2-2 Regulatory stages to zero carbon and CSH levels (derogated)

Code level	% improvement over TER	Implementation
*	10%	
**	18%	
***	25%	2010
****	44%	2013
*****	100%	
******	´zero carbon home´	2016

A closer look to the known as “Fabric First approach” highlights the relevance of the thermal envelope (Taylor et al., 2012). This is corresponded by the substantial change on the minimum U-Value requirements released in the latest England and Wales Building Regulations update (Table 2-3).

Table 2-3 Minimum U-Value requirements for England & Wales Building Regulations (DCLG, 2012)

	U-value (W/m²K)	
	2010	**2013**
roof	0.25	0.13
wall	0.18	0.18
floor	0.20	0.13
window	1.80	1.40

2.1.2 Other international standards

Voluntary low energy building standards have been published around the world for the past 30 years. The standard R-2000 was introduced in 1982 to improve the energy efficiency of new built Canadian homes, as a consequence of the drastic increase of oil prices. This governmental program was initially based in technical guidelines for design, modern technology, good practice and materials. Alongside, the Government of Canada launched a scheme for training builders in low energy construction. Since then, thousands of homes have been built to this standard. Countries such as US, Russia, Germany, Poland or Japan have homes built to R-2000.

In 1988, Prof. Wolfgang Feist and Prof. Bo Adamson came upon the concept of high energy efficiency homes based on building physics. This idea was developed into the Passivhaus (Feist, 1993). The Passive House Institut and the first building, four terraced homes, were established in 1990 in Darmstadt, Germany (Figure 2-2). Only four requirements are needed to achieve this standard:

- Space heating or cooling demand less than 15 kWh/m^2 per year, or
- Space heating or cooling load less than 10 W/m^2
- Primary energy demand less than 120 kWh/m^2 per year
- Building airtightness less than 0.6 air changes per hour (ach) at 50 Pa

Another globally recognised low energy standard is the Swiss Minergie-P® which is a more stringent version of the Minergie baseline standard introduced in 1998 (Minergie, 2008). This registered trademark follows similar principles as the Passivhaus standard, but also it allows for water-based heating and cooling systems for even and efficient air distribution.

Figure 2-2 First Passivhaus building (Darmstadt, Germany)

The Minergie standard limits the space heating or cooling demand to the nature of the building and it varies from 45 kWh/m^2 per year for hospitals to 15 kWh/m^2 per year for warehouses. The standard is popular in Switzerland where more than 18,000 homes have been built to Minergie standards, a thousand of them to Minergie-P. Other Minergie-P projects have been built in Abu Dhabi, France and Japan. The energy requirements of these standards and for new dwellings are summarised in Table 2-4.

Table 2-4 Energy requirements for new homes according to various standards

	E&W (2013)	R-2000	Passivhaus	Minergie-P
Space heating (kWh/m^2)	39/46	40	15	15
Thermal bridges (W/mK)	0.02	minimal	< 0.01	minimal
Airtightness (n50)	6.0$^{(1)}$	1.5	0.6	0.6

(1) For E&W airtightness is measured as q50 instead.

The Passivhaus standard criteria has been adopted as a compulsory building regulation in several European local authorities (Table 2-5) to fulfil the EU future energy efficient requirements. Also, several studies have validated the principles of the standard in other climates (Krainer, 2008 Schnieders & Hermelink, 2006). From the internal Passivhaus database, more than 50,000 dwellings built worldwide are recorded according to these principles. In the UK, McLeod et al. (2012) proposed a method to generate reliable climate data sets for future climate predictions. More recently, the Royal Institute of British Architects (RIBA) has referred to Passivhaus in the UK as an *"emerging popular low-energy standard for housing"* (RIBA, 2015). As a result of this worldwide diffusion, this study follows the Passivhaus criteria as a thermal performance benchmark criterion throughout the research.

Table 2-5 Councils or regions where Passivhaus has been implemented

Council	Country	Year	Type
Vorarlberg	Austria	2007	Public
Wels	Austria	2008	All
Antwerp	Belgium	2013	All
Brussels	Belgium	2014	All
Bremen	Germany	2011	Public
Hamburg	Germany	2012	Public
Luxemburg	Luxemburg	2016	All
Oslo	Norway	2014	Public
Villamediana	Spain	2013	Public
Dún Laoghaire	Ireland	2016	All

2.1.3 Drivers to affordable housing

For the last decades and under successive UK governments, the construction industry have failed to provide a housing supply that matched the demand required (Holmans, 2013). The direct consequence of this housing imbalance has been a continuous growing price for home ownership and rising rents.

In the meantime, the real wages of the average worker have suffered a stagnation for over a decade. During the period 2003-2014, the accumulated average hourly salary rose a 29 % whilst in the same eleven years period, the accumulated inflation rose by 32.5% (Bell, 2015).

The combination of these two aspects housing shortage and the decrease of potential home owner´s purchasing power, lead the drivers to affordable housing. Another financial challenge related to housing is fuel poverty which was defined by the Scottish Government in the Housing (Scotland) Act 2001 as:

"...A household is in fuel poverty if, in order to maintain a satisfactory heating regime, it would be required to spend more than 10% of its income (including Housing Benefit or Income Support for Mortgage Interest) on all household fuel use".

Statistically and in general terms, a household is considered to be in fuel poverty when the total fuel cost is over 10% of the household income. A study on fuel poverty in England (Moore, 2012) published the reduction in minimum income standards (MIS) that a British household had to consider to fully afford the household fuel costs. Taking into account a maximum fuel cost of 10 % of the household´s income, in England there are almost 5.5 million homes in fuel poverty (Table 2-6).

Table 2-6 Households in fuel poverty, England after (Moore, 2012)

Reduction in MIS cost	Households	
	Thousand	Per cent
No reduction required	15,943	74.5
Up to 10%	874	4.1
10 to 20%	870	4.1
20 to 30%	719	3.4
30 to 40%	762	3.6
40 to 50%	741	3.5
More than 50%	1,498	7.0
Total in fuel poverty	5,464	25.5

Another well-defined set of drivers with an emphasis on skilled workforce, efficient and technologically advanced construction industry to exploit the UK construction sector in a global market was published by the UK government in the Construction 2025 report (DBIS, 2013). The clear strategy of this document is to provide a vision of where the UK construction sector will be in year 2025, and which is in line with the research carried out in this thesis, is summarised in the next section.

2.1.4 Construction industry strategy 2025

This document published in year 2013 defines a strategy for collaboration and partnership between the UK government and the industry to provide the basis to exploit its strengths in the global market which is forecast to grow by over 70% by year 2025.

The strategy also estimates four clear figures of the improvement in the industry to be reached by 2025 which are visually presented by Figure 2-3.

Figure 2-3 Summary of industry targets set by Construction 2025 (DBIS, 2013).

Although the publication includes the UK construction industry as a whole, there are strong references to the housing sector as a key market also to support other industries. The document forecasts from 1.7 to 2.5 million new homes by year 2025 which translates to 140-200k homes being built every year. Furthermore, the strategy also identifies the existing housing stock as the biggest potential opportunity where a large retrofit programme will be supported.

The Construction 2025 vision includes five key areas to focus:

- People: talented and diverse skilled workforce
- Efficient industry: productive and technologically advanced
- Sustainable: low-carbon and green construction materials
- Growth: achieving growth across the entire UK economy
- Leadership: reputation of UK construction in the global market

Also, to deliver the industry targets illustrated by Figure 2-3 and in addition to these five key areas for improvement, six drivers of change were required:

- Overall image improvement of the industry
- Increased skills of the workforce
- Understanding the future work opportunities
- Improvement in procurement processes and client satisfaction
- Strong and resilient supply chain
- Effective research and innovation

Outcomes from this thesis contribute to improve key areas and drivers required by this Construction 2025 strategy. In order to facilitate potential improvement in exports by the UK timber frame industry, the Chapter 3 of this research thermal performance of timber frame walls, also considers other different climate areas.

The current and future building regulations reviewed in this section together with the drivers to mitigate the issues of the current housing scenario and with the clear future vision defined in the Construction 2025 strategy must be considered in the fast development of modern methods of construction.

2.2 Modern Methods of Construction

UK government has encouraged the use of Modern Methods of Construction (MMC) to produce a higher quantity and quality of houses (Egan, 1998) with a clear strategy for efficiency and elimination of waste (Office, 2011). The starting point of the UK commission was to tackle the degenerative cycle (Figure 2-4) in the construction industry caused by poor quality products resulting in inefficient processes with low financial profitability and subsequent absence of investment in research and development (Hairstans, 2010)

Additionally, a declining availability of skilled workforce such as carpenters or bricklayers has already been identified by several authors (Johnsson & Meiling, 2009 Kemp, 2010; Green et al., 2014). A recent set of surveys published by the Construction Industry Training Board (CITB) and shown in Table 2-7 identified general labourers, bricklayers and carpenters as the harder-to-fill occupations (CITB, 2011, 2014).

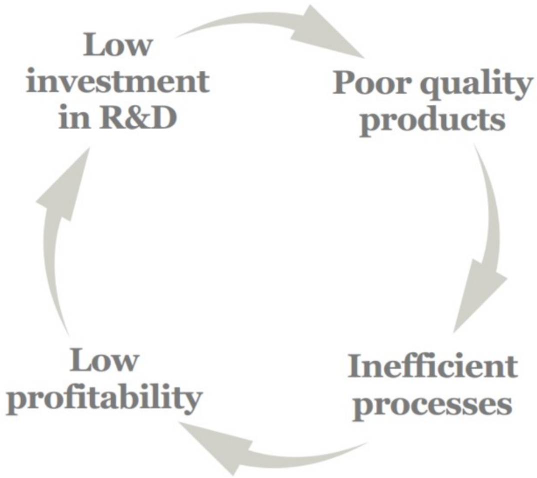


Figure 2-4 Degenerative cycle, adapted from Hairstans (2010)

Although MMC often requires a specialised workforce, the significant shift of building skills from on-site to off-site, the increase in factory mechanisation and the standardisation of internal processes could gear traditional on-site demands to a more multi-skilled workforce.

Table 2-7 Occupations with Hard-to-Fill vacancies in 2009, 2011 and 2014.

Occupation	2014 n= 181	2011 n= 56	2009 n = 70
General labourers	18%	12%	17%
Carpenters	14%	15%	19%
Bricklayers	9%	19%	6%
Technical staff	9%	8%	-
Plasterers	8%	15%	4%
Scaffolders	8%	1%	4%
Plumbers	8%	4%	-
Roofers	7%	1%	-
Machine operatives	7%	3%	15%
Electricians	2%	15%	6%
Others	10%	7%	29%

MMC can be defined as a construction methodology which provides an efficient management process to produce more building products of better quality in less time (Kozlowska et al., 2015). These processes can be made on-site or off-site but the general concept is to use components and assemblies that are faster to install in order to increase the rate of quality and affordable housing supply. The direct consequences of any type of MMC are:

- Reduced waste, skilled labour and reworking on-site
- Greater speed of construction
- Improved logistics and material handling
- Production of quality and affordable buildings

The Office of the Deputy Prime Minister and the Housing Corporation were investing over a £1 billion in 2004-05 on facilitating affordable housing with an implicit focus on four different technologies as shown in Figure 2-5 (Hairstans, 2010).

Panelised

Volumetric

Hybrid solutions

Novel products

Figure 2-5 Different technologies of MMC

- Panelised units fabricated off-site and assemble on-site to produce a volumetric structure. The process of manufacturing the frame may gradually integrate insulation, services, windows and doors or internal and external finishes to produce advanced panelised systems.
- Complete volumetric construction as three-dimensional unit modules produces in factory. In this case, transportation limitations may constraint architectural and other design features.

- Hybrid technologies combining both panelised and volumetric units. These technologies may be of particular interest with volumetric modules for highly serviced rooms such as mechanical rooms or bathrooms whilst the rest of the building are made of 2-D panels.
- Other construction technologies. This may include novel product developments such as floor and roof cassettes, pre-cast foundation solutions, other foam products for in-situ wall formation or thin-joint block work.

2.3 Closed Panel Timber Frame Walls

Timber frame systems, in line with the original definition of MMC, is a potential solution due to its environmental credentials, mechanical properties and light weight (DTI, 2004). Additionally, the environmental performance of wood is a major driver for the use of timber in buildings (Wang et al., 2014).

Considering a number of thermal features such as an adequate insulation or airtight assemblies, offsite timber frame advanced closed panel systems can be considered as a cost-effective MMC for low energy buildings (Hairstans, 2010) as successfully implemented in other countries (Fossdal & Edvardsen, 1995). Although the level of confidence in using prefabricated timber frame systems was reduced in the past (WRAP, 2007), there is a current increase in confidence in off-site timber construction (Hausammann and Franke, 2014)

Timber frame buildings are generally formed by roof, floor and wall systems. Nevertheless, floor systems, except for suspended ground floors, have a minimal contact with the outdoor environment i.e. are not a part of the thermal envelope on its integrity. As a result, the influence of floor systems on low-energy buildings is minimal. For that reason, floors are not investigated in this research.

On the other hand, roof systems are particularly exposed to outside conditions and must fulfil structural, thermal and acoustic requirements. Also, other aspects such as architectural design, local planning permission and building regulations and even the integration with the context and form of the building, result in roof systems to be beyond the scope of this study. Nevertheless, part of the outcomes of this research may be applied to roof systems.

Finally, external wall systems and integrated windows account for almost half of the energy losses in a dwelling (Figure 2-6).

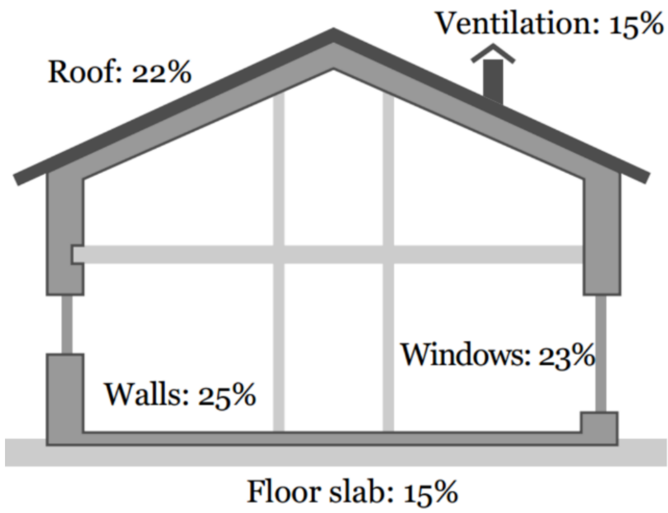


Figure 2-6 Sources of heat loss in a dwelling

Additionally, external walls require sufficient load bearing capacity to withstand gravity and lateral actions. The improvement of standard open panel timber frame systems into advanced designs for structural and thermal performance, while satisfying future building regulations, provides a significant practical challenge to achieve integrated and innovative building design in a cost-effective manner.

One of the outcomes of this research is to provide standardised and modular details that can be incorporated in different architectural designs for a range of applications, under structural and thermal requirements and for the UK context. However, another two different climate regions were considered solely for condensation-risk calculations and presented as a discussion for future work.

2.4 Thermal Requirements for Low-energy Buildings

One of the main aspects of energy efficiency in residential and commercial buildings is based on the reduction of the heating energy demand (Rosenfeld, 1999). The "Fabric First" approach focuses on the external building envelope as a main and most economical driver to reduce heat loss. The results of a recent survey based on information provided by 250 UK architectural practises highlighted that 70% of them forecast for year 2016 a significate increase in advanced insulation products on their projects (RIBA, 2015). There is a clear evidence that current building regulations are integrating improved energy efficient measures into the architectural design.

Another recent report (Lepadatu, Bliuc, & Baran, 2010), published by the Royal Academy of Engineering (RAE), emphasised heat flow through the fabric, control of moisture, air movement, sun, light, acoustics, climate and biology as principal aspects for low carbon design and recommended further research in this topic. For the purpose of this research on timber frame walls, heat flow, moisture as heat transfer by water and airflow through the building fabric are considered as thermal performance variables.

2.4.1 Fundamentals of heat transfer

The first law of thermodynamics (conservation of energy) is the principle for heat transfer processes. In a closed system, the total energy in equilibrium, for an irreversible, reversible or quasistatic process, is the sum of the heat and work added to that system (Warner & Arpaci, 1968):

$$\Delta E^{tot} = Q + W \tag{2.1}$$

Where:

ΔE^{tot}	is the change in internal energy
Q	is the heat added to the system
W	is the work generated by the system

The process of heat transfer can be defined as thermal energy in transit due to a physical temperature difference. This thermal energy in transit can occur in three different heat transfer modes (Figure 2-7) in a stationary medium generating a temperature gradient (conduction), between a surface and a moving fluid (convection) or as a consequence of electromagnetic waves containing energy generated by two surfaces at different temperature (radiation).

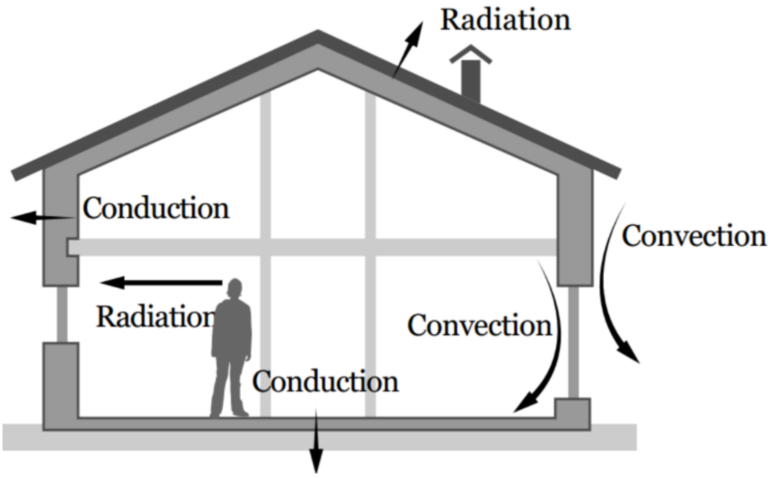


Figure 2-7 Heat transfer modes in a dwelling

The most relevant heat transfer mode associated with Passivhaus buildings is the conduction mode. Heat transfer by convection is rather limited due to the insignificant temperature gradient difference inherent to this construction standard and the absence of interstitial or surface condensation (Schnieders & Hermelink, 2006). The Passivhaus standard includes a soft-criteria where temperature difference shall be lower than 2 ºC and where surface radiant temperatures shall be lower than 4.2 ºC in order to eliminate internal convection (Feist, 1996).

Conduction is a transfer of energy resulting of the interaction from the more excited to the less excited particles that constitute matter like an object or a wall. As higher temperatures present more energetic molecules, the transfer of energy occurs in the direction of decreasing temperatures i.e. from the warm side to the cold side of an external wall. The equation for the heat transfer by conduction (*Fourier´s law*) is given as the amount of energy transferred by unit of time and for a one-dimensional plane is presented in Equation 2.2

$$\ddot{q}_x = -k\,\frac{dT}{dx} \tag{2.2}$$

Where:

$\ddot{q}_x$ is the heat flux in W/m^2

k is the thermal conductivity in W/m K

$\frac{dT}{dx}$ is the temperature gradient

The minus sign is the mathematical representation of the heat transfer direction of decreasing temperature. In case of steady-state conditions where there is a linear temperature distribution, the gradient may be:

$$\frac{dT}{dx} = \frac{T_2 - T_1}{L} \tag{2.3}$$

As expressed in Fourier´s law (Equation 2.2), the thermal conductivity in a given direction-*x*, can also be described as:

$$k_x = -\ddot{q}_x \frac{dx}{dT} \tag{2.4}$$

Thus, for a defined temperature gradient, the conduction heat flux increases if the thermal conductivity also increases. Furthermore, for an isotropic material it is assumed to be:

$$k = k_x = k_y = k_z \tag{2.5}$$

A summary of the thermal conductivities, *k*, for various materials is provided in Figure 2-8.

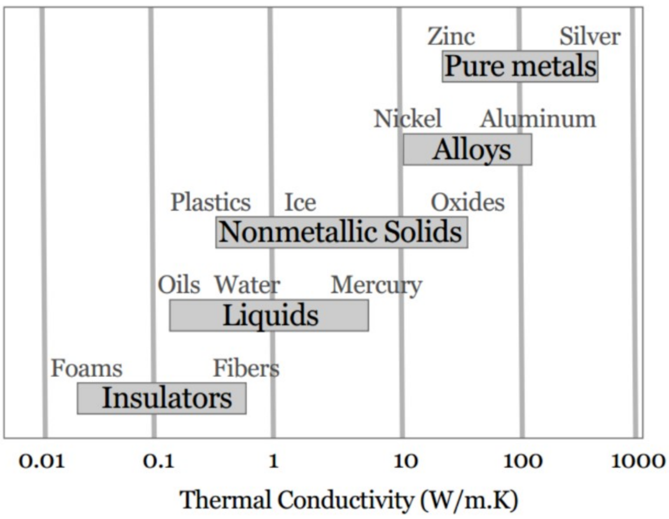
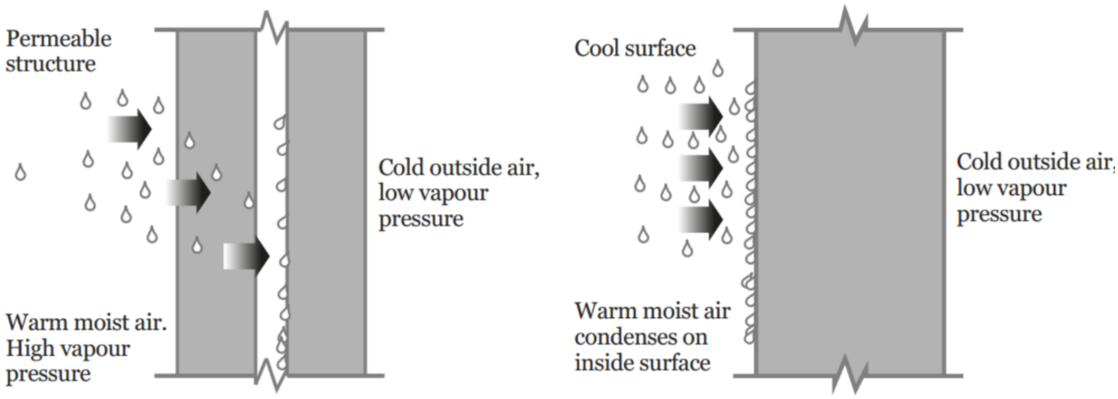


Figure 2-8 Thermal conductivity of different materials (Bergman et al., 2011)

2.4.2 Moisture transfer processes

One of the main causes of deterioration for timber frame buildings is the continuing presence of moisture on the building envelope which can even cause the collapse of the structure (Meklin et al., 2003). An excessive condensation within the timber frame can corrode metalwork, trigger the growth of bacteria and mould and increase the risk of biological attacks by xylophagous (Kalamees & Vinha, 2003 Lamoulie et al., 2012). Moisture within a building can occur as interstitial or surface condensation (Figure 2-9).



a) Interstitial condensation b) Surface condensation

Figure 2-9 Interstitial and surface condensation (TRADA, 2012)

Interstitial condensation occurs when vapour comes to contact with colder temperature conditions and within the structure reaching the dew point. This is particularly dangerous as it may cause wood decay within the timber structure if the water deposition is persistent and moisture content of the timber is greater than 20% for long periods of time (Dinwoodie, 2000). The presence of water also may corrode metal work within the structure. Lingering interstitial condensation also reduces the conductivity of most of the insulation materials causing dew point to appear in an earlier point within the structure.

Alternatively, surface condensation occurs when internal warm and moist air contacts a cold inside surface that are at or below the dew point of that internal air. Usually, surface condensation appears like a patch or damp stain on internal surfaces. This can cause mould growth and material decay. For example, in order to avoid condensation on windows, the surface temperature must be greater than 12.6 ºC (Feist, 1993).

Managing moisture transfer

In terms of building physics, accumulation and transport of moisture for one and two-dimensional building assemblies occur in four different ways (Bergman et al., 2011):

- Condensation as a function of the variable humidity of heat transferred by thermal conduction through different materials.
- Transfer of latent heat caused by the different vapour diffusion of the wall´s build-up materials.
- Accumulation of moisture due to the different vapour sorption curves of the timber frame materials.
- Capillary moisture as a result of horizontal vapour surface diffusion or capillary vertical conduction.

Condensation risk analysis

One and two dimensional hygrothermal numerical models are widely used to accurately assess the moisture behaviour and condensation risk of different wall types and materials even when human interaction is considered (Kalamees & Vinha, 2003).

In order to minimise condensation risk in a timber frame wall, for a constant indoor and outdoor climate, the design of the wall needs to be analysed and optimised. The different parameters affecting the moisture transfer ratio within a construction system are detailed in Table 2-8.

Table 2-8 Factors that affect moisture transfer in timber frame walls

Climate	Components	Properties
Internal temperature	Sequence of materials	Vapour permeability
Internal RH	VCL membrane	Thickness
External air temperature	Wind membrane	Material density
External RH	Thermal bridges	Conductivity
	Ventilation	
	Insulation	

For the research presented in this thesis, the following parameters were investigated:

1. Position and material of the sheathing panel,
2. thickness and material of the insulation between studs,
3. overall vapour permeability of the timber frame wall construction.

In terms of condensation risk assessment, some authors defined various levels of risk depending on building strategies such as moisture vapour buffering or mechanical air renovation rates (Liuzzi et al., 2013; Rosenau, 2009) and even when human interaction is considered (Lamoulie et al., 2012; Ferroukhi et al., 2014). However, in this thesis as energy efficient Passivhaus buildings must be provided with mechanical ventilation which is able to compensate for induced moisture, human interaction on the building is not considered therefore only a “No risk” condensation level is accepted in a design stage. For this condensation level definition, a clear identification of structural and non-structural material is provided:

- The overall humidity of the wall does not increase overtime.
- Non-structural materials are kept below 23% relative humidity at all times.
- Structural materials are kept below 20% relative humidity at all times.
- Surface materials are kept below 20% relative humidity to avoid mould growth.

2.4.3 Air permeability of buildings

Air through the building envelope or through its components can leak from inside to outside (exfiltration) or more commonly from outside to inside (infiltration). This air leakage can be caused by:

- Gaps in any construction joint.
- Cracks around windows, doors, internal or external finishes.
- Construction porosity like brick, blocks or permeable wooden panels.
- Service penetrations like pipes, flues, ducts or wires.

The first three points can be directly influenced by the construction systems and materials used in the building and by the assembly of these different components as part of the construction assembly (Erhorn & Lahmidi, 2009; Sandberg & Sikande, 2005). In addition, the air flow due to air infiltration creates a heat loss ventilation path which reduces the transmittance value of the construction assemblies. Also, the mechanical ventilation system may not operate as designed due to this uncontrollable infiltration causing an increase in the heating demand of the building.

The air permeability is a measured variable of the building fabric performance that takes into account the total air leakage through the envelope. This permeability, in $m^3/(m^2\ h)$, is commonly used to determine the airtightness of the building under a reference pressure differential. Although this pressure can vary, airtightness value, for most of the standards, are related to an applied differential pressure across the building of 50 Pa (Pan, 2010; Sinnott & Dyer, 2012).

In the UK, the test procedure is defined by BS EN ISO 9972:2015 which has recently superseded BS EN 13829:2011. This methodology introduces a fan and a pressure-measuring device to determine the airflow passing through a given area (Figure 2-10).

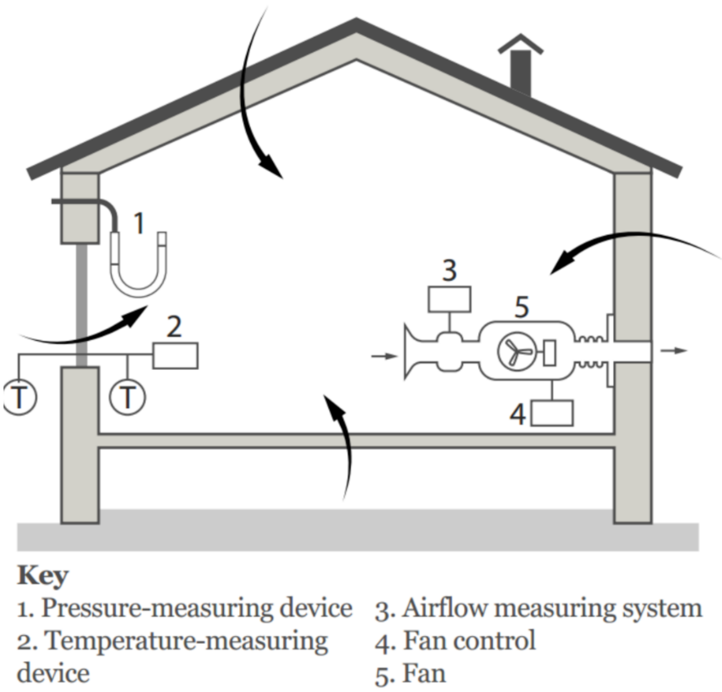


Figure 2-10 Schematic layout of air permeability apparatus

The most common apparatus is a blower door assembly with a variable speed motor to control air flow rates (Equation 2.6).

$$\boldsymbol{Q} = \boldsymbol{C}\,(\Delta \boldsymbol{P})^{\boldsymbol{n}} \tag{2.6}$$

Where:

Q	measured air flow in m^3 / h
C and n	are constants related to tested building
ΔP	is pressure difference

Abundant studies can be found with air leakage test measurements for a variety of buildings (Fraisse et al., 2006; Stephen, 1998; Walther & Rosenthal, 2009). It is apparent from these studies that UK airtightness results are influenced by construction type, dwelling type, year of construction, type of house ventilation and on-site construction quality as suggested also by Korpi et al. (2004).

In terms of a potential change of building airtightness during its service life, different studies concluded that after a period of time homes can either become leakier and less tight or may not change significantly (Antretter et al., 2007; Phillips et al., 1993). However, there are no evidence that any of these construction systems tested were closed timber frame panels.

The required airtightness of the building envelope is achieved by installing a continuous layer made of materials with an adequate permeability such as cross-laminated timber, wet plaster, in-situ concrete, wood based boards or building membranes (Bastian, 2014).

An adequate air permeability K_a is limited between 0.001 and 0.002 $m^3\ m^{-2}\ h^{-1}Pa^{-1}$ which has been suggested for a material to be considered as airtight layer on Passivhaus construction. These recommended values corresponds approximately with an equivalent air layer thickness S_d of 0.5 - 1.0 m respectively (Langmans et al., 2010).

Construction gaps can be avoided by, for example, detailing tight interlocking connections with minimal tolerances between components and with a good implementation of these details both in factory and on-site. On the other hand, the appearance of cracks around openings and other high-stress concentration areas are directly related to the serviceability performance of the structural system. This thesis considers these three factors in order to provide a robust construction detail for resilient low-energy timber frame buildings.

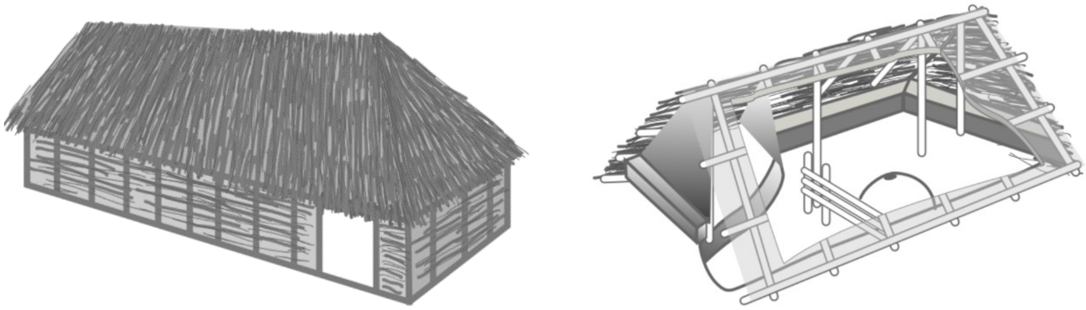
2.5 Structural Requirements for Low-energy Buildings

The structural design of low-energy and Passivhaus timber buildings shall consider design simplicity, thermal continuity and robust airtight details (Leskovar & Premrov, 2012). Nonetheless, thermal bridge free design in timber buildings, although significant, it is not as critical as for steel or concrete buildings due to its intrinsic lower thermal conductivity (Kosny et al., 1997).

Additionally, a timber frame wall shall provide a thermal resistance to the heat flow and also it should perform as an acoustic barrier for sound propagation. The previous section, thermal requirements for low-energy buildings, has highlighted the relevance of the thermal properties of the fabric within current Building Regulations and it has introduced timber frame closed panel as a feasible pre-fabricated construction system. However, the timber frame wall must also be structurally sound. This section describes the synergies and conflicts of closed timber frame wall panels for structural and thermal purposes.

2.5.1 Open panel timber frame systems

The first timber frame houses, known as longhouses (Figure 2-11a), were built by early European farmers between the years 4500 and 3000 BC. The lack of structural design and carpentry knowledge resulted in the longevity of theses constructions generally exceeding no more than twenty years. Other tribes evolved this method of construction to build either stronger or quicker houses. A lighter evolution of the longhouse houses was developed by The Celts after occupying Central Europe in 400 BC. These Celtic houses (Figure 2-11b) were all of similar width and length.



a) Neolithic longhouse b) Celtic house

Figure 2-11 Reconstruction of ancient timber frame houses

Regarding modern timber frame buildings, there are two predominant forms of construction: balloon frame and platform frame. Balloon frame systems present continuous studs over two or more stories with only one top and bottom rail (Figure 2-12 a). The floor joists are supported by horizontal beams fixed internally to the studs.

On the other side, a platform frame system is a storey-by-storey assembly process where the floor joists are supported directly on top of each floor rail (Figure 2-12 b).

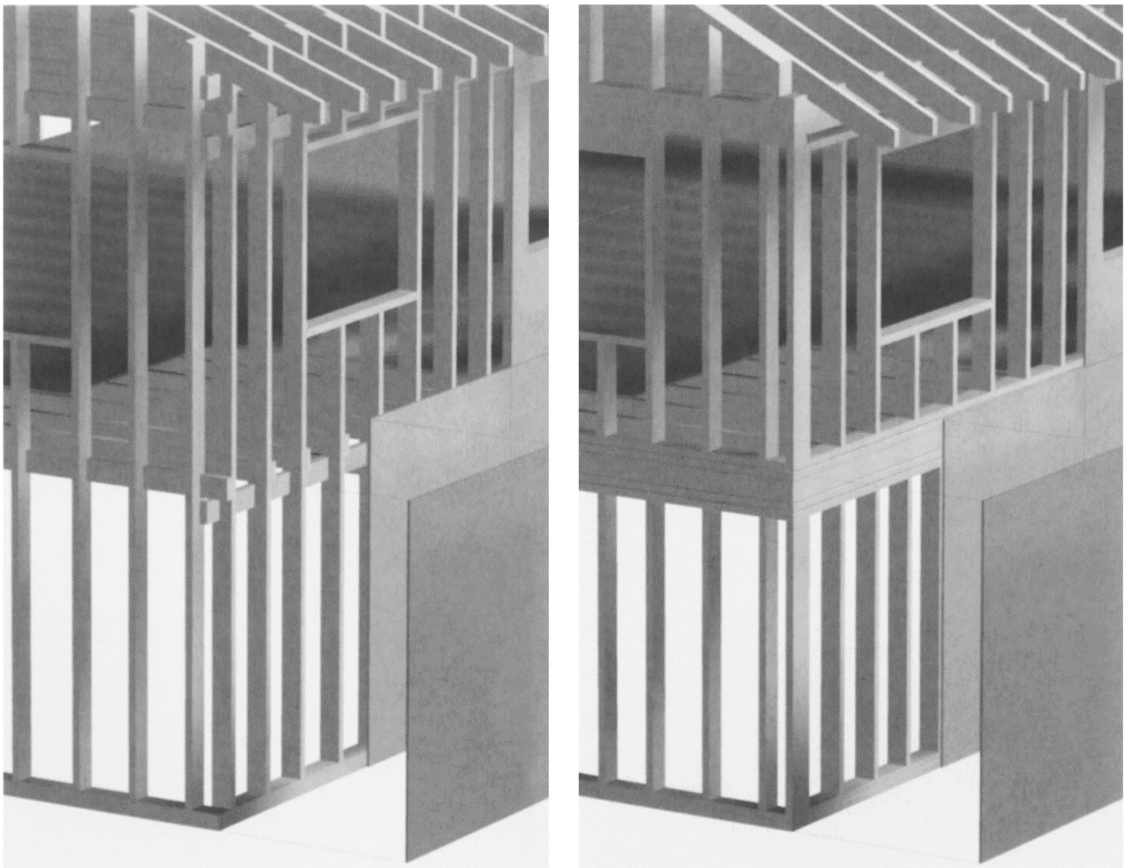
Balloon frame could be more indicative of medium-rise buildings, typically from three to six storey height, due to its better longitudinal structural stability (Cavanagh, 1997).

However, platform frame systems enable for a higher degree of prefabrication and standardisation (Kolb, 2008) which is in agreement with the subject of this research.

In platform timber frame construction, the building designer or engineer, from the architectural layout, must consider a series of structural checks (TRADA, 2007):

- Design of roof elements including common rafters, purlins, ceiling joists and its connectors.
- Design of floor elements including main floor beams, secondary joists and columns from floors above or the roof structure and its connectors.
- Design of wall elements including external wall studs, lintels, columns, façade materials and its connectors.

- Stability of the building including overturning, sliding effects and racking resistance of timber frame walls.



a) Balloon frame b) Platform frame

Figure 2-12 Balloon and Platform frame construction after Kolb (2008)

The structural design of timber frame wall panels for low energy buildings is mostly governed by the racking resistance (Figure 2-13). This is the tendency of the frame to distort from a rectangular to a rhomboid shape under the action of an in-plane force.

Although some literature suggest that the racking resistance of timber frame shear walls is governed by the behaviour of the sheathing to frame connection (Judd & Fonseca, 2005; Pattonmallory et al., 1985; Sugiyama & Uchisako, 1991), the performance of the sole plate fixing detail may also have a significant impact on the global racking resistance of the wall frame (Girhammar & Kallsner, 2004; Leitch, 2013).

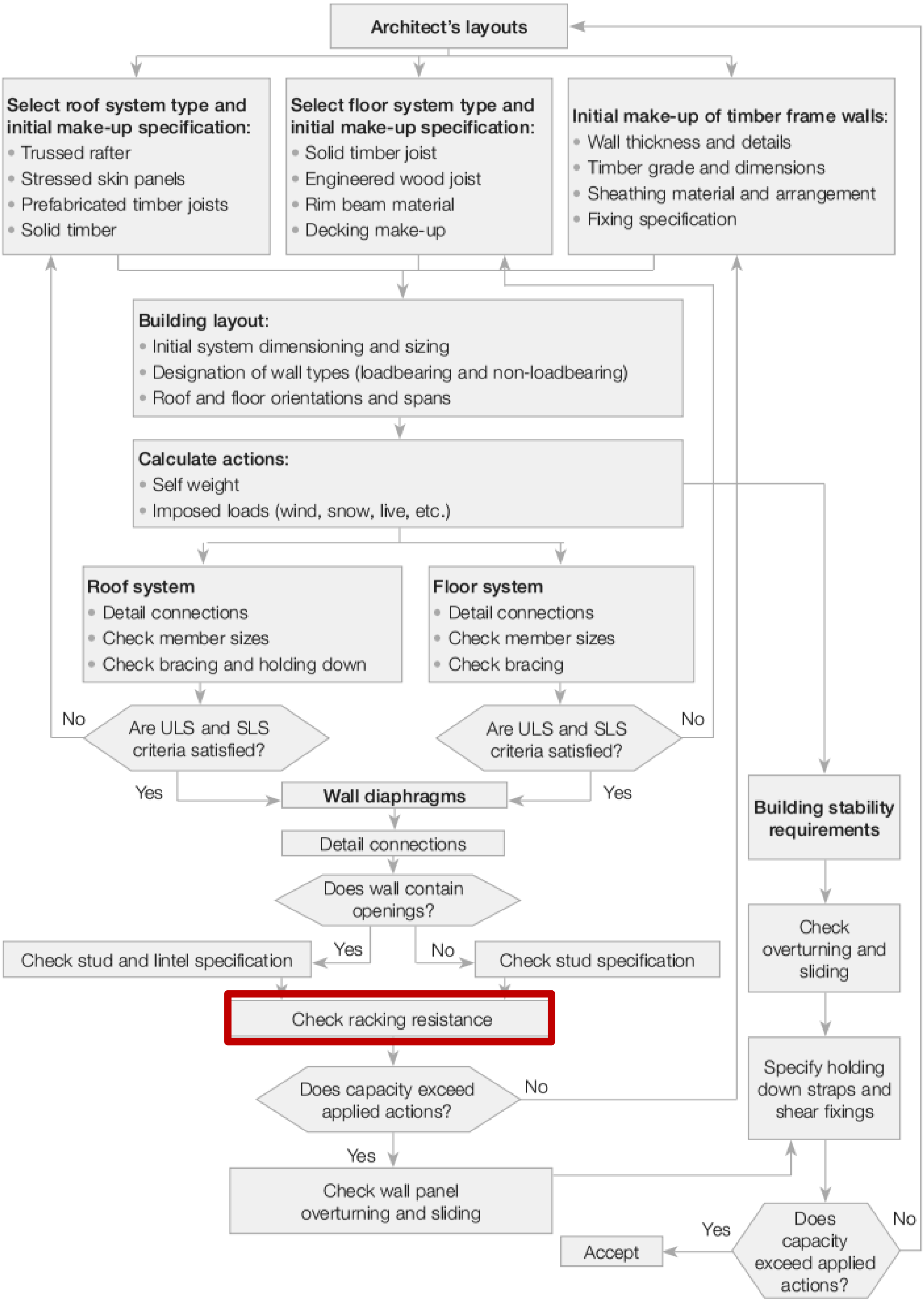


Figure 2-13 Platform timber frame design flow chart (TRADA, 2007)

These two arguments are considered here as the structural performance indicators amongst other structural analyses such as compression strength of a stud, bearing strength of wall panel or bottom rail and head plate strength (Porteous & Kermani, 2007). An example of a full timber frame wall design, as detailed in Table 2-9 and a detailed full timber frame calculation including these structural checks is given in the Appendix I.

Subsequently, structural optimisation of prefabricated timber frame wall panels is focused on racking resistance and on the sole plate base fixing detail.

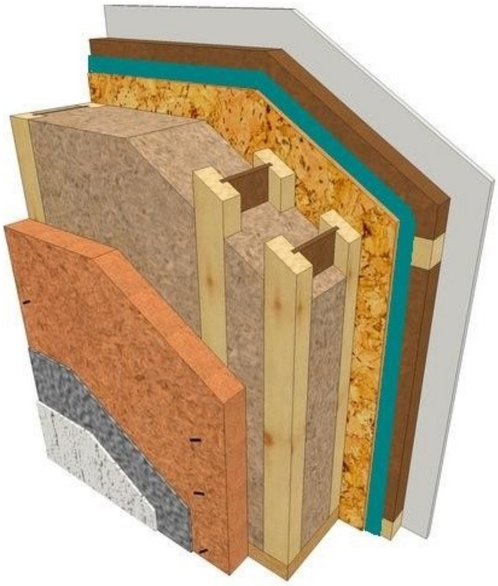
Table 2-9 Structural design requirements for shear walls

	Strength	Stiffness
Stud	Axial compression Axial + Flexural Torsion	0.005 l for Glulam or LVL 0.003 l for solid timber
Head binder *Top/bottom runner*	Bending Bearing Shear	l / 250
Lintels	Bending Bearing Shear Torsion	l / 350
Sole plate	Bearing Uplift	Slip < 0.4mm
Stability	Racking Overturning Sliding	h / 250

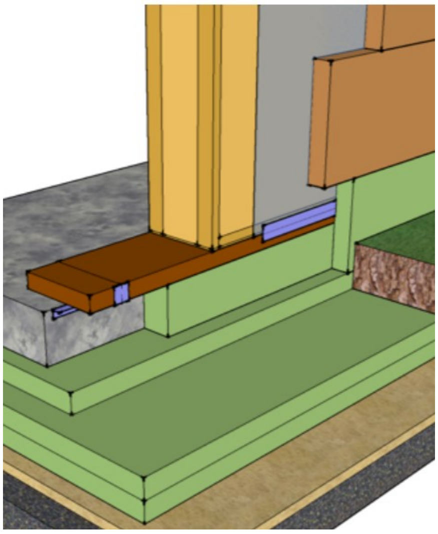
2.5.2 Closed timber frame panels

Several commercial closed timber frame panels have been studied for low-energy buildings, some of them being even Passivhaus certified as the illustrated by Figure 2-14. However, there is no specific literature published on the combined structural and thermal behaviour of these advanced closed panel systems for very low energy buildings.

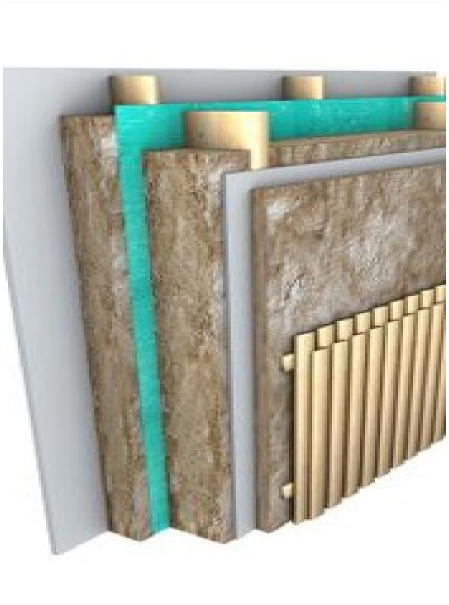
On the other hand, there are a multitude of studies on open panel timber frame racking performance. Nonetheless, only one publication was found with timber frame panels for higher thermal performance. This Canadian study investigated a prefabricated timber frame wall system with an additional sheathing board in the centre of the wall (Ni et al., 2007). The patented system called MidPly Wall presents a superior racking resistance due to the middle board connection acting on double shear but maintains the wall thickness to low values.



Steico Bausystem



Ecococon Straw Panels



Knauf Warm Wand Systeme

Cygnum Passive 350

Figure 2-14 Passivhaus certified advanced timber frame systems from Passivhaus certified components database (www.passivehouse.com)

More recently, a study on the structural and thermal behaviour of timber-concrete prefabricated wall system was published but for mid-rise and tall buildings (Hein et al., 2015).

Furthermore, closed panel shear walls, contrary to open panel construction, must deal with a relatively complex connection system to the underneath construction (TRADA, 2013). This particular collection of connections is referred as the sole plate base fixing detail. On site access to the sole plate detail in prefabricated closed panel units is rather difficult. The solution to this problem is often found in the design of structurally convoluted sole plate geometry and the inclusion of additional shear planes (Menendez et al., 2013).

Therefore, the sole plate design for closed timber frame wall panels presents a decrease of strength and stiffness in comparison with standard open panel construction. Additional design caution must be taken in order to comply with the minimum end and edge connection distances (Leitch, 2013).

2.5.3 Other timber building systems

In this sub-section, a brief critical literature review on different structural timber systems other than timber frame walls is presented. The vast majority of the research found is on open panel timber frames with very few studies on advanced timber frame closed panels, the area of this PhD research. However, some studies on racking resistance of different systems has been undertaken recently, mainly for massive Cross-Laminated Timber (X-LAM).

Massive solid timber

During the last decade, massive solid timber walls, and particularly X-LAM, have become a popular construction system for commercial and even mid-rise buildings (Hein, 2014). Shear walls made of this EWP have been extensively tested in Europe with a great emphasis on seismic behaviour (Dujic et al., 2008). Similarly to timber frame shear walls, the anchorage connection has proven to be especially relevant to the global panel deflection for walls with and without openings.

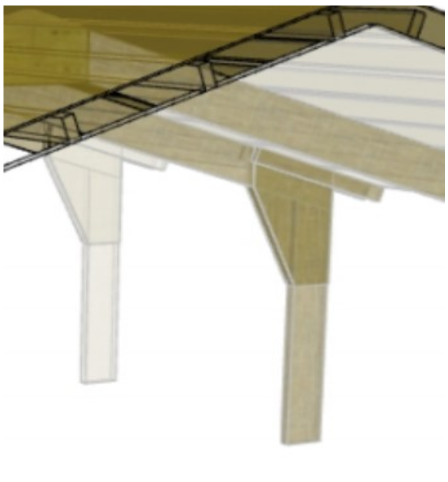
Stiffness of X-LAM shear walls depends on the quality and homogenisation of the internal lamellas, hence a good degree of quality control in the manufacturing process is required (Steiger et al., 2012). Furthermore, excellent seismic behaviour has been shown by X-LAM structures also in full-scale buildings (Ceccotti et al., 2013). Experimental results have been compared with numerical Finite Element Analysis (FEA) models which have been used in the publication of a design provision for seismic design (Parida et al., 2013).

SIP Panel and other timber systems

Structural Insulated Panels (SIPs) are an alternative construction system (Figure 2-15a) for residential and commercial buildings where insulation and wooden boards, most commonly Oriented Stranded Board (OSB), act as a composite element for structural purposes (Kermani, 2006). Beside timber frame construction, Glulam portal frames sheathed with plywood (Figure 2-15b) were also tested to understand and predict the behaviour of this timber system when larger open areas are required (Komatsu, 2004).

Although the thermal performance of this systems can be considered satisfactory for low-energy buildings (Krarti et al., 2007; McCullom et al., 2010), other concerns on embodied carbon and long-term energy performance were raised (Pierquet et al., 1998).

a) SIP Panel



b) Glulam portal frame

Figure 2-15 Other timber structural systems

Regardless of the type of system and material, any structure has to provide resistance to vertical and horizontal loads in order to maintain its structural integrity. A literature review of these actions on timber frame walls is given in the next section.

2.5.4 Actions on the structure

By definition, a shear wall is a structural system that transfers lateral load, acting on the building, to the foundation by diaphragm action (Figure 2-16). The most common lateral loads applied are wind and seismic forces. Other lateral loads such as fluid or earth pressures are seldom on timber structures.

The interaction between the loading and the material properties of the timber building must be understood. Depending on the type of load on the wall panel tested, two loading schedules are commonly performed: monotonic static and cyclic dynamic.

The first test method measures the Ultimate Limit State (ULS) resistance of the wall and the vertical and horizontal deformations under constant pressure and can be approximated to standard wind forces.

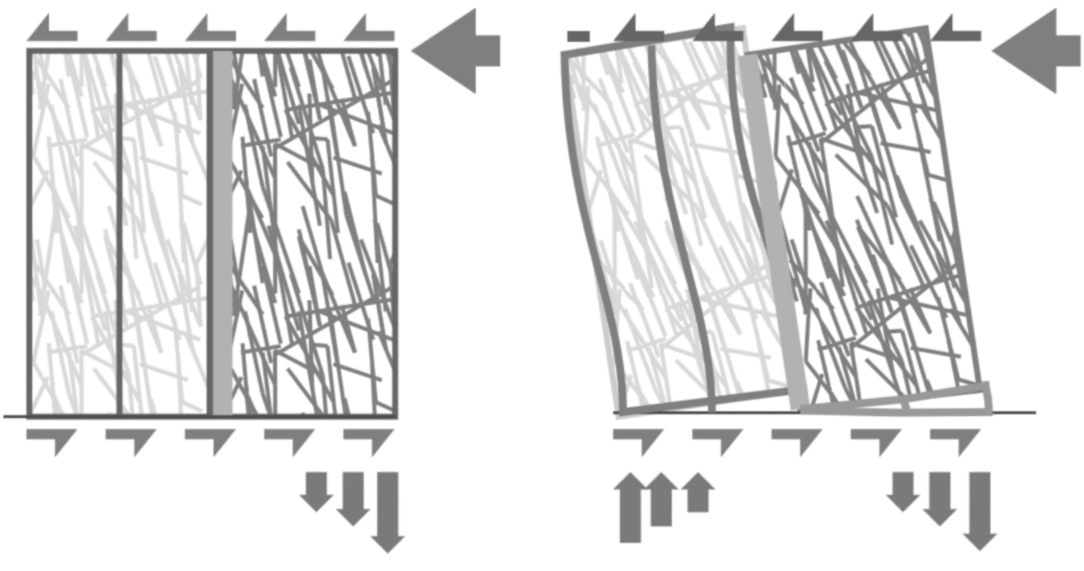


Figure 2-16 Diaphragm action in a shear wall before and during wind load.

The second test method is commonly undertaken for structures subjected to seismic or high wind loads on tall buildings. This is not common in timber structures in the UK and therefore it is excluded on this research.

Vertical or gravity loads

Permanent vertical actions contribute to the resultant equilibrium from destabilising lateral forces and therefore these need to be considered for shear wall design. For example, anchorage systems does not have much influence on racking resistance when high levels of gravity load occurs (Dujic & Zarnic, 2002). Similarly to this conclusion, Ni and Karacabeyli (2005) stated that holding down devices may not be required when considering large dead loads (i.e. considering all the dead loads and returning walls of dwellings with more than two stories and heavy roofing). Nevertheless, wind uplift was not considered in that study published in Canada.

No influence of additional vertical gravity load on fully restrained walls was observed in an experiment with more than 40 tests, under combined lateral and vertical loads and under static and cyclic load protocols (Payeur et al., 2011). However, one of the outcomes of the study stated that lateral resistance of shear walls could not be solely reliant on vertical loads.

Wind loads

Prior to the assessment of the racking performance of a shear wall, a good estimation of the wind load on the structure must be produced. In the UK, Eurocode 1 part 4 (BSI, 2010) provides the design provisions for wind load for low and medium rise buildings.

Load duration has to be considered for timber structures as the strength properties of wood decreases the longer the duration of the acting load is. Eurocode 5 describes wind load as “instantaneous load duration” which can provide the design of the shear wall, depending on the wood material, even with a favourable factor, i.e. k_{mod} greater than 1.0 (Porteous & Kermani, 2005).

2.5.5 Structural concerns on current building design

Architectural modern designs take into account the need of the building to adapt to future floor plan configurations. Frequently, this results in even customer-oriented design layouts with large internal open areas and few partition walls (Stehn & Bergström, 2002).

Contemporary buildings are also defined by other design features from solar passive architecture such as large south-facing glazing areas and compact building shapes (Smeds & Wall, 2007).

However, these architectural features can result in a design with insufficient lateral stability or bracing (Leskovar & Premrov, 2012). A study on structural failures in 157 timber buildings conducted by Frühwald (2011) concluded that half of these structural failures were due to design errors and mostly due to instability issues.

Modern building construction requires economy, efficiency and speed of erection which can be delivered by panelised wall products (Lindow & Jasinski, 2003). Nevertheless, prefabricated construction requires a high level of detailing which not always is produced causing difficulties in the execution phase (Paterson, 2013). A common example of this issue can be found on the sole plate base fixing detail for timber frame construction.

Furthermore, concerns on lateral stability of light-weight low rise timber frame buildings have been reported due to openings (Doudak et al., 2006) and to prefabrication (Toro et al., 2007). A similar conclusion on lateral bracing, but for open panels, was previously published by Liu et al. (1990). In that study, the authors state that structural failure can occur also at roof and top wall level and the connection between these two systems should be detailed to avoid uplift.

Timber frame buildings are highly indeterminate structures with a high capacity to redistribute lateral and gravity forces. The design of this complex and redundant system may either be over-estimated or under-estimated (Kasal et al., 2004). Hence, the designer must assume a correct method to transfer the lateral forces to the substrate. For low and mid-rise timber buildings, shear walls are the predominant structural system to provide lateral bracing and gravity resistance (Vessby, 2008). As a result, structural optimisation can be achieved by accurately predicting the performance of the system and the load transfer paths within the structure (Mi et al., 2004).

As mentioned previously, large openings have indeed a significant impact on the strength and stiffness performance of shear walls. 3-Dimensional modelling investigations undertaken by He, Lam, & Foschi (2001) on regular timber frame cubes and with one side presenting a large opening (75% of wall area) concluded that global shear capacity of the structure was reduced by almost 50%. In the same study, the torsional moment generated on the model caused nine significant different deformations at each of the top corners. Other studies have also noted the occurrence of shear torsional moments on buildings with asymmetric distribution of shear wall stiffness (Ellis & Bougard, 2001 Smith, 1979).

Timber frame wall assemblies formed by multiple wall panels, very common in prefabricated construction, require a vertical connection between two adjacent panels in order to transfer shear forces along the abutted studs (B. Kallsner & Lam, 1995). This design shear strength between panels is set by Eurocode 5 (BSI, 2009) to be at least 2.5 kN/m which is on reasonable agreement with other studies (Girhammar & Kallsner, 2009; Morsefortier, 1995; Vessby, 2011).

Although the latest developments of new engineered wood products such as X-LAM or hybrid LVL elements are initiating the construction of relatively high-rise timber buildings (Hein, 2014; Walford, 2006), timber frame systems are still predominately used in low-rise buildings i.e. three or less stories where research is abundant (Larsen & Munch-Andersen, 2011).

Consequently, the scope of this research and the literature review provided in section 4.2 focuses on timber frame systems only.

2.6 Concluding Comments

This chapter has shown different parameters to consider when designing low-energy buildings. From the high-level and the critical literature reviewed, it can be concluded that thermal and structural performance are key factors to deliver robust low-energy timber frame building solutions. The minimum energy performance has increasingly been updated in recent Building Regulations. Furthermore, special attention was given to building fabric, in terms of thermal transmittance and in terms of airtightness. Similarly, extremely high energy efficient standards, such as Passivhaus are currently being adopted to comply with future building requirements.

In order to mitigate the imbalance between housing demand and number of homes built, the UK government has encouraged the introduction of Modern Methods of Construction (Barker, 2004). Timber frame closed panel systems are a suitable technology for affordable housing. However, the level of confidence in using advanced timber frame systems is reduced depending upon the level of prefabrication where further research is required (WRAP, 2007).

A need for thermal performance research on timber frame walls and for low-energy building design was identified in the literature review. Energy transfer in a wall frame occurs mainly by conduction from the warm side to the cold side according to Fourier´s Law. Managing the moisture transfer ensures a construction free of condensation risk. Thermal performance can be predicted by 1-D and 2-D numerical models which must satisfy with the relevant standards (EN ISO 9972).

The airtightness of a low-energy building also needs to be higher to comply with Passivhaus standard. Construction type and on-site quality procedures were found to be influential on airtightness. However, no publications were found on airtightness durability for advanced closed timber frame systems. This thesis considers robust construction details to provide timber frame buildings for durability and resilience.

The tendency of the timber frame to distort in-plane, also known as racking, is the mechanism to resist lateral loads. For low-energy low-rise buildings, racking frequently governs the structural design. The sole plate connection also significantly impacts on the global racking resistance of a timber frame wall. A research gap on racking and sole plate resistance for closed timber frame systems has been identified. This thesis attempts to add information on this shear wall topic.

No timber frame construction details have been standardised for conformity with regards future regulatory requirements. This research will therefore seek to provide a compilation of informative data sheets containing thermal and structural details for advanced timber frame panelised systems. The energy efficient Passivhaus standard has been selected as a criterion for minimum thermal performance requirements.

The high-level literature review provided in this chapter has identified the Passivhaus standard as a valid construction method to comply with future UK regulations and also has identified closed timber frame panel systems as a construction technology to deliver affordable housing. Moreover, no publications have been found that investigated structural and thermal performance in combination for closed timber frame panels. Furthermore, questions have been raised about the contribution of sole plate base fixing details on the racking performance of timber frame walls which may have also and impact on long term thermal behaviour.

As a conclusion, this chapter has found a knowledge gap for further research on closed timber frame panels and for future building regulations which is investigated in the next chapters.

3 Thermal Performance of Timber Frame Walls

Thermal performance characteristics of timber frame wall panels are presented in this chapter. A background to thermal performance introduces the materials, processes and analyses to be considered in the research. A concise literature review on this topic, complementing that presented in Chapter 2, is also provided.

The next section identifies three different timber frame wall panel configurations for further thermal performance investigation: a standard timber frame as a benchmark scenario, and two commercial advanced closed panel timber frames. The purpose of this exercise is to understand the thermal implications of available closed timber frame panel systems in relation to traditional open timber frame construction.

The Thermal resistance of the three timber frame build-ups combined with two different sheeting materials are calculated by analytical and numerical methods. Also, a timber fraction value for advanced timber frame panels is proposed based on the study of four different house types built. The thermal performance investigations conclude with a thermal bridge simulation analysis for two different sole plate details. The last section investigates the hygrothermal characteristic of the wall assemblies in three different climate zones in order to classify the build-ups as condensation-free.

The organisation of the chapter is illustrated by Figure 3-1.

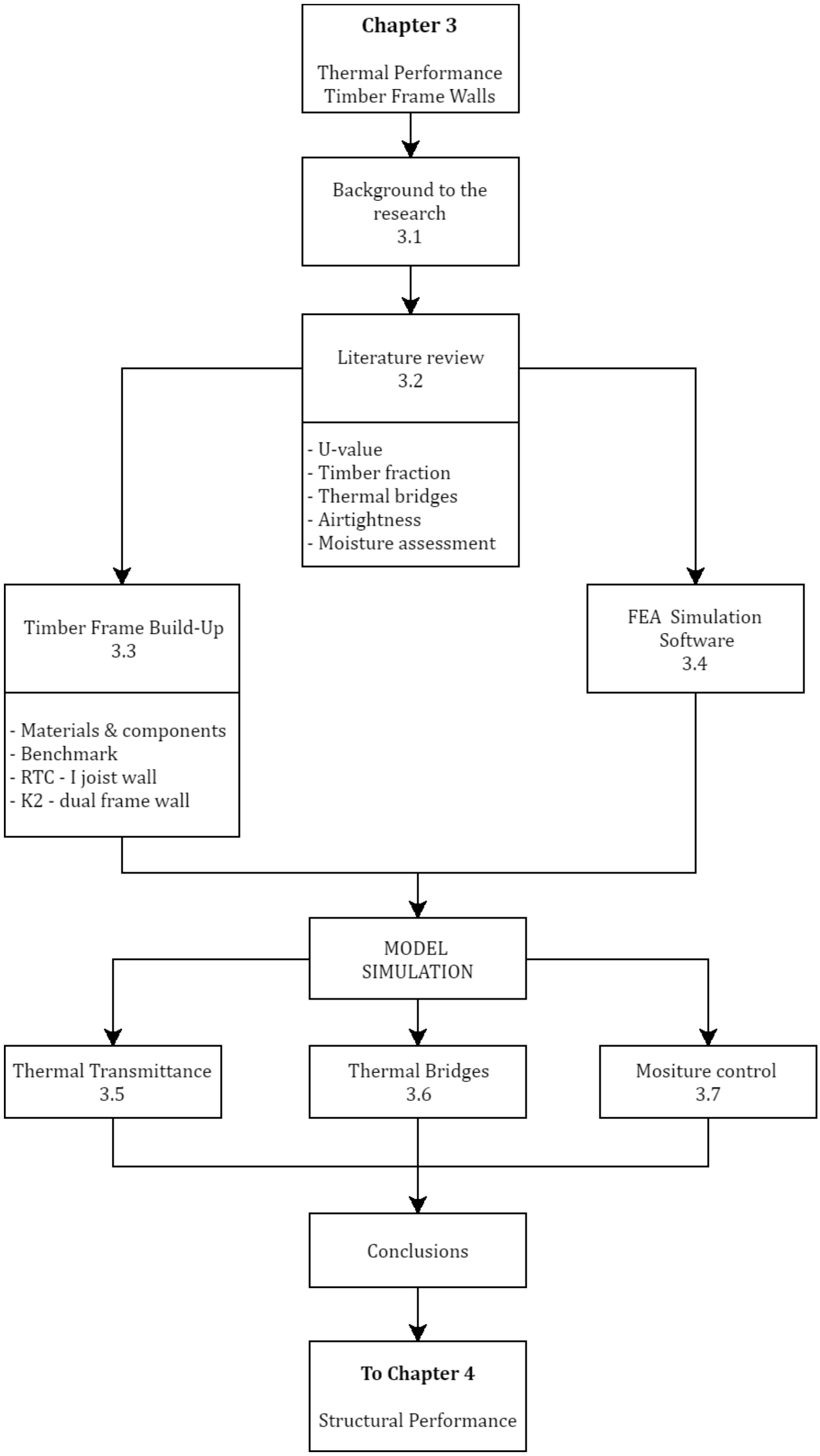


Figure 3-1 Organisation of the Chapter 3.

3.1 Background to Thermal Performance

One of the first measure to achieve a good level of thermal efficiency in any kind of building is insulation. Insulation in the walls, in the floors or in the roof. All over the world, in order to improve building energy performance, building regulations are seeking to reduce the maximum allowable U-values, heat transmittance measured in W/m^2 K. For this reason, the U-value limit for wall panels considered in this thesis was 0.15 W/m^2 K as the maximum value recommended by the benchmark criteria, the Passivhaus standard (Feist, 1996). This standard is seen as a strategy reference for energy efficiency. However, it does not consider other issues like life cycle assessment (LCA) or environmental credentials (Dequaire, 2012) where timber frame construction can perform well.

External timber frame walls can offer a significant level of insulation with relatively slender wall thickness when compared to other construction types such as brick and block, steel framing, cross-laminated timber, rammed earth or even insulated concrete formwork. Only SIPs panels can provide the same level of thermal resistance in thinner wall structures. Nevertheless, the long-term durability of the insulation foamed in SIPs panels has been shown to be less than that of other materials (Pierquet et al., 1998).

It is important to highlight that thicker walls not only require more materials to fabricate but they can reduce the useable floor area of a dwelling on a fixed footprint which can consequently increase the building cost per square metre. In terms of off-site manufacturing, thicker walls frequently cost more to fabricate and the factory layout may require more fabrication space and/or more expensive and powerful equipment.

There are many different types of insulation utilised within a timber frame construction including: rigid batts, flexible blankets or rolls, blown-in or loose-filled with flakes or granules or sprayed foam directly (Table 3-1). Also, in order to deliver even a better thermal U-values, additional insulation can be added on the internal and/or external side of the wall by rigid insulation or flexible insulation supported by a sub-structure i.e. service cavity.

An additional method to improve the global thermal resistance of the building envelope is to use reflective insulation or membranes facing an air cavity (Belusko et al., 2011).

Table 3-1 Types of insulation materials. Table adapted from Pfundstein et al. (2008)

Type	Material	Conductivity λ / k (mW/m K)	Density δ (kg/m3)
Blanket or roll	Fibreglass	32-45	20-200
	Rock wool	35-45	35-150
	Wood fibre	36-45	30-270
	Cotton wool	40-50	20-60
	Aerogel	17-21	60-80
Rigid	Expanded Polystyrene (EPS)	30-35	15-30
	Extruded Polystyrene (XPS)	29-39	25-45
	Polyurethane (PUR)	22-30	30-100
	Polyisocyanurate (PIR)	23-30	30-100
	Wood fibre	38-45	110-300
	Mineral fibre	32-45	30-200
	Compressed straw panel	40-70	70-140
	Cork boards	40-70	90-220
	Vacuum Insulated Panel	5-15	150-300
	Foamed glass	60-90	120-230
Blown-in	Cellulose	40-45	30-80
	Fibreglass	35-38	20-50
	Vermiculite	46-65	70-160
	Cork	40-60	60-100
	EPS pearls	32-45	15-30
Sprayed & foamed-in	Wet-spray cellulose	40-50	50
	Fibreglass	38-45	10
	Icynene	38-44	30
	Polyurethane	25-40	30
	Phenolic foam	22-40	40

The theoretical thermal resistance of the building envelope may be negatively influenced by poor workmanship both off-site and on-site. Thermal by-pass or air moving inside the insulation layer, i.e. creation of air spaces and cracks around the insulation, will degrade the U-value to a certain degree depending on where these gaps are found (Bankvall, 1987). Reduction of thermal performance for insulated walls with 10 mm and 3 mm air gaps at top and bottom was reported as 193% and 158% respectively by Lecompte (1990).

The thermal transmittance of timber frame walls is also influenced by its timber fraction and by any thermal bridges associated with junction detailing. These two factors are particularly relevant on highly insulated walls where bad design or fabrication can make a timber frame perform as poorly as some steel frame designs particularly in busy framing areas (Kosny et al., 1997b).

Airtightness is another parameter associated with thermal performance and where maximum allowable values for air permeability, q_{50}, are specified in the building regulations. In Passivhaus buildings, the infiltration losses are more restrained with an air change rate, n_{50}, value of lower than 0.6 h^{-1} required. Hence, the building must be ventilated mechanically in order to provide fresh air to the occupants and to remove odours, pollutants, moisture and stale air from inside the airtight building. Furthermore, any airtight layer forming part of the construction detailing, regardless its material, must be durable and satisfactory over time (Erhorn-Kluttig et al., 2009). There is a scarcity of literature on the longevity of airtight layers for low-energy buildings and the conclusions of these few studies are indeed contradictory.

A comprehensive literature review on the parameters influencing the thermal performance of timber frame systems (TFS) for low-energy building design is given in the next section.

3.2 TFS Thermal Performance Literature Review

In section 2.4, Thermal Requirements for Low-energy Buildings, an overarching literature review was presented with the basis of heat and moisture transfer processes and air permeability of buildings. In this section, a further critical investigation of research published on thermal resistance, timber fraction, thermal bridges, airtightness and condensation is given from a low-energy building context.

3.2.1 Thermal transmittance U-value of walls

Once the building geometry and design has been established, the next step is to determine precisely the building envelope build-up and the insulation type and thickness. This includes floor, wall, and roof elements and other boundaries to unheated rooms. The objective is to have a first approach to the optimum U-Value in order to satisfy the maximum allowable or the required heating demand.

The thermal transmittance can be determined under steady state or transient condition. Steady state condition is achieved when the temperature is independent of time at each point of the body and this is frequently considered in one-dimensional or two-dimensional coordinate systems. Transient or unsteady state conditions occur when heat transfer is variable driven and time-dependent. This normally occurs when the boundary conditions change i.e. the surface temperatures are altered.

A further mathematical analysis from the Fourier´s law described in section 2.4.1, Fundamentals of heat transfer, to determine heat transfer for different conditions and given in the following equations:

$$\dot{Q} = Q\,(x, y, z) \tag{3.1}$$

$$\dot{Q}_{cond} = -k\,A\,\frac{\partial T}{\partial x} \tag{3.2}$$

$$\frac{\partial^2 T}{\partial^2 x} = 0 \tag{3.3}$$

$$\frac{\partial^2 T}{\partial^2 x} + \frac{\partial^2 T}{\partial^2 y} = 0 \tag{3.4}$$

$$\frac{\partial^2 T}{\partial^2 x} + \frac{\partial^2 T}{\partial^2 y} + \frac{\partial^2 T}{\partial^2 z} = 0 \tag{3.5}$$

$$\frac{\partial^2 T}{\partial^2 x} + \frac{\partial^2 T}{\partial^2 y} + \frac{\partial^2 T}{\partial^2 z} + \frac{\dot{e}_{gen}}{k} = 0 \tag{3.6}$$

$$\frac{\alpha}{\rho\,C_p}\left(\frac{\partial^2 T}{\partial^2 x} + \frac{\partial^2 T}{\partial^2 y} + \frac{\partial^2 T}{\partial^2 z} + \frac{\dot{e}_{gen}}{k}\right) + \frac{\dot{Q}}{\rho\,C_p} = \frac{\partial T}{\partial t} \tag{3.7}$$

Where:

$\dot{Q}$ is the energy transfer for x, y and z planes, in J

k is the thermal conductivity, in W / m K

$\frac{\partial T}{\partial x}$ is the temperature gradient in x plane

$\dot{e}_{gen}$ Is the internal thermal energy generated, in J / kg

$\frac{\alpha}{\rho\, C_p}$ is the energy released per unit energy required to raise the temperature of a unit volume of a body by one degree

$\dot{e}_{gen}$ is the internal thermal energy generated, in J / kg

Equation 3.1 express energy transfer as a heat conduction vector working in a coordinate system. The heat is conducted in the direction of decreasing temperature as indicated in equation 3.2. Equations 3.3, 3.4 and 3.5 describes in one, two and three-dimensions respectively the steady state conditions. When heat is generated inside the body, equation 3.6 applies. Finally, equation 3.7 expresses heat conduction for transient conduction and internal heat generation.

An equivalent thermal series circuit analogy is frequently used to determined one-dimensional heat transfer in composite walls. In Figure 3-2 is represented a wall composed of two external layers and a middle layer with better thermal conductivity (i.e. SIP panel). A thermal resistance $T_{s,1}$ and $T_{s,4}$ for convection and surface radiation is added.

This is the basis for the overall heat transfer U-value analytical coefficient for homogeneous building elements and in accordance with the standard ISO-6946 (2007) where each layer of a building component presents a thermal resistance determined by the thickness and the thermal conductivity of the material (Equation 3.8).

$$U = \frac{1}{\sum R_i + R_{si,se}} = \frac{1}{R_{si} + \frac{d_1}{k_1} + \frac{d_2}{k_2} \dots + \frac{d_i}{k_i} + R_{se}} \tag{3.8}$$

Where:

U is the thermal transmittance in W/m²K

R_i the material, *i* thermal resistance in m²K / W

$R_{si,se}$ the internal and external thermal resistance in m²K / W

d_i the material, i thickness in m

k_i the material, i thermal conductivity in W/m K

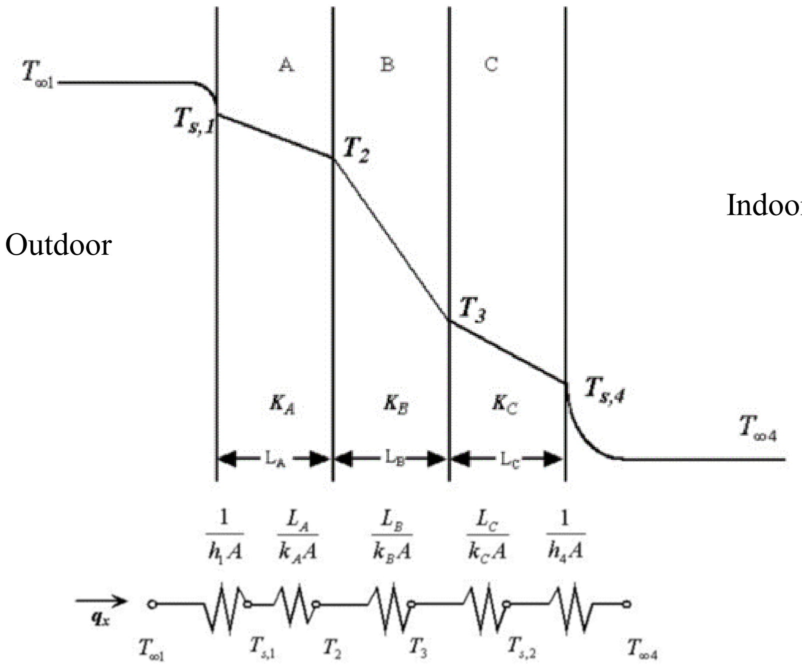


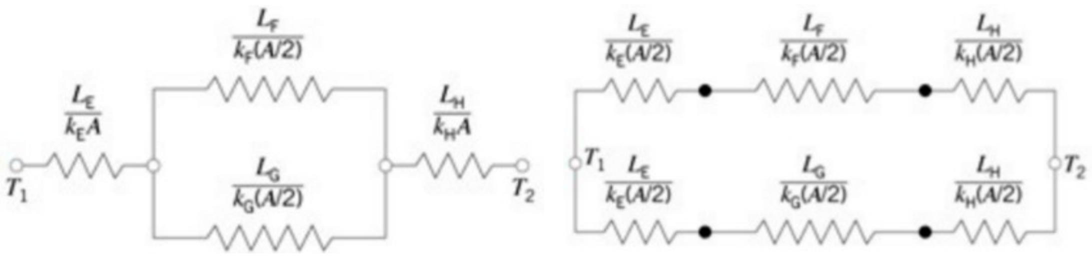
Figure 3-2 Equivalent thermal series circuit analogy for homogeneous walls. Adapted from Bergman et al. (2011)

On the other hand, the thermal resistance for inhomogeneous walls, i.e. timber frame panels, cannot be determined with one-dimensional analysis. In these systems, the heat flow is now multi-dimensional where an equivalent thermal series-parallel circuit analogy can be used (Figure 3-3).

However, for this analogy, the heat flow can reasonable be assumed as one-directional (Bergman et al., 2011). Multi-dimensional effects are significant if the incremental difference between the conductivity of the materials, $k_F - k_G$, are a factor of five or bigger (Anderson, 2006; Feist et al., 2007). In that case, two-dimensional heat flow calculations are required. For the analytical U-value calculation considering only one-dimension heat transfer (Equation 3.9), two estimates are determined:

$$U = \frac{1}{R_T} = \frac{1}{\frac{R'_T + R''_T}{2}} \tag{3.9}$$

- Lower estimate R'_T when surfaces normal to the x-direction are considered isothermal, temperatures remain constant (Figure 3-3 a).
- Upper estimate R''_T when surfaces parallel to the x-direction are considered adiabatic, no heat transfer (Figure 3-3 b).



a) Isothermal surface on y-axis. Change with constant temperature

b) Adiabatic surfaces on x-direction. Here heat does not enter or leave the body

Figure 3-3 Equivalent thermal series-parallel circuit analogy for a timber frame wall. Adapted from Bergman et al. (2011)

Optimised results from the analytical expression to determine the heat transfer of inhomogeneous walls are directly influenced by the timber fraction. This is the amount of timber presented in a wall and placed in the insulation layer without any insulated thermal break. Commonly, this ratio is seldom calculated on a project-specific basis and standard values from literature are often used. As shown in Figure 3-3, this is the ratio between the Area F and the Area G.

The results from experimental research carried out in hot boxes, a calibrated air-conditioned series of chambers, have shown that steady state heat convection through a vertical porous medium are in good agreement with the analytical methodology presented by Langmans et al. (2012). However, two-dimensional transient state simulations may be required to analyse the effectiveness of timber walls for example in renovation projects when they may be affected by rising moisture condensation (Holm & Kunzel, 2003).

3.2.2 Timber fraction

A widely used 15% timber fraction default value for standard timber frame 38 mm studs at 600 mm centres, i.e. from the centre of the stud to the centre of the stud, with noggins (horizontal bracing pieces) and double top plate is used, or 12.5% if the standard wall has one single top rail (upper frame timber) with no intermediate noggins instead (Anderson, 2006; DCLG, 2007). Figure 3-4 represents in red the amount of timber commonly accountable for the wall panel timber fraction for an imperforated and perforated wall.

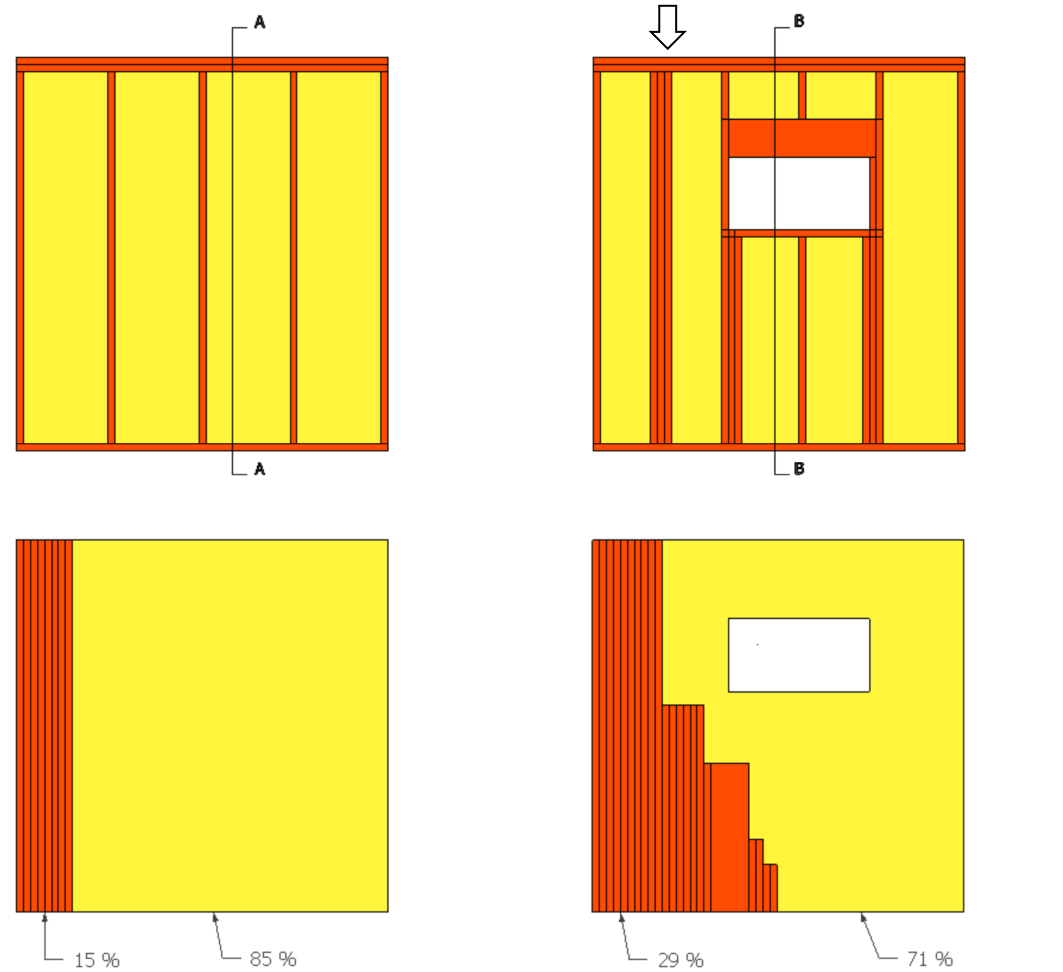


Figure 3-4 Timber accountable for timber fraction

A high timber fraction value is a consequence of a greater amount of wood in the wall panel due to, for example, point loads or openings in the wall panel with no insulation behind the additional wood (as shown on the right image in Figure 3-4). As a result of this larger timber fraction, the actual thermal transmittance of the wall is greater but with a linear thermal bridge higher than predicted in the energy model.

The significance of, higher than predicted, timber fraction values on the overall wall thermal performance has been published (Bell & Overend, 2001; Emmitt & Gorse, 2013) but not for energy efficient timber frame closed panels. Another study was carried out in Canada by Qasass et al. (2014) but no information was given on the stud spacing used in the research. Furthermore, Luxton (2014) concluded that in 20 timber frame houses studied, many of them presented a much higher than predicted timber fraction value and claimed this underestimated value as a contribution to the ´performance gap´ problem in the UK for timber frame houses.

On the other hand, Friedman & Cammalleri (1996) suggested that the common default Canadian timber fraction of 14% could easily be improved to just 9.4%. However, the definition of timber fraction in these studies are not clear for low-energy buildings where thermal bridges in the timber frame may be also accountable as shown later in Figure 3-27.

In the case of a wall panel with window openings, the additional timber surrounding the opening and added to the frame to facilitate its installation, is considered as well as a thermal bridge window installation, φ_{inst} (Schild & Blom, 2010).

The British Research Establishment (BRE) document 443, Conventions for U-value calculations, states that all timber that presents insulation behind the stud or lintel, if placed away from the window and in all of the remainder section may not be accountable for the timber fraction (Anderson, 2006). This agrees with the principle of continuous insulation to avoid thermal bridges but no further information has been found advising how thick this insulation layer should be.

Furthermore, a triple stud resulting from an applied point load as shown in Figure 3-4 (arrow pointing downwards), can be considered either as a timber fraction on that particular panel or as independent linear thermal bridge (Feist et al., 2007).

3.2.3 Critical thermal bridges

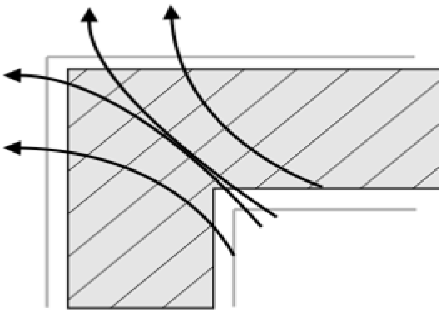
A thermal bridge can be defined as a part of the building envelope where the heat flow density at that point, and typically perpendicular to the surface area, increases or decreases. In that particular point, the temperature-specific heat loss is raised exceeding the corresponded value of the equivalent surface area multiplied by the transmittance of the envelope. The additional heat loss is the effect of the thermal bridge. There may be cases of good detailing where the timber bridge effect is negative resulting in the actual heat loss being smaller than the U-value of the corresponding building envelope.

Two different types of thermal bridges are frequently found in low-energy buildings: construction and geometric thermal bridges (Larbi, 2005). Construction thermal bridges (Figure 3-5a). are found when the insulation layer is partially or totally penetrated by a material with higher conductivity.

This thermal bridge can be linear (*ψ-psi-value*) or point (*χ – chi-value*). Examples of these types can respectively be timber studs within a timber frame wall or fixings for an external thermal insulation composite system (ETICS) façade.

a) Concrete joist on a roof.

Mould growing as a consequence of linear thermal bridge



b) building envelope junction sho Heat flux path in external corner

Figure 3-5 Construction (left) and geometrical (right) thermal bridges

Geometrical thermal bridges occur when the thickness or the material of the insulation layer changes (Figure 3-5b). This is relevant when the bridge is on exposed surfaces. The thermal bridges present a greater impact when insulation levels in the envelope are particularly high.

The effects of cold thermal bridges on the global heat losses for a building can be significantly high. Also, due to design and structural constraints, thermal bridges may be complicated to avoid resulting on additional cost to the project (Kosny et al., 1997a). Conversely, thermal-bridge free detailing is needed not only for project economy (Schnieders & Hermelink, 2006) but for structural safety as the risk of pathologies related to moisture issues are minimised (Hens, 2012).

In Passivhaus design, linear thermal bridge free design is considered if Equation 3.10 is fulfilled (Feist, 1993).

$$\psi_a \leq 0.01 \frac{W}{m\ K} \tag{3.10}$$

Where:

ψ_a is the maximum linear thermal bridge in W/m K

Thermal bridge calculations due to interconnectivity between different building structural elements is commonly analysed by two-dimensional software (Pfluger, 2005). In order to minimise the thermal effect of the load bearing element within the building, new engineering wood products are constantly being redeveloped (Tuomi, 1987).

Point thermal bridges describe the effect of penetrations on the insulation layer such as metalwork fixings on the timber sole plate. Significant point thermal bridges are less common than linear thermal bridges therefore, it is recommended to reduce linear penetrations to only the structurally necessary point penetration (Hopfe & McLeod, 2015).

In Passivhaus design, point thermal bridge free design is considered (Feist, 1993) when Equation 3.11 is fulfilled.

$$\Delta U_{TB} = \frac{\sum \chi}{A} \leq 0.01 \frac{W}{m^2\ K} \tag{3.11}$$

Where:

$\frac{\sum \chi}{A}$ is the maximum summation of all of the point thermal bridges divided by its area in W/m K

A study carried out by Feist (2006) provided a summary of linear thermal bridge for both standard construction practice (from 1990s) and Passivhaus thermal bridge free construction. The conclusions shown in Figure 3-6 highlights the connection between the foundations and the wall as the most critical for heat losses hence. This research examines the significance of the sole plate connection for thermal bridging optimisation.

There are few thermal bridge-free details published in the literature. Accredited construction details have been published in Scotland (SBSA, 2009) and England and Wales (DCLG, 2007) but with declared Psi-values largely exceeding 0.01 W/m K. Pitts and Lancashire (2011) describes some high-performance external timber frame walls but it does not provide any robust sole plate connection detail.

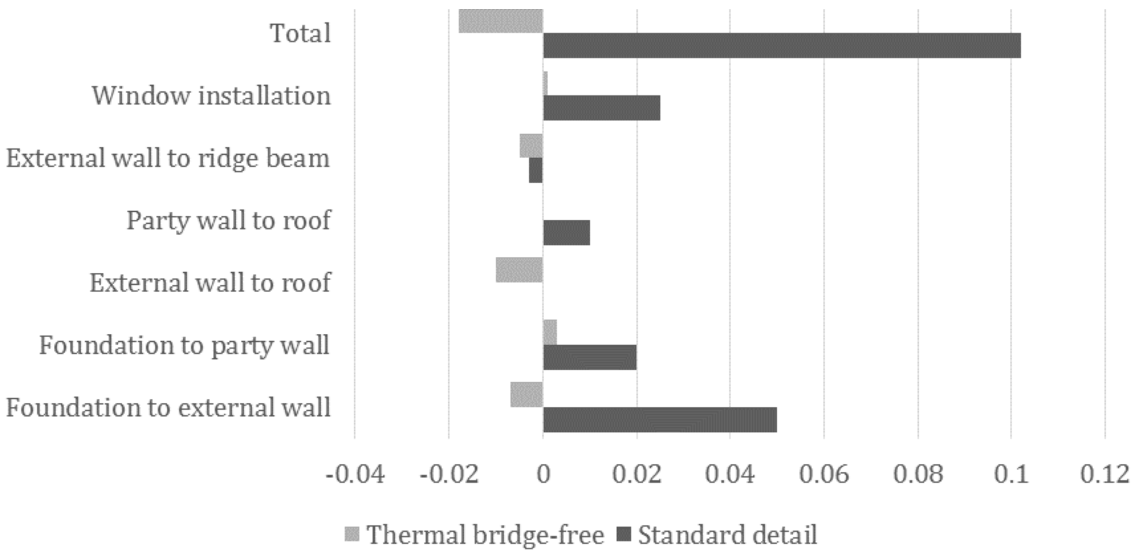
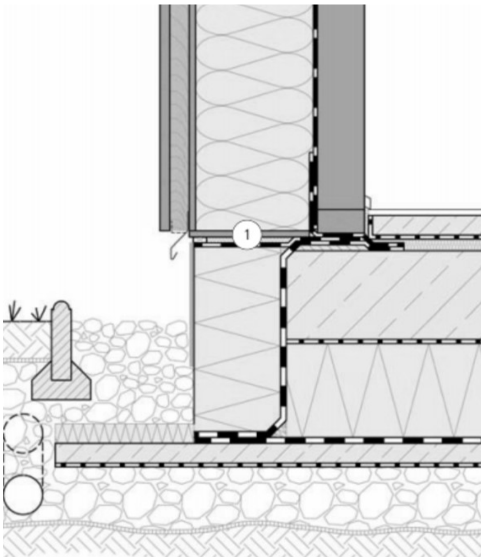
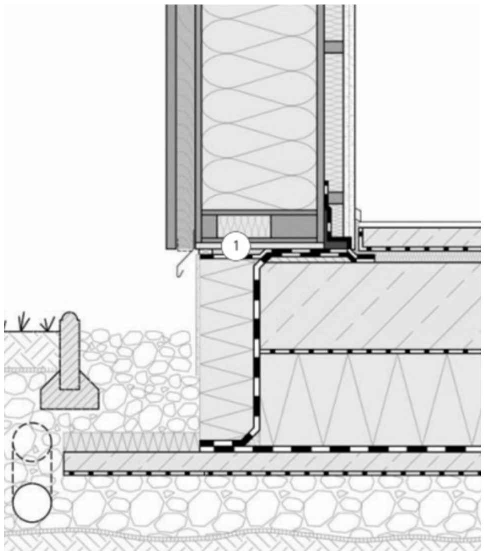


Figure 3-6 Typical thermal bridge values, in W/m K, for standard and Passivhaus construction (Feist, 2006)

On the other hand, Pokorny et al. (2009) suggest a series of different external wall to foundation connections with calculated thermal bridge values less than 0.01 W/m K and recognised by the Passivhaus Institut as thermal bridge-free connections (Figure 3-7). However, these connection details do not provide any structural information. Additionally, modifications on those details may be needed for practical construction.

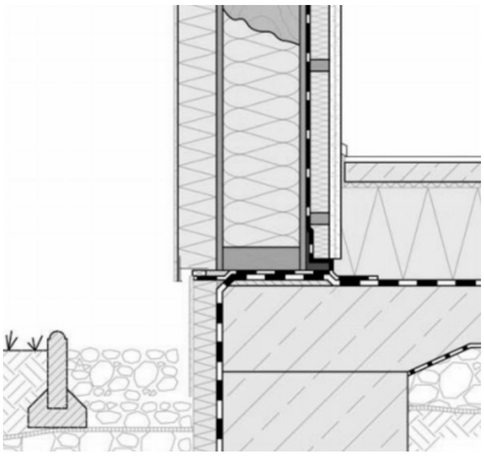
Another descriptive guide for designing energy efficient timber building envelopes was published in Canada (Finch et al., 2013). The comprehensive library of timber frame build-ups and connections, although well illustrated, does not provide any quantitative values for thermal nor structural performance.

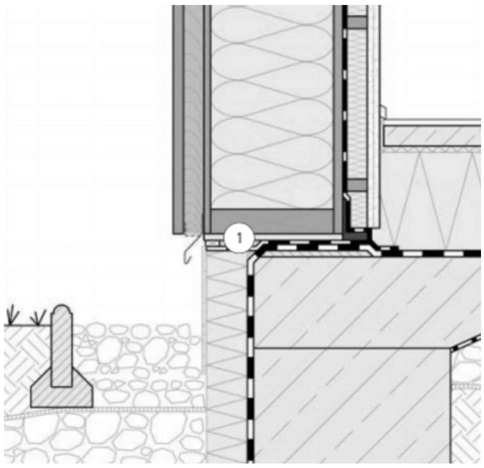




Internally exposed CLT wall with external flexible insulation and rainscreeen facade.
Slab on grade foundation with rigid insulation underneath.
① OSB plate

Dual box beam wall with insulated service cavity and rainscreen façade.
Slab on grade foundation with rigid insulation underneath
① OSB plate





Insulated timber frame wall with insulated service cavity and ETICS façade.
Slab on grade foundation insulated on the upper side.

Insulated I-joist timber frame with insulated service cavity and rainscreen façade.
Slab on grade foundation insulated on the upper side.
① Fibre cement plate

Figure 3-7 Example of thermal bridge free sole plate connections and relevance to airtightness detailing (Pokorny et al., 2009)

3.2.4 Airtightness

The heat loss improvement on the overall thermal performance that airtight construction offers is commonly not perceived. Contrary to this point of view, Doebber & Ellis (2005) state that airtightness is more influential than insulation continuity or thermal mass. Moreover, improvements on the thermal resistance of building envelopes can be counterproductive if inadequate levels of airtightness are in place due to potential moisture-related problems on the interface (Leardini & Van Raamsdonk, 2010).

It has been stated by Asiz (2008) that the butt joint between adjacent shear wall panels showed excessive air leakage in the form of infiltration or exfiltration (depending on pressure differentials), thus causing an important source of heat loss within the building fabric. In that study, a 1D-2D FEA analysis was able to predict with good agreement the thermal performance of a timber wall assembly if the panel-to-panel connection was perfectly sealed.

Geissler (2001) studied the airtightness of 87 timber frame buildings in Germany where only 5% of them presented an adequate level of airtightness in order to install a Mechanical Ventilation with Heat Recovery (MVHR) system. In his study, the average air leakage measured of the old buildings were higher, up to 4 times on average for buildings over 15 years old, than newly built houses.

A study undertaken by Molin et al. (2011) indicated that the airtight layer of a low energy house showed signs of damage on the plastic membrane probably caused by the tenants and by the kitchen fan. In a similar approach, a research project studied the airtightness of 31 Passivhaus dwellings after project completion and two years after that date (Reiss, 2003).

The study concluded that 20 out of the 31 houses showed a certain degree of airtightness degradation and 9 out of 31 houses had a 50% greater air change rate. Indeed, four dwellings presented an airtightness value greater than the maximum allowable by the Passivhaus standard (0.6 ach).

Another similar study carried out more recently on 25 Minergie-P standard low-energy dwellings (Bossard & Menti, 2013) concluded that 12 out of 25 buildings presented a worse airtightness after a period of time but only four out of 25 houses had a 50 % air changes more per hour and this was attributed to a defected window installation. Figure 3-8 and Figure 3-9 represent the variation on airtightness data for these two studies.

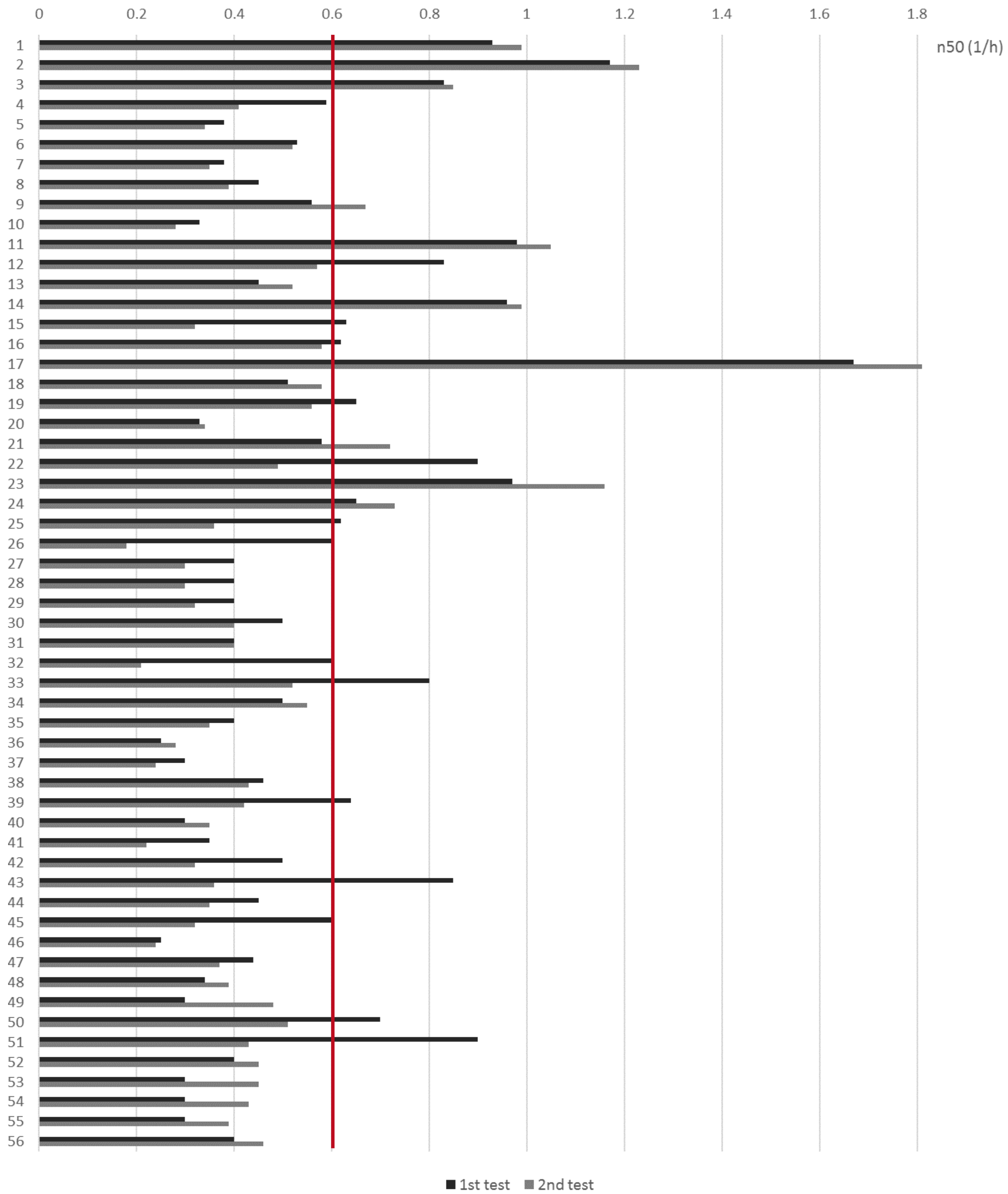


Figure 3-8 Building airtightness test results for 56 dwellings after Bossard & Menti (2013) Reiss & Erhorn (2003). Red line shows Passivhaus criteria.

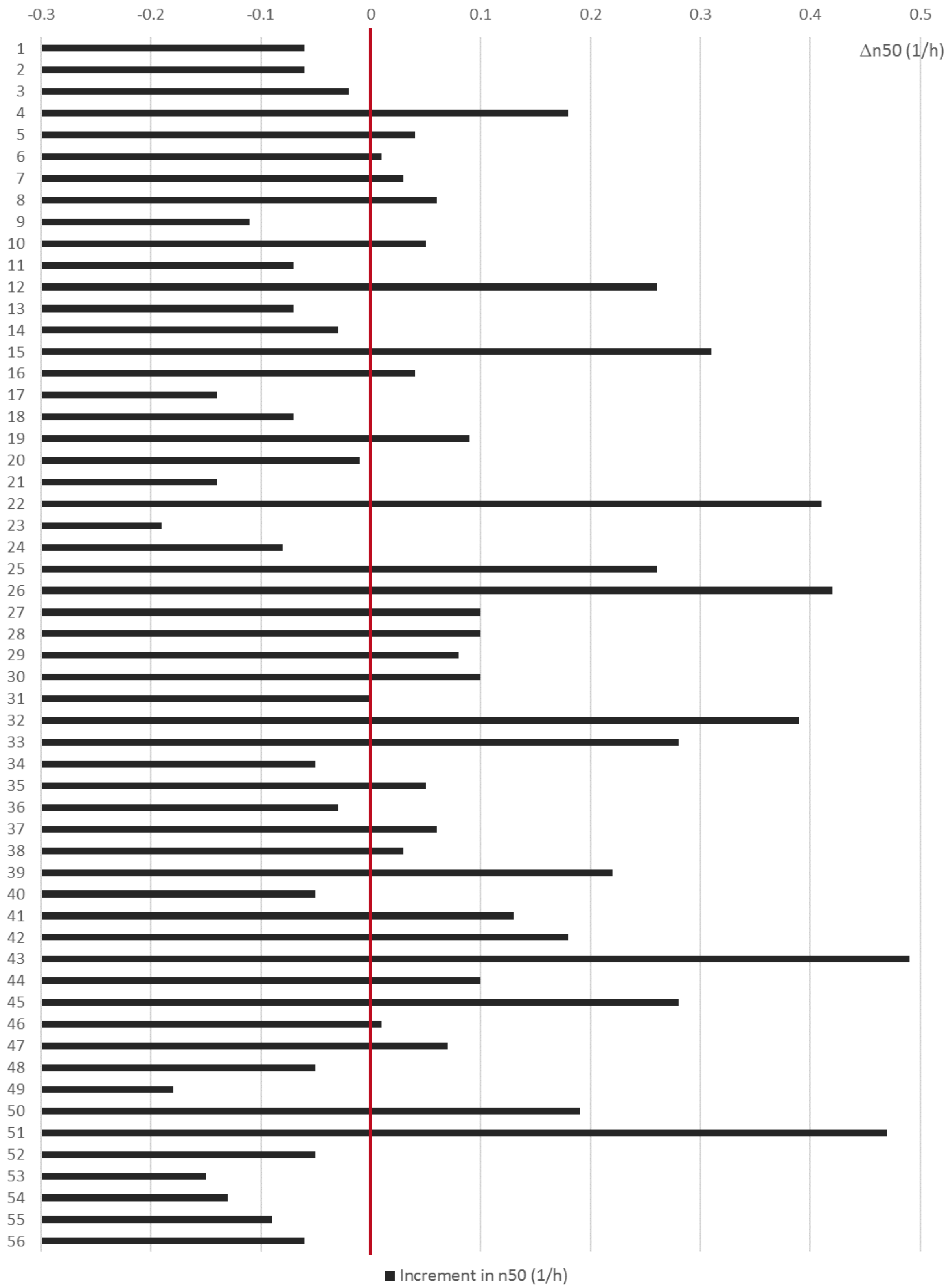


Figure 3-9 Airtightness difference from first and second test for 56 dwellings after Bossard & Menti (2013) Reiss & Erhorn (2003). Red line shows no variation.

The controversy about the results for the integrity of airtightness in energy efficient buildings over time should be explored in future work with contribution also for closed timber frame panel systems.

3.2.5 Moisture assessment

When designing any type of structure, the risk of any type of interstitial condensation and mould growth must be assessed. There are several methods for interstitial condensation risk analysis but the most extensively used is the "Glaser method". This mathematical method assesses the amount of interstitial condensation that can be stored over the coldest month of the year and the amount of water that is able to evaporate in a cold summer.

If the amount of water generated during winter is lower than the evaporation limit in summer, no condensation risk is considered (ISO-13788, 2012). This method can only be applied in assemblies where steady state conditions are met such as standard light weight timber frame (Ojanen & Kumaran, 1996).

However, transient-state condition models with moisture load issues such as construction moisture, driven rain, rising damp or summer condensation cannot be determined using the standard steady-state Glaser calculations (Künzel, 2000).

For these cases, numerical simulation software like WUFI© or Delphin can be used to simulate the transient moisture transfer generated (DIN-4108-3, 2014). Although this method is not reliable for internally highly-insulated buildings, with high sorptive properties, this practice is not recommended on new low-energy buildings where insulation is preferable to be placed on the external side of the wall (Kalamees & Vinha, 2003).

Additionally, these software packages allow for the transient assessment of the mould growth risk on the surface of several building materials according to the Viitanen Mould Index (VMI) methodology (Viitanen & Ojanen, 2007) and further described in Table 3-2.

Table 3-2 Viitanen Mould Index

Mould index	Growth	Description
0	No growth	Spores inactivated
1	Microscop amount on surface	Initial stage of growth
2	<10% microscop coverage	-
3	<10% visual coverage	New spores produced
4	10-50% visual coverage	Moderate growth
5	>50% visual coverage	Plenty of growth
6	100% visual coverage	Very heavy growth

This dynamic time-dependent model suggests a classification for an accumulative mould growth index depending on critical relative humidity for germination, temperature, moisture and material. An example for the accumulative Viitanen Mould Index methodology for a storey ceiling dividing a heated and unheated room is shown in Figure 3-10.

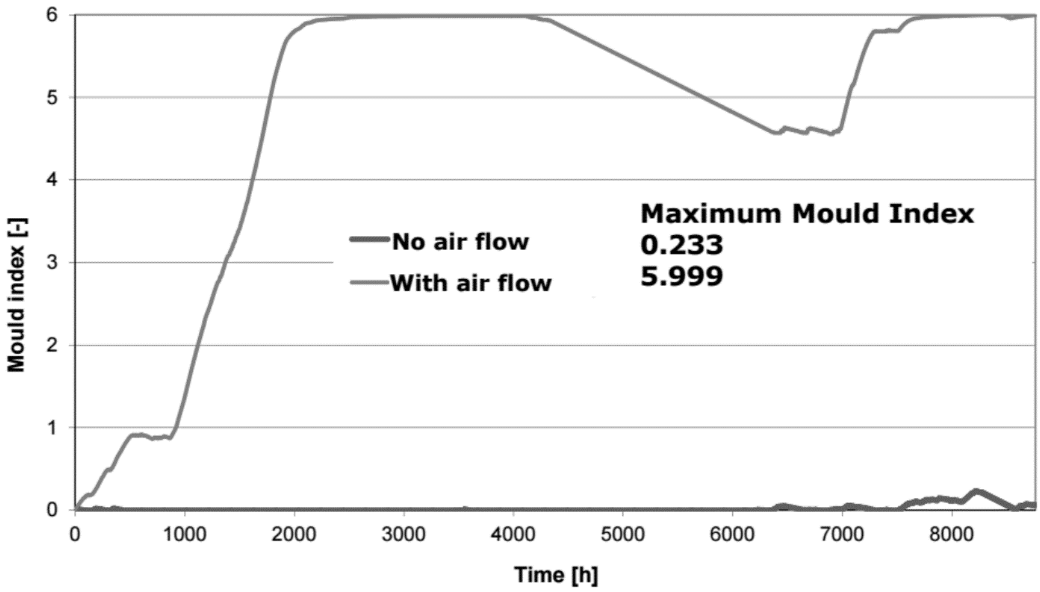


Figure 3-10 VMI for a storey ceiling dividing unheated and heated room

Frequently, the moisture dynamics of the 1-D or 2-D detail is studied for a period of three years where the total water content of the constructions should not show a gradually increasing trend under standard building conditions (Figure 3-11)

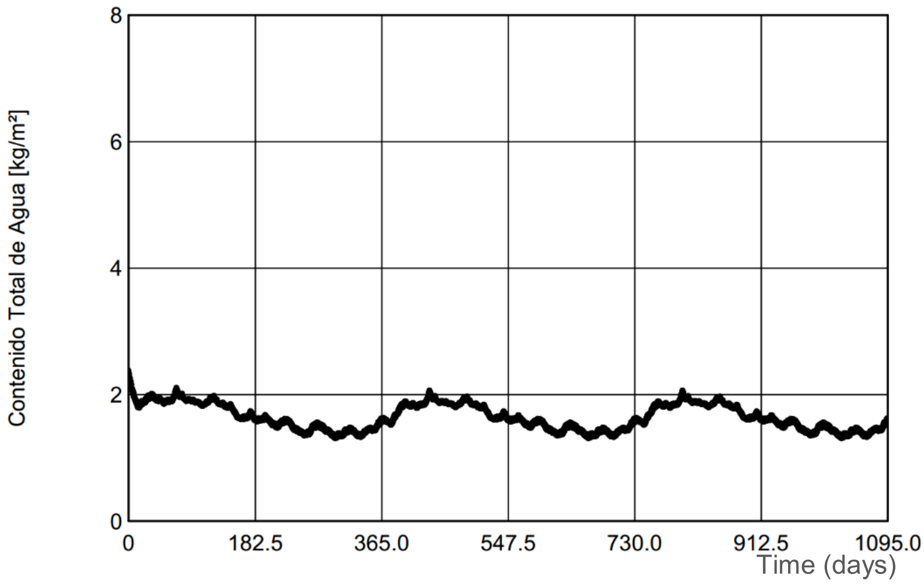


Figure 3-11 Total water content of construction by WUFI pro 5.0

The hygrothermal and durability of X-LAM floor details, on highly-insulated basements for floating houses, was investigated by C. Buxbaum, Seiler, and Pankratz (2007) using numerical methods due to concerns of this particular detail in Passive Houses. This research concluded that moisture accumulation on the XPS insulation layer was low after a 30-year simulation period. However, the author highlighted that any cracks and gaps in the vapour layer could lead to large moisture masses on the floor side. Another experiment undertaken by the same author, in this case for highly insulated I-joist flat roof, concluded that this type of construction, with the right choice of vapour retarders and wind membranes, no moisture was retained within the structure (Christoph et al., 2008). These research conclusions highlight the need for special attention to any hygrothermal issues on highly insulated timber buildings. Attention must should be taken during onsite building stage when exposed construction elements may absorb high-levels of water which cannot subsequently escape through the building fabric.

Further research on this area has successfully proven the accuracy of transient-state numerical methodology to determine time-dependant heat and moisture conditions (Karagiozis et al., 2001; Lengsfeld & Holm, 2007; Marian & Pavol, 2010; Teasdale-St-Hilaire & Derome, 2007).

In this research, one-dimensional heat, air and moisture (HAM) software, WUFI® Pro 5.0, was utilised due to its extended use in this research field to model the proposed timber frame walls in order to predict their hygrothermal behaviour for three different climates. The hourly record of the humidity of each material of the build-up was monitored to assess the condensation risk of each construction system.

As a moisture content assessment reference, the timber frame wall panel is considered risk-free of mould growth if both the structural wood-based materials and other elements of the wall do not exceed more than 20% and 23% moisture content (MC) respectively, more than eight weeks per year. These eight weeks are the addition of any period longer than a week where the MC limit is surpassed (Lamoulie et al., 2012). However, a more conservative range of 18 % and 20% respectively is recommended by WUFI® developers (Karagiozis et al., 2001)

Another commonly used methodology to analyse the risk of mould growth is by determining the temperature factor of the internal surface f_{Rsi} (Equation 3.12). If the internal envelope presents a temperature factor greater than 0.75 the construction is considered safe with no damage to the structure from condensation (BRE, 2006).

$$f_{Rsi} = \frac{\theta_{si} - \theta_e}{\theta_i - \theta_e} \tag{3.12}$$

Where:

f_{Rsi} temperature factor of the internal surface

θ_{si} temperature of the internal surface, in ° C

θ_i indoor temperature, in ° C

θ_e outdoor temperature, in ° C

In case of the presence of gaps in the insulation layer, natural convection may increase moisture load within the building envelope even with just a 3 mm air gap (Siddall, 2009). This issue has been also reported where an external vapour control layer is installed and small air gaps are presented within the internal insulation (Langmans, Klein, & Roels, 2013).

The thermal parameters involved in low-energy building design have been reviewed. In the next section, the research has collated all of the findings to propose two thermally efficient closed panel timber frame configurations.

3.3 Description of Timber Frame Wall Build-Ups

Thermal transmittance is frequently the governing design parameter for low-energy timber fame walls (TRADA, 2011). Hence, a suggestion for a timber frame build up is based primarily on the thermal resistance of the component. Once all the hygrothermal requirements are fulfilled, the research concentrates on the structural performance.

In this section three different walls are presented a standard “six inches” timber frame panel used as a benchmark scenario and two advanced timber frame systems using I-joists and dual insulated studs respectively. Although the type of stud on the walls is maintained, insulation and sheathing materials were altered.

The depth of the advanced timber frame closed panels presented varies in order to obtain different tabulated U-values and hence, better thermal transmittance. The studs are presented in three different formats of 195 mm, 245 mm and 300 mm. Stud spacing, in order to minimise timber fraction, is set at 610 mm centres. Sheathing of the frame is also presented in two different formats: OSB sheathed both sides and OSB sheathing on the internal side and rigid wood fibre board on the external side.

In terms of internal finishing, the study considers two scenarios; a service aired cavity and an insulated service cavity both with plasterboard finish. In terms of external finishing, in all cases a ventilated façade, also called a cavity or drained wall, with no significant impact on thermal performance was used (CIBSE, 2006; Feist et al., 2007). Further details of the panel build-ups are given in Table 3-3.

The technical specifications of the materials used in the research are based on commercially available products and for the advanced panel systems, based on the actual specimens tested. A summary of the materials, types and related standards is given in Table 3-4. Figure 3-12 provides a comparative image of the different I-joist and K2 materials utilised in the timber frames.

Table 3-3 Summary of timber fame build-ups.

(units in mm)		Benchmark	I-Joist PassiveWall	Dual frame K2
Studs	Depth	150	195 245 300	(45+60+90) (45+110+90) (45+165+90)
	Width	45	47	45
	Spacing	610	610	610
Insulation		Mineral wool Wood fibre		
Sheathing	Internal side	OSB/3 9mm	OSB/3 OSB/3	9mm 15mm
	External side	OSB/3 9mm Wood fibre 60mm		
Membranes	Airtightness	VCL	VCL Taped OSB	
	Windproof	Breathable n/a		
Insulated	Service cavity	Mineral wool 45mm		
External	finishing	Ventilated façade		

Table 3-4 Timber frame material properties

Material	Type / Grade	Norm	Conductivity λ_d (W/mK)
Wood products	C16	BS EN 338	0.13
	OSB/3	BS EN 300	0.13
	I-Joist	ETA-05/0224	0.13-0.18
	LVL	BS EN 14374	0.13
Insulation	Mineral wool	BS EN 13501-1	0.032
	Flexible wood fibre	BS EN 13171	0.038
	Rigid wood fibre	BS EN 13171	0.046
	XPS	BS EN 13164	0.037
Membranes	VCL	BS EN 1849-2 BS ISO 12572	n/d
	Windproof	BS EN 12310-1 BS ISO 12572	n/d
	Tapes	BS EN 13984	n/d

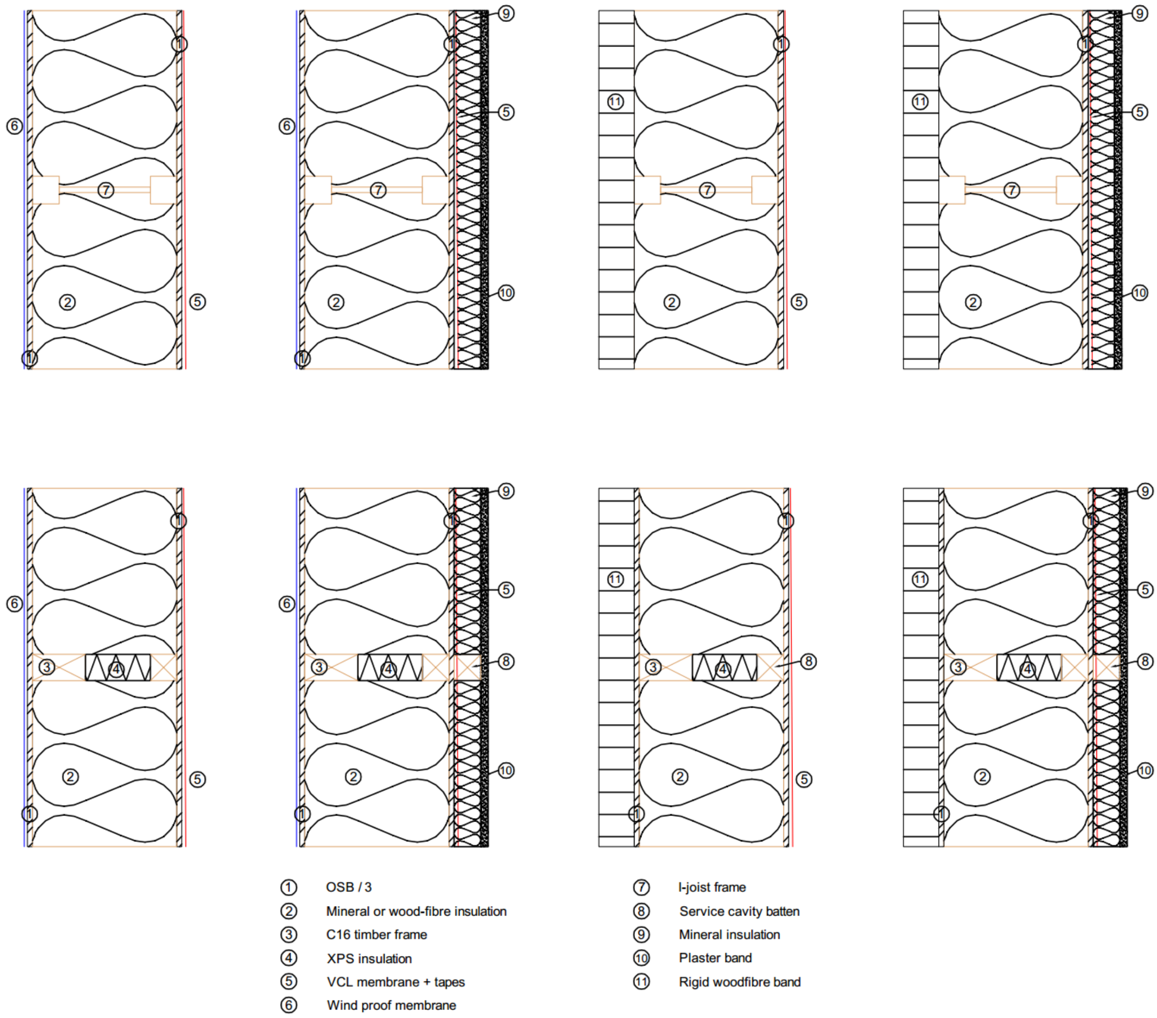


Figure 3-12 Comparative illustration of the I-joist and K2 timber frames

Timber frame and sheathing metal fixings are not included in this section as the impact can be considered negligible in the overall thermal resistance of the timber frame wall system (Anderson, 2006 Pokorny et al., 2009). This agreed with the research exercise carried out in section 3.6.2.

The boundary between the timber frame system and the foundation is the sole plate connection. The components and geometries of this detail, for each wall build-up proposed, is a thermally improved detail from standard construction practice and is tested for structural optimisation purposes in chapter 4. Although the standard dimensional tolerances for this type of connection are recommended to be ±10 mm (TRADA, 2013), the potential gap created would cause negative thermal effects at a critical point. For this reason, the tolerance suggested in the research is ±5 mm and the gap filled with flexible-type foam. However, in order to minimise the linear thermal bridge of this critical connection, a modified version is also suggested. The benchmark timber frame is also considered, in terms of on-site connectivity, as a closed panel system.

Further detailed information and drawings for all possible timber frame build-up combinations, including the sole plate base detail fixing, is provided in the following sub-sections and in the Appendix II.

3.3.1 Description of timber frame components

Timber frame panels are fixed together in factory or off-site to make a timber frame wall. The components used in this process are typically framing material (studs, rails, lintels and noggins), sheathing, membranes and fasteners. Although timber frame structures can be erected on any type of foundation, the most common substrates are suspended floor systems over concrete blocks or concrete slab (TRADA, 2013). This must be also considered within the thermal bridge calculations.

Framing material

Structural framing studs, runners or rails and lintels, made of softwood shall be individually graded with a strength class C16 or better according to BS EN 338 (BSI, 2003). Although structural calculations shall determine section sizes and grades, the most common timber sizes in UK are 38 x 89(90) mm and 38(45) x 145(150) mm what is typically known as "four and six inches" with nominal sizes on brackets (TRADA, 2007).

Studs can be also made of engineered wood products (EWP) such as Glulam, Laminated Veneer Lumber (LVL) or I-Joist. In this research, one of the advanced closed panels systems presents I-joist as stud and top and bottom rail framing members.

The moisture content for use in buildings under service class 1 and 2 shall not be greater than 20%. A preferable and recommended moisture content of the timber when installed, in order to reduce shrinkage, is 12-15% MC (Williamson, 2002). The timber should also be marked with its moisture content followed by the letters DRY (if naturally dried) or KD (if kiln dried).

Structural softwood used on the wall frame, which is of durability class 5 or lower, should be treated for both durability and for insurance purposes (such as NHBC compliance scheme). The treatment is essential in the following areas:

- Sole plates
- Joinery resting directly on the damp proof course (DPC)
- Timber used for external cavity barriers (battens for ventilated cladding).

Sheathing material

Eurocode 5 states that only wood-based board products complying with EN 13986 (BSI, 2004) and EN 12871 (BSI, 2010) are suitable for racking resistance of timber frame walls. However, PD 6693-1 states that plasterboard-only sheathed timber frame walls can be designed for racking resistance. Nevertheless, the use of a wood-based panel is recommended (TRADA, 2006). For external walls, the panel should be suitable for structural use in service class 2 conditions. Therefore, the wood-based materials which are deemed fit for purpose is shown in Table 3-5. However, OSB/3 is by far the most common sheathing material in the UK.

Table 3-5 Suitable sheathing materials for external racking wall panels

Material		Service Class
LVL		2 / 3
Plywood		
	636-2	2
	636-3	3
OSB		
	OSB/3	1 / 2
	OSB/4	1 / 2
Particle board		
	P5	1 / 2
	P7	1 / 2
Hardboard		
	HB.HLA1	1 / 2
	HB.HLA2	1 / 2
Medium density board (MDF)		
	MBH.HLS1	1 / 2
	MBH.HLS2	1 / 2
Medium density fibre (MDF)		
	MDF.HLS	1 / 2

Fasteners

Metal dowel-type fasteners are used to connect the timber frame members and to fix the sheathing panel to the frame. The fasteners used must present CE marking in accordance with EN 14592 (BSI, 2008a). The lateral load carrying capacity depends on the bending behaviour of the fastener and the bearing stresses of the timber or panel in contact along the shank of the fastener. Friction and axial pull-out resistances may contribute also to the lateral shear capacity. In line with the CE marking, the fastener manufacturer must declare values for the characteristics shown in Table 3-6.

Table 3-6 Characteristics of fasteners according with CE marking

Parameter	Fastener type
Wire specification	All
Geometry	All
Yield moment	All
Withdrawal resistance	Nails / Staples / Screws
Head pull-through	All
Tensile capacity	Nails / Screws
Head twist off	Screws
Torsional resistance	Screws
Corrosion resistance: type and thickness	All

Eurocode 5 provides an analytical model based on Johansen (1949) equations but slightly modified to consider also the combination of friction forces and withdrawal in the connection (commonly referred as *"rope effect"*). The fastener's shear capacity prediction, achieved by this analytical method, is considered satisfactory by numerous studies.

Membranes

Two types of membranes are commonly used for modern timber frame systems: a vapour control layer (VCL) membrane on the internal side of the insulation and a windproof breathable membrane on the external side of the insulation. Properties for these materials are given in Table 3-7. The mission of these membranes is to provide airtightness to the system (vapour retarders) and to control moisture through the building fabric (wind barriers). The equivalent air layer thickness, S_d, measured in metres, indicates the thickness of a static layer of air that presents the same water vapour resistance. This parameter ordinarily relates to the *"breathability"* of the building envelope.

The VCL membrane can be eliminated if an airtight sheathing material is taped at every joint. However, the economy of these membranes and the issues regarding the permeability of thin OSB (Peper, 2014) make membranes a popular choice.

Table 3-7 Technical specifications for membranes (DAFA® systems)

	Sd-Value (m)	Tear resistance (N)	Tensile strength (N/50mm)	Mass (g/m²)
HiFoil adaptive retarder	0.2 - 10	59	130	100
AluFoil vapour barrier	300	200	250	150
Difoil vapour retarder	2	90	90	100
ProFoil vapour barrier	140	60	130	184
WindFoil wind barrier	0.02	130	95	105
UV Façade wind barrier	0.12	n/d	170	160

On the other hand, a permeable membrane, such as a wind barrier, can be disregarded if another suitable material protects the timber frame from moisture. In this case, rigid forms of water-repellent insulation can be placed instead.

There are bio-materials like high density rigid wood fibre or expanded cork insulation boards which are suitable to protect the timber frame. Other non-natural insulation products that can perform as a wind barrier are EPS, XPS or foamed glass. These products are frequently installed at the sole plate level as they are more durable than the natural insulation materials described previously.

3.3.2 Benchmark wall panel

There are over seventy timber frame manufactures registered as members of Timber Research and Development Association (TRADA), the UK´s largest organisation dealing with timber and wood products. Although this is a large number of fabricators, the vast majority of them produce standard four or six inches timber frame kits in a very similar approach (Figure 3-13).

Figure 3-13 External open panel timber frame wall

In this research, four companies based in Scotland were visited in order to propose a documented benchmark timber frame scenario:

- RTC Timber Systems, Elgin.
- Oregon Timber Frame, Selkirk.
- CCG Construction Group, Cambuslang
- Alexanders Timber Design, Troon

A summary of the characteristics of the materials commonly used for these companies are defined in Table 3-8 below:

Table 3-8 Materials to fabricate benchmark timber frame

	RTC	Oregon	CCG	ATD
Studs	45x145	38x140	38x140	45x145
Sheathing	9mm OSB/3 2 layers	9mm plywood 2 layers	9mm OSB/3 2 layers	9mm OSB/3 2 layers
Insulation	Mineral wool	Mineral wool	Rock wool	Mineral wool
Membranes	Yes	Yes	Yes	Yes

Regarding the sole plate base fixing detail, this differs considerably if the timber frame wall is manufactured as open or closed panel. Figure 3-14 shows two examples of sole plate details for standard closed panel systems. However, these details are seldom used in practice as most of the production manufactured in UK is open panel.

Figure 3-14 Sole plate detail for closed panel (in red)

As an alternative, two common open panel sole plate details are described for the purpose of the research: a sole plate on top of a concrete raft foundation and a timber frame on top of a suspended timber floor cassette (Figure 3-15). This facilitates a comparison between the current foundation type practice and the proposed advanced timber frame systems.

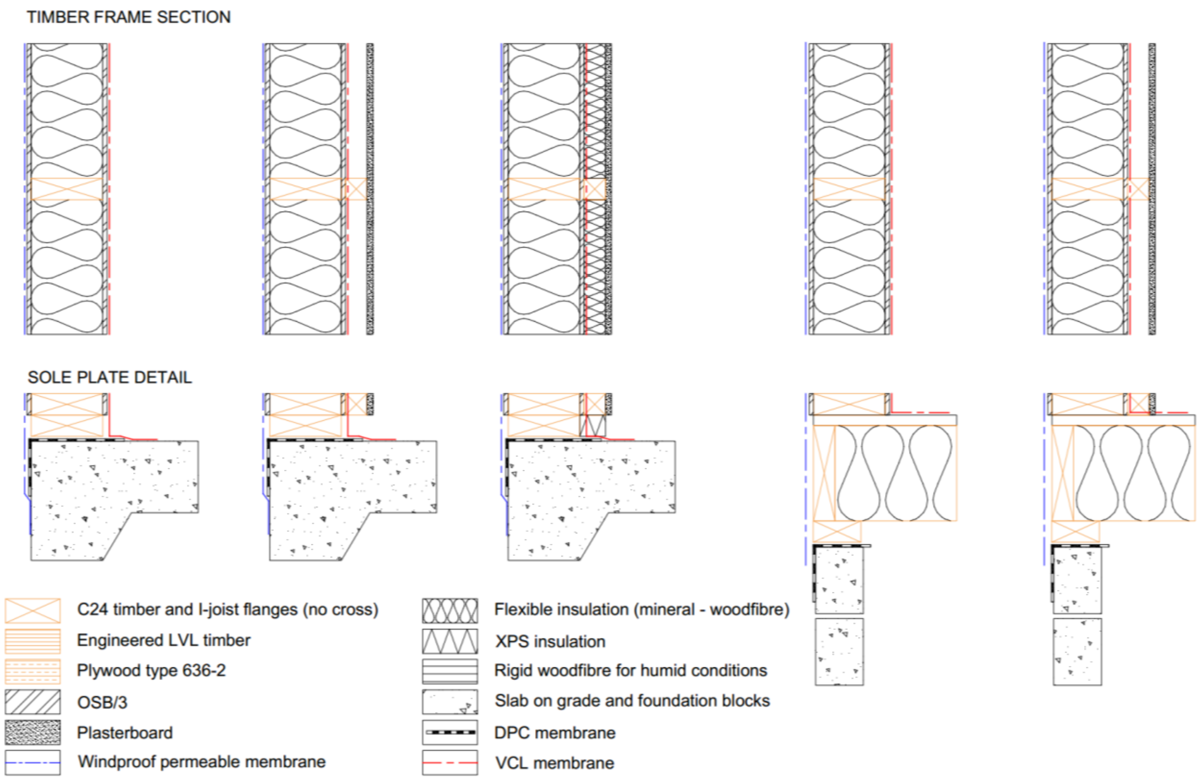


Figure 3-15 Benchmark timber frame elevation and section drawings

3.3.3 I-joist PassiveWall™ – RTC panel

The first advanced closed panel system proposed was studied at early stages of this research as part of a Knowledge Transfer Partnership (KTP) project between Edinburgh Napier University and the Scottish company RTC Timber Systems. Although, the KTP project was not successfully completed due to the liquidation of the company, the concept of a super-insulated timber frame made of I-joist was included in this research.

The RTC PassiveWall™ was an industrialised panel to be installed as wall, floor or roof cassettes. The wall panel was formed by I-Joist of 2 different depths, 245mm and 300mm, one 9mm OSB board at each side of the panel and insulated core of recycled mineral wool fibre glass. The panels had a vapour barrier on the inside and a breathable membrane on the outside. Panels were simply fitted by the fixing system called PassivePlate at foundation and inter-storey level (Figure 3-16).

Figure 3-16 RTC PassivePlate on foundation and on lintel junction

The most common panel dimensions were 263 mm and 318 mm in width, wall lengths up to 12 m and 2900 mm in height if transported flat or 3950 mm if transported vertically (toast rack) on the trailer. These are common maximum dimensions limited by transportation. This timber frame panel also includes in this research some modifications to the original PassiveWall™ design for thermal optimisation. The core insulation can be made using mineral wool or wood fibre. This may be insufflated rather than being semi-flexible batts or rolls in order to fill the wall panel entirely around the I-profile. Alternatively, if insulation products are batts or rolls, it is recommended to pre-insulate the web of the I-joist by gluing the same insulation material. The purpose is to have a rectangular compound stud to avoid air pockets in the panels. The sole plate, in slab on grade and raft foundations, is formed by LVL instead of plywood to improve the structural performance with a packer of XPS insulation glued on top to form the tongue and groove system with the wall. In the case of suspended timber floors, the XPS packer is glued on top of the floor cassette. An alternative to OSB sheathing at each side of the panel is also presented to improve the thermal performance. The external OSB sheathing board is substituted by 60mm of rigid high-density wood fibre board which also it acts as a windshield barrier. Finally, the internal cavity for services can be insulated to improve the overall U-value. In this case, it is recommended to place the internal battens horizontally. A section of the different wall panel configurations is shown in Figure 3-17 where core insulation maybe mineral or wood fibre wool.

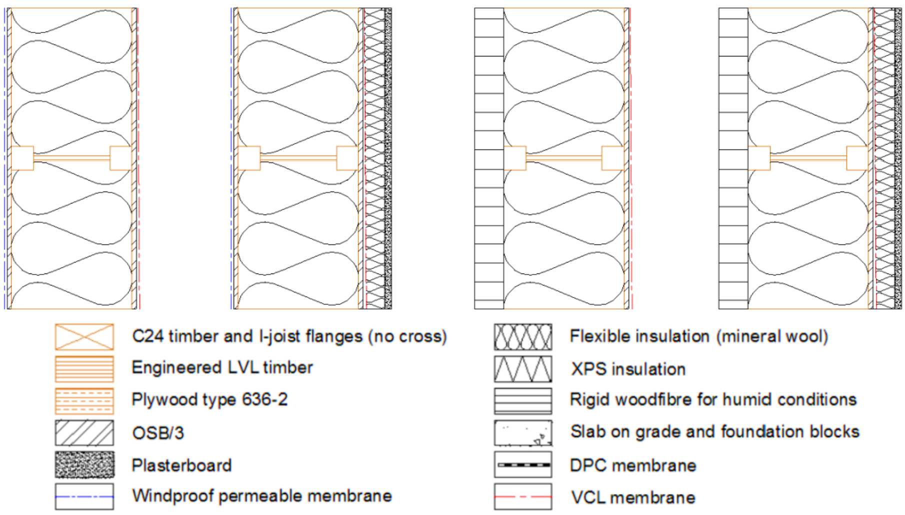


Figure 3-17 RTC wall build-up for different core insulation

The different panel configurations and materials for the RTC timber frames are described in Table 3-9.

Table 3-9 Materials to fabricate RTC timber frame panels

	RTC I-beam panel materials		
Studs	47x195	47x245	47x300
Sheathing	9mm OSB/3 9mm OSB/3	9mm OSB/3 Wood fibre 60mm	
Insulation	Mineral wool	Wood fibre wool	
Membranes	VCL + Windshield	VCL only	
Service cavity	Insulated		

Similar to the benchmark wall panel, the two sole plate details used in this research are timber frame on top of concrete raft and timber frame on top of suspended timber floor cassette. In this case rigid wood fibre board insulation is placed instead of OSB/3 sheathing in the outside. It is also recommended to protect the edge of the foundation with XPS rigid insulation board (Figure 3-18).

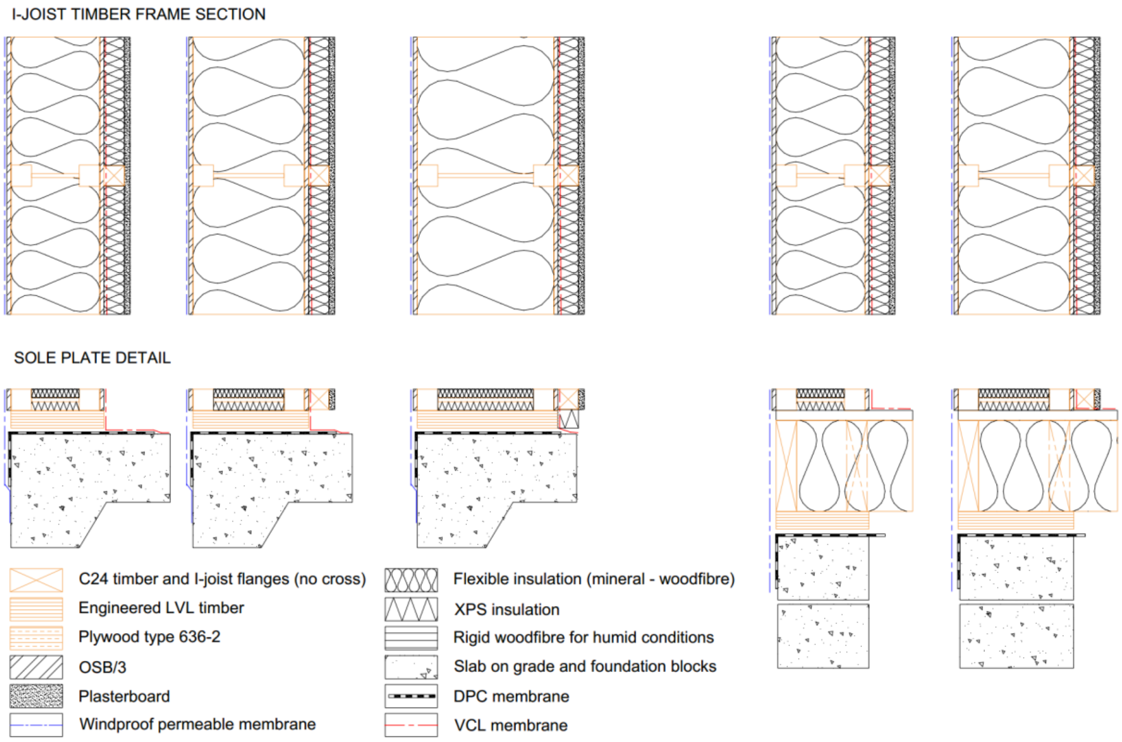


Figure 3-18 RTC I-beam timber frame elevation drawings

3.3.4 Dual Frame – K2 panel

The second advanced closed panel system proposed was also studied at early stage of the research as part of the European Regional Development Fund (ERDF) project titled Wood Products Innovation Gateway between Edinburgh Napier University and the Scottish company Kraft Architecture. This project focused on the feasibility of Scottish home-grown timber for the fabrication of advanced closed panel systems. However, in this research, the source of the timber is not considered and the grade of the framing material is classified as C24.

The Kraft architecture K2 wall system is a dual frame comprising of external and internal studs manufactured from sawn lengths of structural timber of standardised crosssectional sizes. In order to provide in-plane rigidity, sheathing material is secured to both the external and internal frame using pneumatically driven mechanical fasteners. The external frame forms the primary load bearing structure of the wall and is therefore sheathed with a structural board material, typically OSB, to provide the required racking capacity.

The internal frame is typically non-load bearing, carrying only the internal finishes and therefore can be sheathed using 12.5mm plasterboard in order to provide fire resistance and an internal finishing. In situations where a high degree of racking resistance is required, OSB may also be secured to the internal face in addition to the plasterboard. The cavity between the two sheathing materials can be of varying depth according to insulation requirements. The completed wall assembly is sealed top and bottom using 22mm weather and boil proof (WBP) plywood plates (Figure 3-19).

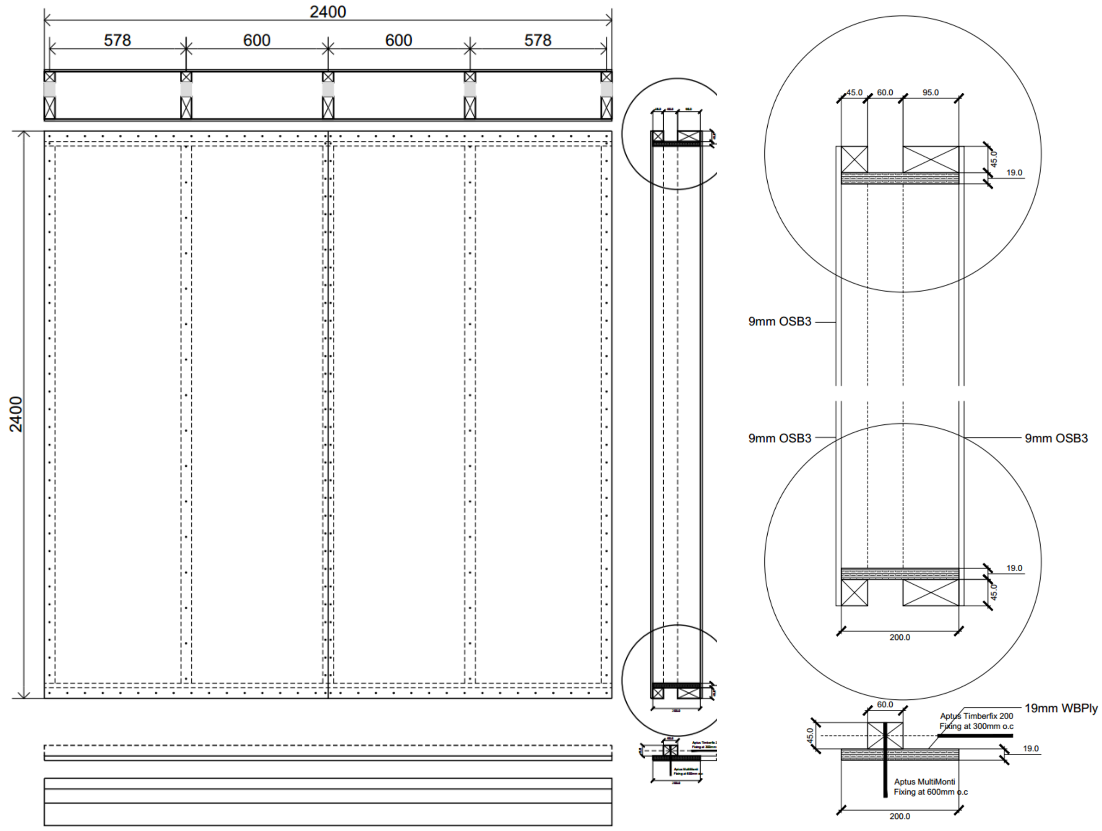


Figure 3-19 K2 panel and sole plate detail

The wall is secured to the foundation substrate using a combination of smooth nail and screw type fasteners. These act collectively to provide resistance to sliding and, if they possess a sufficient withdrawal capacity, overturning forces. However, a modification to this sole plate detail has been considered in this research due to structural issues of the slant fasteners as EC5 minimum spacing distances may not be fulfilled if the angle and the penetration depth is not adequate. The timber frame panel also includes some modifications to the original K2 design for thermal optimisation. The core insulation can be made using mineral wool or wood fibre in the form of batts, rolls or insufflated. The insulation between internal and external frame is glued XPS to optimise manufacturing.

The sole plate, in concrete slab on grade and raft foundations, is formed by plywood with a timber packer glued to it to facilitate the line-up and fixing of the wall to the sole plate. In the case of suspended timber floor cassette construction, the plywood over the concrete blocks is substituted by LVL whilst the timber packer is fixed directly on top of the floor. Contrary to the I-joist timber frame, the 60 mm of rigid high-density wood fibre board is added to the external OSB/3 sheathing to provide racking resistance to the external load bearing frame. The rigid insulation is placed nonetheless to evaluate the thermal improvement.

The internal cavity for services can be insulated to improve the overall U-value. In this case, it is recommended to place the internal battens horizontally. A section of the wall panel configurations is shown in Figure 3-20 where core insulation is glued XPS.

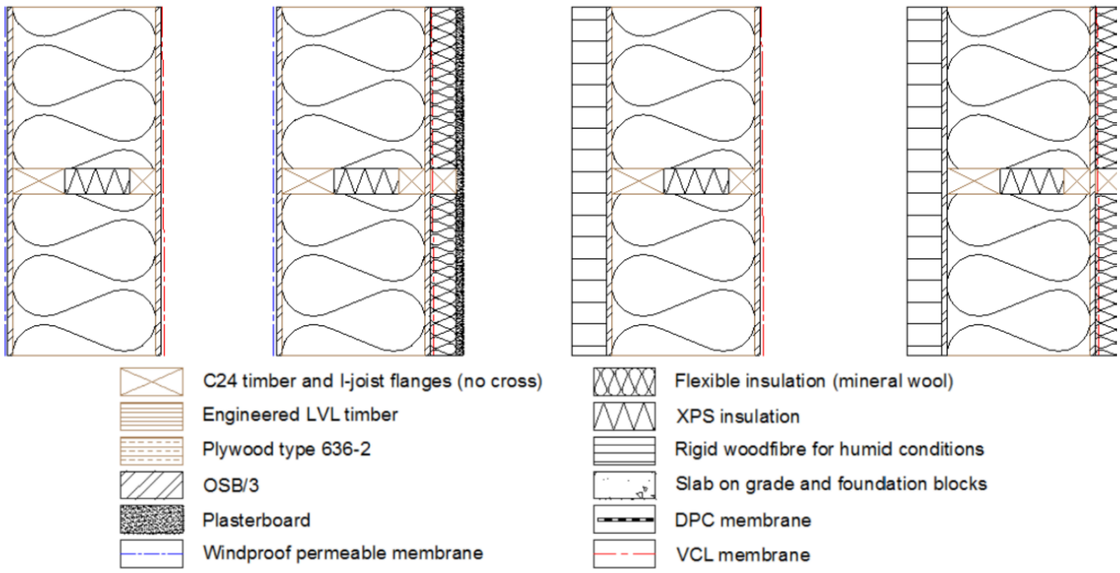


Figure 3-20 K2 wall build-up for different core insulation

The different panel configurations and materials, for the purpose of this chapter, are described in Table 3-10. Note that a 9 mm sheathing board was also included if external wood fibre board was used.

Table 3-10 Materials to fabricate K2 timber frame panel

K2 wall			
Studs	45x195(135)	45x245(135)	45x300(135)
Sheathing	9mm OSB/3 9mm OSB/3	9mm OSB/3 9mm OSB/3 Wood fibre 60mm	
Insulation	Mineral wool	Wood fibre wool	
Membranes	VCL + Windshield	VCL only	
Service cavity	Insulated		

Similarly to the benchmark wall panel, the two sole plate details used in this research are timber frame on top of concrete raft and timber frame on top of suspended timber floor cassette.

In case rigid wood fibre board insulation is placed on the OSB/3 sheathing board in the outside, it is recommended to protect also the edge of the foundation with XPS rigid insulation board (Figure 3-21).

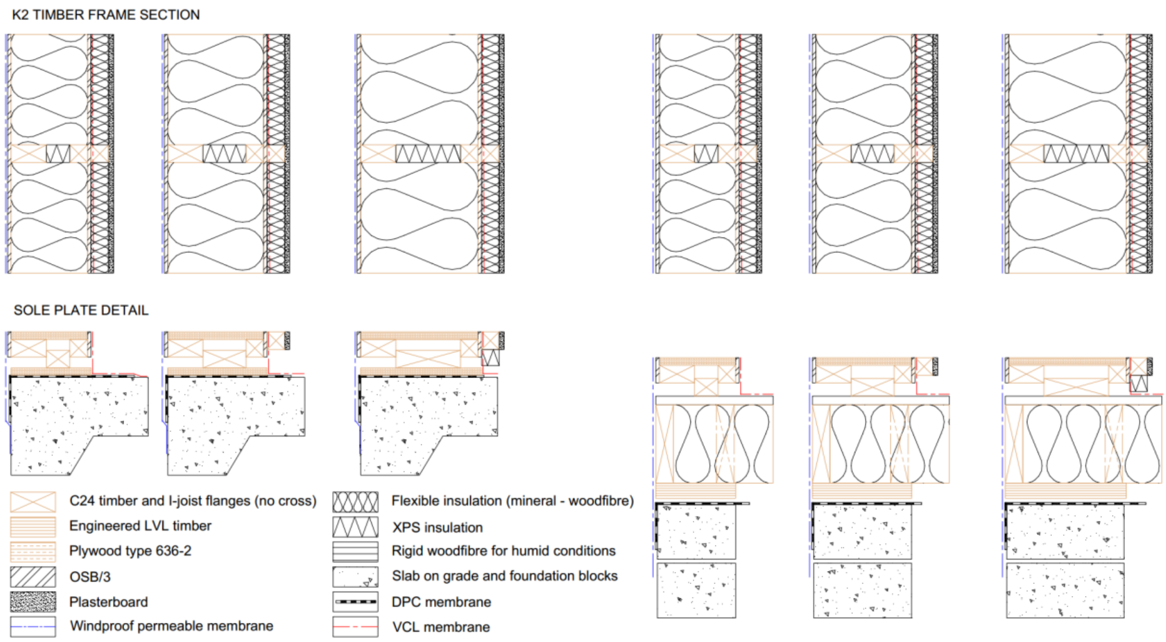


Figure 3-21 K2 timber frame elevation drawings

Once all the timber frame walls were well-defined, the corresponding hygro-thermal investigation was carried out. Prior to undertaking the study, a comparison of different thermal modelling tools was performed in order to determine the preferred software application.

3.4 Comparison of FEA Software for Thermal Simulation

There are different standards used to numerically determine linear thermal transmittance in building construction (Anderson, 2006). In the UK, the BRE 497 document describe the conventions for calculating thermal bridges but it is not valid for Passivhaus as it considers internal dimensions rather than external (Ward & Sanders, 2007). The procedure recommended by the Passivhaus Institut to determine thermal bridges is the EN ISO 10211 (ISO, 2007).

Three different software packages, for two-dimensional simulation of steady-state heat transfer for building physics, were evaluated to assess the accuracy of their engines with regard of the ISO 10211 method (Table 3-11). A fourth 2-D method, a German online platform - www.u-wert.net, developed by Dr Ralf Plag and validated in several studies (Capener at al., 2014; Weber et al., 2015) was also evaluated. Furthermore, these values were compared with the analytical simplify method (BSI, 2008b).

Table 3-11 List of 2-D simulation heat transfer software

Software	License	Type	Country	Validation	CAD import
Therm 7.4	Free	Heat	US	ISO 10211-2:2007 ISO 10077-2:2012	Yes
Flixo	Commercial	Heat	Switzerland		Yes
HTFlux	Commercial	HAM	Austria		Yes
U-wert.net	Free	Heat	Germany		No

The heat transfer analysis process flowchart for the four programs is almost identical. A schematic representation is shown in Figure 3-22. The maximum percent energy norm error setting for EN 10211 compliance is documented to 2%. This had to be set manually in Therm 7.4.

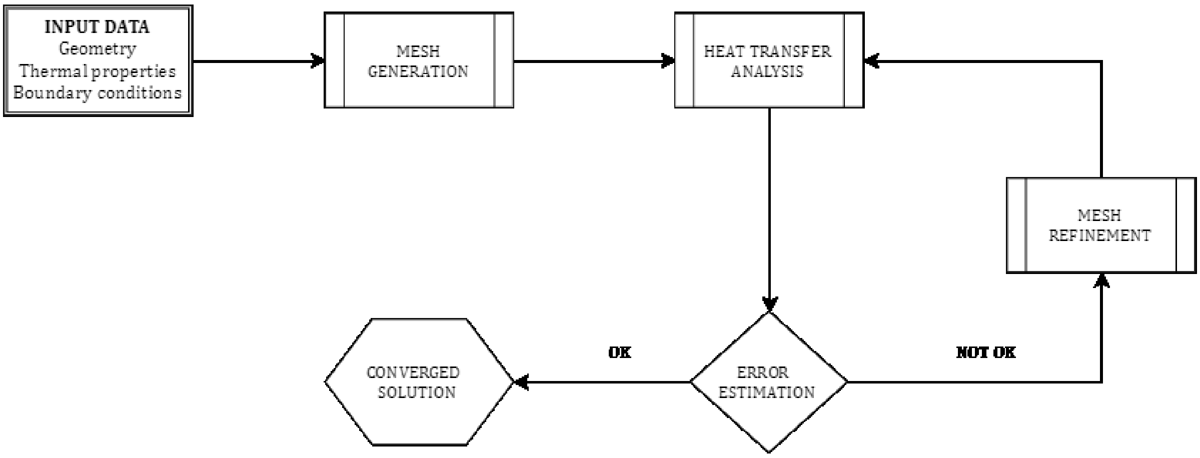


Figure 3-22 2D Heat transfer procedure flow chart

3.4.1 Materials and geometry reference values

In order to obtain a valid result for the comparison between the different software packages analysed, the thermal properties and the geometry of the external wall details must be equal in all three packages. Table 3-12 presents the thermal conductivity of the materials employed in the wall and their source. It is important to bear in mind that the thermal conductivities for the wood-based materials were considered isotropic with physical property declared perpendicular to the grain.

Table 3-12 Thermal properties of timber frame wall materials

Material	Thermal conductivity λ_D (W/mK)	Vapour diffusion μ (dry)	Source
Timber C16	0.130	50.0	(Dinwoodie, 2000)
OSB/3 board	0.130	50.0	(Williamson, 2002)
Mineral wool	0.032	1.0	BBA AC 95/3212
XPS	0.037	150.0	BBA AC 95/3102
Woodfibre board	0.046	10.0	DoP No 01-0006-03
Wood wool	0.038	2.0	(STEICO, 2009)
LVL I-flange	0.130	50.0	(STEICO, 2009)
MDF I-web	0.180	30.0	(STEICO, 2009)

The boundary conditions of the wall were set as for the standard surface film resistance coefficients given by Feist (2006) and in accordance to BSI (2008b). In this particular case, for the different wall sections, three different boundary conditions were maintained and specified:

- Adiabatic condition, i.e. no heat transfer across the thermodynamic system and its surroundings (Equation 3.14)
- internal film resistance at 20 ºC and 0.13 m2K/W
- external film resistance at 0 ºC and 0.13 m2K/W

$$\left(\frac{\partial T}{\partial n}\right)_w = \dot{q}_w = 0 \tag{3.14}$$

Where:

$\frac{\partial T}{\partial n}$ normal derivative of the temperature at the wall

$\dot{q}_w$ heat transfer on the wall

The geometries of the three different wall build ups are shown in Figure 3-23. It corresponds to a section of one intermediate stud at spacing 610mm centre to centre. The geometry is symmetrical and the resulting isotherms for the wall section with higher U-value are, as expected, perpendicular to the adiabatic boundary conditions.

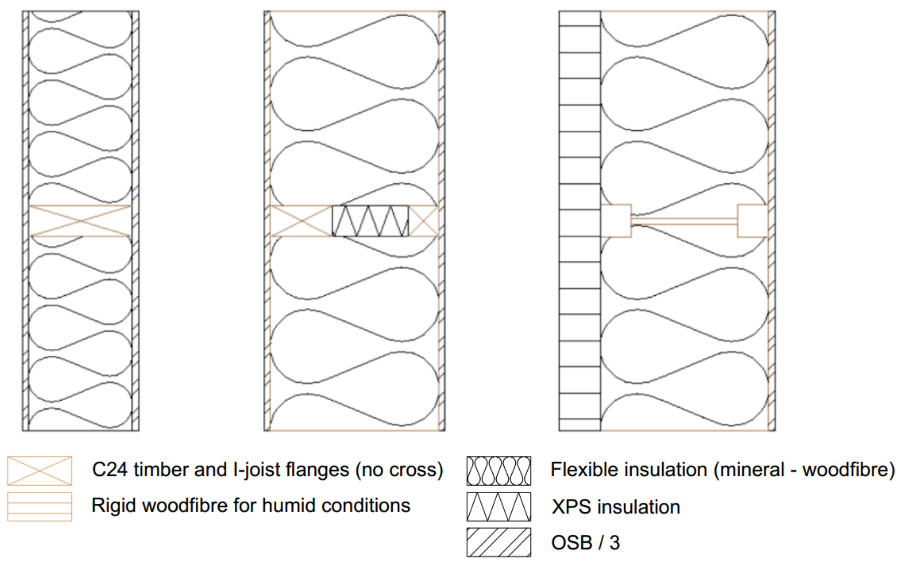


Figure 3-23 Geometry of walls studied for U-value correlation

3.4.2 Results

The U-value calculations according to the analytically simplified BSI method (BSI, 2008b) were determined by three independent numerical tools in order to corroborate this methodology independently of the tool selected. The PHPP also integrates a U-value calculation tool which complies with ISO 6946 (Feist et al., 2007) and was also used in the study (Figure 3-24). The calculation results from the different programs are presented in the Appendix III.

Assembly no.	Building assembly description	Interior insulation?
1	Benchmark	

Heat transfer resistance [m²K/W] interior R_{si}: 0.13; exterior R_{se}: 0.13

	Area section 1	λ [W/(mK)]	Area section 2 (optional)	λ [W/(mK)]	Area section 3 (optional)	λ [W/(mK)]	Thickness [mm]
1.	OSB/3	0.130					9
2.	Knauf Earthwool	0.032	45x150 KVH C24	0.130			150
3.	OSB/3	0.130					9
	Percentage of sec. 1	93%	Percentage of sec. 2	7.4%	Percentage of sec. 3		Total 16.8 cm

U-value supplement: W/(m²K) U-Value: 0.233 W/(m²K)

Assembly no.	Building assembly description	Interior insulation?
2	K2 panel	

Heat transfer resistance [m²K/W] interior R_{si}: 0.13; exterior R_{se}: 0.13

	Area section 1	λ [W/(mK)]	Area section 2 (optional)	λ [W/(mK)]	Area section 3 (optional)	λ [W/(mK)]	Thickness [mm]
1.	OSB/3	0.130					9
2.	Knauf Earthwool	0.032	90x45 KVH C24	0.130			90
3.	Knauf Earthwool	0.032	110x45 XPS 37	0.037			110
4.	Knauf Earthwool	0.032	45x45 KVH C24	0.130			45
5.	OSB/3	0.130					9
	Percentage of sec. 1	93%	Percentage of sec. 2	7.4%	Percentage of sec. 3		Total 26.3 cm

U-value supplement: W/(m²K) U-Value: 0.135 W/(m²K)

Assembly no.	Building assembly description	Interior insulation?
3	RTC Passivewall	

Heat transfer resistance [m²K/W] interior R_{si}: 0.13; exterior R_{se}: 0.13

	Area section 1	λ [W/(mK)]	Area section 2 (optional)	λ [W/(mK)]	Area section 3 (optional)	λ [W/(mK)]	Thickness [mm]
1.	WoodFibre board	0.046					60
2.	Knauf Earthwool 32	0.032	LVL 47x45	0.130			45
3.	Knauf Earthwool 32	0.032			MDF 9x155	0.180	155
4.	Knauf Earthwool 32	0.032	LVL 47x45	0.130			45
5.	OSB/3	0.130					9
	Percentage of sec. 1	91%	Percentage of sec. 2	7.7%	Percentage of sec. 3	1.5%	Total 31.4 cm

U-value supplement: W/(m²K) U-Value: 0.115 W/(m²K)

Figure 3-24 U-values of sample walls according to EN 6946 from PHPP 8.5

The U-values calculated, for all of the methods, were equal within three decimal places. The values provided by the analytical PHPP methodology were almost the same (Table 3-13). These results must be interpreted with caution because no statistical analysis was considered. However, the results confirm that for timber frame wall panel thermal transmittance calculation, the simplified approach given by EN ISO 6946 is adequate.

Table 3-13 U-values compared with four different 2D FEA software

U-value (W/m^2K)	Analytical (EN 6946) PHPP	2D FEA (EN ISO 10211) Therm	Flixo	HTFlux	U-wert
Benchmark	0.233	0.232	0.232	0.232	0.232
K2 wall panel	0.135	0.134	0.134	0.134	0.134
RTC I-joist	0.115	0.116	0.116	0.116	0.116

Although, the outcomes of the five tools were almost identical, the U-wert has been the preferred tool for the subsequent U-value calculation of the different wall combinations due to the additional thermal information provided (Figure 3-25). These are: interior surface temperature, moisture content, thermal phase shift and heat storage capacity and are presented in section 6.1.

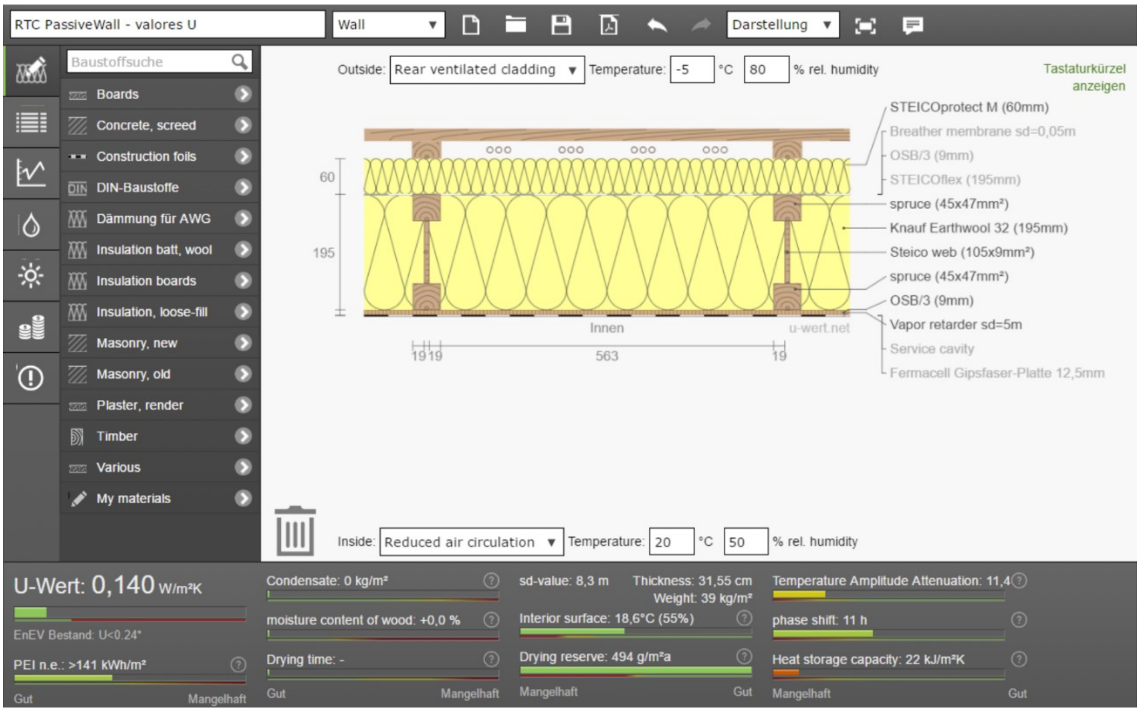


Figure 3-25 Screenshot of U-Wert.net with an I-joist wall analysis

Similarly, the FEA software HTFlux has been chosen as a preferred tool for thermal bridge calculation due to the additional capabilities with regard to condensation risk analysis (HTFlux, 2015).

3.5 Thermal Transmittance of Walls

The U-value baseline, for the creation of a wall build-up benchmark, was designed to comply with the minimum value stablished by building regulations when this research section started (2013) of 0.25 W/m^{2}K. For the object of this thesis, the minimum performance was set to maximum Passivhaus standard recommendation of 0.15 W/m^{2}K.

Additionally, the Passivhaus standard for refurbishments, EnerPHit, has published (Bastian, 2014) a suggested U-value for different climates ranking from arctic to very hot (Figure 3-26). The U-value for artic (light blue) , cold (blue) , cold-temperate (dark blue) and warm-temperate (green) areas, where the advanced timber frame panel systems may be economically feasible, are 0.09 W/m^{2}K, 0.12 W/m^{2}K, 0.15 W/m^{2}K and 0.25 W/m^{2}K respectively.

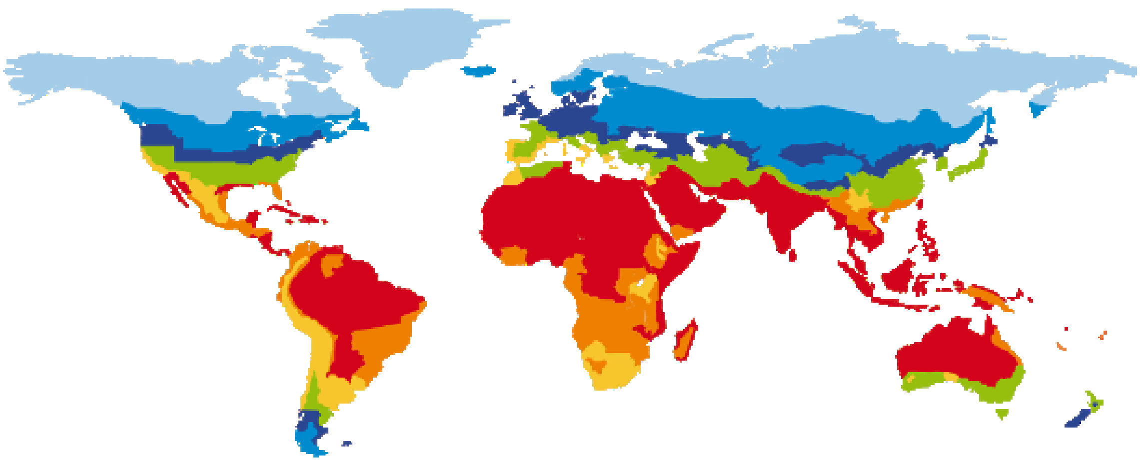


Figure 3-26 Passivhaus climate zones (Bastian, 2014)

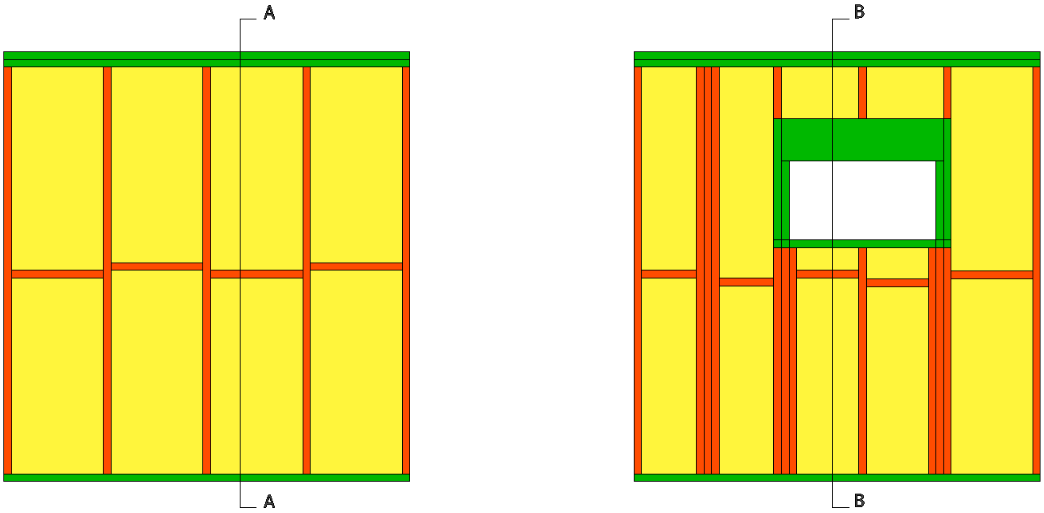
In this section, the U-values for all of the possible wall combinations, as described in Table 3-3, are given. The online tool U-wert has been used to perform the calculations. However, for the accurate determination of the thermal transmittance, and as result of the disparity of information gathered in the literature review, a comprehensive study of the timber fraction is detailed in the next sub-section.

3.5.1 Determination of Timber Fraction

The standardised timber fraction information reported from the section 3.2.2 of the literature review was rather controversial, and there were no general agreement on the methodology employed to determine this factor.

Low-energy buildings require to use a building fabric with high thermal resistance. Additionally, the design of this buildings must account for thermal bridging if this cannot be avoided. Public Passivhaus projects include a complete report of thermal bridges including timber frame wall to foundations, to roof and to floor elements. Also, there is an additional thermal bridge to be appraised in the design that considers the installation of windows and doors.

All these issues resulted in the development of a different quantitative methodology to determine the actual timber fraction of energy efficient timber frame walls when thermal bridging is known (Figure 3-27).



Yellow: insulation   Red: Accountable for timber fraction   Green: Accountable for thermal bridge

Figure 3-27 Timber accountable for timber fraction in a wall panel

In this figure, in red is noted additional timber that contributes to the lower thermal resistance of the insulation layer whilst in green is represented additional timber that contributes to a (linear) thermal bridge. Additional considerations must be taken in case the window joist lintels are placed immediately underneath the top rail and when the width of the joist equals to the full depth of the timber fram frame. In that case, area related to the lintel must be added to the overall timber fraction.

In order to provide an accurate estimation of timber fraction for the thermal transmittance of the suggested closed panel timber frame build-up models, the manufacturing drawings of four different real low energy projects were used to determine this value. The design of the timber frame walls was carried out with particular care to not use more timber than required for the assembly with timber fraction on mind but also considering structural compliance. Every panel for each wall and for each project was considered and timber area related to total area of the wall was analysed. Panels with a noteworthy opening area were recorded. The results of the four different projects with the theoretical timber fraction calculated, as per Equation 3.14, are presented in Table 3-14.

$$TF_i = \frac{b_i}{s} \times 100 \tag{3.14}$$

Where:

TFi	timber fraction for wall i
bi	stud breadth
s	stud spacing

Table 3-14 Typology of projects for Timber Fraction calculation.

	No. storey	Stud size b×h (mm)	Spacing s (mm c.c.)	Theoretical TF %	Measured TF %
Project 1	1	45x220	600	7.5	9.2
Project 2	1	45x220	600	7.5	6.9
Project 3	1	60x120	625	9.6	9.3
Project 4	2	60x160	625	9.6	10.0

The statistical results of the analysis and a reference to values from literature are given in Table 3-15. In conclusion, a Timber Fraction of 10 % it is a sensible approach and therefore, this is the suggested value on the calculation for the compound timber frame wall U-values. This percentage is used for the benchmark and the K2 panel. For the RTC I-joist timber frame, a value of 10 % is used for the flanges and a proportional value of 2 % is considered for the web.

Table 3-15 Statistical and Timber Fraction reference values

n=42	Experiment	Friedman & Cammalleri (1996)	Val-U-Therm
Mean	10.36 %	9.4 %	10%
SD	4.68 %		
Median	9.65 %		

Further information on the methodology and timber frame manufacturing layouts to determine this timber fraction is attached in the Appendix IV.

In summary, the results from Figure 3-28 shows a weak correlation between timber fraction and area of the wall.

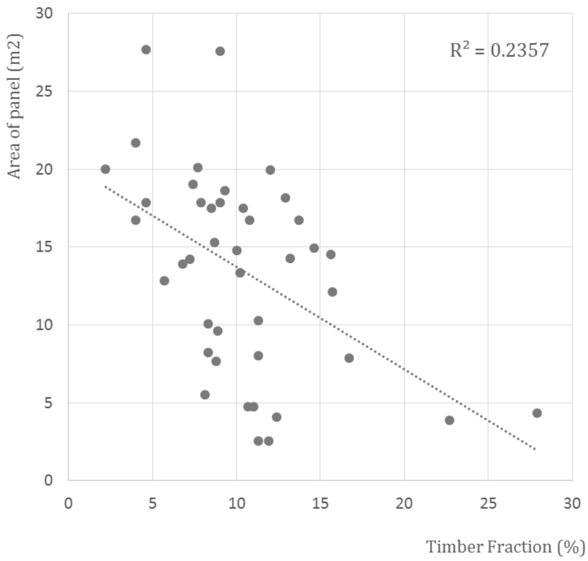


Figure 3-28 Relationship between Timber Fraction and wall area

3.5.2 Calculation of U-values

In this sub-section, the U-value for each wall build-up configuration is determined by numerical methods. The final purpose of this exercise is to present a series of timber frame configurations for direct prescription according to a required U-value. Moreover, the optimal thermal performance for every different wall is the reference target scenario for next chapter: structural performance optimisation.

The 1-D u-wert online tool has been utilised to perform the U-value calculations with the following boundary and modelling conditions:

- Geometry of the building wall element: Minimum one period, $T \geq 1$
- Timber fraction of frame: TFs and TF_{flange} = 10% for studs and I-joist flange and TF_{web} = 2% for I-joist web
- Timber fraction of internal insulated service cavity: TFsc =7.5%
- Internal thermal film surface resistance: Rsi = 0.13 m2K/W
- External thermal film surface resistance: Rse = 0.13 m2K/W

The properties of the materials used on the timber frame walls are described in Table 3-4 Timber frame material properties. In order to account for the corresponding timber fraction in the software, with a value of 10% (all types of timber studs and I-joist flanges) and 2% (I-joist webs), the equivalent timber frame stud spacing suggested is:

- 45/450 mm for studs and I-joist flanges and 405mm clear stud spacing
- 9/450 mm for I-joist web and 441mm clear web spacing

For each wall panel build-up, and with the same core thickness, there are four different configurations as shown in Figure 3-29:

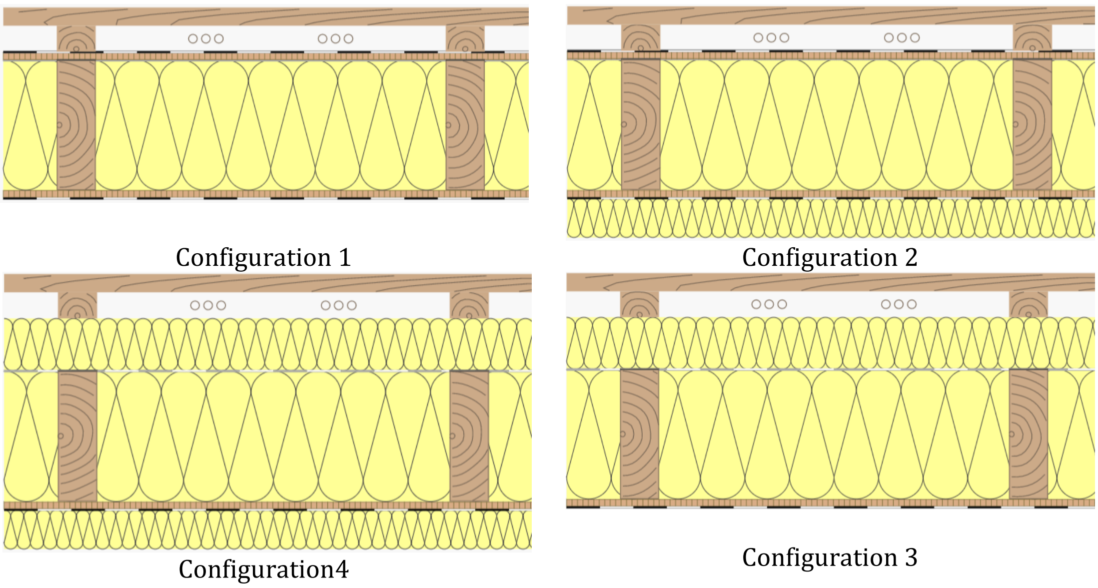


Figure 3-29 Panel configurations for each wall type.

An additional explanation of configurations 1, 2, 3 and 4 (clockwise from top left) are presented below:

1. Intermediate insulation and frame core with sheathing boards at both sides.
2. As configuration one plus insulated service cavity in the internal side.
3. Intermediate insulation and frame core with sheathing in the internal side and rigid wood fibre board insulation on the external side. In case of the K2 wall panel, there is sheathing board at both sides for structural purposes and also rigid wood fibre board insulation on the external sheathing board for thermal enhancement.
4. As configuration 3 with an insulated service cavity in the internal side.

Finally, mineral glass wool (MW) and flexible wood fibre (WF) insulation batts for the intermediate core layer were investigated. The U-values for the resultant 56 different wall panel configurations were calculated. The results are presented in Table 3-16.

Table 3-16 U-values for timber frame wall build-ups

Core depth h (mm)	Config	K2-Panel MW	K2-Panel WF	RTC-Panel MW	RTC-Panel WF	Benchmark MW	Benchmark WF
195[1]	1	0.173	0.200	0.175	0.201	0.245	0.270
	2	0.144	0.162	0.145	0.163	0.189	0.207
	3	0.141	0.158	0.143	0.160	0.185	0.203
	4	0.121	0.133	0.123	0.135	0.152	0.164
245	1	0.136	0.158	0.140	0.161		
	2	0.117	0.133	0.120	0.136		
	3	0.115	0.130	0.119	0.134		
	4	0.102	0.113	0.104	0.116		
300	1	0.110	0.128	0.115	0.133		
	2	0.098	0.112	0.101	0.115		
	3	0.096	0.110	0.100	0.114		
	4	0.086	0.097	0.090	0.100		

[1]Core depth for benchmark panel is 150mm

<0.25	E&W Building regulations	<0.12	Passivhaus cold
<0.15	PH cold-temperate	<0.09	Passivhaus artic

From the results, it is evident that the benchmark timber frame configuration is not suitable for Passivhaus construction, even for configuration 4 with external rigid wood fibre board insulation and internal insulated service cavity. Furthermore, the benchmark wall panel without additional insulation and with wood fibre insulation in the core layer does not even comply with minimum Building Regulation requirements.

It is also clear that a 195 mm core timber frame for advanced panels is also not suitable for Passivhaus standards. Only two wall configurations are suitable for artic Passivhaus climate both having external insulation, internal insulated service cavity and mineral glass wool as core insulation. Timber frame walls for cold-temperate climate are easily achieved with additional insulation on the intermediate 195 mm wall.

An interesting observation was the impact of timber fraction on the global wall U-values. Figure 3-30 shows that when timber fraction is added to the frame U-value, it has more impact on less insulated timber frame walls. This observation was even more remarkable for the benchmark scenario. The impact of timber fraction on the frame configurations 3 and 4 were almost negligible.

The analyses of the U-values depending on the build-up configuration and for each of the insulation cores mineral wool and wood fibre insulation, are presented in Figure 3-31. The results show an almost identical U-value for the K2 dual frame and RTC I-joist frame for each of the intermediate core layers and insulation types. In general, insulating the internal service cavity and externally by a wood fibre rigid board improves the global U-value by 30 % to 35%, independently of the insulation core type.

The impact on the thermal resistance of the walls by insulating the 45mm service cavity is almost identical than by insulating externally the frame with a 60 mm rigid wood fibre board. This conclusion is related to the differences on the thermal conductivities of both materials.

The nominal thermal conductivity, λ_D, were considered in the calculation of the timber frame U-values. This value is also known as the rounded up to the nearest 0.001 W/m K conductivity value, $\lambda_{90/90}$. This conductivity is declared when the production is not exceeded by at least 90% with a 90% confidence interval.

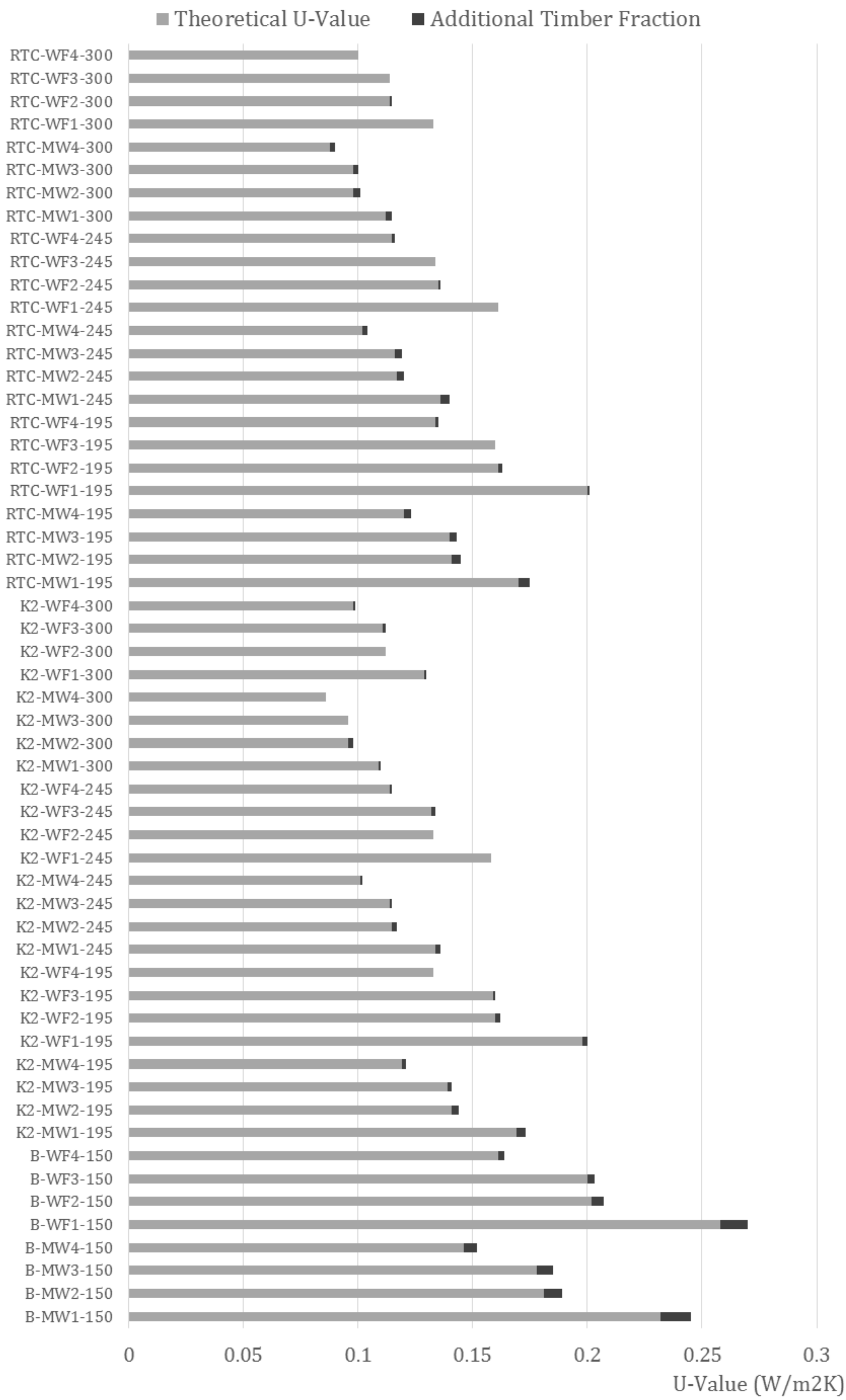


Figure 3-30 Timber frame U-value (grey) and timber fraction added (bold)

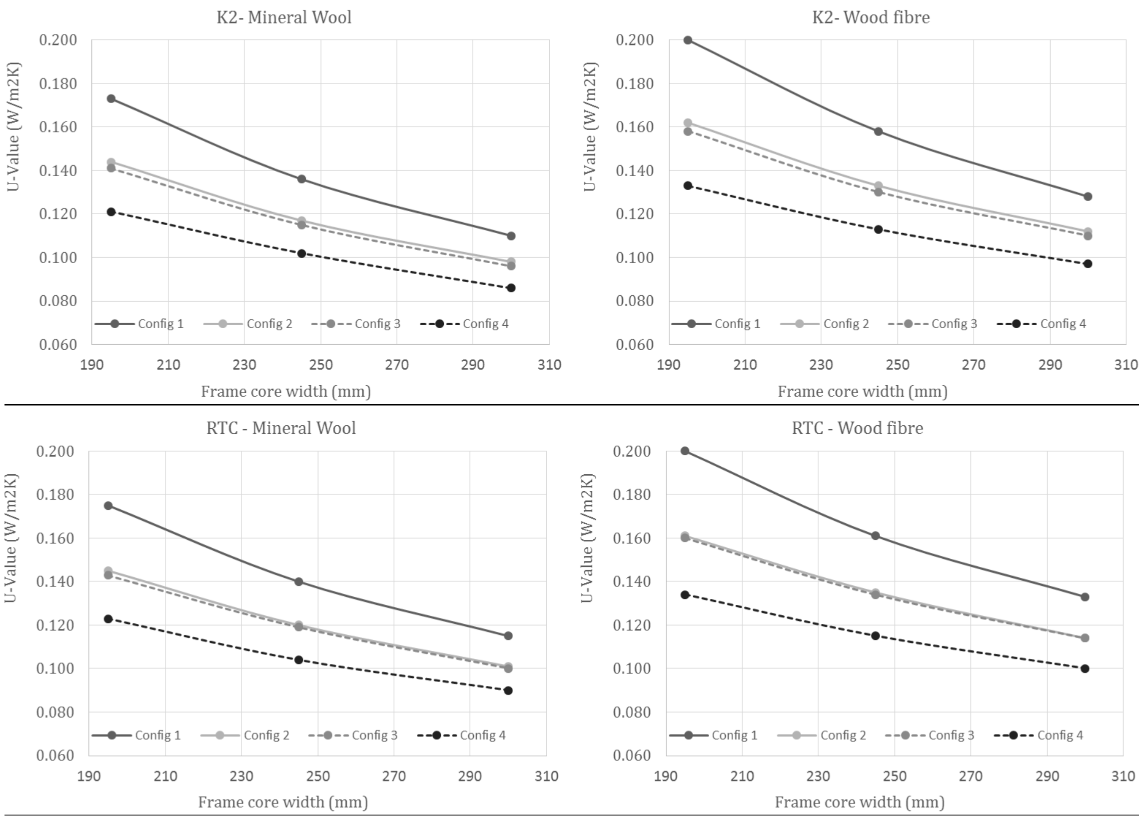


Figure 3-31 U-value comparison for insulation core and wall type

3.6 Analysis of Thermal Bridges

The largest heat flow was found close to the perimeter of the floor. The conversion of the thermal bridge resulting from a three-dimensional model to a two-dimensional model when the dimension of the floor is similar to the width of the building can be considered acceptable (BSI, 2009d). In this case, the steady-state ground heat transfer coefficient can be expressed as per Equation 3.16 where the ground thermal bridge, φ_g, is determined by numerical simulation.

$$H_g = A\,U + P\,\varphi_g \tag{3.16}$$

Where:

H_g steady-state ground heat transfer coefficient, in W / K

A area of floor, in m²

U transmittance between internal and external lay, in W / m² K

P exposed perimeter of floor, in m

φ_g ground thermal bridge, in W/ m² K

In this section, one hundred and forty linear thermal bridges for three different timber frame panels and for two different foundation types were undertaken to determine optimised sole plate details. Furthermore, a qualitative assessment of a point thermal bridge simulating was carried out by using 2-D FEA analysis to understand the thermal significance of metal fixings within the sole plate detail.

3.6.1 Methodology and boundary conditions

The reference documents to consider for analysing 2-D thermal bridge calculation of heat flow and surface temperatures were the recommended by ISO 10211:2007:

- ISO 6946, building components and building elements for the thermal resistances of the surfaces.
- ISO 13370:2007, Thermal performance of buildings, for the methodology to simulate heat transfer via the ground.
- ISO 13788, Hygrothermal performance of building components and elements to determine internal surface temperatures to analyse surface and interstitial condensation issue.

The two-dimensional planes of the ground and soil for the sole plate to the foundations thermal bridge calculation, according to EN ISO 10211, is as shown in Figure 3-32. For this dimension, $0.5 \times b$ should be considered at least, the greater value as per Equation 3.17 (Feist, 2006):

$$0.5\,b = \begin{Bmatrix} 1\ m\ clear\ consistent\ construction \\ 4\ \times wall\ thickness \end{Bmatrix} \tag{3.17}$$

The widest wall, according to the results from the previous U-value calculation exercise, was achieved on a 300 mm thick central core for a total width of 475mm. In order to simplify the thermal bridge simulations for all of the wall configurations, the wall was considered to be 500 mm thick. Hence, the floor width *b*, was taken as 2 m.

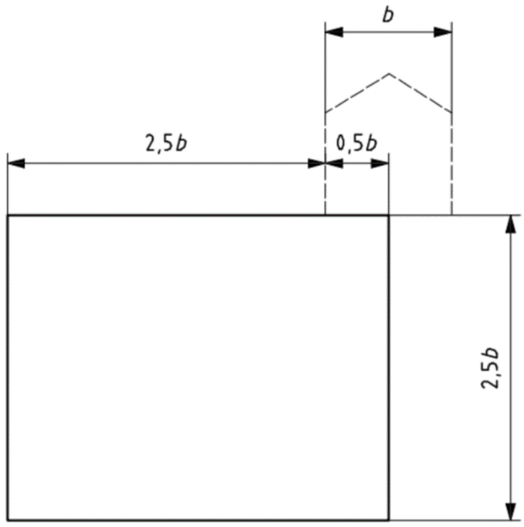


Figure 3-32 Ground geometry and dimensions for 2-D thermal bridge model

The resultant soil geometry is a rectangle of 6 m long by 5 m deep. The wall and the floor are projected 2 m from the external base point. This dimension covers all the cases for any Passivhaus envelope system suggested in this research.

The Austrian FEA software HTFlux was selected to perform the thermal bridge analyses. Previously, the optimal resolution of the mesh for the thermal simulation was determined, according to the stability of the results and simulation time (Figure 3-33), at 3.5 mm resulting in over 2.5 million cells for each of the models.

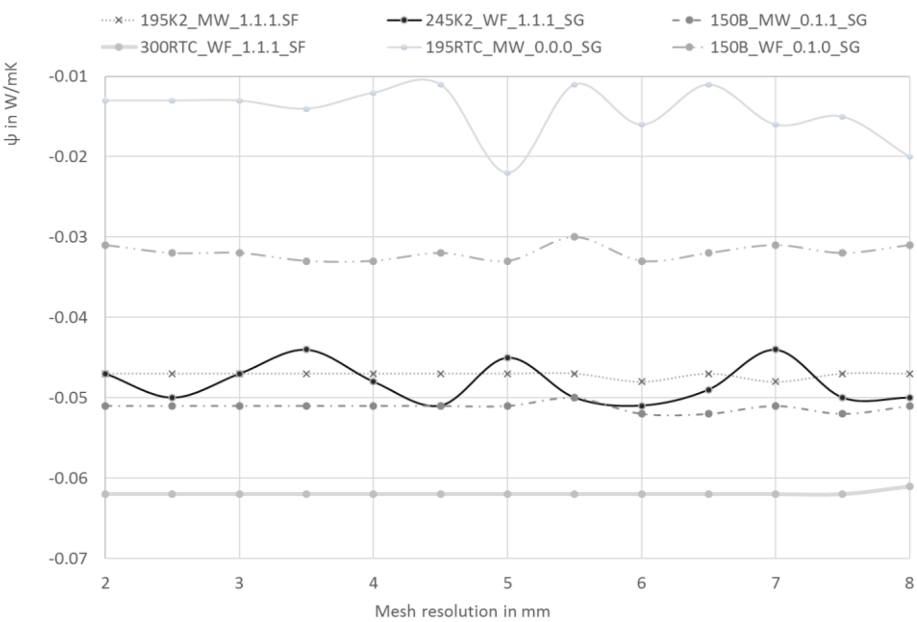


Figure 3-33 Determination of FEA mesh resolution

The average simulation time for an Intel Core i5 processor and 8 GB of memory was 120 s. An example of the model definition, geometry and materials input created in this tool and for the K2 wall panel is shown in Figure 3-34.

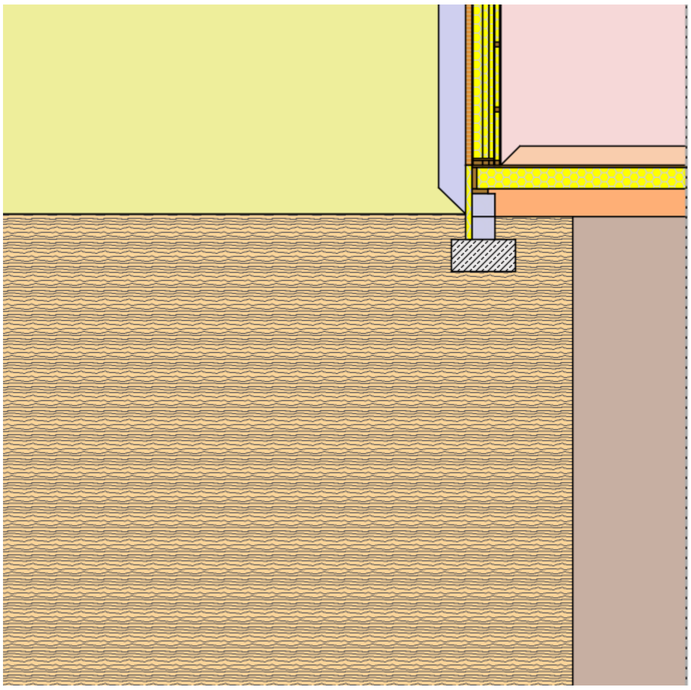


Figure 3-34 HTFlux geometrical definition with no cladding simulated

The thermal properties of the timber frame and insulation materials considered in the analysis were described in section 3.4.1 and summarised in Table 3-12. The rest of materials not declared previously are detailed in Table 3-17.

Table 3-17 Thermal properties of non-timber frame materials

Material	Thermal conductivity λ_D (W/mK)	Vapour diffusion μ (dry)	Source
Gypsum plasterboard	0.250	10	EN 12524
Cement screed	1.330	37	HTFlux
Solid concrete block	1.150	100	EN 12524
RF concrete 2%	2.500	130	EN 12524
RF concrete 1%	2.300	130	EN 12524
Soil	2.000	1	ISO 13770
Sand	2.000	1	ISO 13770
Gravel	2.000	1	ISO 13770

The boundary conditions of the spaces surrounding the timber frame, the suspended floor and the soil elements are defined as the recommendations provided by the Passivhaus Institut (Feist et al., 2007) and the three documents presented at the beginning of this section.

Although the temperature of the boundary conditions does not have a direct impact on the thermal bridge calculation, they do in terms of accuracy of resulting solution. Also, the minimum surface temperature T_{min} and the temperature factor f_{Rsi} are dependant of the temperature of the internal and external boundary conditions as detailed in Table 3-18.

Table 3-18 Simulation model boundary conditions

BC	T (ºC)	RH (%)	$R_{e,i}$ (W/m²K)
PH internal wall	20	50	0.13
PH rainscreen wall	0	80	0.13
PH external wall	0	65	0.04
PH floor void	2.2	65	0.17
PH internal down	20	50	0.17
PH internal up	20	50	0.10
PH soil conditions	10	80	0.00

3.6.2 Sole plate detail: point thermal bridge

Significant point thermal bridges shall be determined by three dimensional analyses. This is a labour-intensive task that requires complex and time-consuming meshing models.

In the case of timber frame fixings at the critical point, the sole plate base connection, the significance of this effect can conservatively be analysed in 2D by considering a connector of a given diameter but with infinite length along the wall, i.e. simulating the fixing as a metal plate of thickness equal to its diameter.

A thermal bridge simulation of the K2 wall panel sole plate connection with a horizontal screw or bolt of diameter 10 mm protruding 160 mm into the depth of the construction was analysed by the software HTFlux.

The K2 panel was selected for this simulation as the defined bolt was found in real sole plate detail examples whilst for the RTC panels, the current practice utilised slant screws for fixing the frame to the sole plate.

Furthermore, another identical bolt but fixed vertically was simulated to determine the thermal influence of the connection between the sole plate and the foundation. In order to maximise the thermal bridge effect, the concrete slab was insulated with 240 mm of XPS insulation underneath.

The comparison between this -linear- thermal bridge and a clear junction free of metal parts are determined by equations 3.18 and 3.19 and shown as heat flux (left) and isotherms (right) in Figure 3-35:

$$\varphi_{bolt,v} = \varphi_{eqv,bolt,v} - \varphi_{eqv,clear} \tag{3.18}$$

$$\varphi_{bolt,v,h} = \varphi_{eqv,bolt,vh} - \varphi_{eqv,clear} \tag{3.19}$$

Where:

$\varphi_{bolt,v,h}$	thermal bridge of the bolted detail, v (vertical), h(horizontal)
$\varphi_{eqv,bolt,v,h}$	thermal bridge of a linear equivalent bolt (vertical and horizontal)
$\varphi_{eqv,clear}$	thermal bridge of the detail with no bolts

It can be seen in red colours the greater heat flux caused at the bolted connection but this heat flux is not proportional across the sole plate. The resultant effect of the 10x160 mm bolt connected vertically and horizontally in the sole plate, as a linear thermal bridge along the full length of the wall was:

$$\psi_{bolted} = 0.011 \text{ W/(mk)}$$

This thermal bridge heat loss, derived from a point thermal bridge approximation, can be considered almost as a Passivhaus thermal-bridge free detail.

As a result, it can be concluded that the thermal impact of the mechanical fasteners within the sole plate, only if they are not penetrating the full depth of the connected element, may not be considered as a heat loss in the energy balance.

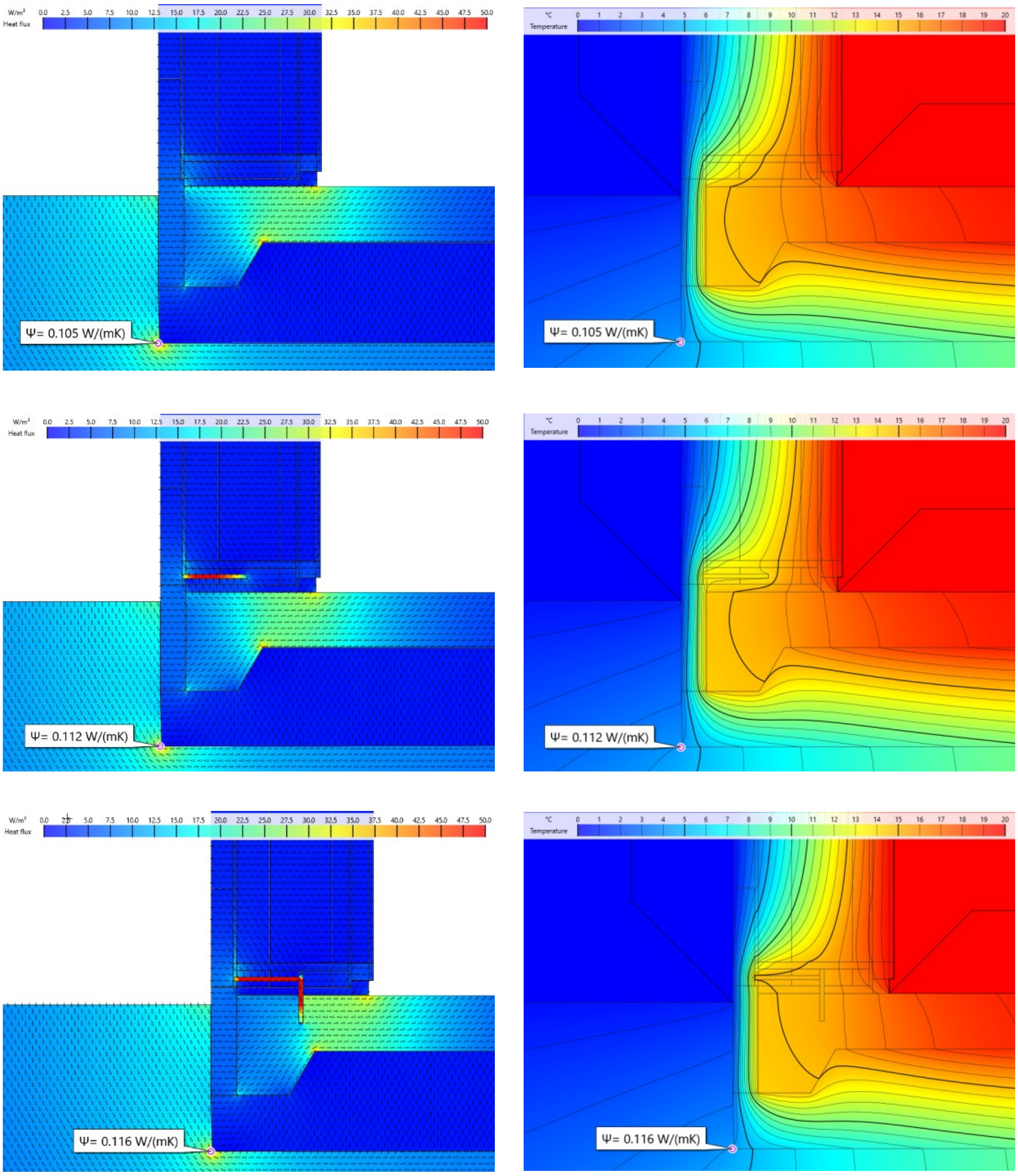


Figure 3-35 Point thermal bridge due to bolted connection

3.6.3 Sole plate detail: linear thermal bridge

The thermal performance of the suggested three wall panel configurations was determined in terms of U-value in section 3.5.2. However, the global thermal performance of the wall panel must consider the thermal bridge between the wall panel and the substrate including the sole plate base fixing detail.

A series of thermal bridge analyses were carried out for the three wall panel configurations with several stud depths and for two different types of foundations: slab on grade and suspended timber floor cassettes. Two different internal finishing were considered: insulated service cavity and non-insulated service cavity.

No effect on the thermal simulation was considered for the non-insulated service cavity. Finally, each of the wall panels presented two different external configurations: OSB/3 and windshield membrane and rigid wood fibre board apt for external applications. It must be noted that for the case of the K2 wall system, the OSB/3 must also be installed at the external side of the frame to provide the structure with racking resistance.

The three different wall panels also utilised two materials as timber frame insulation: mineral wool and flexible wood fibre. Finally, the perimeter of the substrate for both, the slab on grade and the suspended timber floor was simulated with and without 60 mm thick vertical XPS insulation.

The nomenclature of each wall panel subjected to thermal bridge analysis, and described in Table 3-19, is as follows:

$$000AA\_BB\_C_{01}.D_{01}.E_{01}\_YY$$

In total, over 140 linear thermal bridges where analysed in HTFlux. The representation of four different wall panel and substrate configurations, including the nomenclature proposed, is presented in Figure 3-36. The detailed calculations and the report of the thermal bridge simulation for the RTC timber frame panel is given in the Appendix V.

As stated in the previous sections, there are different procedures to perform thermal bridge simulations. One of the greatest difference is the wall and floor reference lengths which can be measured either internally or externally.

Internal measurements are generally used in the UK as required in the Standard Assessment Procedure (SAP 2012) whilst external thermal bridge measurements are compulsory for Passivhaus design. In this investigation, only external dimensions are considered.

Table 3-19 Thermal bridge wall panel nomenclature

Ref	Attribute	Parameters
000	Timber frame depth	150 – 150 mm 195 – 195 mm 245 – 245 mm 300 – 300 mm
AA	Wall panel configuration	B – Benchmark K2 – K2 panel RTC – RTC panel
BB	Type of timber frame insulation	MW – Mineral wool WF – Flexible wood fibre
C D E	External Rigid Insulation Insulation on substrate perimeter Insulated service cavity	0 - No 1 - Yes
YY	Type of substrate	SG – Slab on grade SF – Timber suspended floor

This difference has a direct impact when calculating thermal bridges. If the same construction detail is calculated, the resultant linear thermal bridge is greater when measured internally, as the heat losses on the junction area are not accounted from the surrounding building envelope.

On the other hand, there is a relationship between the U-value of the components and the resultant thermal bridge of the inter-connection. Typically, the higher the U-value is for the walls and floors, the greater the influence of the linear thermal bridge on that sole plate connection.

In this case, the criteria followed in the research established variable wall U-values for each of thicknesses proposed and for the three panel configurations. In terms of characteristic U-values for the floor systems, three different approaches were considered:

- U-value of suspended floor system of 0.20 W/m^{2}K, equivalent to a timber joist with 200 mm mineral wool insulation in between.
- U-value of slab on grade foundation system of 0.34 W/m^{2}K, equivalent to a

100 mm of XPS insulation on top of the slab.

- U-value of slab on grade foundation system for comparison of 0.18 W/m^2K, equivalent to a 200 mm of XPS insulation on top of the slab.

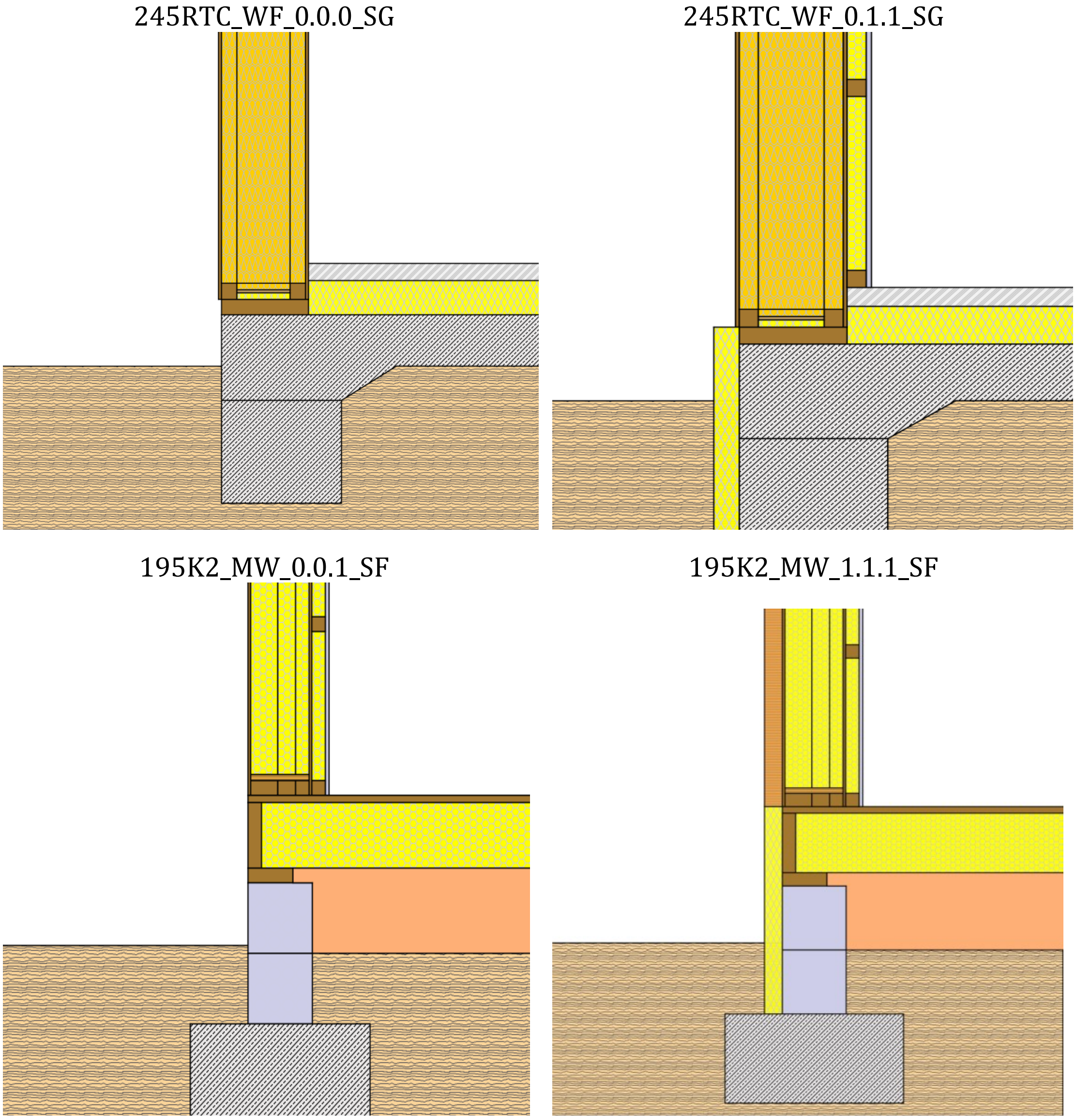


Figure 3-36 Representation of four different wall panel configurations

The results of the linear thermal bridge simulations for the advanced closed panel timber frames are presented by Figure 3-37.

W/mK

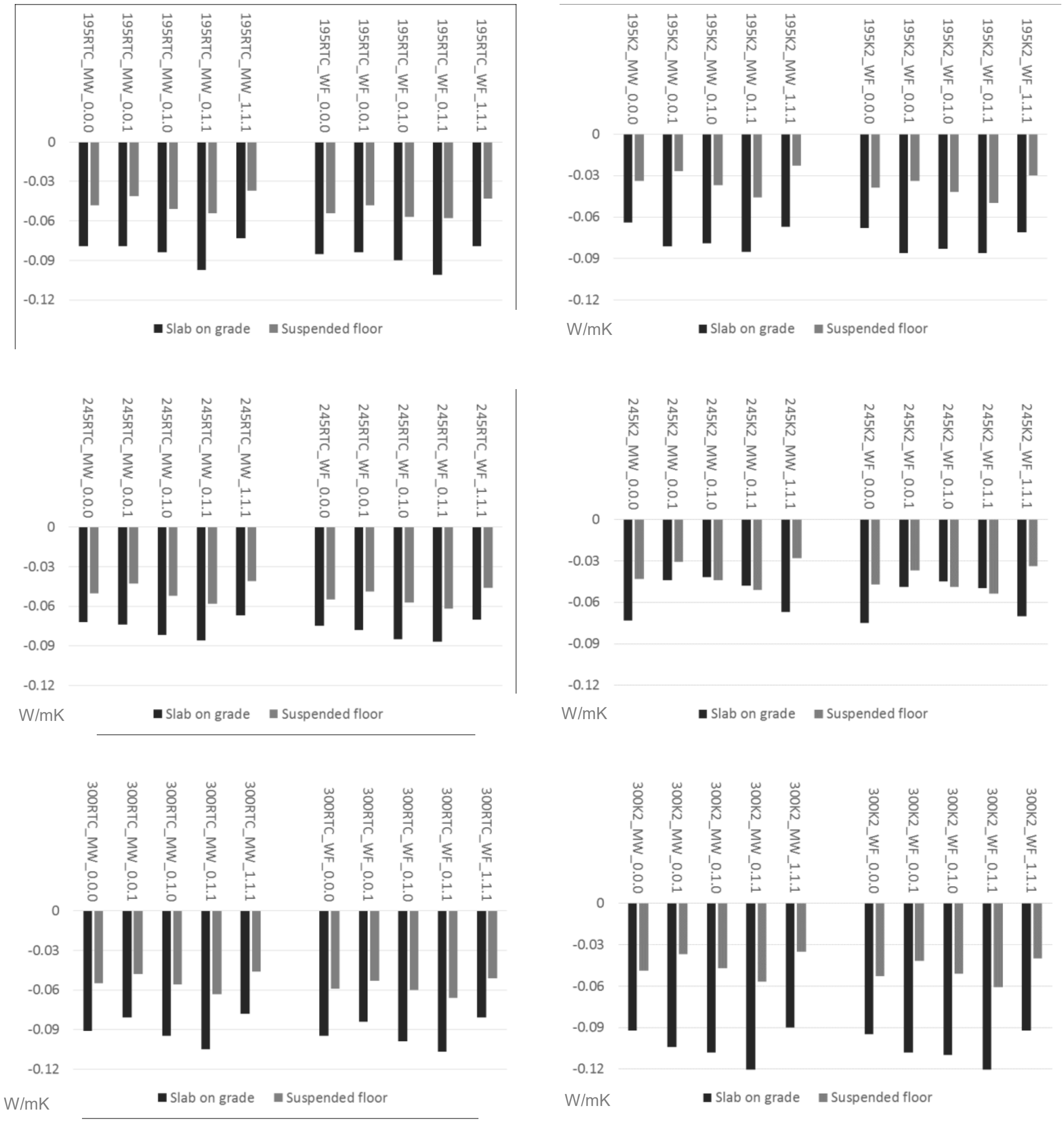


Figure 3-37 Thermal bridge simulation, in W/mK for advanced closed panels

The results of this study indicate that advanced wall panel to foundation detail provides no additional heat loss to the building construction as all of the thermal bridge analyses were negative. The research also concludes that for slab on grade foundation type, the thermal bridge for the RTC it is slightly more favourable than the K2 panel for frame depths of 195 mm and 245 mm but the thermal performance of the K2 is better for a timber frame depth of 300 mm.

In the case of suspended timber floors, the RTC wall system behaves slightly better, in terms of thermal bridging, than the K2 panel for all three timber frame depths.

The installation of 60 mm of XPS insulation on the perimeter of the foundation for both the slab on grade and the suspended floor is the individual parameter that has the greatest influence on thermal bridging. This is an important consideration as this measure does not provide any direct improvement on the wall and floor U-values, but can reduce the heat loss of the whole building.

As expected, there is no difference in the thermal behaviour of the timber frame walls insulated with mineral wool and the timber frame walls insulated with flexible wood fibre. The thermal bridge simulations for the later are slightly less favourable due to the higher thermal conductivity of the wood fibre. However, this difference is more significant for 195 mm timber frame walls whilst it is almost negligible for 300 mm timber frame walls.

In the case of the benchmark, open timber frame panel, a different thermal behaviour is observed. In this case, the suspended timber floor provides better thermal bridge values if no additional insulation is installed or if only the service cavity is insulated. Also, in this case, the timber frame with mineral wool insulation provides a greater reduction in thermal bridging for both slab on grade and suspended timber floor system.

The addition of 60 mm XPS insulation on the perimeter of the floor improves itself the thermal performance of the detail as much as 0.03 W/m K over the timber frame with no additional thermal measures (Figure 3-38).

The insulated service cavity in the thermal simulations was modelled with horizontal battens spaced every 610 mm centres and mineral wool insulation between them. A research finding was that this approach, for the K2 and benchmark panels, performed thermally slightly worse than simulating a vertically fixed batten at the same stud spacing distance.

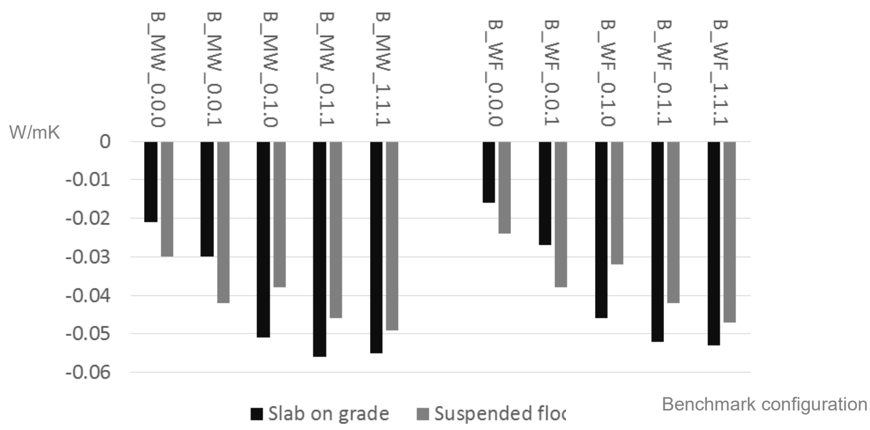


Figure 3-38 Thermal bridge simulation for benchmark timber frame

This could be explained by the negative effect of the first horizontal batten where the skirting board is fixed. However, this factor had little influence for 300 mm thick walls (Table 3-20).

Table 3-20 Effect of insulated cavities on thermal bridge value, in W/mK

Wall panel	Hor. batten	Ver. battens	Difference
150B_WF_0.1.1_SF	-0.025	-0.042	+0.017
150B_WF_0.1.1_SG	-0.027	-0.041	+0.014
195RTC_WF_1.1.1_SF	-0.058	-0.053	-0.005
195RTC_MW_1.1.1_SF	-0.054	-0.050	-0.004
195K2_WF_1.1.1_SF	-0.050	-0.054	+0.004
195K2_MW_1.1.1_SF	-0.047	-0.050	+0.003
245RTC_MW_0.0.1_SG	-0.074	-0.070	-0.004
245K2_WF_1.1.1_SF	-0.055	-0.058	+0.003
300RTC_WF_1.1.1_SG	-0.054	-0.053	-0.001
300K2_MW_0.0.1_SG	-0.092	-0.092	+0.000

3.7 Moisture Control of Building Assemblies

In order to assure a durable timber frame building and to avoid any future damage to the structure, it is essential to control moisture within the assemblies. The influence of the moisture in the building site was not considered in this research. Prefabricated advanced closed timber frame panels present shorter on-site building time. Hence, the risk of additional moisture in the assembly due to wet building site conditions is minimised.

The surface condensation risk analyses carried out in this research include two steady-state methodologies: the temperature factor and an evolved version of the one-dimensional Glaser method. These two simulations do not consider other water transportation or water absorption processes, such as driven rain or water splashing off. Furthermore, only rain-screen ventilated façade systems are modelled in order to disregard potential water transportation from the outside to the inside.

A third methodology is included at the end of the section which includes one-dimensional dynamic hygrothermal simulation. This methodology not only evaluates the effect of built-in assembly moisture but driving rain, capillary transport, or summer condensation.

Contrarily to thermal bridge simulation, condensation risk analysis requires, for any methodology type, clearly defined climate conditions. The next section defines three climate boundary conditions to limit the moisture control to a set climate scenario.

3.7.1 Climate conditions

In this research, no specific building site was established. Nevertheless, the suggested advanced closed panel systems provided a U-value from 0.09 W/m^2K to 0.20 W/m^2K which related to timber frame walls from 195 mm to 300 mm thick respectively.

According to the Passivhaus recommendations, these U-values correspond approximately to climate zones from artic to warm-temperate climates (Figure 3-26). However, only 2 out of 60 timber frame wall configurations studied (Table 3-16) are just adequate for artic climate conditions and for this reason, this climate is discarded in this study.

Three climates are considered in the condensation risk analysis of the building assemblies for each of the method described in the following sections: cold, cold-temperate and warm-temperate. Furthermore, three cities were identified within these categories and their available climate data were used in the simulations (Table 3-21).

Table 3-21 Climate conditions considered

City (Country)	Climate	Min. T (°C)	HR (%)	f_{Rsi} criteria
Warsaw (Poland)	Cold	-10	80	≥0.75
Edinburgh (UK)	Cold-temperate	-5	80	≥0.70
Bilbao (Spain)	Warm-temperate	0	65	≥0.65

3.7.2 Temperature factor f_{Rsi} method

This methodology it is widely used to determine if a building assembly is subjected to potential mould growth and surface condensation. The relative humidity of an external wall may exceed 80% when the indoor air relative humidity is greater than 70%. If this event occurs more than 72-96 h, mould is likely to develop on that surface. This method, based on EN ISO 13788, considers the internal moisture supply, the internal air temperature and the temperature factor to calculate the relative humidity of the surface.

In this research, internal moisture supply is not considered as Passivhaus buildings require a mechanical ventilation with heat recovery systems that are able to help control and remove the excess of indoor moisture generation. Furthermore, for climates where the relative humidity is considered high all year round, an enthalpy heat exchanger can regulate the amount of indoor relative humidity by the use of hydrophilic membrane cores which allows for moisture diffusion from the membrane to the air and vice-versa (Peper et al., 2005).

The temperature factor of the internal building surface considers both the thermal resistance and the geometry of the materials present in the assembly and the internal surface resistances R_{si}. This dimensionless unit refers to the temperature difference between the indoor and outdoor air that is present at the interior surface of the building assembly. Together with the temperature factor, the minimum temperature at the internal surface for 80% relative humidity also is an indicator of mould growth. This temperature for mould growth not to occur was determined as 16.7 ºC in a study carried out in Denmark (Green, 1979).

In Passivhaus, certification criteria for construction systems establishes a minimum temperature factor depending on the climate zone. This hygiene criterion shown in Table 3-21, considers an internal heat transfer surface resistance of 0.25 m^2K/W.

The temperature factors obtained for the less favourable RTC, K2 and benchmark timber frames are shown in Figure 3-39. Although all the timber frame walls studied comply with the minimum f_{Rsi} required by the Passivhaus Institut, in some cases the minimum temperature delivered was too low.

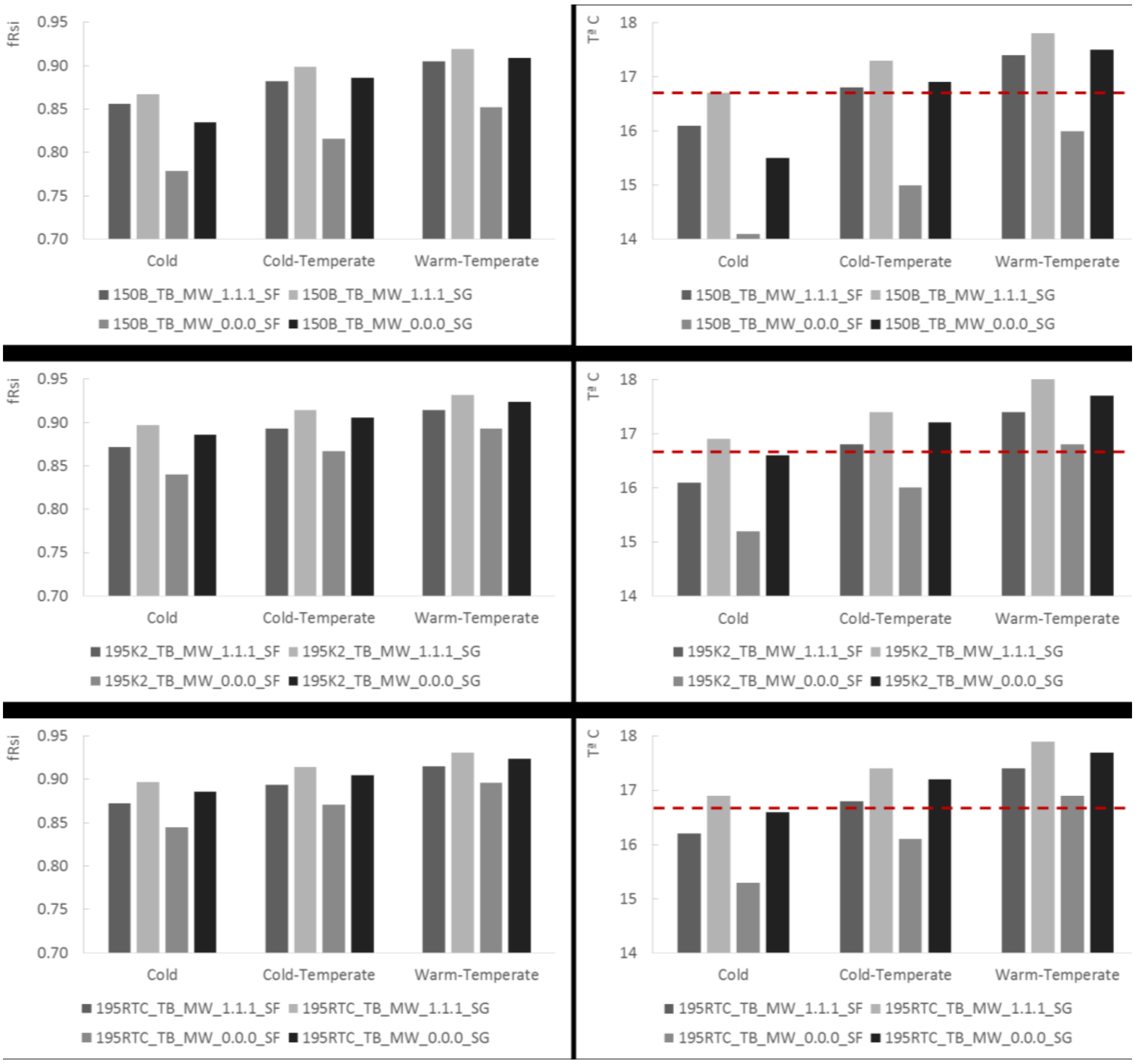


Figure 3-39 Temperature factor and minimum surface temperatures for benchmark (top), K2 (middle) and RTC (bottom) timber frames.

Similarly to the thermal bridge analysis, slab on grade foundations provided a better temperature factor and a greater minimum surface temperatures. As expected, the results obtained for K2 and RTC advanced timber frame walls were almost identical. Both building systems can be rated as condensation-risk safe for cold-temperate and warm-temperate climates except for the suspended floor foundation with no additional insulation. Lastly, the benchmark timber frame with no additional insulation and with a suspended floor system was not safe even for warm-temperate climates.

An extended full report of the temperature factor for the advanced timber frame assemblies is included in the Appendix VII.

3.7.3 Glaser 2-D method

The Glaser method (ISO-13788, 2012) determines the drying capacity of a construction detail in order to analyse the potential risk of interstitial condensation by considering steady-state conditions in one dimension. This methodology calculates the amount of condensation formed in a cold winter and the theoretical evaporation capacity in also a cold summer. If the evaporation capacity is greater than the condensation formed during winter and this does not exceed a certain limit, the construction is considered condensation risk free.

As stated in section 3.2.5, this method is worldwide adopted for light-weight construction assemblies and does not take into account liquid migration nor hygroscopic sorption of the building materials.

The 2-D Glaser approach is an evolution of the one-dimensional Glaser method developed by HTFlux (HTFlux, 2015) which extends the Glaser algorithm on to two-dimensional planes. This method determines the vapour diffusion, partial vapour pressure and humidity based on constant parameter for building assemblies. The software runs a defined maximum number of iterations to converge to a stable solution. However, the steady-state model, as the 1-D Glaser method, does not consider liquid migration processes nor storage of moisture. Therefore, this method may not be as accurate on strong condensation formation in a construction assembly. However, this is seldom to occur on highly insulated Passivhaus assemblies (Feist, 1993).

In order to proceed with the two-dimensional Glaser simulation, two material well-known constants are required:

- Thermal conductivity, λ
- Water vapour diffusion resistance, sd-value, μ

The information and source on properties for the 2-D Glaser simulation carried out on this research are detailed in Table 3-12 for timber frame materials, in Table 3-7 for construction membranes and in Table 3-17 for other non-timber frame products.

These 2-D Glaser qualitative analyses were carried out for the 195 mm K2 and RTC wall and for the suspended floor foundation type, 195K2_MW_0.0.0._SF and 195RTC_MW_0.0.0_SF respectively.

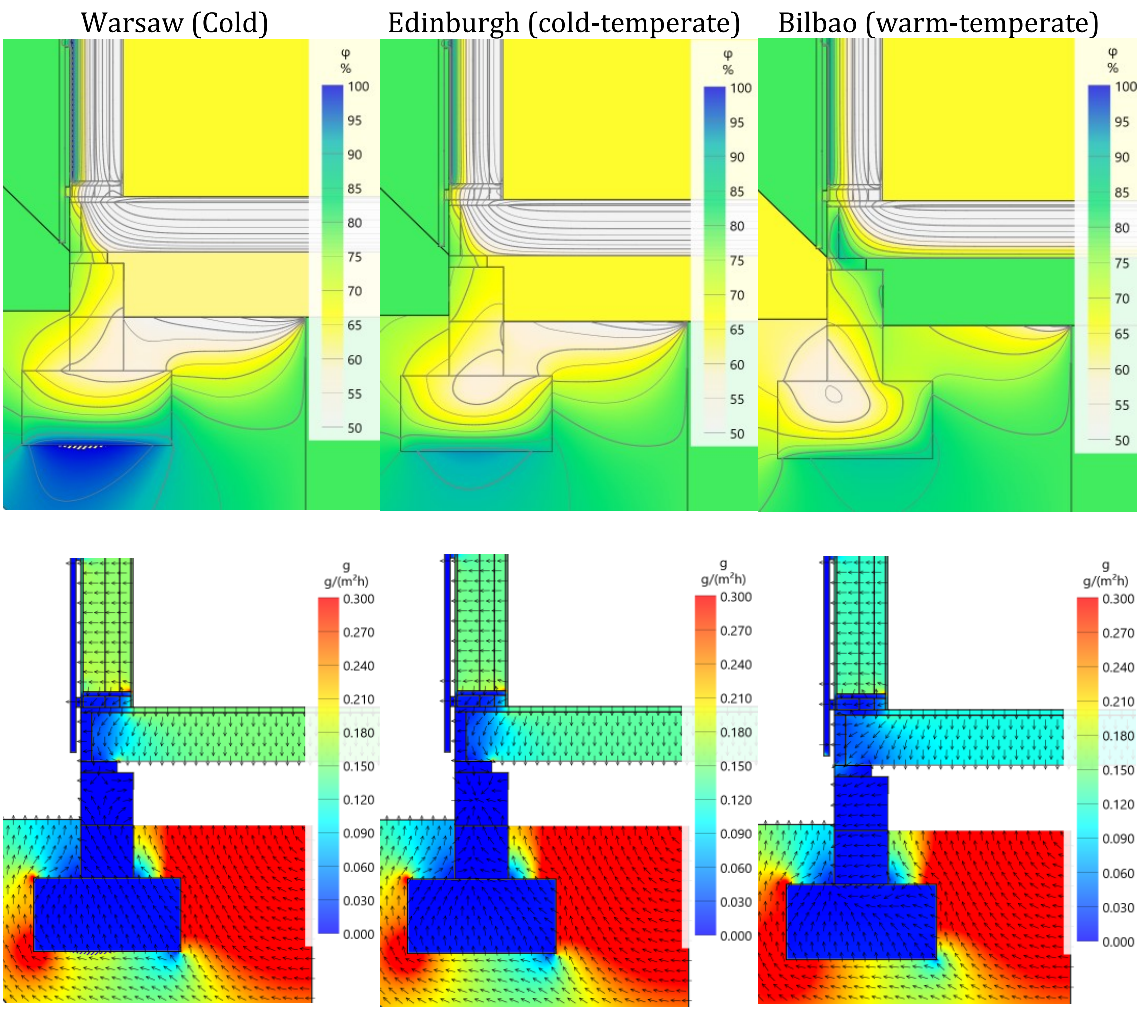


Figure 3-40 Humidity (top) and vapour flux (bottom) for 195 mm K2 panel.

Figure 3-40 shows the hygrothermal result for the K2 advanced panel and for three different climates: cold (left), cold-template (middle) and warm-template (right). The results do not provide any condensation risk in the construction assembly when an Ethylene Propylene Diene Monomer (EPDM) damp proof course is provided on top of the concrete block; a windproof shield membrane is installed in the external OSB/3 board and a VCL layer runs continuously on the internal OSB/3 board and through the internal floor cassette.

On the other hand, Figure 3-41 shows the hygrothermal results for the RTC closed panels and also for three different climate scenarios: cold (left), cold-temperate (middle) and warm-temperate (right). These results also do not show any evidence of condensation risk in the construction assembly when an EPDM damp proof course (dpc) is installed on top of the concrete block, a windshield membrane is fixed in the external OSB/3 board and a VCL layer runs continuously on the internal OSB/3 board and through the internal floor cassette.

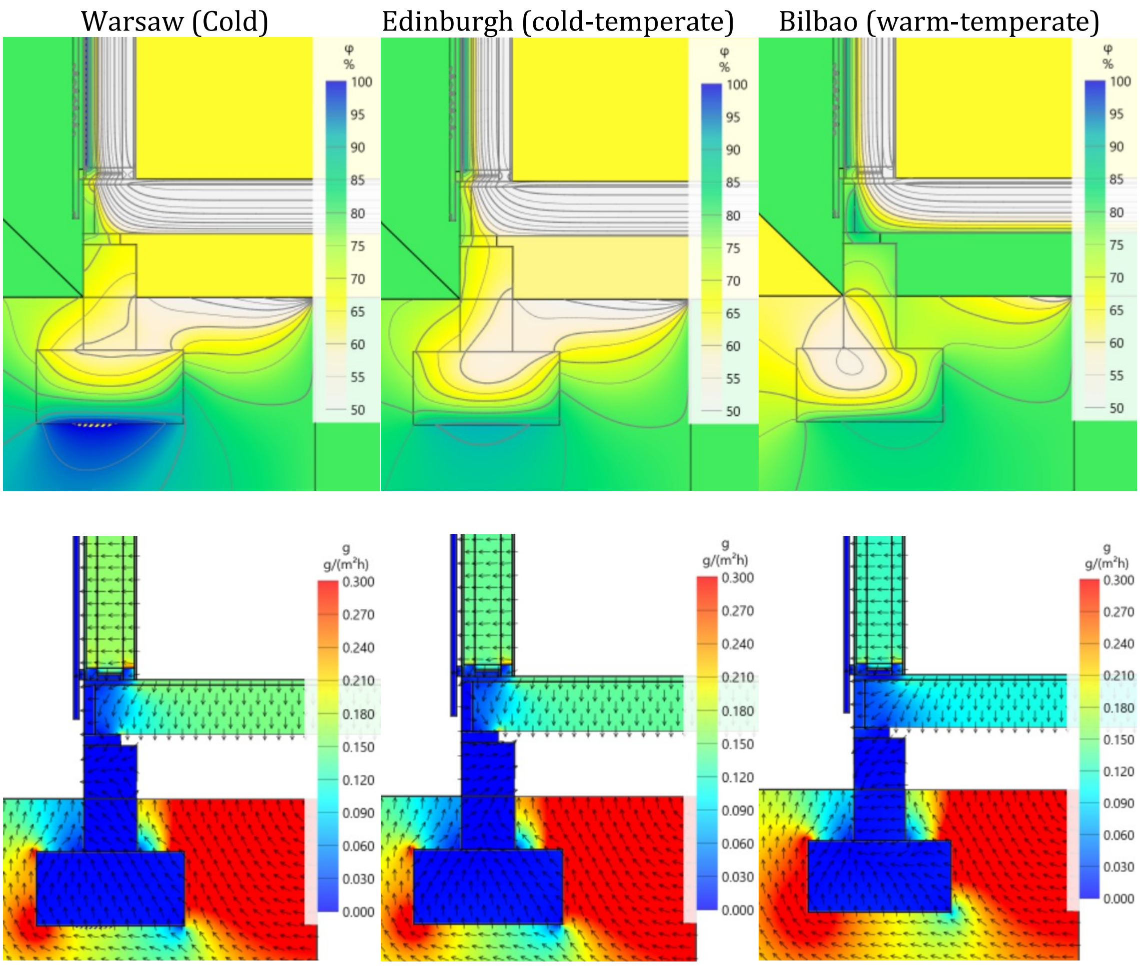


Figure 3-41 Humidity (top) and vapour flux (bottom) for 195 mm RTC panel.

3.7.4 One-dimensional hygrothermal transient method

The aim of this dynamic hygrothermal procedure is to model more realistic conditions. Hence, drying-out moisture of the building assembly, water vapour condensation and the effects of precipitation over a timber cladded ventilated rain-screen are considered by in one-dimensional WUFI® models. The simulation period for all of the timber frame walls was three complete years.

WUFI® Pro 5.0 already includes the climate data for two locations considered in the previous section: Bilbao and Warsaw. However, the climate for Edinburgh was obtained from Meteonorm v5.1 and exported in TRY format to WUFI®. The temperature, relative humidity, solar radiation and precipitation profiles are shown in Figure 3-42.

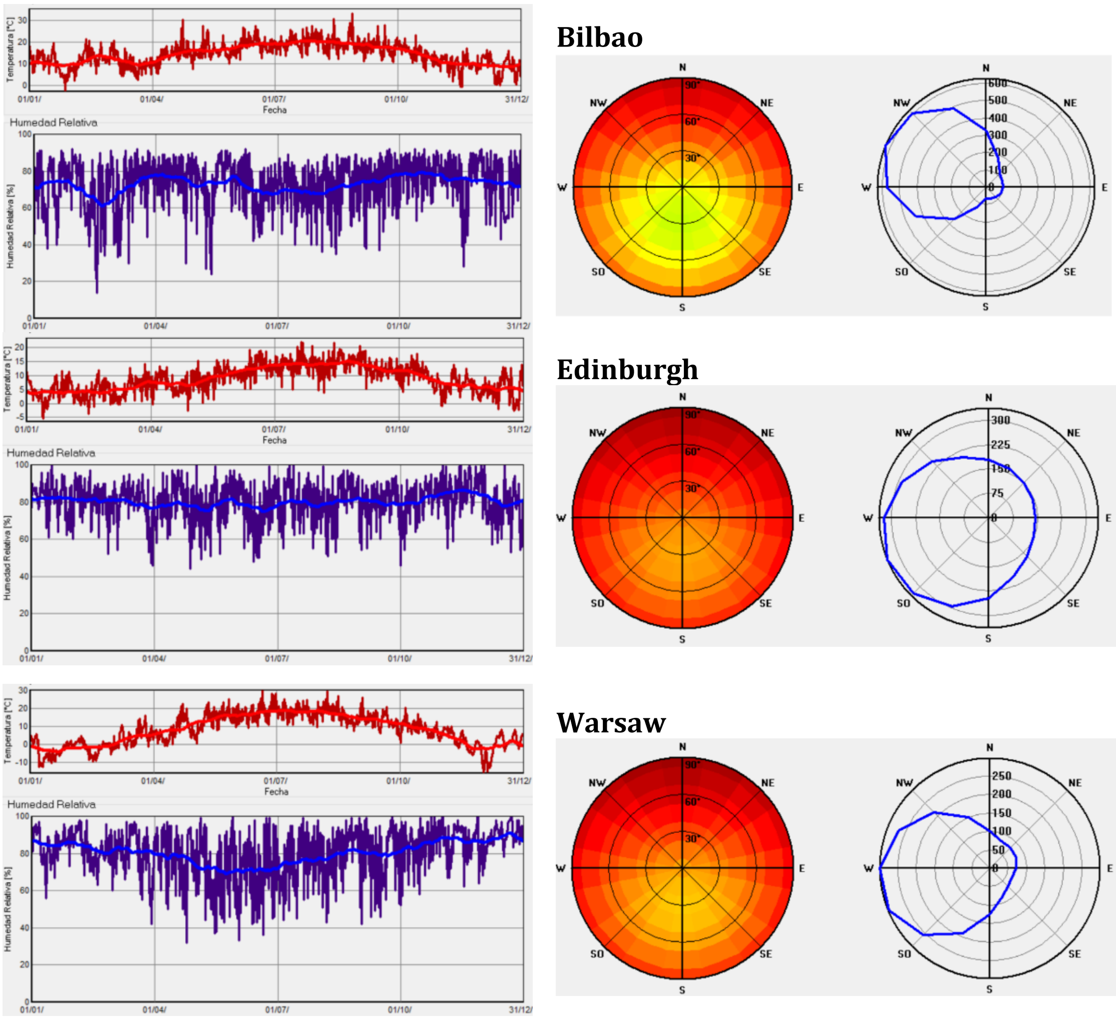


Figure 3-42 Climate profiles for Bilbao, Edinburgh and Warsaw. Tª (top left), relative humidity (bottom left), solar radiation and precipitation (right)

Other boundary conditions and initial considerations required to perform the hygrothermal calculations for the wall panels are detailed below:

- Inclination of the wall 90 º vertical
- Height of the building above ground less than 10 m
- Exterior surface heat transmittance of 17.0 W/m2K
- Internal surface heat transmittance of 8.0 W/m2K
- Internal indoor climate sine-curve low moisture load 45±15 % and sine-curve temperature 21±1 ºC
- West orientation of the wall due to dominant precipitation for all of the climates.
- Constant initial temperature profile through the assembly components. Initial temperature in the assembly 20 ºC
- Constant initial moisture profile through the assembly components. Initial water content of the assembly 80 %
- Moisture Content of wood based boards in % at the last period peak
- Isopleths as % RH on the interior surface

The total water content is measured at beginning of year zero and after the set three-year period. The aim of this activity is to consider all of the proposed building assemblies safe from a long-term increase of the water content within the envelope. A temporal short-term increase is acceptable if the humidity is lower than the initial water content.

Also, the moisture content of the individual layers containing wood-based materials was reported. The objective is to analysis the MC of the external OSB or wood fibre board insulation and the internal OSB board and to report any building assembly with more than 18% MC even for short periods of four days in order to avoid degradation and further decay processes (Dinwoodie, 2000; Faherty & Williamson, 1998).

Water content results did not show evidence of a long-term increase in the building assembly except for the benchmark timber frame build-up with additional insulation and for a cold climate scenario (Figure 3-43). The results plotted as an irregular line show a slightly higher water content within the assembly the colder the climate is.

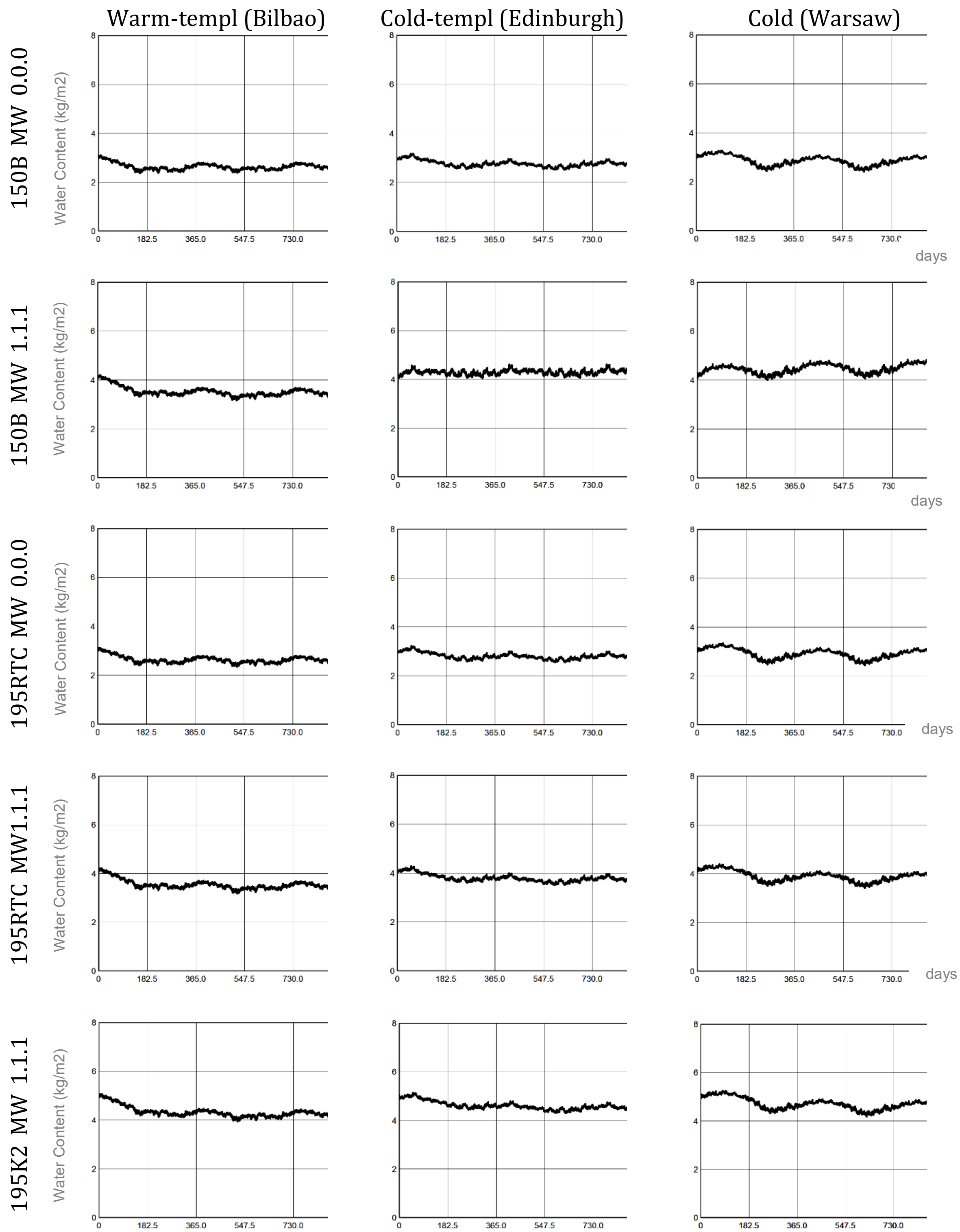


Figure 3-43 3-year period water content in kg/m^2 for construction assemblies.

In terms of humidity for individual components there was no wood based material exposed to moisture content greater than 18 % for external OSB/3 boards or even wood fibre insulation boards (Figure 3-44). The simulation was performed on the thinner wall panels as they are more able to accumulate water at colder temperatures.

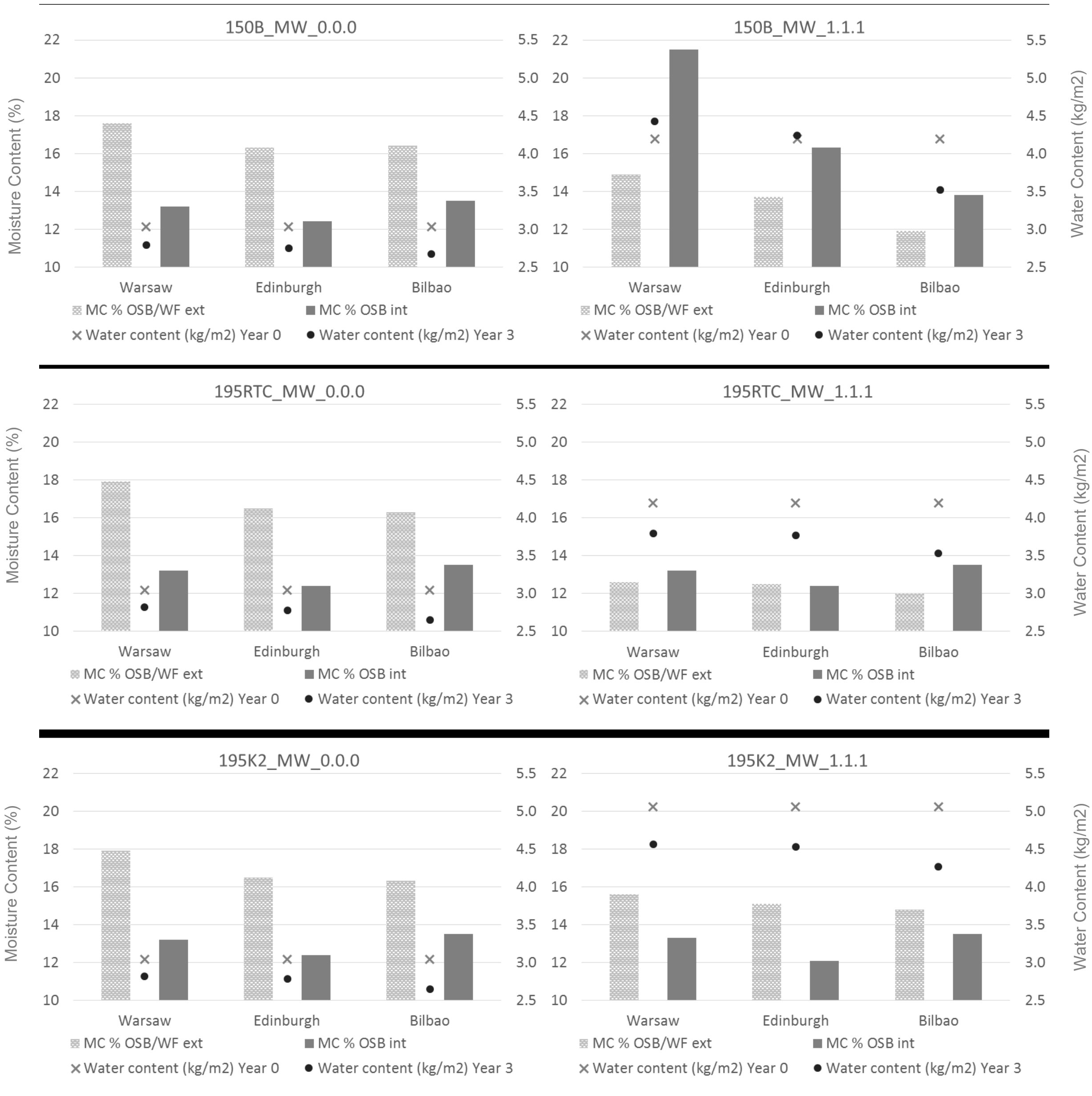


Figure 3-44 Total water content and variation of wood based boards

However, for internal OSB/3 board humidity, the benchmark building assembly with additional insulation provided a MC greater than 20 % for extended periods of time (~3 months) within a cold climate. This timber frame build-up is therefore, not recommended. Additionally, this timber frame build-up is the only suggested assembly that delivers a final water content higher than the initial one for cold and cold-temperate climates.

Another observation found was that additional insulation on the frame causes an increase of the water content of the building assembly which is climate independent. This is a common effect observed in WUFI as thicker assemblies tend to retain more moisture than thinner walls (Karagiozis et al., 2001).

The last parameter studied is the Isopleths diagram which provides the condensation occurring combination of relative humidity and temperature on the internal surface – Lowest Isopleth for Mould (LIM). The WUFI output for this parameter includes two limiting isopleths for building materials, LIM-B1 and LIM-B2, below which mould to occur is not expected. For the purpose of this research, LIM-B1 is the identified limit as the internal surface is a biodegradable substrate.

Supporting the conclusions obtained from these two parameters, water content and moisture content of internal OSB/3, the isopleths resulted (Figure 3-45) agree on the risk of mould growth occurring for the benchmark timber frame with additional insulation. Indeed, the building assembly corresponding to the panel 150B_MW_1.1.1 determines positive mould growth of that particular frame for the climates of Edinburgh and Warsaw. There is no risk in the other timber frame panels according to the temperature and relative humidity isopleths.

A full WUFI® report containing the full hygrothermal transient-state analysis for K2 and RTC closed panel timber frame systems and for the cold climate is included in the appendix VI.

The results of the WUFI® Pro analysis were coherent with the temperature factor and minimum surface temperature determined in the previous sections. However, for the transient-state methodology, the influence of the sole plate and the type of foundation was not considered.

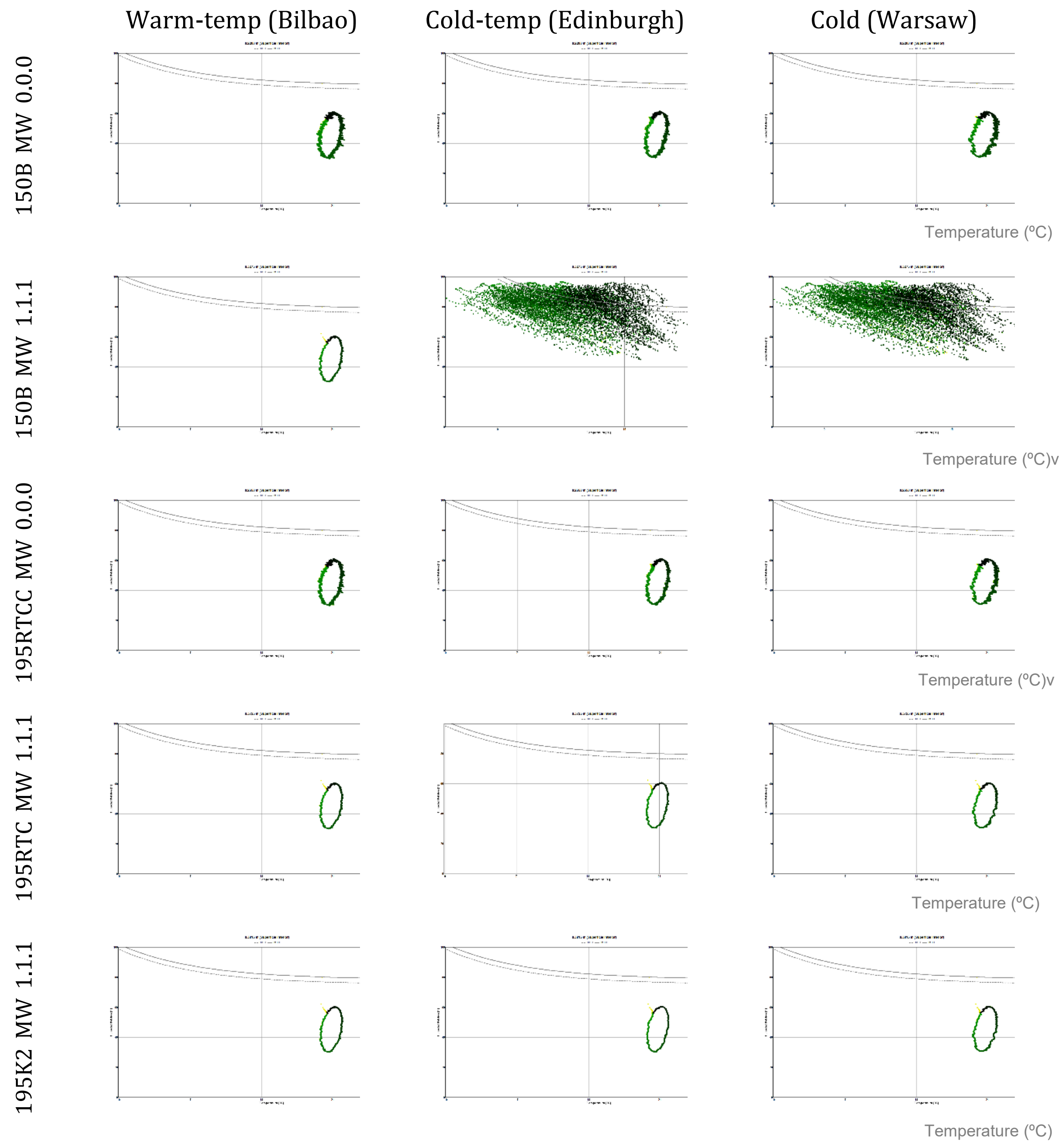


Figure 3-45 Isopleths for condensation occurring temperature and relative humidity.

3.8 Summary

This chapter proposed a series of advanced closed panel timber frame assemblies for low-energy buildings in different climate zones. The thermal properties of these wall panels were investigated in terms of timber fraction, heat transmittance, thermal bridging and moisture control.

Timber frame construction may accommodate a great level of insulation between studs, one of the most relevant factors to achieve low-energy buildings, with slender walls. This is a factor to consider when dealing with extremely energy efficient buildings if the design does not want to compromise the usable floor area. Mineral wool and flexible wood fibre insulation has been investigated in the wall assemblies with absence of gaps and air pockets. Sole plate details were suggested for two types of commonly found foundations: concrete slab on grade with XPS insulation on top and suspended timber floor insulated with mineral wool.

Specific literature on thermal performance was reviewed which identified a series of procedures on which to perform hygrothermal calculations on timber frame walls. From this review, different analytical and experimental methodologies in order to determine the thermal transmittance, U-value, were evaluated for both steady-state and transient conditions. In this research, for highly insulated new built timber frame buildings, two-dimensional steady-state simulations were found in good agreement with past experimental investigations.

Thermal resistance, U-value, was identified as the governing design parameter for low-energy and Passivhaus timber frame walls. Two advanced closed panel wall assemblies of 195 mm, 245 mm and 300 mm and one open timber frame of 150 mm were presented with two different sheathing configurations. The external façade, for simulation ease, was considered as timber clad rain-screen. The assemblies were internally finished by a service cavity which may be insulated. Commercially available materials with declared isotropic thermal properties were used in all the hygrothermal simulations. A complete description of the timber, sheathing, fasteners and membranes materials and its properties was provided.

Timber fraction values provided by literature may not be considered accurate or reflective of modern construction for highly insulated walls when geometrical and window installation thermal bridges are also accounted for. In most of the cases, timber fraction values taken from literature were found to be conservative. A methodology to determine timber fraction on energy efficient buildings was presented. Furthermore, the timber fraction determined for four different low energy projects estimated a mean value of 10 % which was on agreement with some published values.

The most significant thermal bridge found in traditional construction was the foundation to external wall detail which accounted for 50 % of the total linear thermal bridges in a typical dwelling. A comprehensive thermal bridging analysis on one hundred and forty details was carried out. The result of this investigation showed that the proposed closed panel configurations can be considered thermal bridge free. However, the action that contributed the most to mitigate thermal bridging was to place insulation on the foundation edge. The findings of the study also suggested that point thermal bridges at the sole plate, when no fully penetrating the detail, can be neglected.

The literature review also identified and helped collated the airtightness results of fifty-six dwellings where tests were undertaken after completion and over a set period of time. In two-thirds of the cases, the airtightness of the buildings increased after it was occupied. This conclusion was in agreement with the as measured “Performance Gap” theory.

Several methodologies for moisture assessment were reviewed. Minimum temperature factors, minimum internal surface temperatures, the Glaser method and one-dimensional dynamic hygrothermal simulations were all considered valid approaches for light-weight timber frame construction. Three climates corresponding to cold (Warsaw), cold-temperate (Edinburgh) and warm-temperate (Bilbao) were identified. All the methods highlighted the risk of the benchmark case scenario, and particularly the panel with additional insulation, in cold and cold-temperate climates. The closed timber frame panels, in all the cases, were found safe from moisture ingress and long-term storage.

This chapter has investigated the hygrothermal performance of two proposed closed timber frame panels. It has provided different wall panel configurations suitable for Passivhaus standard in three different climates. Also, the research has provided sole plate base details considered thermal bridge and condensation risk free. The following chapter will discuss the structural behaviour of the proposed solutions.

4 Structural Performance of Timber Frame Walls

In this Chapter, the structural performance of the two-advanced closed panel timber frames, regarded in the previous sections as optimal solutions, are analysed as shear walls. The investigation includes analytical and experimental methodologies to understand the structural performance of the wall and the sole plate connection detail in isolation and then in combination.

Once the results of the experiments were corroborated with the latest analytical approach, defined by PD 6693 and included in the UKNA of the Eurocode 5, a timber frame racking application software was developed to provide structural engineers with a design tool for direct specification and with BIM enabled capabilities. The organisation of the chapter is illustrated by Figure 4-1.

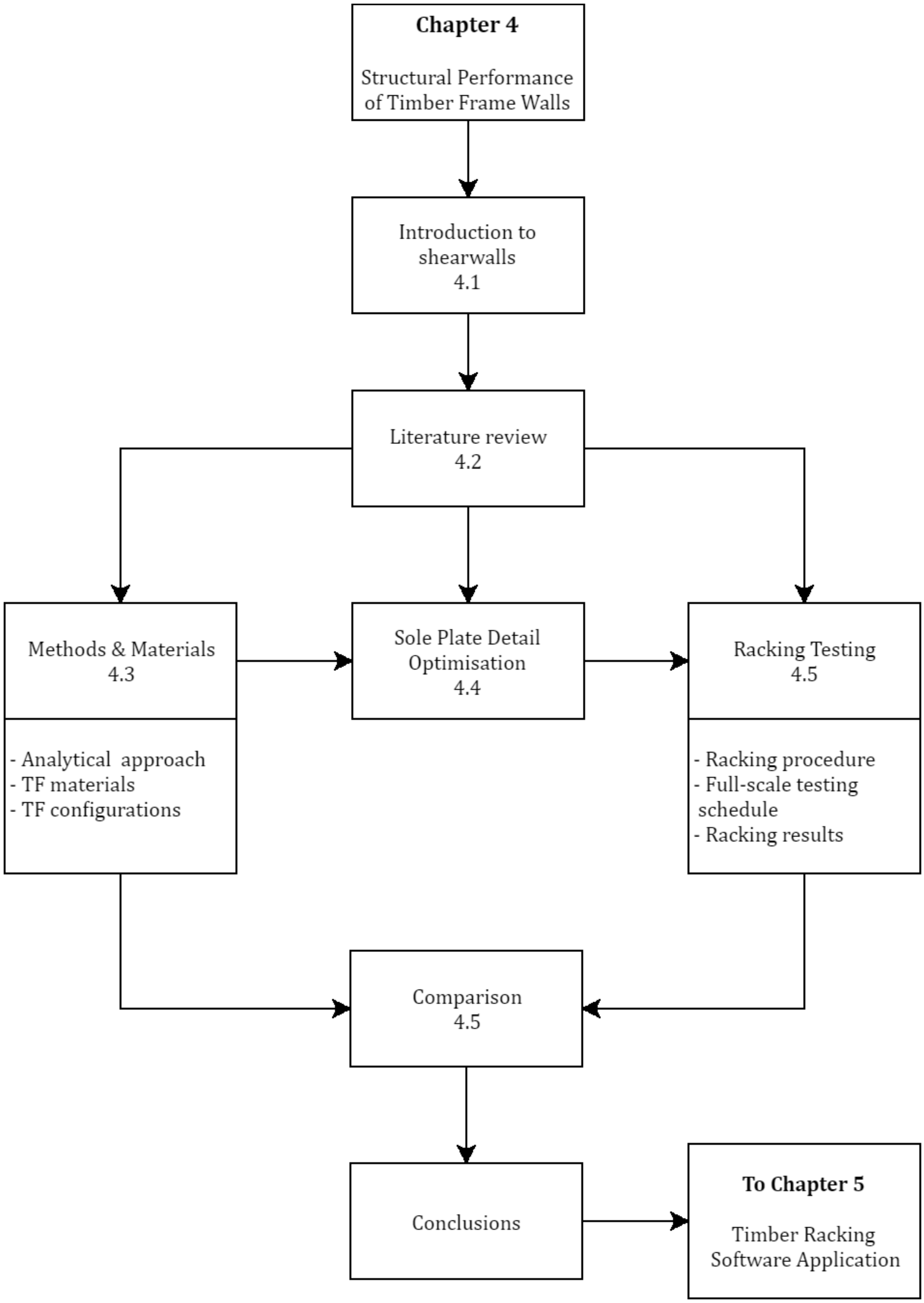


Figure 4-1 Organisation of the Chapter 4

4.1 Introduction to Shear Walls

Timber frame shear wall diaphragms are subjected to three major structural actions: vertical loading, out-plane actions due to horizontal loading, and in-plane lateral loading (TRADA, 2006). The capacity of the shear wall to withstand under the effect of these actions is determined by its strength. The structure will fail if the final action, alone or in combination, exceed the strength of the shear wall.

A shear wall is a structural sub-assembly able to act as a diaphragm in order to transfer horizontal building loads to the foundation. It is also convenient to note that an external shear wall is also a physical boundary element between the outdoor and the internal environment (ASTM, 2006).

The shear resistance of racking walls can be determined as load per unit length (kN/m) assuming that the load is distributed uniformly along the total length of the wall (Salenikovich & Dolan, 2003). In order to calculate shear capacity of timber frame walls, the permissible stress code BS 5268 (BSI, 2002) has been superseded by ultimate limit state i.e. Eurocode 5 (BSI, 2009b) design codes.

A paper published by Dietsch and Winter (2012) states the unsatisfactory acceptance of the Eurocode 5 in the timber sector. Previously, Silih and Premrov (2010) concluded that the afore mentioned method B, generally delivers higher strength values. These statements are also supported by IStructE (2007) as the two Eurocode 5 design methods for racking performance, method A & method B, were defined as incompatible with standard UK timber frame construction practice.

Abundant papers, related to ultimate strength of wood shear wall panels, have been published. Liu, Gopalaratnam, and Nateghi (1990) identified lack of wall resistance to both uplift and racking and lack of proper designed wall anchorage to foundation, as the main cause of excessive wood-frame house damage under high winds. The lack of structural redundancy, traditionally achieved by on-site continuous sheathing construction, results in a need of careful engineering design (Morsefortier, 1995). In order to ensure load transfer continuity, adequate anchor bolts or lag screws in the sole plate detail is identified as critical element in shear walls (Scott et al., 2005). This chapter provides a literature review on shear walls and investigates the structural performance of the advanced timber frame panels, K2 and RTC, from a global perspective including design methodology, materials and sole plate base fixing details.

4.2 TFS Structural Performance Literature Review

Timber wall panels are defined by multiple variables which relate to both internal and external boundary conditions. Substantial research has been done in the field of shear walls mostly in North America, Europe, Japan and New Zealand, especially during the last two decades.

In order to overview the existing literature in an organised and structured manner, related variables influencing the behaviour of racking walls were gathered together and classified according to the boundary conditions and limitations of the research undertaken. This classification is visually represented in Figure 4-2 where variables considered in this thesis are noted in bold.

A recent Scandinavian publication by Labonnote (2013) summarises state-of-the-art research on timber frame wall diaphragms. In this report, the author suggested the creation of a master database containing latest research on timber shear walls grouped by different domain areas (see Table 4-1). Furthermore, a collection of timber frame racking tests carried out at Edinburgh Napier University are provided in the Appendix VIII.

Table 4-1 Database format proposed by Labonnote (2013). Adapted.

Source	**Date**	**Sheathing**	**Boundary conditions**	**Loading**	**Analysis**
Author	1900	OSB one sided	Nail Vertical load Fully anchored	Static EN 694	ULS SLS Failure types
…					

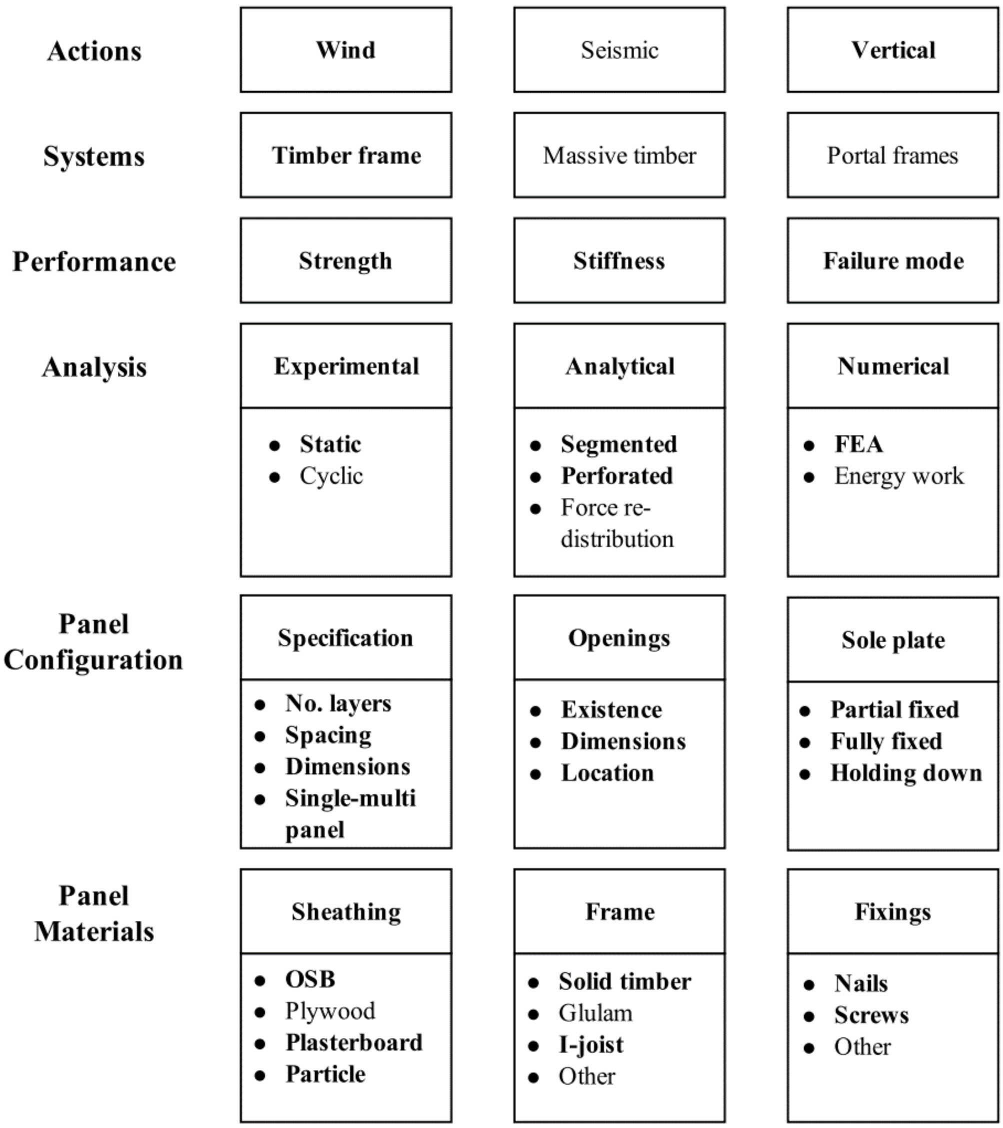


Figure 4-2 Variables influencing the design of shear walls

4.2.1 Structural wall diaphragms

The strength and stiffness of shear walls is usually determined either by experimental testing of individual wall panels, by analytical equations derived from engineering principles and by numerical methods (Alsmarker, 1995; Dolan, 1991; Doudak, 2006; Salenikovich, 2000).

Eurocode 5 currently provides two different methods to determine the racking capacity of shear walls. Method A is based on equilibrium where holding down devices are required at both stud ends. Method B, compulsory in UK and optional in Denmark only, is based on a large series of tests where holding down is not used. In this method, the anchorage force to resist uplift is provided by the bottom rail fasteners which also prevent lateral sliding. This process generally delivers lower shear resistance than method A.

Method B is based on the empirically design method established on the now superseded British Standard BS 5268 which relates to the test behaviour of nearly two hundred wall panels (Griffiths, 1987). However, this methodology has been questioned due to misinterpretations from the original British code (Kallsner & Lam, 1995).

The fact there is not a clear analytical rationale, in line with the Eurocode approach behind the outcome of method B, resulted in the recent publication of an alternative method informally named method C and published as PD 6693-1 (Griffiths, et al. 2005). This analytical method does not require full tie-down of lead and trail studs (opposite to method A). However, this method accounts for other indirect holding down devices such as returning walls or vertical loading. Moreover, PD 6693-1 considers racking contribution for the sheathed area around openings (as in method B). Contrary to both method A and B, the new design methodology allows the racking contribution of plasterboard-only sheathed timber frame walls.

According to these Eurocode methodologies, the full-height wall panel area enclosed by a window opening does not contribute to racking resistance in method A but it does in methods B and PD-6693.

Finally, another method named *"force transfer around openings"* has been recently researched and tested in the US (Yeh et al, 2010). In this method, the shear wall is reinforced by metal straps or other plate material i.e. plywood around the opening. The method then analyses the re-distribution of forces around the opening.

For the analytical methodology, the now published PD6693-1 has been followed through-out this research (BSI, 2012).

4.2.2 Timber frame panel materials

Before a review of the structural shear wall performance as a component, it is necessary to consider the mechanical properties of wood-based materials and metal work components used in the fabrication of timber frame wall panels. This section collates relevant information on sheathing boards, sheathing and frame fasteners and anchorage metal work.

Sheathing plates

Sheathing boards have a direct impact on the racking performance provided by its shear modulus and particularly, by the contribution to the lateral load carrying capacity as part of a connection together with sheathing fasteners. According to EC5, sheathing materials shall be wood-based materials and must ensure a minimum distribution of shear forces from the board to the frame. These wood-based panels ensure a minimum level of ductility characterised on the load versus strain curve. For a certain fastener specification, three different equations are given in EC5 for plywood, hardboard and particleboard or OSB. Hence, the ductility of the sheathing connection is directly proportional to the embedment strength of the connection: Equation 4.1 to Equation 4.3 (Buckingham, 1914).

$$f_{h,k,plywood} = 0.11\ \rho_k\ d^{-0.3} \tag{4.1}$$

$$f_{h,k,hardboard} = 30\ d^{-0.3}\ t^{0.6} \tag{4.2}$$

$$f_{h,k,particle-OSB} = 65\ d^{0.7}\ t^{0.1} \tag{4.3}$$

Where:

ρ_k is the characteristic density of the sheathing board

d is the diameter of the fastener

t is the thickness of the panel

Different publications agree that shear walls sheathed only with plasterboard can transfer little in-plane shear forces. If this is assumed, then the contribution to the overall stiffness of the three-dimensional structure can be ignored (Asiz & Smith, 2011; Kozem, (2016); Premrov, 2012; Thomas, 2010). However, these studies completely neglect the overall contribution of plasterboard and under estimate the global stiffness of the structure. On the other hand, the UK Non-contradictory complimentary information (NCCI) document PD6693-1 provides conservative design shear capacities for different configurations of plasterboard sheathed walls. Other experimental studies observed a modest racking resistance contribution of gypsum-based boards (Yasumura & Kawai, 1997).

Similar conclusions were reached in two other different studies by (Serrette, Encalada, Juadines, & Nguyen, 1997) where the shear strength of light-gauge steel frame sheathed with OSB only and with OSB and plasterboard combined were presented for static and cyclic tests. Table 4-2 compares the static test results of six light-steel frames sheathed with OSB only and OSB and 12.7 mm plasterboard. The results indicate that the contribution of plasterboard as additional sheathing layer is greater on shear walls with lower shear capacity.

Table 4-2 Summary of static tests, after Serrette and Ogunfunmi (1996)

Ref no	Sheathing thickness (mm)	Screw spacing (mm)	Shear OSB (kN/m)	Shear OSB+GYB (kN/m)
1A2, 1A3	11.1	152.4	1.235	1.649
1D3, 1D4	11.1	101.6	1.914	2.115
1D7, 1D8	11.1	50.8	2.592	2.554

Furthermore, an analytical study showed that fibre-plaster materials, in seismic or windy areas, present a significant loss of structural stiffness resulting in the creation of cracks in the plasterboard (Dobrila & Premrov, 2003). This outcome, as investigated in the previous chapter, could have a significant impact on the dilapidation of airtightness over time for Passivhaus and very low-energy buildings. In relation to other wood-based materials, a study with small-scale tests showed lower shear capacity on walls sheathed with OSB rather than with plywood (Beall et al., 2006).

The EC5, and particularly the PD 6693-1, include a comprehensive set of application rules for the adequate arrangement of the sheathing boards within the timber frame. Also, minimum board width dimensions depending the location of a sheathing sheet within the panel are given to provide sufficient stiffness. Experimental work confirmed that sheathing arrangement has an impact on the wall stiffness (Cheung et al, 1988). Additionally, an investigation undertaken by Lam et al., (1996) concluded that timber frame sheathed with oversized OSB panels under monotonic loading experienced a significant increase in both strength and stiffness.

Apart from the sheathing material, research undertaken in the United Kingdom suggests that brick and block skin façades of mid-rise timber frame buildings can increase the shear capacity of the walls due to its contribution as a system to the racking resistance and to the wind shield (TRADA, 2006). This outcome is now included in the non-contradictory complementary document to Eurocode 5 for the UK (BSI, 2013).

Although most recent experiments are performed with OSB as sheathing material, earlier studies on this subject were performed with plywood (Ni et al., 1999). However, the economy and availability of plywood for structural use nowadays is limiting its specification on timber frame panels. A study on wood fibre insulation boards conducted by Gebhardt & Blaas (2009) published embedment strength, pull-out, pull-through values and stiffness values of wood fibre connections to staples and nails. These values may be used to determine the shear resistance of timber frame walls sheathed with wood fibre insulation boards for roof, floor and wall systems, as suggested in the previous chapter.

Frame members

Structural floor diaphragm design includes timber blocking between joists to ensure the system performs as a true diaphragm. However, shear wall diaphragms not always present timber blocking at transversal sheathing edges. This situation is more commonly found when sheathing boards are installed horizontally to the frame. Ni & Karacabeyli, (2005) investigated horizontally sheathed unblocked shear walls and concluded that a reduction factor applies to the design shear strength depending on stud and nail spacing.

The orientation of the OSB sheathing boards in the timber frame members is not arbitrary. In order to maximise the racking resistance and minimise horizontal deflections, panels should be orientated parallel to the frame, especially if the frame is unblocked (Leskela, 2005).

Sheathing and frame fixings

Nail or screw spacing is a key variable influencing the design of shear walls. Nail perimeter spacing is restricted by Eurocode 5 to a maximum of 150 mm and a minimum of 7 times the fastener of the diameter (BSI, 2006). The spacing of internal fasteners is, by common practice, specified as twice the perimeter fastener spacing. This assumption is corroborated by past studies (Cheung et al., 1988) and thus followed on this research. Nevertheless, an investigation from North America was undertaken with an internal nail spacing of four times the perimeter spacing which was considered apt to be used for design rules (He et al., 2001).

Moreover, Chung & Yu (2002) concluded that the most relevant parameter influencing the structural performance of timber shear walls is the connection between the frame and the sheathing material rather than the mechanical properties of both the frame and the sheathing material.

On the other hand, contribution of the vertical shear capacity by the frame fasteners is most commonly ignored. As part of research carried out in Sweden, the model proposed by Girhammar et al., (2004) achieved, in partially anchored walls, a 15% increase on the shear strength capacity due to the inclusion of the vertical shear contribution from the frame fasteners. However, this model relies on the lower bound plasticity of the fasteners where the conditions for equilibrium are not always fulfilled.

The lateral load carrying capacity of both sheathing and frame fasteners is directly related to the yielding capacity of the fastener. Plenty of research in this area has been done during the last fifty years. Mechanically fixed sheathing-to-frame fasteners, with plastic behaviour, are utilised in most of the analytical models discussed in the literature review.

4.2.3 Timber frame panel configuration

The understanding of the components in isolation for timber frame closed panels is the first step in order to characterise the mechanical properties of the system in combination. A comprehensive review of past research regarding panel specifications and openings is presented in this section. Research about the anchorage method of the wall to the substrate is also included.

In the previous section, an overview of timber frame racking panel specifications and naming conventions was introduced. This sub-section provides further literature of timber frame shear walls for both conventional and non-conventional material specifications. It must be mentioned that, in a building with different timber frame shear wall configurations and lengths, the stiffness of the walls is not equal. This assumption may lead to over-designed or under-designed load bearing walls (Kasal et al., 2004).

Openings

There is a wide agreement that openings have a significant impact on the strength and stiffness performance of shear walls. Three-dimensional modelling investigations undertaken by He et al. (2001) on regular timber frame cubes and with a large opening in one side (75% of wall area) concluded that global shear capacity of the structure was reduced by almost 50%. In the same study, the torsional moment generated on the model caused significant differential deformations at each of the top corners. Other studies have also noted the occurrence of shear torsional moments on buildings with asymmetric distribution of shear wall stiffness (Ellis & Bougard, 2001).

A conservative approach, with no contribution of the area of the sheathing above and below of the opening, is generally considered in many countries. However, analytical (Ge et al., 1991) and numerical (Guan & Zhu, 2009) methods considering this area were compared with empirical results with good agreement. A recent study performed by Skaggs & Martin (2002) introduced a new model based on the result of transferring forces around openings.

The results of another study performed by Yasumura (2000) lead to the conclusion that non-linear analytical models can underestimate considerably the racking resistance of perforate timber frame walls. Other simplified methods for designing shear walls with openings consider the wall as the addition of multiple panel segments when holding-down devices are installed at the end of each wall segment (BSI, 2009b; Ni, Karacabeyli, & Ceccotti, 1999).

Sole plate base fixing anchorage

Modern methods of timber off-site construction involving shear walls rely heavily on the structural behaviour of the connection to the substrate. Two different approaches on the sole plate detail are found in the literature: partially and fully restrained timber walls.

Fully anchored walls include holding down devices at the leading and trailing studs of the wall assembly. Perforated walls with a design methodology based on multiple segment walls also require holding down devices on cripple studs.

Partially anchored walls do not present holding down devices. Partial vertical restrained is provided by positive vertical loads and by a percentage of the lateral load carrying capacity contribution from the bottom sheathing fasteners. Most of the studies and analytical models are validated for fully anchored panels (Kallsner & Girhammar, 2009). However, potential difficulties on the constructability of prefabricated sole plate details resulted in the need for partially restrained solutions. In line with this demand, the release of PD 6693-1 provides the engineers with a well-defined and validated methodology. This analytical method is supported by previous research undertaken mainly in Sweden (Girhammar & Kallsner, 2004). The basis of the design accounts for some degree of vertical uplift resistance by the bottom runner sheathing fasteners.

Conventional anchorage construction practise also includes the installation of a series of bolts at regular intervals on the sole plate. A recent study undertaken by Yeh et al. (2010) integrated the concept of Optimal Value Engineering (Bell & Overend, 2001) and resulted in design tables for different bolt spacing depending on combined shear and uplift forces.

A stress distribution, for fully anchored shear walls and according to the plastic lower bound theory, was derived by Kallsner & Girhammar (2009). This theory assumes flexible frame members and fasteners acting a full plastic capacity causing a parallel force distribution in the sheathing material. This conclusion was developed from previous studies where a simplified plastic model for the design of partially and fully anchored shear walls were proposed and then corroborated (Girhammar & Kallsner, 2004; Kallsner & Lam, 1995).

An alternative approach to partially restrained wall panels was published by Ni et al. (1999) as a guidance for the withdrawal Canadian Standard for engineering Design in Wood (CSA) in the 2001 edition. One of the conclusions of this study is the importance of understanding load paths in timber frame building systems.

Apart from the degree of anchorage restraint, the specification of the anchor bolts on the sole plate may influence the global structural performance. When combining bi-axially lateral and uplift forces, brittle failure of the timber on the tangential to the grain direction were observed (Yeh et al., 2009). This is especially relevant on areas with very high wind loads. Furthermore, Girhammar and Kallsner (2009) also observed cross-wise bending and further splitting failures at the sole plate. In order to mitigate this effect, the authors suggested washer specifications and end and edge distances in the sole plate.

A numerical expression, from fracture mechanics, was suggested by Serrano et al. (2011) to estimate the design load for vertically loaded bottom rails. Good agreement between the FEA model and the experiment was found for partially anchored walls.

4.2.4 Analytical Methods

As mentioned previously, Eurocode 5 currently presents two simplified analytical approaches to determine the racking resistance of timber frame shear walls: Method A and Method B (BSI, 2009c).

Method A is based on a linear elastic model (Figure 4-3) where the walls are fully restrained by holding down devices at the leading studs and at cripple studs around openings. Design verification for this method requires substantial time as a check for each full sheathing sheet within a perforated panel shall be carried out.

PD 6693-1 is a design method based on the withdrawal BS 5268-6.1 where test-based values were converted to factors and formulas. However, this method did not follow the analytical principles of Eurocode 5 and un-conservative values were reported (J. Porteous & Kermani, 2007).

This method PD6693-1 was published after several years of research as a non-contradictory complementary document that support the Eurocode (BSI, 2012). This analytical method is now adopted in UK and, at the time of writing, it is being introduced in other countries like France or Spain. In this case, the racking analysis methodology is an adaptation of the simplified plastic model developed by Griffiths et al. (2005) and Kallsner and Girhammar (2004).

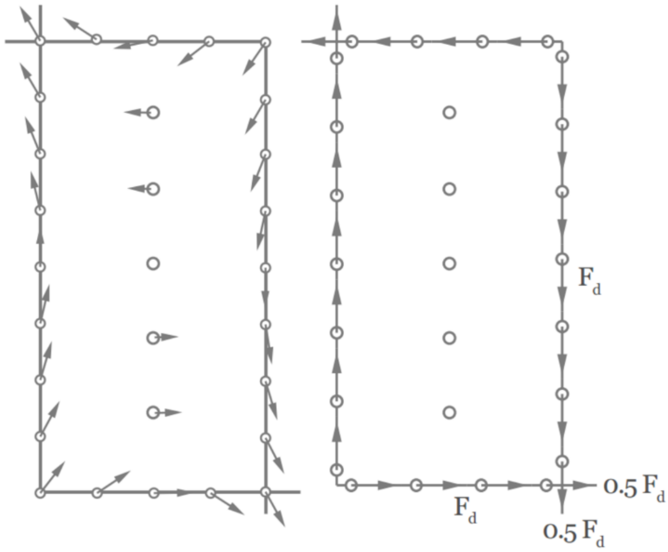


Figure 4-3 Linear elastic (left) and pure plastic (right) force distribution theory

The analytical study carried out in this research focuses on this method C. A further explanation of the basis for timber frame racking design according to this theory is provided in section 4.3.1: Racking design methodology.

Apart from Eurocode 5, other standards providing racking resistance of timber walls can be found (ANSI, 2011 IBC, 2009 NZS 3603, 1993). The methodology included in these standards is the result of previous broad research. In this section, a comprehensive review of research on analytical models for the design of timber frame shear walls is provided.

The analytical models for the design of wall panel segments with perforated openings may provide racking capacity or may be disregarded for that purpose (BSI, 2009b). A desktop study conducted by (Kozem Šilih & Premrov, 2012) concluded that timber frame walls with door openings could take up to 50% of the ultimate racking resistance of the equivalent imperforated wall.

A study comparing two different analytical methods, multiple shear wall segments with and without openings, based on Phillips et al. (1993), was performed by Ni et al., (1999). This study concludes that the analytical methodology including openings provides a better prediction when compared with experimental past results.

In terms of deformations, PD 6693-1 does not provide any analytical procedure to determine racking stiffness (BSI, 2012). Nonetheless, clause 21.5.2.3 provides a condition to limit racking deflection (Equation 4.5). In terms of testing, shear stiffness is commonly evaluated at displacements measured at 20 % and 40 % of the ultimate load (Dujic et al., 2008; Porteous & Kermani, 2005) and determined in N / mm in Equation 4.6.

$$K_{i,w}\, f_{p,d,t} \leq\ 8\,(1 + K_{comb})\left(\frac{L}{H}\right) \tag{4.5}$$

$$R = \left[\frac{F_4 - F_2}{\nu_4 - \nu_2}\right] \tag{4.6}$$

4.2.5 Experimental Methods

Testing timber frame wall panels for racking resistance in isolation may not represent the actual boundaries of the design in service. Issues such as a three-dimensional construction, floor diaphragm action, upper loading contributing to stability and an overall understanding of the systems requires a more holistic approach. Stiffness under cyclic responses can be also measured as the relationship between maximum and minimum forces. Their corresponding displacements are determined for the seventh cycle as the previous six cycles are used to check the equipment and settlement (Fonseca et al., 2009)

Relationship between the nail slip and shear wall deformation direction, out of plane, was investigated by Kamiya (1987). In this case, it was concluded that shear walls have a higher buckling capacity even considering these eccentrics applied loads. Also, the rigidity of the sheathing to frame joints positively impact the composite frame-to-sheathing action. As a consequence of the different wall stiffness, there is an offset between the centre of the stiffness and the centre of the building impinging upon the torsional stability. In this experimental study, the most accurate method to predict the load distribution was the finite element analysis followed by the plate model. However, the determination of the different wall stiffness proved to be rather difficult.

A mathematical model combining non-linearity of fasteners and vertical loading and corroborated by laboratory testing was performed by Dujic & Zarnic (2002). The authors, in this case, suggested that non-linear behaviour of sheathing fasteners and anchors govern the racking behaviour for low and medium racking forces.

The effect of vertical loads and holding-down restraint were tested in cyclic behaviour by Johnston, Dean, & Iii (2006). The authors concluded that lateral stiffness increases up to 80% with walls with vertical loads of 25 kN/m whilst holding-down anchors have minimal impact on the stiffness of the shear walls.

In Canada, seven different test schedules were performed by Doudak et al. (2006).The investigation takes into account multiple uniformly distributed load (UDL) on top of the top runner, the inclusion of openings and different types of anchorage (Nelson et al., 1985). Similar to most of the studies reviewed, this test does not take into account the boundary conditions representing the action of the rest of the building elements like shear walls at the other direction or the effect of floor diaphragm.

The past UK TF2000 research project (Ellis & Bougard, 2001) experimentally evaluated the stiffness of a timber frame platform building according to the different levels of internal and external finishing. It was demonstrated that finished buildings are much stiffer than the sheathed frame alone, although this incremental effect, is difficult to calculate. Also, frequent poor correlation between the experimental and the predicted analytical stiffness on timber frame shear walls on 3D wall-to-floor systems was observed by Filiatrault et al. (2002) On the other hand, a pseudo-dynamic test was carried out to estimate 3D behaviour of shear wall structures (Silih & Premrov, 2010). Parallel, Andreasson et al. (2002) concluded on his study that the behaviour of shear walls, at low levels of displacements, is best described by non-linear analysis. This contrast other studies where a linear relationship is assumed for nailed joints at low levels of load (Kamiya, 1987). On the other hand, according to He et al. (2001), the linearity of the displacements at low levels may be assumed for shear walls of low stiffness whilst stiffer shear walls present non-linear displacements even at low applied loads.

An extensive racking test program was carried out by Leitch (2013) at Edinburgh Napier University as part of a doctoral research. Over forty different wall types were tested for a total number of 112 individual panels. As a result of the large variability of the samples, the author proposed a naming convention to fully describe the wall panel specification (Table 4-3). This naming convention is followed throughout this research.

Table 4-3 Naming convention for racking test, after Leitch (2013)

Specification	Naming	Comments
Restraint conditions	P	Partially restraint
	F	Fully restraint by means of holding down
Wall length	L[ ]	Length of wall. Default 2.4 m
Nailing spacing	s[ ]	Internal spacing twice perimetral. Default 150 mm
Vertical load	V[ ]	Point load considered as UDL. Default 10 kN/m
Layers	DS[ ]	Material of double sheathed specimen (if any)
Stud spacing	st[ ]	Indicates stud spacing. Default 600 mm c/c
Sheathing width	sh[ ]	Minimum sheathing width. Default 1200 mm
Opening	w[a x b]	Area of opening and located at Windward end
	l[a x b]	Area of opening and located at Leeward end
	c[a x b]	Area of opening and located at Centre

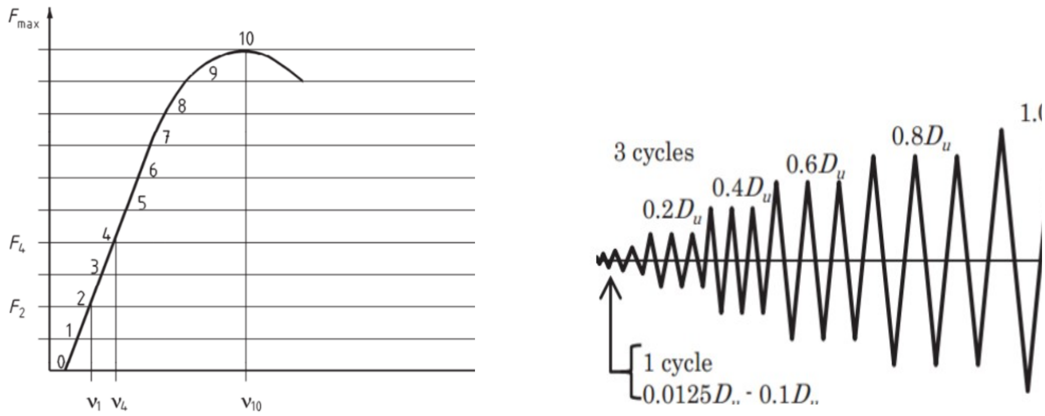
The large dataset of experiments gathered in this thesis shows significant differences between analytical and experimental stiffness on racking wall panels with high aspect ratio. The analytical linear stiffness is based on the maximum lateral deflection of a cantilever beam theory. Here, it can be observed deformations, δ_v, due to shear and deformations, δ_m, due to bending according to the transformed section method (Parida et al., 2013).

Nevertheless, alternative experimental studies have showed non-linear relationship between stiffness and wall lengths (Girhammar & Kallsner, 2009).

Furthermore, an experimental study on the structural performance of scaled racking wall panels provided good agreement to predict the maximum load and initial stiffness (Lee & Hong, 2002). However, the load-strain deformation curves were unsatisfactory. Additionally, a recent study compared a series of forty-three static and dynamic racking test with several analytical methods. In particular, EC5 - method C resulted the most conservative method although the overall prediction was a good estimation (Salenikovich & Payeur, 2010).

Review of international test methods

In Europe, apart from Eurocode 5, the racking resistance of timber frame shear walls can be determined by test undertaken to EN 594 (BSI, 2011). The loading schedule specified in this standard has been recently updated (Figure 4-4 (a)).



a) EN594 / ASTM E564 b) ISO 21581

Figure 4-4 Loading protocols of selected test methods

In this update, although some recommendations were taking into account e.g. transverse bending of bottom plate, other suggestions from previous research were not included. The most notable suggestions were about the adequacy of testing the wall in pure shearing (Wu et al., 2002) or the adoption of more neutral configurations for considering vertical loading (Dujic & Zarnic, 2002).

Another optional standard is the ISO 21581 Timber structures - static and cyclic lateral load test methods for shear walls which is the outcome of other investigations for racking behaviour in seismic areas (Yasumura & Kawai, 1997). This methodology and the loading protocol (Figure 4-4 (b)) was followed to predict strength and stiffness of 3D diaphragms (Kamada, Yasumura, Yasui, Davenne, & Uesugi, 2011). The ASTM E564-06 is another static monotonic method to determine the shear capacity of light-frame walls on a rigid foundation and the loading protocol is similar to BS 594. This standard is not exclusive for timber frame and allows for light-gauge steel frame walls to be tested. However, ASTM E564 but does not evaluate combined bending and shear.

In order to use shear strength values from test results, a minimum of two tests of the same specimen are required if the shear resistance of each sample is within 10% of each other. Otherwise, at least three samples must be tested of the given construction detail (Labonnote, 2013). This was not considered in the experimental research due to the availability of wall samples. Hence, the experimental results presented in the next section of this research should be interpreted with caution if the shear resistance difference between same samples is greater than 10 % of each other.

4.2.6 Other Research Methods

In this section, a brief critical review of other two common methodologies is also given as they were referenced for future work.

Numerical Methods

The most popular numerical method to understand and estimate structural, solid and fluid mechanics is Finite Element Modelling (FEM). An overall review of research on this topic related to wood both as a construction material and as a product with more than three hundred entries can be found in Mackerle (2005). Nevertheless, the application of FEM techniques within current structural engineering practises is rather limited due to the time and cost involved in the definition of both boundary conditions and material properties. This has been reported even in the automotive industry (Bylund, Isaksson, Kalhori, & Larsson, 2004). However, in this sub-section a review of research on shear walls, involving FEA, is presented.

A parametric analysis based on a FEA model was performed by Dujic et al. (2008) on twelve shear wall timber frame and X-LAM panels with different opening-to-area ratios. The maximum story drift was set to h / 200 or 0.5 % but specimens with the larger openings failed at lower drift. An interesting outcome of this study states that although the strength of X-LAM panels is significantly greater than the equivalent size timber frame wall, the difference in stiffness, for any opening ratio, is less significant.

Another comparison study between a numerical methodology based on both a FEA shear wall model and laboratory tests concluded that a good agreement between both methods was achieved for the overall system deformation but significant differences occurred for horizontal forces only (Mi et al., 2004). Similarly, Davenne, Daudeville, Kawai, & Yasumura (1997) concluded that 3-D models with coupling devices need to be developed for better prediction of reality. Also, for accurate predictions, the mathematical model for the mechanical fasteners introduced in the FEM analysis should be derived from experimental results (Dujic & Zarnic, 2002).

Energy work

Another less common mathematical method to estimate the performance of structural systems is by the law of conservation of energy in an isolated system. The energy absorption of the sheathing board and fasteners is the based for other numerical models. Some studies on this topic are introduced below.

An elastic energy work model for predicting shear strength under racking load was suggested by Tuomi & McCutcheon (1978). Similar models were proposed assuming a pin-jointed timber frame with infinitive support stiffness (Källsner, 1984). A refined model of the previous study, including non-linear slip behaviour of the sheathing fasteners and linear shear deformation of the sheathing plate, was presented years after by McCutcheon (1985).

4.2.7 Timber Frame Failure Modes

Apart from the obvious consequences of structural collapse, timber frame wall panel failure modes are also investigated to understand the potential impact of structural movements on the performance of the building. Partial or even minimal failure of the system can cause discomfort to people and ultimately damage to the structure. Furthermore, partial and unnoticeable failure of the structure can cause air infiltration leakages within the building envelope.

It can be found in several studies, for partially restrained panels, that the most common modes of failure occur in the form of uplift of leading stud and specially, nail withdrawal from the sheathing board (Dinehart & Shenton, 1998). Also, for perforated wall panels, significant and predominant sheathing tearing can be observed around the corners of the opening due to high stress concentration levels. This failure mode is attenuated if sheathing board is fixed flush to the frame. Similarly, Kawai & Okiura

(2003) identified and prevented premature failure of sheathing fasteners at the corners by means of reinforcements. Another method to strengthen the racking resistance of shear walls with openings under cyclic load was to sheath the timber frame with oversized OSB panels (He, 1997).

A general failure mode was also described by Leitch (2013). Horizontal deflection or “drift” as result of rotation of the sheathing plate and the subsequent yielding of the sheathing fasteners was followed by a pull through of the head of the fastener at the bottom windward edge of the sheathing. This type of failure was also observed by Salenikovich & Dolan (2000) and described it as “unzipping”. This brittle failure is also common when the sheathing board is fastened by screws instead of nails (Kobayashi & Yasumura, 2011).

In partially restraint shear walls, Leitch (2013) also observed yield fasteners at the bottom rail due to uplift reaction. As a result of this behaviour, a lead stud to bottom frame connection tend to fail prematurely. This failure has been also commented by other authors under different loading protocols (Caprolu, Kallsner, Girhammar, & Vessby, 2012 U. A. Girhammar & Kallsner, 2009). However, the failure mode of racking timber frame panels, in terms of fastener behaviour, can vary significantly under monotonic or cyclic test conditions (Lam et al., 1996).

In order to disregard failure of the timber frame wall due to buckling of the sheathing material, PD 6693 only provides just a simple check (Equation 4.7). Alternatively, Kallsner & Lam (1995) provide an equation to determine the critical stress, as if the sheathing material is subjected to a constant shear stress, before buckling occurs (Sugiyama & Uchisako, 1991).

$$\frac{b_{net}}{t} \leq 100 \tag{4.7}$$

Where:

b_{net} is the clear distance between studs and,

t is the thickness of the sheathing board

As largely presented in this literature review, connections and particularly yielding of the fasteners are responsible for most of the failure mode observed in timber frame walls. Hence, a short review of the yielding theory applied to metal fasteners to wood is given below.

Yield theory for timber connections

The theory of linear-elastic beam foundation or Winkler theory of beams is a simple method to predict the behaviour of connections. However, this mode, in attempt to fit a linear-elastic load-slip relationship, is not very accurate (Jumaat et al., 2004).

The European Yield Theory considers a perfect plastic approach by assuming a uniformly distributed reaction to wood crushing along the length of the fastener. This theory is based on Johansen (1949) for bolted connections and was completed by Möller, (1950) for nails and by Larsen (1969) for screws. Furthermore, the model considers two effects on the connection: embedment or crushing of the wood fibres and yielding of the fastener forming zero, one or two hinges. Although the model is widely used due to its relative simplicity and accuracy for the determination of lateral load capacity of dowel and bolts connections, the theory is incomplete as it does not predict deformations. Subsequent studies have identified the dowel diameter and the member density as the key variables on the determination of connection stiffness (Heine & Dolan, 2001; Porteous & Kermani, 2005).

Over-strength is usually necessary if ductility shall govern the design. This stiffness connotation implies avoiding brittle failure mechanisms for timber structures. A general connection over-strength ratio of 1.60, corresponding to utilisation factor of 0.625, was proposed by Jorissen & Fragiacomo (2011).

According to the different failure modes for panel to timber connection, the fastener aspect ratio is considered the principal parameter (Tjeerdsma et al., 1998). This geometrical ratio is the coefficient between the member thickness and the diameter of the fastener (Equation 4.8).

$$a_{c,i} = \frac{t_i}{d_{c,i}} \tag{4.8}$$

The greater the ratio, more yielding occurs in the fastener. Likewise, the lesser the ratio, more brittle failure on the wood (Heine & Dolan, 2001). Certainly, an optimal connection design requires higher levels of yielding, not only in seismic areas for greater energy dissipation, thus nail fasteners are commonly preferred over screws (Carradine et al., 2006)

Non-linear elastic foundation was derived for finite element models with a greater degree of accuracy by Foschi (1974). This theory has been followed by several authors (Chang et al., 2009). Alternatively, another analytical method to determine fastener yielding behaviour based on the Foschi method was proposed (Jumaat & Murty, 2004). This procedure is known as the 5% diameter offset method.

Other methodology to determine the load-bearing capacity of joints is the virtual work approach (Aune & Patton-Mallory, 1986). Alternatively, simplified numerical methods based on diameter and density of the timber members have been published also by Jumaat & Murty (2004).

Finally, in terms of optimisation, a study conducted by Anderson & Leichti (2007) concluded that increasing the yielding moment of the fasteners beyond a certain point, shear strength and stiffness did not improve the racking behaviour of timber frame shear walls.

Once reviewed all of the shear wall aspects related to the geometry of the panel, the materials involved, the structural performance methodologies and the potential failure modes, this thesis examines analytical and empirical methodologies in order to understand the behaviour of the proposed closed panels. The findings of this study are also the basis for the development of a robust timber frame racking software application.

4.3 Research Methods and Materials

Contained within this section are the research methodology carried out in order to assess the structural performance of the suggested advanced closed panels and the characterisation of the timber frame and sheathing materials used in the fabrication of the tested timber frame walls. The empirical test results of the panels are presented in the section 4.5: racking test results.

The last sub-section describes the approach carried out in this study to understand the structural performance of the sole plate base fixing detail for advanced timber frames.

4.3.1 Racking design methodology

According to the PD 6693-1, only a wall which is fully restrained against overturning presents a racking strength equal to the shear capacity of the sheathing fasteners along the length of the bottom rail. If the wall is only partially restrained, a proportion of the fasteners will be diverted in order to contribute to the restorative overturning moment. Since it is assumed that fasteners can only provide racking resistance or tension resistance, not both in combination (Girhammar & Kallsner, 2009; Lam et al., 1996), the capacity of the wall is directly proportional to the additional hold down resistance provided.

The maximum racking capacity will only be provided where the wall is fully restrained against overturning by means other than the sheathing fasteners along the bottom rail. Vertical point loads and uniformly distributed load shall be considered as restorative overturning moments. A schematic representation of the force distribution is shown in Figure 4-5.

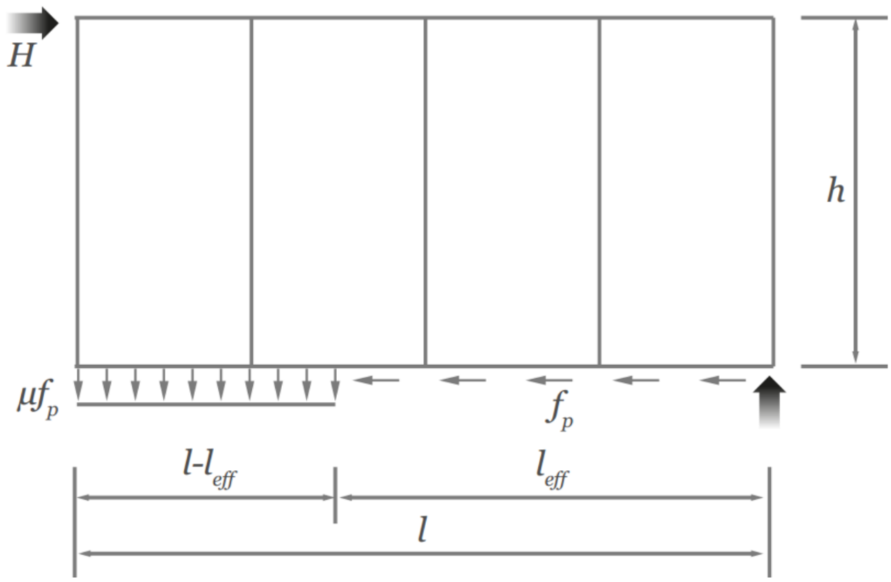


Figure 4-5 Shear wall distribution of forces

In order to satisfy equilibrium, the conditions in 4.9 must be met. The summation of moments at bottom rail must be zero. Furthermore, the equilibrium of forces at top rail and bottom rail must satisfy equations 4.10 and 4.11 respectively.

$$\sum F_x = 0 \sum F_y = 0 \sum M = 0 \tag{4.9}$$

$$r_{t,n} \cdot L = F \tag{4.10}$$

$$r_{t,n} \cdot \alpha \cdot L = F \tag{4.11}$$

In this unified method, the factor α depends directly from the geometry of the wall panel and by mathematical derivation of Equations 4.12, 4.13 and 4.14 the aspect ratio factor can be simplified as per Equation 4.18:

$$F \cdot H = \left[(1-\alpha)L \cdot r_{b,n}\right] \cdot [0.5L(1-\alpha)] \tag{4.12}$$

$$F \cdot H = 0.5 \cdot r_{b,n} \cdot L^2[(1+\alpha)(1-\alpha)] \tag{4.13}$$

$$F \cdot H = 0.5 \cdot r_{b,n} \cdot L^2 \cdot (1-\alpha)^2 \tag{4.14}$$

$$\alpha = 0.5 \cdot \left(\frac{L}{H}\right) \cdot (1-\alpha)^2 \tag{4.15}$$

$$0 = 0.5 \cdot \left(\frac{L}{H}\right) \cdot \alpha^2 + \alpha - 0.5\left(\frac{L}{H}\right) \tag{4.16}$$

$$\alpha = \frac{-1 \pm \sqrt{1 + 4\left(\frac{0.5L}{H}\right)\left(\frac{0.5L}{H}\right)}}{2\left(\frac{0.5H}{L}\right)} \tag{4.17}$$

$$\alpha = \frac{\sqrt{1 + \left(\frac{L}{H}\right)^2} - 1}{\frac{L}{H}} \tag{4.18}$$

The adaptation of the simplified plastic model in PD6693-1 results in a racking strength design for a shear wall as per Equation 4.19

$$F_{i,v,Rd} = K_{i,w} \cdot f_{p,d,t} \cdot K_{opening} \cdot L \tag{4.19}$$

The modification factor $K_{i,w}$ accounts for the aspect-ratio in Equation 4.18, for the vertical applied loads and for the holding-down devices and it is determined by Equation 4.20:

$$F_{i,v,Rd} = \min_{K_{i,w} \leq 1} \sqrt{\left[1 + \left(\frac{H}{\mu L}\right)^2 + \left(\frac{2M_{d,stb,n}}{\mu \cdot f_{p,d,t} \cdot L^2}\right)\right]} - \left(\frac{H}{\mu L}\right) \tag{4.20}$$

Where $M_{d,stb,n}$is the design net stabilizing moment from design permanent loads reduced by the vertical component of any design wind load in kN m. The μ factor is given by Equation 4.21:

$$\mu = \min_{\mu \leq 1} \frac{f_{w,d}}{f_{p,d,t}} \tag{4.21}$$

and where $f_{w,d}$ is the design withdrawal capacity of the sole plate to the substrate connection in kN/m.

The total design shear capacity per unit length of the perimeter sheathing fasteners is determined as per Equation 4.22 where $f_{p,d,2} \leq\cdot f_{p,d,1}$:

$$f_{p,d,total} =\cdot f_{p,d,1} + K_{comb} \cdot f_{p,d,2} \tag{4.22}$$

with K_{comb} a sheathing combination factor with values of 0.50 or 0.75 depending on the side of the second sheathing layer and on the sheathing material and schedule in comparison to the first layer.

In order to account for the mean load carrying capacity of the sheathing fasteners along the edge of the perimeter of the frame and for the reduced non-linear behaviour of the fastener depending on the spacing, the design shear capacity of the sheathing fasteners per unit length, $f_{p,d}$, is calculated from Equation 4.23:

$$f_{p,d} = \frac{F_{f,Rd} \cdot (1.15 + s)}{s} \tag{4.23}$$

Where $F_{f,Rd}$ is the design lateral capacity of the individual fastener and s is the sheathing perimeter spacing in meters.

Finally, to determine the design racking strength and if the wall present a framed opening not considered as discontinuity (4.24), then the opening factor, $K_{opening}$, is determined by Equation 4.25:

$$discontinuity = if \begin{vmatrix} V_{opening} & \leq 0.65H, or \\ H_{sill\ to\ soleplate} & \leq 0.25H \end{vmatrix} \tag{4.24}$$

$$K_{opening} = 1 - \frac{1.9Area}{HL} \tag{4.25}$$

This approach does not consider any serviceability criteria other than a limitation derived from the panel geometry and the load ratio (Equation 4.26)

$$K_{i,w} \cdot f_{p,d,t} \leq 8 \cdot (1 + K_{comb}) \left(\frac{L}{H}\right) \tag{4.26}$$

Once the analytical model is determined for strength and stiffness of shear walls, the next process is to investigate the mechanical properties of the timber frame components and the sole plate base fixing detail which were considered to corroborate the analytical model with the empirical racking tests.

4.3.2 Characterisation of timber frame components

One of the greatest benefits of Eurocode implementation is the inclusion of innovative materials and solutions into the design process. Current engineered wood products to be used in the timber frame are twin stud walls, I-joists, or metal-web studs. This implementation of innovative products also applies to sheathing materials such as fibre plasterboards or particle boards. Nevertheless, from 1st July 2013, all construction products for which a harmonised Product Standard exists, must be CE marked and manufacturers must also provide data compatible with Eurocode 5 (CPR, 2011).

An overall view of all available elements, connection types and components that may be used in the fabrication of advanced closed timber frame systems is given in Figure 4-6. Also, a brief explanation of different timber frame products and sizes utilised in the fabrication of the K2 and RTC closed panels was given in Table 3-9 and Table 3-10.

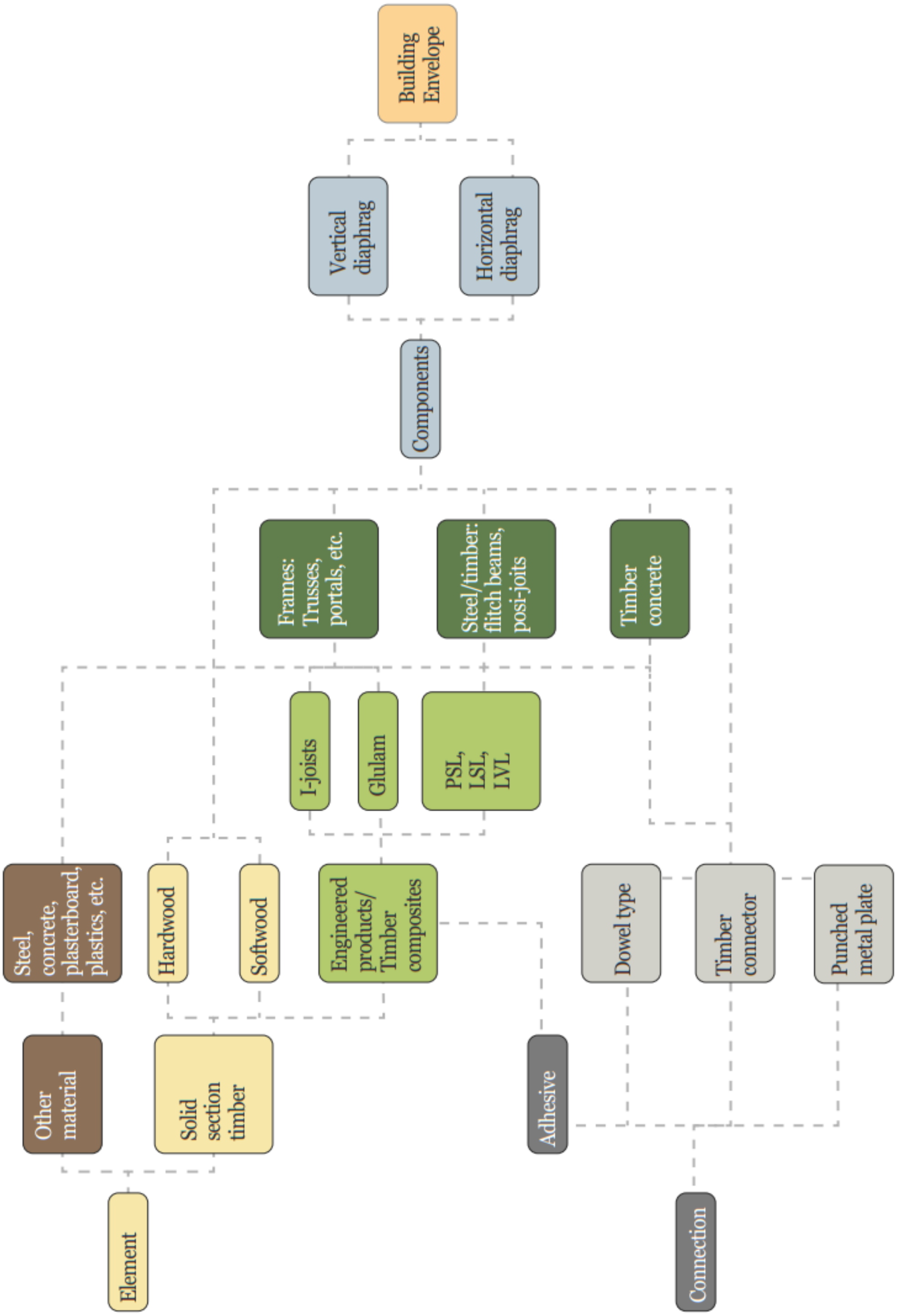


Figure 4-6 Products map for timber frame systems

Sheathing and sheathing fasteners

Different sheathing materials and sheathing fasteners were used in the experiments in order to compare the resultant empirical racking strength with the analytical approach published by PD 6693 (BSI, 2012) and adopted in the software calculation developed in this research.

The average dry densities for all of the sheathing boards used in the experiment were measured and compared with the manufacturer declared values. The characterisation of the sheathing materials is given in Table 4-4.

Table 4-4 Characterisation of sheathing materials

Material	Producer	Thickness (mm)	Measured density (kg/m3)		Declared density (kg/m3)
			ρ_{mean}	n_{sample}	ρ_N
OSB/3	Norbord	9.0	598	6	620
MDF	Panelvent	9.2	767	2	720
Plasterboard	Fermacell	12.5	1175	2	1150

Different commercially available screws and nails were used to fix the sheathing boards to the timber frames. The technical data provided by the manufacturers were considered in the analytical approach and in the software application developed. Different spacing fastener schedules were also investigated with further information detailed in section 4.5.1. The materials characterisation of the sheathing fasteners utilised in the experimental research is presented in Table 4-5.

Table 4-5 Characterisation of sheathing fasteners

Fastener Id	Type	Producer	Diameter (mm)	Length (mm)	Shank
S1	Screw	Timberfix	4.50	60	Threaded
S2	Screw	Rothoblaas	5.00	50	Threaded
N1	Nail	DuoFast	2.90	50	Smooth
N2	Nail	Paslode	2.85	50	Smooth
N3	Nail	DuoFast	3.10	50	Twisted

Timber frame materials and fasteners

The timber materials used in the production of the frames were commercial C16 timber, acoustically graded home-grown Sitka spruce (*Picea sitchensis*) and Finnjoist I-beam from Metsa Wood. This 240 mm composite beam was made by LVL – Kerto® in the flanges and OSB/3 in the web. Due to several availability issues, the I-beam was slightly different to the simulated in the thermal performance, James Jones JJI-joists, which were 245 mm deep and flanges made by C24 timber. However, this has not a significant effect in neither the structural nor the thermal performance.

In terms of measured density, both C16 imported timber and home-grown Sitka spruce were measured after the test. Due to its little used, the density of the other materials provided by the manufacturers was considered in the analytical method. The materials used in the interface between the timber frame and the testing rig and identified as the modified sole plate were also characterised and provided together with the timber frame materials in Table 4-6.

Table 4-6 Characterisation of timber frame and sole plate materials

Material	Producer	Dimensions (mm)	Measured density (kg/m3)		Declared density (kg/m3)
			ρ_{mean}	n_{sample}	$\rho_{N,mean}$
C16	Imported	45x89	412±33	24	420
Home-grown Sitka spruce	Fakland State (Sco)	45x95 ext 45x45 int	424±27	104	370[1]
I-Beam	Metsa	45x240	-	-	510[2]
Kerto[3]	Metsa	45x400	-	-	510
WBP Ply	n/a	18x200	-	-	600
WBP Ply[3]	n/a	3x15x400	-	-	600

[1] Timber acoustically graded as C16 was manually selected to produce the timber frames although its related density was classified as C24.

[2] Flange density of the I-beam. The characteristic density of Kerto, for the analytical methodology, is 480 kg/m3.

[3] Kerto and plywood used as sole plate to fix the RTC wall to the testing rig

Different commercially available screws, bolts and threaded nails were used to assembly the timber frames and to fix the sole plate to the rig. The technical data provided by the manufacturers were considered in the analytical approach and in the software application. The materials characterisation of the timber frame fixings utilised in this experimental research is presented in Table 4-7.

Table 4-7 Characterisation of timber frame and sole plate fasteners

Fastener	Producer	No.	Diameter (mm)	Length (mm)	Shank
Nail N4	Paslode	2	3.10	90	ring
Nail N5	Paslode	1	3.10	90	ring
Screw S2	Rothoblaas	2	5.00	80	threaded
Screw S3	TimberFix	2	3.00	60	threaded
Screw S3	Rothoblaas	18	5.00	40	threaded
Bolt B1	n/a	-	M20	n/a	threaded
Bolt B2	n/a	-	M16	n/a	threaded

Further explanation on the particular connection related to the fasteners for the sole plate details, including lateral load carrying capacity, is given in the section 4.5.1.

4.3.3 Acoustically graded home-grown timber for K2 panels

A total number of 104 studs of dimensions 45 x 95 x 2400 mm were sawn and marked at Living Solutions in Cowdenbeath, Scotland. The timber was locally sourced and conditioned in the same company.

Preceding the grading process by the MTG grader, a visual inspection of the timber was undertaken in order to mainly detect dead knots and other wood imperfections. Furthermore, timber studs were also measured with a calliper and a measuring tape to check cross-sections and lengths were within a ±2 mm tolerance.

The moisture content of the wood was measured with a MD-812 digital meter in order to correct the MOE (Figure 4-7). A problem on the custom-made kiln was identified due to the irregular moisture content of the timber batch. The boards presented a large variation (6 % to 20%) in the moisture content after the drying cycle. The timber studs were also weighted to correct the MOE.

The assumed mean density at 12% from literature on Sitka spruce was 390 kg/m^3 (Moore, 2011). However, the measured mean density of the timber graded as C16 and with a measured MC between 10% and 14% was slightly greater (409 kg/m^3) The outcome of this research is detailed in Table 4-8

Figure 4-7 Equipment used for grading: MTG grader (left), moisture meter (middle) and scale (right)

Table 4-8 Visual inspection and MC of timber studs

No. sample	Studs MC>20%	Studs MC<10%	Studs defects
104	32	10	3

The three studs rejected due to defects were, in all the cases, for the presence of transversal dead knots across the full depth of the timber. However, they were also acoustically graded to check if they would have been rejected by the MTG grader.

After acoustically grading the 104 timber studs and correcting the MOE from the current density and moisture content, the 42 studs which presented a either high or low moisture content were also rejected. From the remaining 59 studs, 35 studs were classified as class C16 (including strength grade C16, C18, C20 and C22), 14 studs were classified as C24 (including strength grade C24 or better) and 10 studs were rejected (13 studs in total if the visually rejected ones are considered).

Furthermore, if moisture content had not been considered, 56 studs, 33 studs and 15 studs would have been classified as C16, C24 and rejected respectively. A summary and a full distribution of the resulted grading investigation undertaken is presented in Table 4-9.

Table 4-9 Strength class distribution from home-grown Sitka spruce

Strength Class	**MOE Boundary** (kN/mm^2)	**Count**	**Percentage of Batch** (%)
Reject	1	5	4.8
C14	7	10	9.6
C16	8	17	16.3
C18	9	7	6.7
C20	9.5	11	10.6
C22	10	21	20.2
C24	11	10	9.6
C27	11.5	4	3.8
C30	12	7	6.7
C35	13	7	6.7
C40	14	3	2.9
C45	15	2	1.9
C50	16	0	0.0
	total	104	100

As the total number of C16 home-grown studs was just insufficient to fabricated, the closed K2 panels and the two single structural layer frame K2 panels (P1, P2, P7, P8, P13, P14, P15 and P16 from Table 4-17), the intermediate studs were C24 as they relevance on the racking performance is less significant than in the perimeter studs (Figure 4-8).

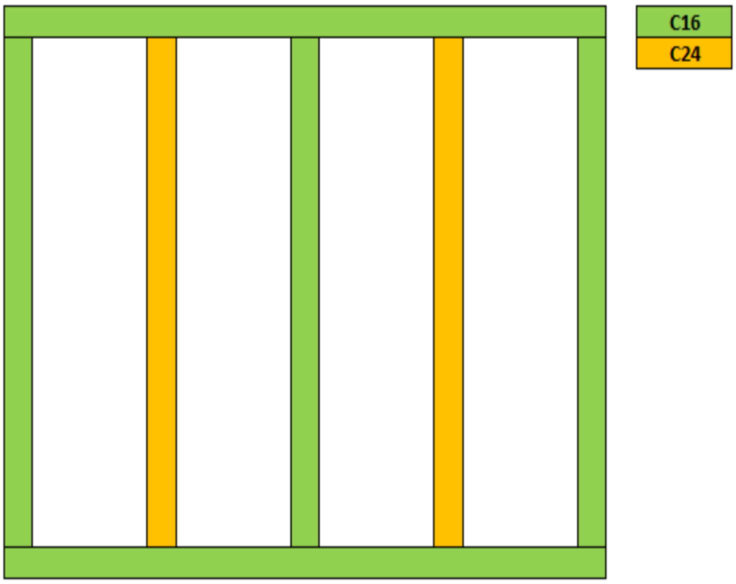


Figure 4-8 C16 / C24 combined assembly of K2 panels

This sub-section, research methods and materials, provides the analytical methodology to determine the racking performance of timber frame walls and investigates the characterisation of the materials used in the fabrication of the closed timber frame walls.

The home-grown timber used in the K2 panel was also acoustically graded in order to understand the product specific influence on the timber frame racking performance.

Before proceeding to investigate the experimental racking behaviour of the proposed closed panels, it is necessary to explain the research carried out in order to optimise the structural performance of the sole plate base fixing details and also under a thermal criterion as identified in the previous chapter.

4.4 Sole Plate Base Detail Optimisation

There is a degree of uncertainty around sole plate base fixings for closed panel timber systems as supported by a lack of literature on this topic. Furthermore, as presented in the previous sections, the sole plate base fixing detail is critical from both, a thermal and structural perspective.

A description of the isolation and combination theory, as a methodology to structurally optimise this detail, and the investigation of the theory on the suggested sole plate details is presented in this sub-section.

The structural performance of the sole plate fixing detail for timber frame panels was initially described in a series of laboratory tests carried out previously at Edinburgh Napier University (Leitch, 2013; Menendez et al., 2013) and it has been developed further on this research.

4.4.1 Isolation-combination theory for sole plate detailing

The methodology of this investigation assessed the structural performance of closed panel sole plate components in isolation as a valid approach to provide structural information of the combined detail.

Information of the closed panel sole plate details studied in the previous research (Leitch, 2013, Menendez et al., 2013) is shown in Figure 4-9 where the different shear plane connections are highlighted individually. Each specific isolated connection named a) b) c) d) and e) is explained in detail in the next paragraphs.

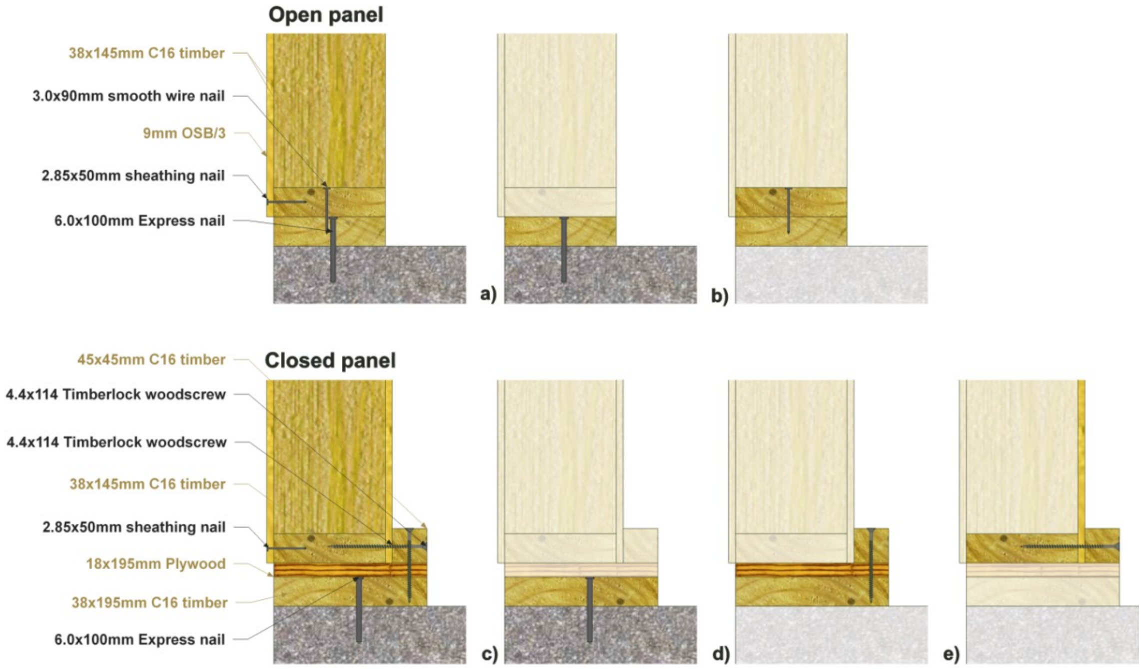


Figure 4-9 Detail of the two sole plate base fixings studied and their isolated connections.

Table 4-10 summarises the relevant European Standards and bespoke methods used in the experiment. For fastener determination of yield moment, observed deformations did not comply with BS EN 409 recommendations (BSI, 2009a) as the loading points from the test apparatus moved along the dowel. As a result, the double plastic hinge deformation model developed by Coste (2010), and based on the previous work of Jorissen & Blass (1998) was adopted instead. Furthermore, it was observed that embedment tests caused also bending of the fastener. BS EN 383 standard invalidates embedment test results if bending of the fastener occurs (BSI, 2007).

Table 4-10 Description of the tests methodology

Test	Method	Equipment	Applied load
Lateral load capacity	BS EN1380	100kN SCHENK	BS EN26891
Tensile strength	BS EN ISO898-1	30kN Lloyd R30k	1mm/min
Yield moment	BS EN409	30kN Lloyd R30k	1mm/min
Embedment	BS EN383	30kN Lloyd R30k	BS EN26891
Pull through	BS EN1383	100kN SCHENK	2mm/min
Withdrawal	BS EN1382	100kN SCHENK	2mm/min

The current standard for the design of racking walls in the UK, PD 6693-1 (BSI, 2012), allows the sole plate to provide resistance against overturning moments. Therefore, fastener withdrawal tests were also undertaken in the previous experiments in order to assess its mechanical properties.

The overall strength performance of the sole plate details was determined by the capacity of the weakest sub-connection as a revised model of the weakest link theory (Madsen & Buchanan, 1986). On the other hand, the overall stiffness of the sole plate base fixing was defined by the accumulative displacement occurred at each shear plane (Leitch, 2013).

In order to determine the level of influence and characterise the structural performance of the sole plate detail for inclusion in the analytical methodology, the closed timber fame sole plate details were investigated in isolation and in combination.

An open panel and a closed panel sole plate connection was suggested as per Figure 4-9. The following connection components from the detail were identified and tested according to the methodology described in Table 4-10:

a) c) Timber to concrete: The study of this connection for both open and closed panel systems include 7.5x100 mm express nail type fastener. Substrate material was dense aggregate block of 7 N/mm^2 compressive strength.

b) Timber to timber: Apart from joining the wall framing members, this connection is found at the base of the open panel and it is critical in terms of transferring the racking forces from the wall to the foundation. Fasteners tested include 3.0x90 mm smooth wire nail.

d) Timber to plywood to timber: This non-standard double shear connection comprised of 45x70 mm timber batten to the sole plate packer through 18 mm plywood by 4.4x115 mm self-tapping screw.

e) Timber to OSB to timber: Again, this non-standard double shear connection horizontally secured the interlock timber to the closed panel by 4.4x115 mm self-tapping screw.

This sole plate testing schedule was carried out at Edinburgh Napier University in collaboration with another doctoral study and already published by Leitch (2013) and Menendez et al. (2013). The complete test results with images of the apparatus and equipment utilised and the load versus displacement diagrams are provided in the Appendix IX. The summary of the results for the isolated connection tests are given in Table 4-11. Due to the ductile nature of the connections, the ultimate strength was based upon the measured resistance at 15 mm of displacement. It must be noted from Figure 4-9 that connection details a) and c) were identical.

Table 4-11 Structural test results summary for sole plates in isolation

Connection	Ultimate strength f_{max} (N)	Slip modulus[1] k_{ser} (N/mm[2])
a) c)	4087	1594
b)	3159	1500[2]
d)	5727	1297
e)	11470	1542

[1] Slip modulus taken as linear stiffness between 0.1 - 0.4F_{max}
[2] Slip modulus in accordance with BS EN 26891

In order to confirm the performance of the sole plate fixing detail in isolation with the overall performance of the complete detail, the full sole plate fixing details for open and closed panel were tested in the same study (Menendez et al., 2013). These tests were performed according to a modified version of the BS EN 1380 test set up (BSI, 2009b) so as to replicate the shear load being transferred from the wall panel to the substrate. Table 4-12 presents the test results in terms of strength and stiffness of the open panel (OP) and closed panel (CP) sole plate base fixing detail respectively. The maximum ultimate strength value, f_{max}, was determined when 15 mm displacement of the bottom rail relative to the substrate was reached. Stiffness, k_{ser}, was based on the displacement of the bottom rail at 40 % of its ultimate strength.

Table 4-12 Sole plate structural results in combination.

Sole plate	Ultimate strength f_{max} (N)	Slip modulus k_{ser} (N/mm[2])
OP	2841	1364
CP	4036	745

As stated before, the overall performance of the sole plate detail is dictated by the weakest connection. The investigations provided a direct comparison between the performance of the detail in combination and as the performance of the isolated components. The experimental results are presented by Table 4-13.

Table 4-13 Isolation and combination sole plates strength

Connection (N)	Isolation					Combination	Ratio
	a)	b)	c)	d)	e)		
OP	4087	3159				2841	0.90
CP			4087	5727	11470	4036	0.99

Therefore, as the combination results are approximately equal to that of the weakest connection within the sole plate detail, the isolation-combination methodology followed in that research study was corroborated. The validation of this approach allowed for the optimisation of the sole plate fixing detail in MMC by designing effective shear planes of similar strengths. Nevertheless, the spacing of the fasteners for each component also influences the racking design as the strength capacity of the sole plate is given in resistance per meter run, kN/m. This approach is followed in the next section in order to determine an optimised sole plate detail solution.

4.4.2 Sole plate structural performance of RTC and K2 frames

One of the research outcomes from chapter 3 was that is more thermally efficient to have fewer but stronger fasteners at the sole plate. Additionally, another outcome stated that the heat flux of fasteners fixed perpendicular to the foundation can be neglected if they do not fully penetrate the sole plate.

These outcomes have been considered in the development of robust sole plate details for the proposed closed timber panels. Nevertheless, it is important to bear in mind that in countries where earthquake action needs to be considered, a rather large number of ductile fasteners shall be preferable as that detail is able to dissipate more energy.

Figure 4-10 and Figure 4-11 illustrate the sole plate base fixing details suggested for the K2 and RTC panels and used throughout this research. However, in conditions when a holding down tie cannot be placed, i.e. narrow solid concrete block walls, an alternative detail with an external metal plate is provided (Figure 4-12).

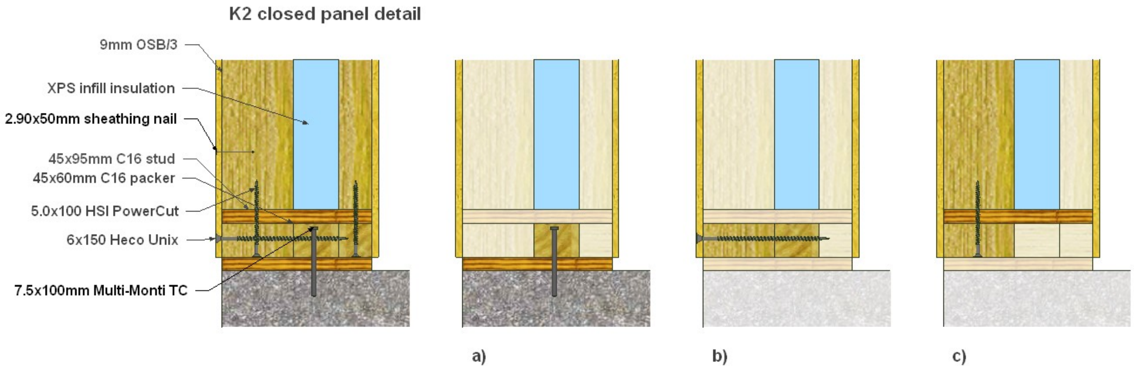


Figure 4-10 K2 closed panel sole plate, isolated tests represented a) to d)

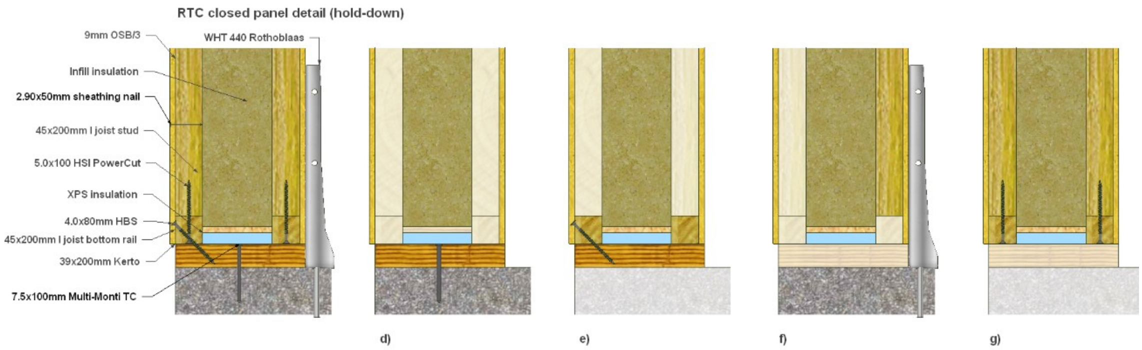


Figure 4-11 RTC closed panel sole plate, isolated tests represented d) to g)

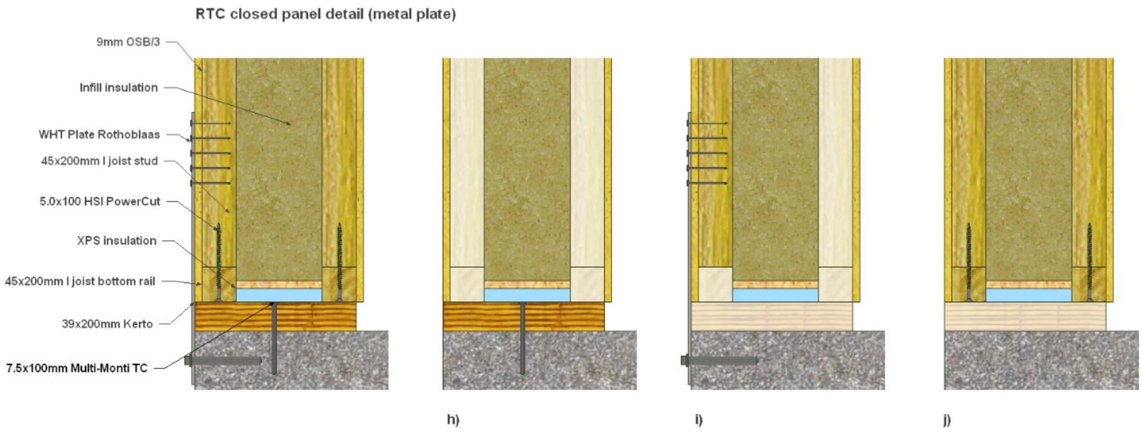


Figure 4-12 RTC alternative sole plate, isolated tests represented h) to j)

These different individual sub-connections named from a) to j) are further detailed in Table 4-14 where these results were obtained according to Eurocode 5.

Furthermore, the structural performance of the sole plate components in isolation and also determined as per PD6933-1 methodology is presented in Table 4-15.

As per weakest link theory and with an optimised fastener spacing, the ultimate limit state and the maximum withdrawal capacity of each connection type for the sole plate base details studied is also presented in Table 4-15.

Table 4-14 Description of closed panel sole plate components.

Plane	Element 1	Element 2	Structural properties: Ultimate strength f_{max} (N)	Structural properties: Slip modulus k_{ser} (N/mm^2)	Fastener (mm): No	Fastener (mm): dia	Fastener (mm): length
a)	45mm C16	Concrete C20/25	1348	1856	1	7.5	100
b)	95mm C16	60 mm C16	995	1175	1	8.0	160
c)	45mm C16	parallel C16	1508[1]	1129	1	5.0	100
d)	39mm Kerto	Concrete C20/25	1685	3004	1	7.5	100
e)	45mm C24	39mm Kerto	568[2]	1004	1	4.0	80
f)	Frame C24	Concrete C20/25	38600	5705	WHT 440 Rothoblaas[3]		
g)	45mm C24	parallel C24	3016	1175	2	5.0	100
h)	39mm Kerto	Concrete C20/25	1685	3004	1	7.5	100
i)	Frame C24	Concrete C20/25	17350	3596	WHT Plate Rothoblaas[4]		
j)	45mm C24	parallel C24	3016	1175	2	5.0	100

[1]Axial withdrawal capacity of leading stud
[2]Lateral carrying capacity considered at ¾h of sole plate connection with minimum 36 mm into Kerto.
[3]Partially fixed with n20 4x60mm nails and M16 to foundations with no washer. Experimental K_{ser} for no 20 4.0x60mm nails in WHT holding down strap.
[4]Partially fixed with n10 4x60mm nails and M16 to foundations with no washer. Neff = 100.7.

Table 4-15 Structural performance of closed panel sole plate in combination.

Connection	Spacing (mm) a)	b)	c)	d)	e)	f)	g)	h)	i)	j)	ULS $F_{max,k}$ (kN/m)	Withdrawal $F_{ax,Rk}$ (kN/m)
K2	800	600	900								1.66	2.47
RTC				900	300	2ud	610				1.87	2.45
RTC (alternative)								900	2 ud	610	1.87	4.23

Once the optimised sole plate base fixing details were identified and defined, a series of twenty racking tests were carried out. The results of this investigation are presented in the next section.

4.5 Racking Test Programme.

The information provided in this section includes the racking test results of the advanced closed timber frame panels described in the previous sections. The information on the racking experiments is expressed in both a quantitative (failure load, deformations) and qualitative manner (failure modes).

Partially and fully restrained timber frame walls of different dimensions were tested for both the RTC and K2 assemblies. The BS EN 594:2011 is the experimental racking strength and stiffness methodology followed in this research. The advanced closed panel testing program and the number of tests carried out for each frame type were determined by the financial resources available. The test schedule is detailed in Figure 4-13.

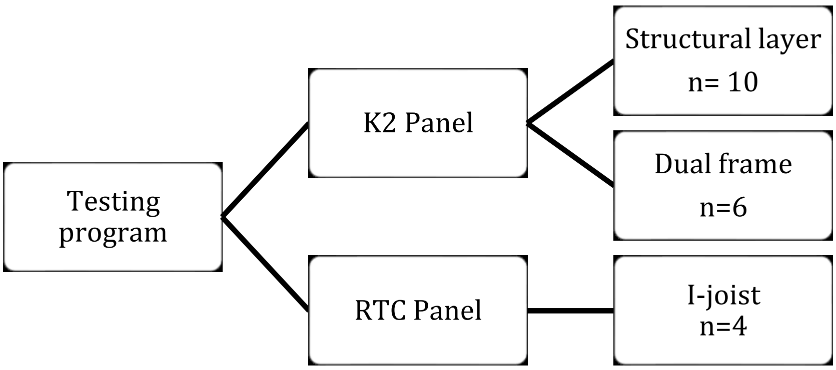


Figure 4-13 Experimental tests and number of panel samples.

The dual frame K2 panel presents an external structural layer and an internal non-structural frame. The connection between both frames is considered to be non-structural due to the properties of the XPS and the bonding agent. The testing program for this panel type comprises of two different configurations: one included only the structural layer whilst a second program tested the complete dual frame in order to investigate the contribution of the non-structural internal frame and the failure mode.

The performance of the benchmark timber fame has been extensively reported in pass testing programs by Edinburgh Napier University (Leitch, 2013). This testing program was the initial baseline for this research and therefore, a summary of the results is provided in the Appendix VIII.

4.5.1 Racking test procedure

In this sub-section, information related to the empirical methodology followed in the research is detailed.

Twenty timber frame wall panels were tested in accordance with BS EN 594:2011 (BSI, 2011). This involved securing each panel in turn to a custom-built test rig and then applying a horizontal racking load as per the specified rate given by clause 6.4 of the afore mentioned standard. The particularity of the racking rig used was its horizontal setting instead of the more common vertical lay out (Figure 4-14).

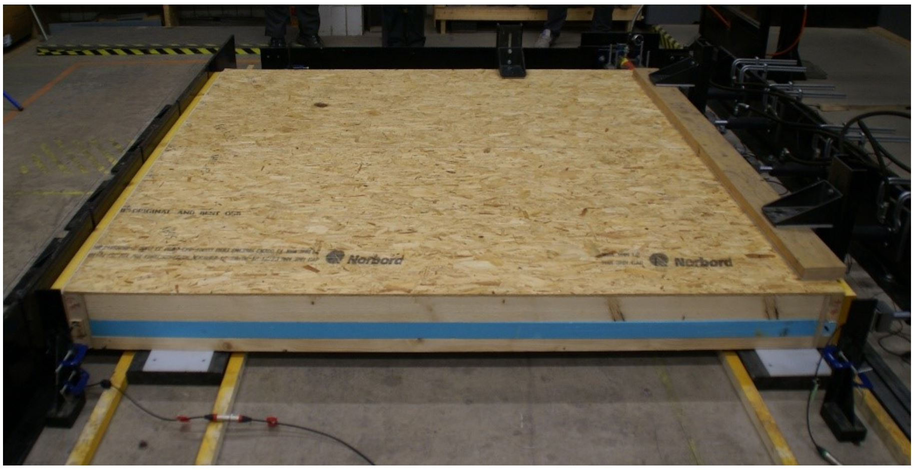


Figure 4-14 Racking rig lay-out

The timber frame panels, due to several logistic and time issues, were tested in three different locations around Scotland. Nevertheless, all the timber frame panels, except one RTC panel, were tested in the same rig to the same method and with the same level of calibration as per United Kingdom Accreditation Service (UKAS) standard.

The racking load was applied until the panel was deemed to have breached the failure criteria, F_{max}, given by clause. 6.4.2 of the related standard except for two partially restrained RTC timber frame panels where large deformations occurred before.

In order to obtain test results from the I-joist timber frame for fully and partially restrained walls, the partially restrained RTC panels were firstly tested until the estimated 0.7 F_{max} then, they were turned over and fully restrained by holding down devices. These panels were finally tested to ultimate failure. It was ensured that all of the panels failed totally as the racking load began to reduce and as the panels did not show any recovery by redistribution of the load to the remaining fixings.

Prior to the test procedure, and for all the panels except for the RTC , panels were subjected to 5 kN vertical loads. These loads, F_v, were applied to the head binder at the stud positions as stated in Figure 4-15, a modified horizontal rig from the test apparatus recommended in BS EN 594:2011.

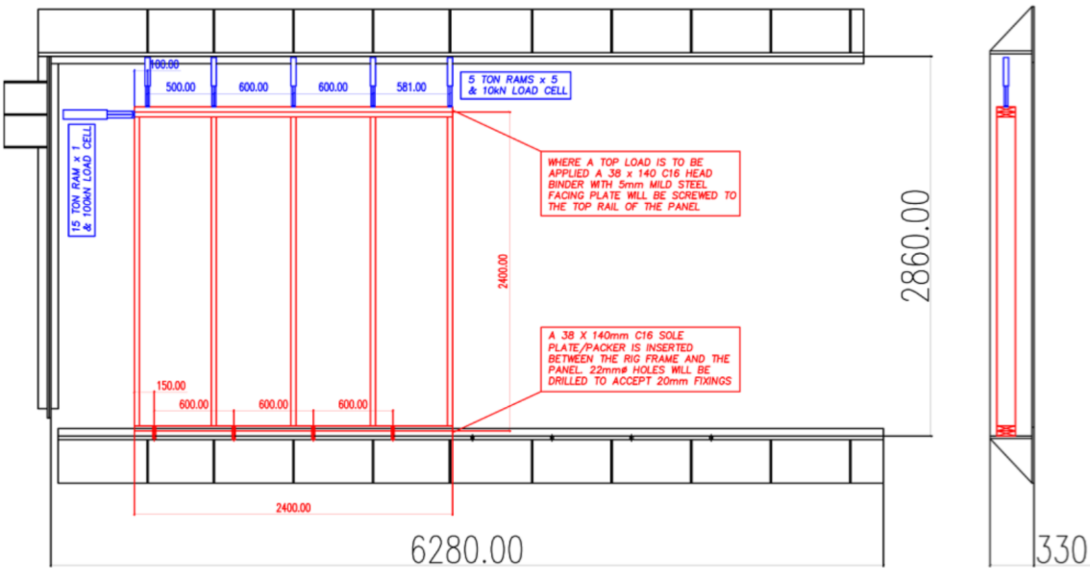


Figure 4-15 Plan and end-elevation of UKAS accredited racking test rig modified from BS EN 594:2011

A stabilising load cycle with vertical loads of 1.0 kN was applied to the head binder at the stud positions and maintained for 120 s. After this period of time, the load was removed and the panel was allowed to recover for 600 ± 120 s before continuing the load procedure. Then, a vertical load F_v of 5.0 kN at each stud position, according to Figure 4-15, was applied to the head binder and maintained for the full duration of the test. The load protocol followed in this research is described in Figure 4-4a and detailed in BS EN 594:2011 (BSI, 2011). Note that the loading protocol in this standard was modified from its previous version BS EN 594:1996.

Data from the load cells and displacement transducers was transferred to a 4 channel control box. Table 4-16 shows the channel allocation. Data was recorded at 1 s intervals and transferred to a MS Excel spreadsheet. The vertical imposed load when applied is connected to a vertical ram which maintains the load constant regardless the panel deformation.

Table 4-16 Allocation of data channels during testing

Channel	Allocation
1	Load cell - Applied racking load, F_R
2	Transducer - Deflection of frame at point 1
3	Transducer - Deflection of frame at point 2
4	Transducer - Deflection of frame at point 3

The apparatus was supplied by a 15 ton capacity hydraulic ram connected to a 100 kN capacity load cell. In order to simulated the imposed vertical loads, up to five 5 ton hydraulic rams were acting over the top runner through a steel backed timber. The pressure to actuate the ram was delivered via a hydraulic compressor governed by a control unit. In the rig, the panels were restrained out of the plane. The frictional effects were reduced by using a 3 mm Polytetrafluoroethylene (Teflon) sheathing.

Once the rig was ready, the full-scale timber frame wall panels were tested as per configurations and materials detailed in next section.

4.5.2 Full-scale racking testing

The twenty-advanced closed panel timber frame walls were manufactured as per specifications detailed in Table 4-17.

The standard wall specification was 2.4 m long panel with 38x140 mm timber frame fixed using 3.25x90 mm threaded nails and sheathed with one board 9 mm OSB/3 fixed to the frame by 2.9x50 mm smooth nails. The nomenclature and the parameters to define each wall reference utilised in this study is presented in Table 4-18 which follows Leitch (2013) methodology for racking testing reports.

[restrain] [length] [nail spacing] [sheathing] [load] [open WxL] [hold-down]

Table 4-17 Timber frame closed panel specifications

Series name	Panel	Reference	Sheathing fastener	Wall type
Structural layer imperforated K2	P1-P2	P[L1.2].75	S1	D
	P3-P4	P[L1.2].75.PanelVent	S1	D
	P5-P6	P[L1.2].75.FermaCell	S1	D
	P7-P8	V5.75.	N1	A
Structural layer perforated K2	P9-P10	P[L3.6].75 [1060x2070W]@670	N1	B
	P11-P12	P[L4.8].75 [1060x2070W]@670	N1	C
Complete assembly imperforated K2	P13-P14	V5.75	N1	A
	P15-P16	V5.75.DS	N1	A
Complete assembly imperforated RTC	P17-P18	P[1.2].150.DS	N3	D
	P19-P20	[1.2].150.DS.HD	N3	D

Table 4-18 Nomenclature of racking walls

Code	Parameters
[length]	2.4 m 1.2 m 3.6 m 4.2 m
[restrain]	P (partially restrained) F (fully restrained)
[nail spacing] S indicates screwed	50 mm 75 mm 100 mm 150 mm
[sheathing]	DS (Double sheathing) SS (Single sheathing)
[load]	V5 (Vertical 5kN/m) V10 (Vertical 10 kN/m
[opening WxL] [Lee or Wind end]	900x1280W 900x1280L 1900x1365W
[hold down]	HD (Holding down strap)

As an example, [OP.P.S75.SS] refers to an open panel [name] configuration partially restrained [P] where the sheathing is fixed by screws at 75 mm centre to centre in the perimeter [S75] and with a single sheathing layer [SS].

The fully restrained RTC imperforated panel presented a holding down strap at leading and trail stud, WHT-440 from the Italian company Rothoblaas, which was connected to the frame by 18 no S3 screws to the frame and by M16 bolts to a 60 mm Kerto® packer (Figure 4-16).

Figure 4-16 Holding down metal strap to plywood (left) and to KERTO (right) prior to fully restrained the panel to the rig.

This holding down strap was originally glued and screwed to 45 mm plywood. However, the stiffness of the detail was unsatisfactory and the test had to be re-arranged with Kerto® instead as shown by Figure 4-16 (right).

The Scottish company CCG manufactured the advanced K2 closed panels from home-grown Sitka spruce and Nordbord Sterling OSB/3 boards. Carbon Dynamic, another Scottish company, fabricated the RTC panel from commercially available I-beams and also from Nordbord Sterling OSB/3 boards. The lay-out of the K2 panels were types A, B, C and D whilst the lay-out of the RTC panels were type D according to Figure 4-17.

Although the timber frame panels were produced by commercial organisations, special indications in terms of quality assurance procedures and checklist were facilitated to maintain a level of academic rigour (Appendix X). Nevertheless, the wall panels were exhaustively inspected prior testing.

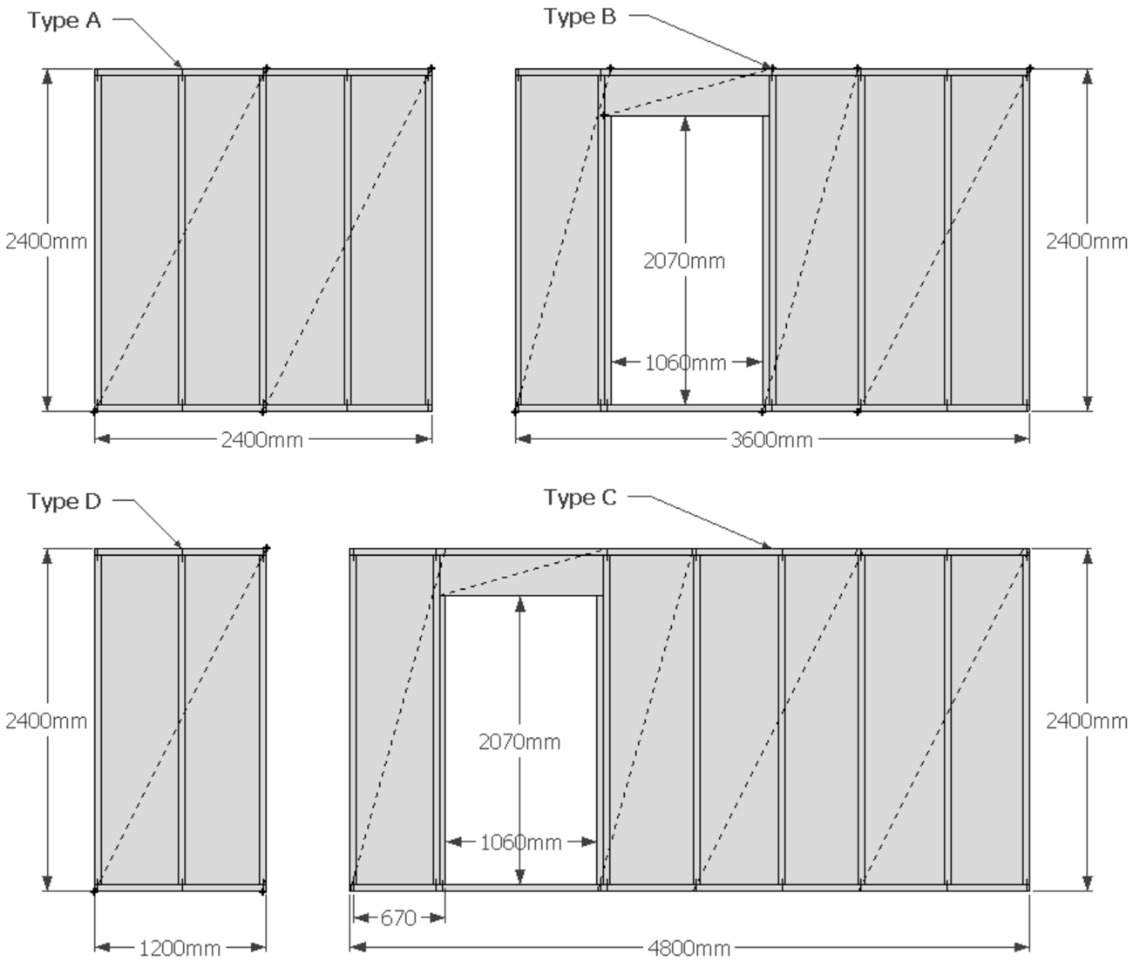


Figure 4-17 Lay out of tested advanced closed panels

The magnitude of the applied racking load and the resultant racking displacement in the top rail were recorded during the duration of the test.

From this resultant load-displacement curve, wall strength at ultimate load and wall stiffness were determined. Furthermore, the horizontal displacement of the bottom rail and the vertical displacement of the leading stud near the bottom rail were measured (Figure 4-18).

Figure 4-18 Transducers to measure displacement and horizontal racking load cell

A modified wall head displacement to neglect timber frame slip at sole plate was determined according to Equation 4.24:

$$\delta_{1,mod} = \delta_{point,1} - \delta_{point,2} \tag{4.24}$$

The racking test results for the advanced K2 and RTC advanced timber frame walls are discussed in the next sub-section.

4.5.3 Experimental racking results

The result of the twenty individual racking tests encompassing ten 2.4 m high timber frame wall types of different specifications is reported in this section. The racking test program was intended to investigate the effects of the following variables on the in-plane strength and stiffness of advanced closed panels of several lengths:

- Sheathing materials.
- Sheathing fastener schedule.
- Applied vertical loading.
- Restrained sole plate conditions.
- Openings.

Furthermore, special attention was considered to the failure mode mechanisms and its potential impact on a serviceability criterion based on airtightness. This exercise was carried out for mere qualitative purposes which must be interpreted with caution.

The structural performance criteria for each tested wall was defined by a series of strength and stiffness parameters derived by the single load-displacement curve and in agreement with the standard BS EN 594:2011. This criteria is shown in Table 4-19 and it is and adaptation of the approach from previous research on racking walls carried out at Edinburgh Napier University (Leitch, 2013).

Note that referred loads are considered by metre run of wall panel (kN/m) in order to facilitate direct comparison between similar racking wall types.

Table 4-19 Criteria used in the reporting of the racking test results

Criteria	Description	Designation	Unit
Racking performance curve	Load$_{(y\text{-}axis)}$ vs modified displacement$_{(x\text{-}axis)}$ at wall head	-	-
Ultimate strength	Maximum racking strength	Fmax	kN/m
Displacement at F_{max}	Displacement measured at the wall head and corresponding with F_{max}	δFmax	mm
Load at SLS	Load at 40% of F_{max}	0.4Fmax	kN/m
Displacement at $0.4F_{max}$	Displacement measured at the wall head and corresponding with $0.4F_{max}$	δ0.4Fmax	mm
Displacement SLS	Maximum displacement considered as 0.003 H or 7.2 mm for 2.4 m height walls	δSLS	mm
Load at Displacement SLS	Load measured at δ_{SLS}	F7.2mm	kN/m
Load at Displacement AT	Load measured at 10 mm displacement and referred as thermal serviceability criteria	F10mm	kN/m
Ultimate failure load	Load at 80% of F_{max}	0.8Fmax	kN/m
Racking stiffness	Racking stiffness calculated as Equation 4.6	R	N/mm

Racking results for K2 single structural frame

The six-pairs of single structural layer K2 panels presented a combination of different sheathing materials, different wall lengths and different sheathing fastener schedules. Also, shear walls P7 and P8 were tested under 5 kN vertical point load at each stud axis. A summary of the racking test results is described by Table 4-20.

Table 4-20 Summarised racking results for K2 single structural frame panels

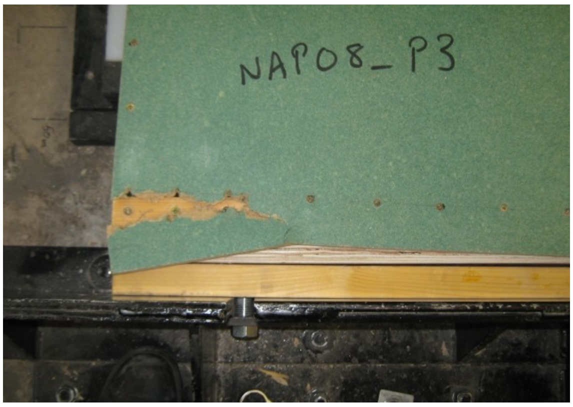
	F_{max} kN	F_{MAX} kN/m	δ_{Fmax} mm	$0.4F_{max}$ kN	$\delta_{0.4Fmax}$ mm	$F_{7.2mm}$ kN	F_{10mm} kN	$0.8F_{max}$ kN	R N/mm
P1	4.55	3.79	35.3	1.8	9.87	1.25	1.83	3.66	182
P2	6.06	5.05	46.79	2.41	9.48	1.98	2.58	4.85	209
P3	5.98	4.98	45.64	2.37	12.23	1.73	2.13	4.79	139
P4	6.77	5.64	44.24	2.7	12.8	1.68	2.19	5.41	180
P5	3.30	2.75	17.75	1.35	6.74	1.48	2.11	2.63	162
P6	3.00	2.50	15.1	1.19	4.15	1.65	2.09	2.39	208
P7	21.33	8.89	41.43	8.53	4.19	n/a	n/a	17.06	1451
P8	23.08	9.62	32.96	9.23	5.25	n/a	n/a	18.46	1514
P9	26.13	7.26	54.8	10.43	11.17	7.32	9.65	20.90	767
P10	23.77	6.60	46.82	9.53	10.23	7.26	9.36	19.04	782
P11	47.71	9.94	36.07	18.89	6.58	20.19	24.51	38.17	2071
P12	41.48	8.64	43.74	16.65	8.8	14.17	17.62	33.18	1385

A relative low strength and stiffness was found on the partially restrained timber frame panels P1-P6 in comparison with the vertically loaded timber frame panels P7 and P8. This may be caused, apart from the restrained effect of the vertical loads at the stud point, by the brittle failure observed on the closely spaced sheathing screw fasteners where no much energy was dissipated by the sheathing connection (Figure 4-19).

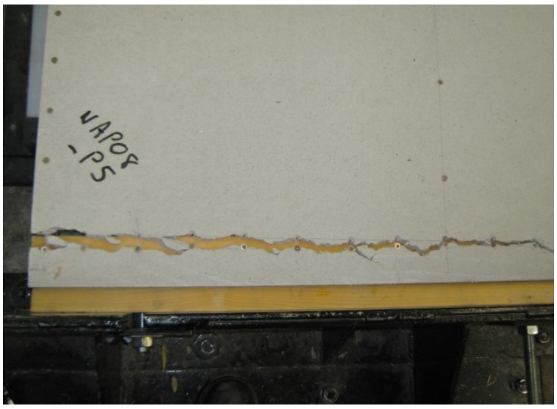
Figure 4-20 illustrates, for direct visual comparison between the same wall panel types, the racking strength for ultimate load in kN (F_{max}), the ultimate load expressed in kN/m (F_{MAX}), the load in kN (F_{10}) for a 10 mm displacement and the racking stiffness derived from the load-modified displacement at Point 1, head binder, in N/mm.

A qualitative outcome observed during the length of each testing protocol, for the screw fastener schedules P1 to P6, indicated an early crack initialisation. This effect may not be adequate if a serviceability criterion, based on low-energy resilient airtight buildings, is required.

Failure observed in P1-P2



Failure observed in P3-P4



Failure observed in P5-P6

Failure observed in P7-P8

Figure 4-19 Sheathing failure modes obeserved in K2 single panel

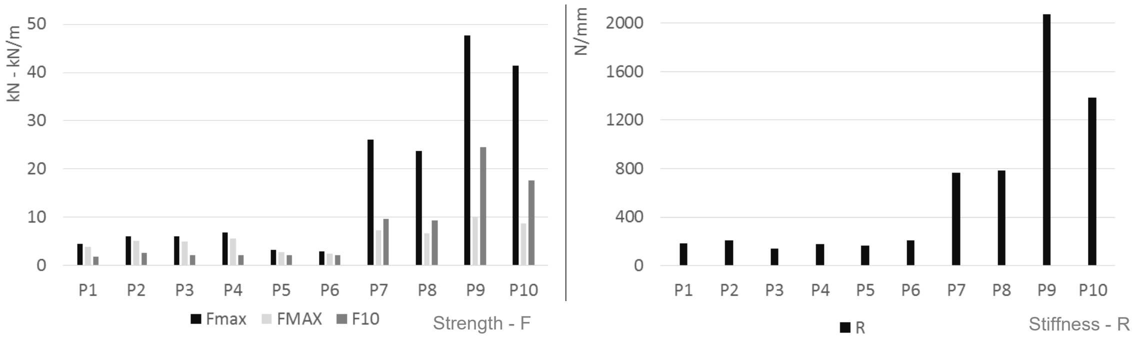


Figure 4-20 Comparative strength and stiffness performance of K2 single framed panels

Racking results for K2 double frame

The two pairs of double structural layer K2 panels, P13-P16, presented identical sheathing fastener schedules and an identical timber frame specification. The four walls were tested under 5.0 kN vertical point load at each stud axis and the sole plate was partially restrained against overturning moments. The only difference was panels P13 and P14 had a single sheathing layer on the structural frame whilst panels P15 and P16 had a sheathing layer on each frame with identical sheathing fastener schedule. The aim of this testing schedule was to understand the contribution to racking of the non-structural internal frame layer. A summary of the racking results is presented by Table 4-21.

Table 4-21 Summarised racking results for K2 double frame panels

	F_{max} kN	F_{MAX} kN/m	δ_{Fmax} mm	$0.4F_{max}$ kN	$\delta_{0.4Fmax}$ mm	$0.8F_{max}$ kN	R N/mm
P13	17.01	7.09	33.43	6.8	3.31	13.61	1484
P14	16.93	7.05	29.35	6.77	2.84	13.54	1937
P15	16.56	6.90	25.14	6.63	4.1	13.25	1180
P16	20.25	8.44	30.06	8.1	6.89	16.20	874

The racking strength for ultimate load in kN and also expressed in kN/m, for direct comparison between the same wall panel types, and the racking stiffness in N/mm were derived from the load-modified displacement at Point 1 head binder. The strength and stiffness of the single structural layer P7 and P8 were also included for comparison purposes (Figure 4-21).

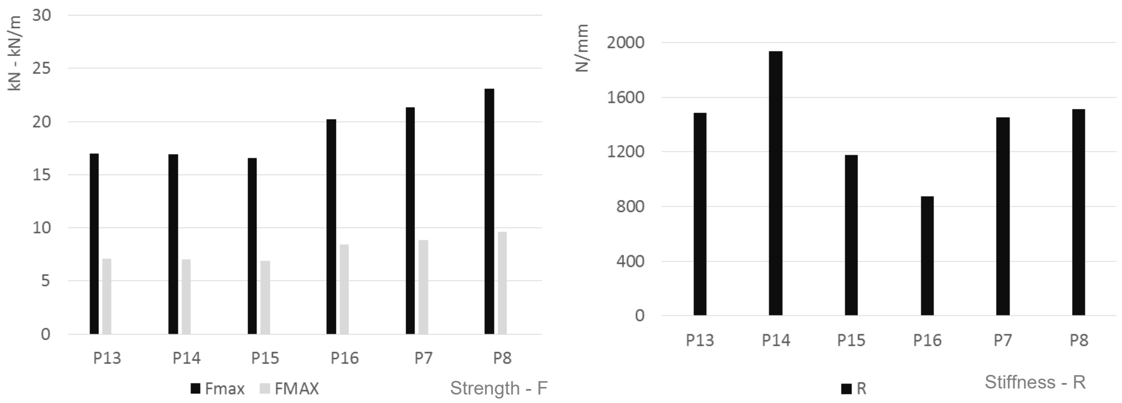
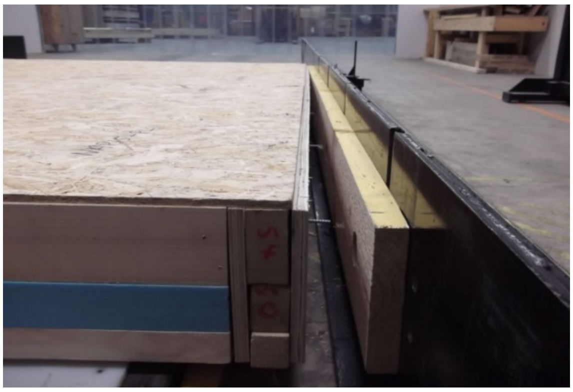


Figure 4-21 Comparative strength and stiffness performance of K2 double framed panels

A relative high strength and stiffness can be considered for the double frame panels with failure loads greater than 7.0 kN/m run and stiffness greater than 800 N/mm in all the cases. However, although there is not sufficient data to provide a rigour conclusion, there is enough evidence from the test results to state that the contribution of the secondary structural layer has an equal or positive impact on the global strength of the panel but this secondary layer has no contribution to the stiffness of the dual frame.

The characteristic failure mode observed for the dual frame K2 panels corresponded to the sole plate connection (Figure 4-22). A withdrawal of the fasteners connecting the strip of 19 mm plywood to the 38 x 200 x 2400 mm sole plate timber packer was observed in all of the specimens. This withdrawal was recorded for as much as three-quarters of the length of the panel. The loading configuration for these tests correspond to 5 kN load cell at each timber frame stud.

Sheathing failure observed in P13-P14



Sole plate failure observed in P13-P14

Sheathing failure observed in P15-P1

Sole plate failure observed in P15-P16

Figure 4-22 Sheathing and sole plate failures obeserved in K2 double frame

Racking results of RTC I-beam structural frame

The two pairs of RTC I-beam timber frame panels presented identical sheathing fastener schedules and an identical timber frame specification. The four walls were tested with no vertical loading acting on the studs but walls P19 and P20 had a holding down strap at leading and trail studs.

Hence, the investigation was focused on the comparison of a closed timber frame panel fully restrained against overturning moments against partially restrained wall.

The testing protocol was slightly different from the previous K2 panels as only two 1.2 x 2.4 RTC walls were provided to undertake four racking tests. Firstly, panels P17 and P18 were fully restrained by connecting the sole plate packer to the frame with 12no 6.0 x 100 mm SDS screws and the sole plate packer to the testing rig by 2 no M20 bolts.

These panels were carefully tested to 70 % of a predicted ultimate load of 6.2 kN. However, the test was terminated at 4.0 kN racking load as the leading stud was starting to lift off from the bottom rail and the bottom rail beginning to lift from the sole plate.

A summary of the racking test results is presented by Table 4-22 where the structural performance information regarding panels P17 and P18 is provided considering the tests aborted at 70 %, and a possibly more realistic 85 %, of the ultimate failure load.

Table 4-22 Summarised racking results for RTC I-beam frame panels

	F_{max} kN	F_{MAX} kN/m	δ_{Fmax} mm	$0.4F_{max}$ kN	$\delta_{0.4Fmax}$ mm	$F_{7.2mm}$ kN	F_{10mm} kN	$0.8F_{max}$ kN	R N/mm
P17[1]	5.87	4.89	n/a	2.35	6.02	2.65	3.25	4.70	298
P17[2]	4.84	4.03	n/a	1.93	4.67	2.65	3.25	3.87	325
P18[1]	5.94	4.95	n/a	2.38	8.55	2.15	2.62	4.75	205
P18[2]	4.89	4.08	n/a	1.96	6.29	2.15	2.62	3.92	224
P19	6.06	5.05	62	2.64	18.2	1.32	1.70	4.85	126

[1]Information reported for test ended at 0.70Fmax
[2]Information reported for test ended at 0.85Fmax

The characteristic failure mode observed for the RTC panels corresponded to the sole plate connection for the fully restrained P17 and P18 panels and to the holding down straps for the partially restrained P19. RTC panel P20 was decided not to test due to instability issues of the holding down connection to the rig (Figure 4-23).

Failure observed in P17

Failure observed in P18

Failure observed in P19

Failure observed in P19

Figure 4-23 Sheathing and sole plate failure modes obeserved in RTC panels

Figure 4-24 illustrates, for direct visual comparison between the same wall panel types, the racking strength for ultimate load in kN (F_{max}), the ultimate load expressed in kN/m (F_{max}), the load in kN (F_{10}) for a 10 mm displacement and the racking stiffness derived from the load-modified displacement curve at Point 1, in the head binder, in N/mm.

The results of the RTC and K2 advanced timber frame panels were provided within this section. These empirical results are compared with the PD 6693-1 results in the sub-section 4.6.

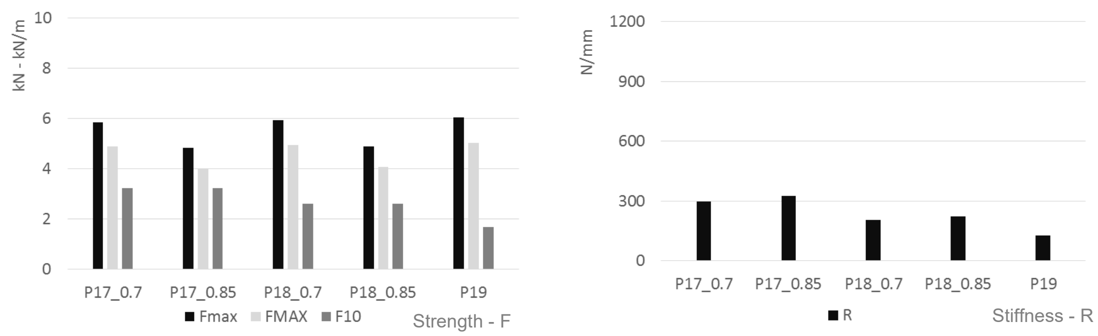


Figure 4-24 Comparative strength and stiffness performance of RTC panels

4.6 Comparison of Experimental Results with PD 6693-1

In this section, a direct comparison between the experimental racking tests results obtained in the previous section and the PD 6693-1 design is provided. The results of this research exercise delivered further information to the accuracy of the analytical approach considered for inclusion in a timber frame racking design software application detailed in next chapter.

In order to compare a wider sample, the previous experiments carried out at Edinburgh Napier University by Leitch (2013) as initial groundwork for this thesis are included. The data of these tests are presented in Appendix VIII.

The experimental versus analytical EC5 comparison of 19 open panel timber frame racking walls is provided in Table 4-23. The experimental versus PD 6693-1 comparison of 20 closed panel timber frame racking walls is provided in Table 4-24. The nomenclature of these racking timber frame walls was described previously (Table 4-18).

It can be concluded from Figure 4-25 that for all the open panel racking tests performed in the study, the characteristic racking strength provided by PD 6693-1 is conservatively lower than the resultant 0.8 F_{max}. However, the SLS criteria check failed in four samples due to a combination of wall panel dimensions (short wall and walls with door opening) and spacing of the sheathing fasteners (50 mm and 75 mm spacing).

This failure of the SLS check under these considerations agrees with the outcome of the results obtained in the study by Leitch (2013) particularly in situations where a dense nailing spacing was adopted.

Table 4-23 Experimental (Leitch, 2013) vs PD 6693-1 racking comparison for open panel timber frame walls (values in kN).

Wall reference	F_{max}	0.8 F_{max}	0.4 F_{max}	PD6693 (k)	SLS check
1.2.F.75.	17.02	13.62	6.81	9.17	Fails
F.75.	36.14	28.91	14.46	18.33	OK
F.50.	49.37	39.50	19.75	26.95	Fails
F.50.DS.	82.48	65.98	32.99	47.17	Fails
V10.50	34.86	27.89	13.94	18.87	OK
V10.150	13.89	11.11	5.56	9.73	OK
V10.50.DS.	41.94	33.55	16.78	27.54	OK
V10.150.DS.	35.6	28.48	14.24	14.42	OK
V10.150..3	10.23	8.18	4.09	9.73	OK
V5.75.	21.17	16.94	8.47	11.54	OK
V10.150.3	15.53	12.42	6.21	9.73	OK
V5.150.900x128W	15.61	12.49	6.24	4.82	OK
V5.150.900x128L	11.43	9.14	4.57	4.82	OK
3.6.V5.150.1380x1250L	20.16	16.13	8.06	8.79	OK
3.6.V5.150.1900x1365L	14.54	11.63	5.82	6.08	OK
3.6.V5.50.1900x1365L	23.83	19.06	9.53	12.30	OK
3.6.V5.150.2x1380x1250WL	28.47	22.78	11.40	9.64	OK
V10.50.1200x1200C	18.53	14.82	7.41	9.91	OK
V10.50.1200x2100C	14.58	11.66	5.83	3.18	Fails

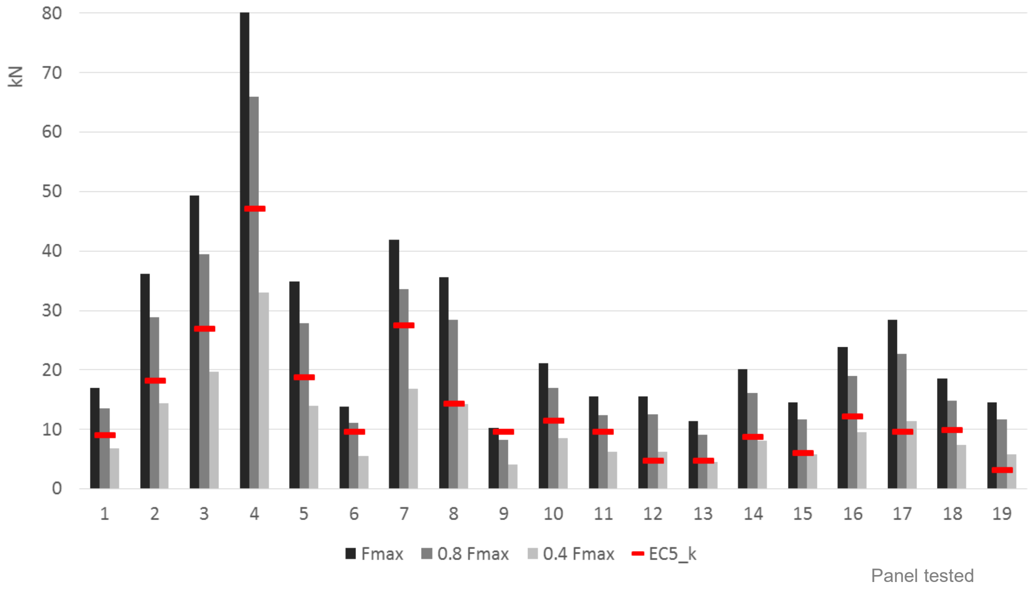


Figure 4-25 Open panel racking comparison: Test vs PD 6693-1

Conversely, although the resultant 0.8 F_{max} was greater than the analytical characteristic value provided my method PD 6693-1, a larger deformation of the timber frame wall panel was observed which could lead to other thermal considerations as detailed in Chapter 3.

This thermal consequence in double sheathed and partially restrained timber frame panels could significantly be undesirable if the splitting of the bottom rail due to transverse bending forces occur at low racking loads. It is important to bear in mind that this conclusion for serviceability must be interpreted with caution as it was observed at ULS.

For the case of the closed panel racking tests (Table 4-24), the analytical characteristic racking resistance was also in great agreement with the empirical 0.8 F_{max} racking resistance. However, the analytical PD 6693-1 characteristic racking resistance in three samples was greater than 0.8 F_{max} but the analytical results did not pass the SLS check. Figure 4-26 illustrates a comparison between the analytical racking design and the test results for the closed panels.

It can be observed that for the K2 closed timber frame panels (P13, P14, P15 and P16) that the second sheathing layer of OSB/3 is not providing any further significant racking resistance to the timber frame. It can be assumed therefore that little or even no shear is transferred by the XPS stud webs from one sheathing layer to the opposite sheathing layer.

The racking strength for the fully restrained RTC closed timber frame panels (P19 and P20) observed was about 40 % greater than the obtained by the partially restrained RTC panels (P17 and P18).

Table 4-24 Experimental vs PD 6693-1 racking comparison for closed panel timber frame walls (values in kN)

Wall reference	Fmax	0.8 Fmax	0.4 Fmax	PD6693 (k)	SLS check
P1 - P[L1.2].S75.SS	4.55	3.66	1.8	3.34	OK
P2 - P[L1.2].S75.SS	6.06	4.85	2.41		OK
P3 – P[L1.2].S75.PV.SS	5.98	4.79	2.37	3.36	OK
P4 – P[L1.2].S75.PV.SS	6.77	5.41	2.7		OK
P5 – P[L1.2].S75.FC.SS	3.30	2.63	1.35	1.521	OK
P6 – P[L1.2].S75.FC.SS	3.00	2.39	1.19		OK
P7 – V5.S75.SS	21.33	17.06	8.53	19.33	Failed
P8 – V5.S75.SS	23.08	18.46	9.23		Failed
P9 –P[L3.6].75.1060x2070W	26.13	20.90	10.43	15.38	OK
P10 –P[L3.6].75.1060x2070W	23.77	19.04	9.53		OK
P11 –P[L4.8].75.1060x2070W	47.71	38.17	18.89	32.31	OK
P12 –P[L4.8].75.1060x2070W	41.48	33.18	16.65		OK
P13 – V5.75.SS	17.01	13.61	6.8	12.962	OK
P14 – V5.75.SS	16.93	13.54	6.77		OK
P15 – V5.75.DS	16.56	13.25	6.63		OK
P16 – V5.75.DS	20.25	16.20	8.10		OK
P17 – P[1.2].150.DS	4.11	3.29	1.64	3.52	OK
P18 – P[1.2].150.DS	4.16	3.33	1.66		OK
P19 – [1.2].150.DS.HD	6.06	4.85	2.42	7.57	Failed
P20 – [1.2].150.DS.HD			n/a		

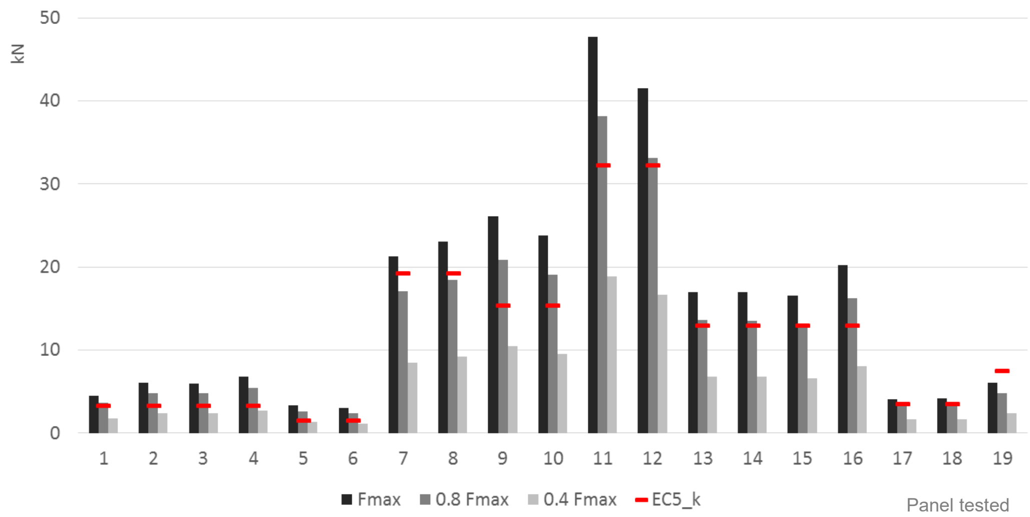


Figure 4-26 Closed panel racking comparison: Test vs PD 6693-1

However, the last two RTC closed panels were tested with holding down straps secured to a 75 x 360 mm LVL plate. The holding down straps fixed to the frame outperformed the sole plate connection to the rig as shown in Figure 4-23. This type of failure may also be found in real practice where the timber frame wall is secured to the substrate or foundations by holding down straps only (Figure 4-27).

Figure 4-27 Timber frame closed panel secured to substrate by holding down straps only at every other stud.

The investigations related to the experimental racking behaviour of the walls carried out in this section concluded that PD 6693-1 is a conservative approach to determine racking strength. However, it was noted that the analytical methodology was more conservative for open panels than for closed panels where the characteristic racking strength values was closer to the 80 % of the empirical F_{max}.

These results provided further support for the theory that partially restrained walls, although are able to resistance racking loads, presented a poor stiffness even for very low applied lateral loads. The next section provides specific recommendations for serviceability criteria with a focus on low energy building design.

4.7 Recommended Shear Wall Serviceability Criteria

Serviceability criteria is often related to the point where a part of the structure or the building needs to be repaired or replaced due to aesthetics or a loss of service. There is a direct cost implication for this limit state criteria but it does not consider a loss in thermal performance (Cook, 1984).

Heat loss due to air leakage is becoming more relevant on current and future building regulations in terms of building energy efficiency. This can be critical on very efficient and Passivhaus buildings without conventional heating installation where heat loss due to the apparition over time of cracks or gaps within the thermal envelope, the boundary timber frame, may impinge upon the adequate supply of warm air into the building through mechanical ventilation systems.

During the tests undertaken in this research, the initiation of cracks on sheathing materials was observed to start as low as 10 % of ultimate failure load and the fissure developed very quickly. This was already observed in other studies (Dobrila, 2003). Furthermore, this author also concluded that reinforcing wall panels by adding a second sheathing layer practically did not improve final deflections. This phenomenon is noticeable when the connection between sheathing and frame is particularly brittle.

On the other hand, imperfections such as gaps in the studs and uplift are relatively frequent on the manufacturing and construction of on-site timber frame wall panels. As a result, the stiffness of the wall panel decreases particularly for panels of short length.

Based on the tear capacity of commercially available tapes and construction membranes, a maximum instantaneous racking displacement for building airtight serviceability criteria of 10 mm is proposed.

4.8 Summary

In this Chapter, the analytical and empirical structural performance of two advanced closed timber frame systems has been reported. The investigation included full-scale testing according to European standards and an assessment on different sole plate base fixing details. The results were compared to the analytical methodology implemented in the latest revision of the UK National Annex to Eurocode 5.

The empirical results obtained for partially restrained and for fully restrained walls agreed with the analytical theory implemented as published document PD 6693-1. The shear wall distribution of forces at the bottom runner presented two components: a vertical restorative overturning moment contribution and a lateral shear resistance component.

The strength grade for the timber of eight K2 wall panels was characterised by non-destructive acoustic methods. No significant shear strength differences were found in the racking test results between the acoustically graded and non-graded timber frame panels. This was unexpected as the measured mean density of the timber for the K2 type A and D tested panels was 409 kg/m^3 which corresponds to strength class C22. The shear strength of the wall was determined, for all types, assuming strength class C16. Therefore, a lower density was used on the analytical EC5 connection design strength between the sheathing and the stud frame. Further work needs to focus on the relationship between strength class and mean density, particularly on lateral load shear connections, for Scottish Sitka spruce.

A series of sole plate base fixing details for closed timber frame wall panels were provided together with structural performance information. This investigation set the basis for the isolation-combination methodology, based on the weakest link theory, to determine the overall structural performance of a complex sole plate detail based on the performance of its individual connections. The isolation-combination methodology also provides an optimised design tool when determining optimal spacing distances as the individual connections should present a similar shear capacity per meter run.

As expected from the literature, no substantial racking transfer resistance was provided between one sheathing layer of the dual K2 panel and the other sheathing layer. However, it was observed a slight reduction on the global stiffness of the dual frame panel when a second sheathing layer was fixed. Low strength and stiffness performance was found on all the partially restrained timber frame panels. The use of holding down straps or returning walls and vertical shear transfer fixings increase the strength and stiffness of closed timber frame walls. This was even more evident for the RTC I-joist wall panels.

The failure mode mechanisms for the tested closed panel systems were also visually reported to understand possible potential consequences on the building thermal performance, especially in terms of air leakage formation. Based on the tear capacity of commercially available tapes and construction membranes, a maximum instantaneous racking displacement for building airtight serviceability criteria of 10 mm was proposed.

PD 6693-1 has proved to be a safe design methodology for the closed timber frame panels investigated in the study. However, the analytical PD 6693-1 is a time-consuming design code which allows for the inclusion of multiple parameters. In order to perform a parametric multi-variate analysis and facilitate design optimisation, the need of a racking software application is required. Furthermore, this software tool could be a means for disseminating research findings to the general public.

5 Timber Racking Design Software Application

The conclusions from research are generally disseminated in the form of publications (scientific books, journal papers or articles), seminars or conference proceedings. Therefore, although this work is of great value, there is an elongated timeline to utilisation, due to research dissemination by these traditional methods. In addition to this, the application of innovation in the construction industry is difficult, often due to a lack of available information, confidence in the product or construction detail and technical compatibility issues. The timber sector is not an exception and any innovative solution frequently requires a long transitional process through all of the industry levels.

In this chapter, the development of a timber frame racking software application for commercial purposes and based on the outcomes from the previous chapter is presented. The objective was to create a design tool for calculation and specification of timber frame walls. The organisation of this chapter is illustrated by Figure 5-1.

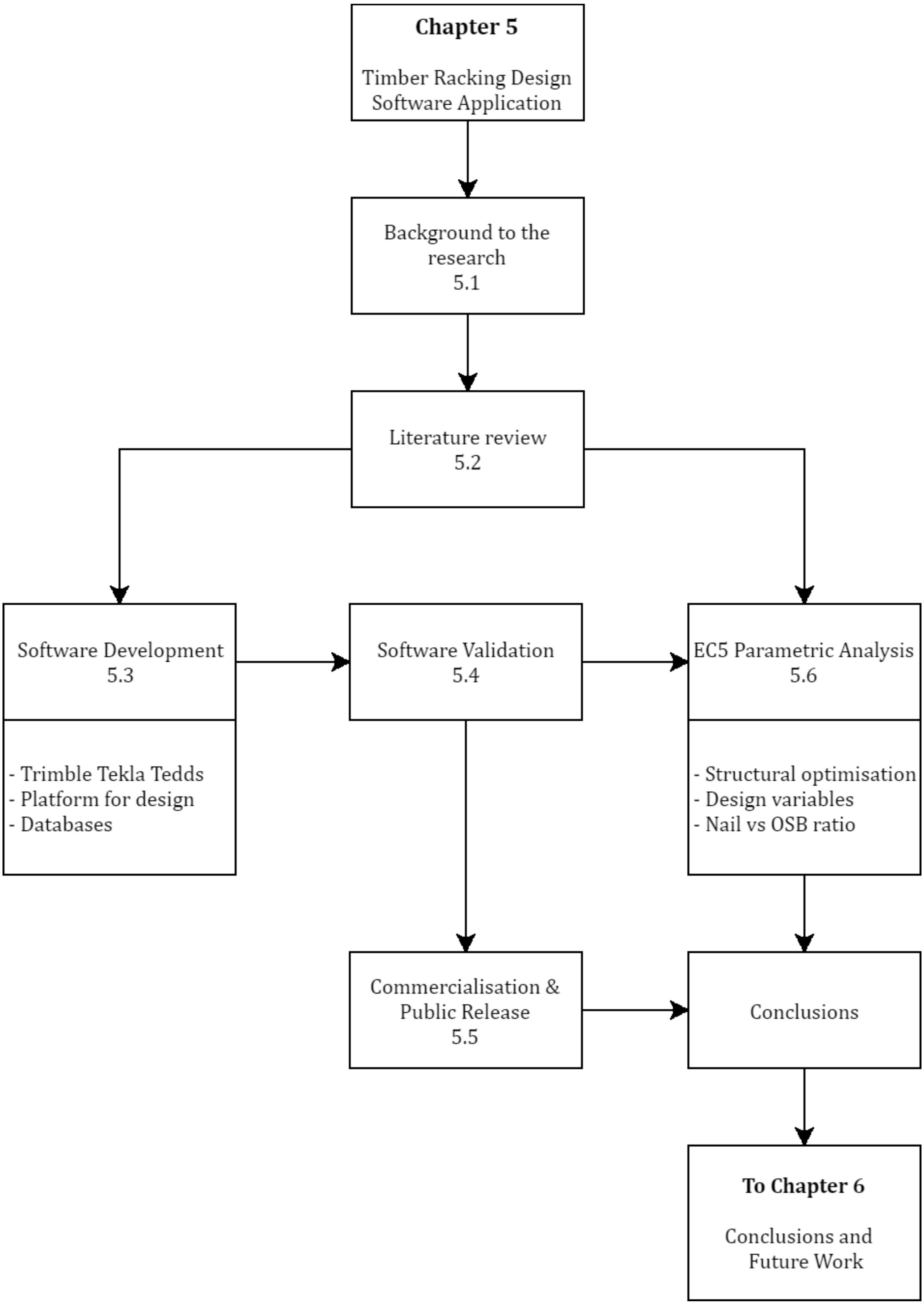


Figure 5-1 Organisation of the Chapter 5.

5.1 Background to the Research

The basis of this research began as a Knowledge Transfer Partnership (KTP) project between The Roof Truss Company Ltd (Scotland) and The Centre for Offsite Construction and Innovative Structures (COCIS) at Edinburgh Napier University. One of the main project aims was to develop a knowledge transfer mechanism from analytical and empirical research to commercial applications of use by general UK structural engineers.

The application Tedds® for Word (Tedds), from CSC (UK) Ltd and now Trimble Tekla, was selected as a timber design platform, due to its accessibility to engineers; availability to source programming code and the possibility of integrating research in the design process by means of databases.

5.2 Knowledge Transfer into Practise

Although there is a general public and private agreement about the need to transfer knowledge from academia to the industry, there is still a lack of high quality knowledge transfer mechanisms into action in a fast and productive manner (Ward, 2009).

The British government through programmes such as Knowledge Transfer Partnerships (KTP) is helping organisations to improve competitiveness and productivity through a better use of knowledge, skills and technology from research organisations. Similarly, governmental research council agencies such as the Engineering and Physical Science Research Council (EPSRC) are also funding projects proposed by higher education institutes in relationship with industrial partners.

5.2.1 Timber research knowledge transfer

Research related to structural timber engineering commonly results in new design methods, new structural timber based materials or innovative methods of timber construction (Hu, 2004). The findings of the research are generally disseminated in the form of publications: scientific books, journal papers or articles; seminars or conference proceedings. Therefore, although this work is of great value, there is an elongated time-line to utilisation due to traditional dissemination being employed.

In addition to this, the application of innovation in the construction industry is difficult often due to a lack of available information, confidence in the product or detail and technical compatibility issues (acoustic and thermal performance or production and construction processes). The timber sector is not an exception and any innovative solution frequently requires a long transitional process through all the industry levels.

5.2.2 AEC timber related software and prospects

There is a large catalogue of Architectural Engineering and Construction (AEC) software products concentrated mainly on steel and concrete building design in contrast with the limited range of products capable of providing an element of structural timber design (Table 5-1).

Table 5-1 List of design software for timber buildings

Name	Design code	Language	Full timber design	2D/3D CAM	BIM
IES VisualAnalysis	NDS	E	No	2D/3D	Via Revit
Bautext (Wood module)	DIN, EC5	E, G	No	No	No
Dlubal (TimberPro.X)	DIN, EC5	E, G	No	2D/3D	No
Dietrich's (D-Wall)	DIN, EC5	Multiple	Yes	2D/3D CAM	Export IFC
Weto (Viskon V5)	DIN	G	Yes	2D/3D CAM	Export IFC
Technosoft (AxisVM)	EC5	Multiple	No	2D/3D	Export IFC
Autodesk (Robot Analysis)	Multiple	Multiple	No	2D/3D	Via Revit
CSC Inc (TEDDS)	Multiple	E	No	2D	No
TRADA (TimberPro)	EC5	E	No	2D	No
TimberTech	EC5	Multiple	Yes	3D	Export IFC

Timber design code: NDS (National Design Specification) DIN (DIN 4074, Germany), EC5 (Eurocode 5). Language: E (English), G (German).

The use and application of these tools is rather limited when considering the use of modern timber connections, components and systems such as propriety metal work, EWP and advanced closed panels such as those incorporating services or renewable energy technologies. Correspondingly, there is modest use made of advanced computer aid tools in the timber sector (Palmer, 2000) and indeed the use is normally fragmented.

In addition to the above, the publication of the Eurocodes and its inclusion in the UK building regulations requires timber design processes to be Eurocode compliant. The shift to Eurocodes is more onerous in the UK when considering timber design given the need to migrate from a permissible stress approach (BS 5268-1, 1996) to a limit state design approach (BS EN 1995-1, 2006). Eurocode is also a more analytical approach to design facilitating innovation and system evolution however, it is also a more time-consuming code to use requiring the need for easy to use yet transparent design tools.

Although suggestions were made to simplify verification of standards (Dietsch, 2012), particularly Eurocode 5 due to its high technical content, other actions to facilitate timber design are possible. An internal survey carried out by COCIS on a sample of 77 structural engineers (Figure 5-2) concluded that software solutions and continuous professional development (CPD) seminars are the most relevant actions to be taken on the timber engineering community. However, internal and external teaching activities promoted by Edinburgh Napier University such as seminars or practical laboratory exercises for undergraduates successfully combined theoretical content with software demonstration.

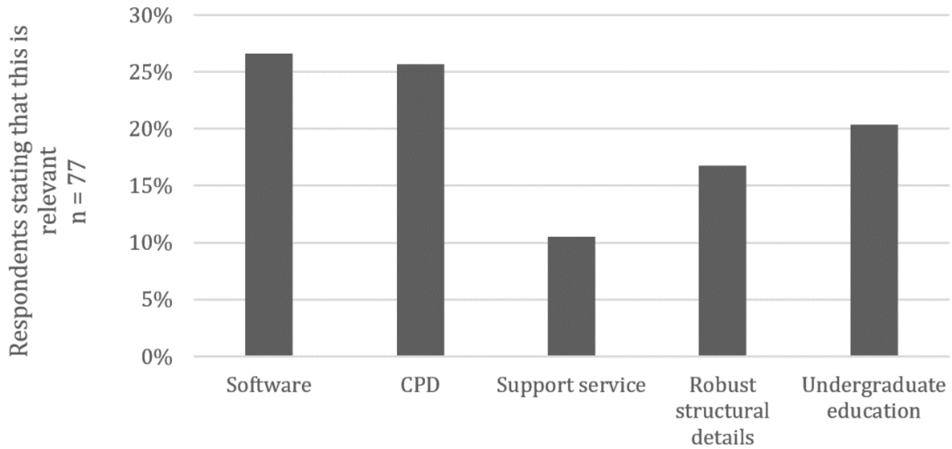


Figure 5-2 Routes to facilitate the use of Eurocode 5

It is evident that a software platform for timber design it is an appropriate tool for knowledge transfer into the timber engineering community. This software platform shall include not only proprietary materials and components but conclusion obtained from other non-commercial research activities.

5.2.3 Whole House Engineering platform

The structure of a Whole House Engineering mechanism, concept can be defined as a group of components or proformas that use a core centralised database of collated information (Osterrieder et al., 2004). The database retains the material, component or system performance information and the proformas process this information according to the different structural function to be designed for i.e. roof, floor or walls which when combined provide a Whole House Engineering platform (WHE). There are similar approach but for energy consumption simulation, none for timber structural design (Holst, 2003).

For structural timber design, Figure 5-3 shows different components that may be part of an integrated WHE concept (Menendez et al., 2012). In this particular model, the components are able to both stand alone and to be interlinked with other WHE pro-formas.

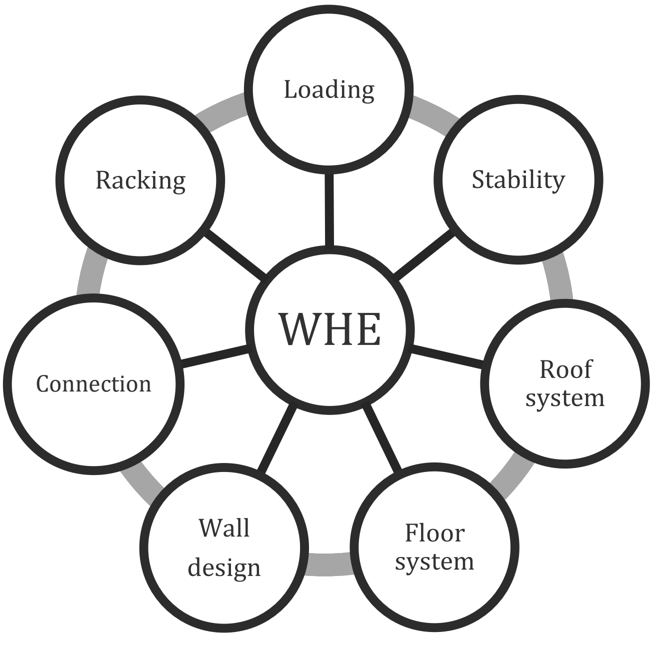


Figure 5-3 Whole House Engineering concept

5.2.4 Data sharing

Open and normalise models are a step forward on the construction industry and shall enable a great degree of interoperability between the different parts involved in the AEC sector. Since 1994, an industry consortium has been working on the Industry Foundation Classes (IFC) to standardise Information Technologies (IT) as an open data model in the construction sector (Kiviniemi, 2012). The aim of IFC is to facilitate Building Information Modelling (BIM) between cross-compatibility software platforms.

Osterrieder et al. (2004) concluded that the AEC industry together with the timber industry must be provided with viable design aided tools in order to facilitate the knowledge transfer from the research stage to the final market and enable data sharing instead of data exchange.

The model provided in Figure 5-4 follows the same principles from the current BIM theory. On this model, the authors propose a sequential migration from a management system with a high number of bilateral processes (a) to a data exchange system with fewer standard templates (b) and software systems to a data sharing model managed by a single model (c).

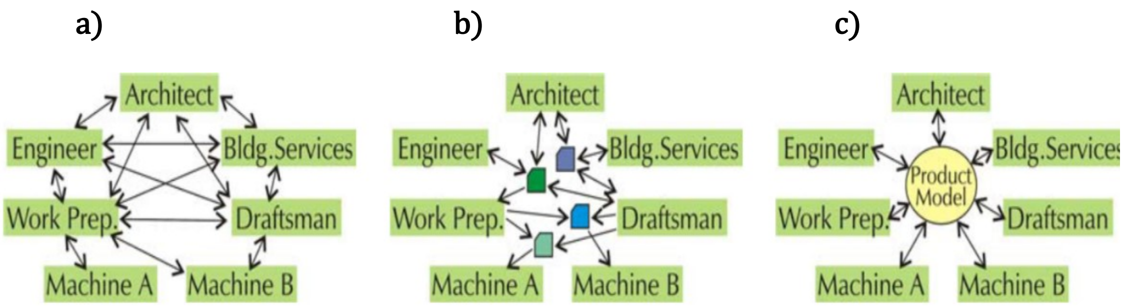


Figure 5-4 Data exchange model, after Osterrieder et al. (2004)

The standardisation of construction data management through BIM technologies, some of them driven by the government, have been recently implemented in other building materials such as steel or pre-cast concrete (Jeong et al., 2009).

In the WHE concept presented in the previous section, the database can include product specific, product generic or research specific information in order to facilitate the subsequent parametric analysis process to optimise the final structural design and interconnected with the different proformas or calculation libraries (Figure 5-5). It is also recommended that the database is BIM enabled as information can be imported or exported from other software or even in a spreadsheet format (Lucas, Bulbul , & Thabet, 2013).

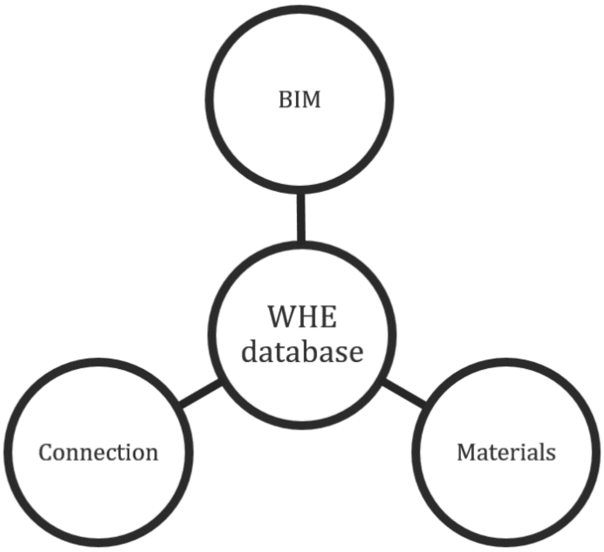


Figure 5-5 WHE database information

5.2.5 Integrated software for timber building design review

The use of advanced computer programs for design (CAD) and manufacturing (CAM) is relatively common in timber construction (Haller & Menzel, 1998). There are many suppliers that provides software packages able to deliver complete 3D-simulation, cut list of materials, take-off quantities or cost (Figure 5-6).

These tools are not only able to process graphical data but numerical and text information which can be used for warehouse inventory, purchasing orders or volumes and transportation requirements. MMC benefit especially of this technology as the manufacturing process and preparation of materials is streamlined. However, only few commercial software is available for the analysis and design of prefabricated timber structures with a self-explanatory Graphical User Interface (GUI) and even fewer applications where direct input from empirical data of timber components and systems can be integrated in the structural design.

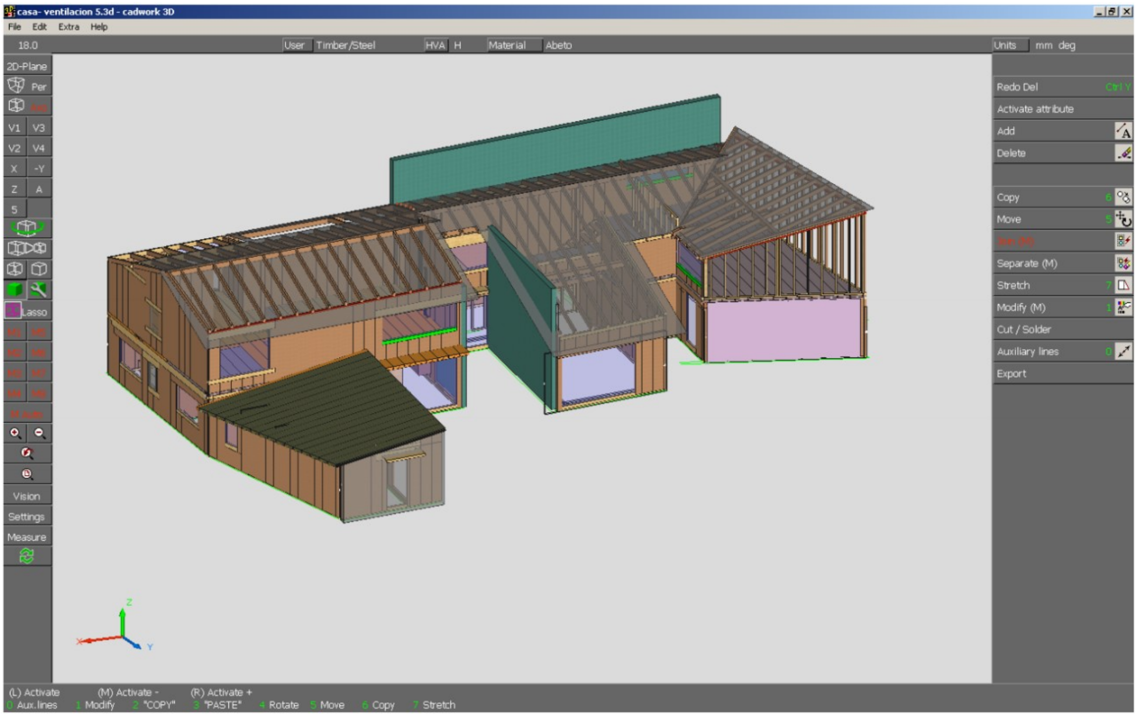


Figure 5-6 Snapshot of a CAD/CAM software program

Other available timber engineering related proprietary software used in the UK is currently provided, usually free of charge, by manufacturers of EWPs or metal plate connections. These design tools do not provide design flexibility as they are restricted to the products of the manufacturer portfolio only. The internal programming system is a 'black box' where the material selection, the method of construction or the design code of practice is normally hidden and not open for amendment. This restricts engineering judgement and commoditises the process. Finally, the capability to import or export information across other software platforms is very limited and as a result, Building Information Modelling (BIM) processing and the capability to apply a holistic design approach is restricted (Khalili & Chua, 2013).

At the same time, the market share of timber in construction is growing due to future building regulation requirements, government sustainability policies, the advantages of off-site timber MMC (Hairstans, 2010) and the increasing demand from architects and clients for EWP (Wilson, 2007b).

In addition to the above, the recent publication of the Eurocodes and its inclusion in the UK building regulations requires timber design processes to be Eurocode compliant. The shift to Eurocodes is more onerous in the UK when considering timber design given the need to migrate from a permissible stress approach (BSI, 1996) to a limit state design approach (Griffiths et al., 2005). Eurocode is also a more analytical approach to design, facilitating innovation and system evolution. However, it is also a more onerous code requiring the need for easy-to-use yet transparent design tools.

5.3 Development of Racking Wall Design Software

A key objective of the EPSRC project “Structural Optimisation of Timber Offsite Modern Methods of Construction” awarded to the Centre for Offsite Construction and Innovative Structures (COCIS) at Edinburgh Napier University was to develop structural details for advanced panelised system that were able to conform with future building regulations and offsite construction.

Outcomes of this research contributed to the EPSRC project by developing a software application. This tool is a mechanism to impact which is now cross-correlated with Eurocode design procedures and capable of exploiting a comprehensive database that can be potentially interfaced with other software applications in a BIM environment.

The work carried out in this research started in collaboration with the software company CSC (UK) Ltd, now Trimble Navigation Limited, in order to develop a mechanism to streamline the release of research findings to the AEC sector. In particular, the collaboration was established on work undertaken on new UK timber components and systems and their associated details. The WHE mechanism identified in the EPSRC project is shown in Figure 5-7 where the Racking Application was the first pro-forma to be developed

The analytical methodology described in the previous chapter and published as a complementary information document to Eurocode 5 was fully integrated in the program. The development of the racking wall design presented in this chapter involved a substantial C++ code and software programming learning processes.

The material databases included in the application can be populated with either generic information from available standards and technical data sheets or obtained through United Kingdom Accreditation Service (UKAS) laboratory testing. The information then is ready to be distributed directly to a targeted audience hence streamlining the access of practicing engineers to innovation. In this case, the material databases developed for the timber frame racking application contains generic data from standards and generic information from non-disclosure manufacturers.

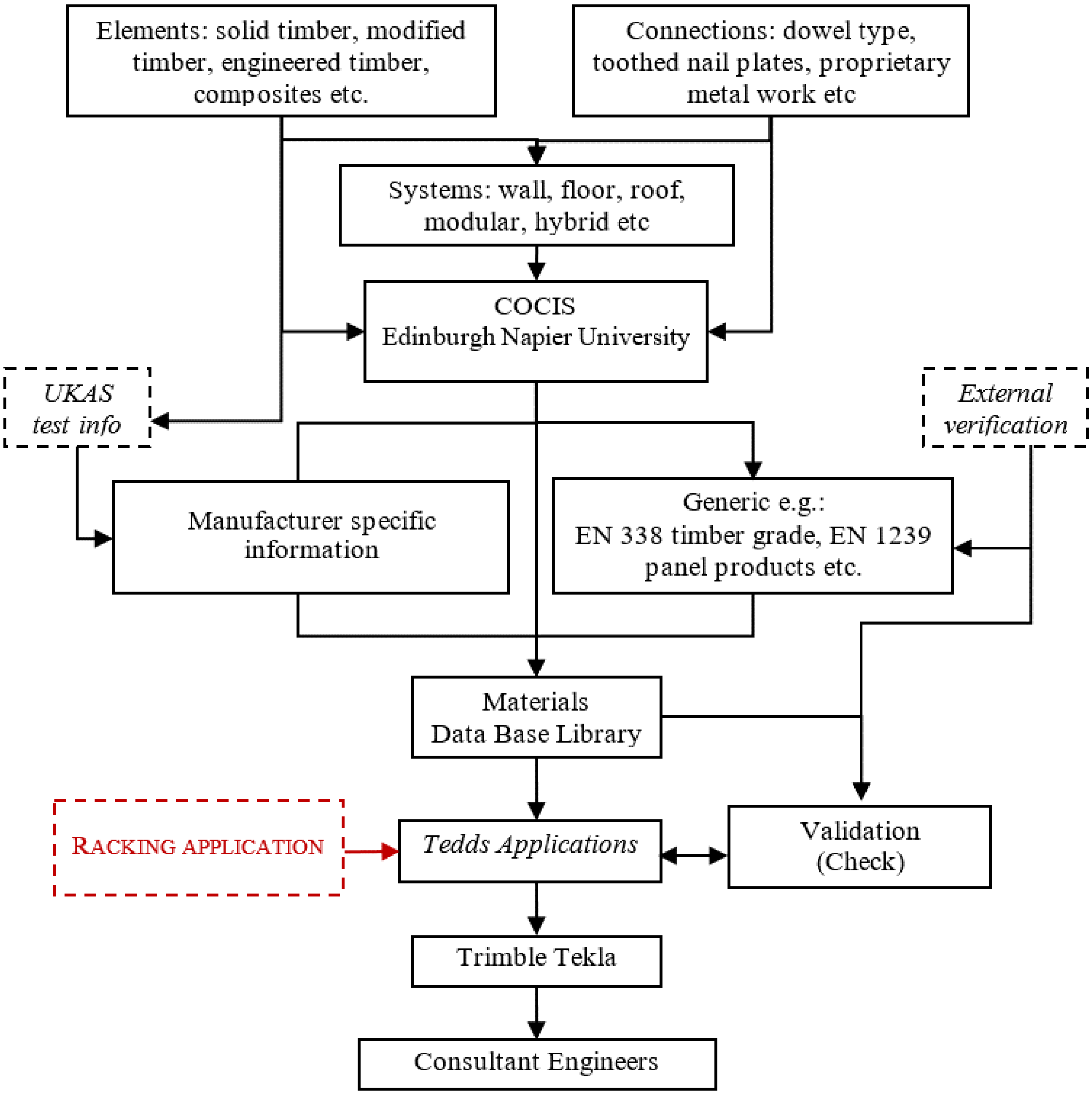


Figure 5-7 Methodology to deliver WHE mechanism via Trimble Tedds

5.3.1 The Trimble Tekla Tedds environment

Tedds – *The Calc Pad for structural Engineers* – is a software program developed by CSC (UK) Ltd. that allows the user to run engineering calculations from a comprehensive generic library and also to create and edit custom calculations. The timber design application has been developed under this proprietary platform for the following key reasons:

- Wide distribution across UK engineering practices: Trimble Tedds has sold more than 4,000 Tedds licenses in the UK, including network licenses. According to internal Trimble Tedds data, the estimated number of engineers using this tool within the UK is of around 14,000 (Figure 5-8).
- Access to the programming source code: Trimble Tekla Tedds provided the author with the Professional Development Package (PDP) tool, which enables the developer to produce, from scratch, professional calculations directly from the source code.
- Integration of databases. The software allows for the creation of Databases which can be populated with either generic information from available standards, manufacturer literature, or obtained through research and accredited laboratory testing.
- Creation of structural reports as MS Word documents. Trimble Tedds can also be installed and executed as a MS Word add-on. In this environment, the application also allows the user to attach other documentation, such as images, 2-D and 3-D sketches, tables or other informative notes.

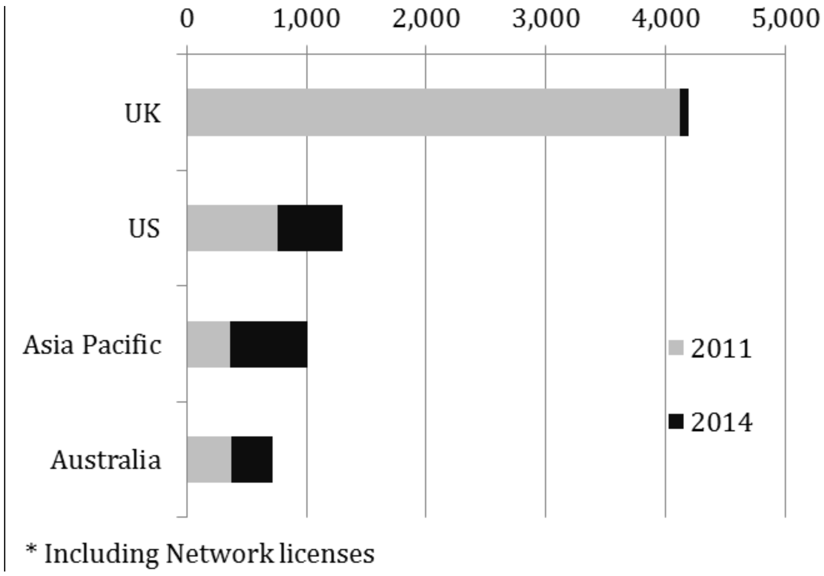


Figure 5-8 Trimble Tedds licenses sold worldwide

5.3.2 Platform for structural design

The structure of the timber design mechanism, based can be defined as a group of pro-formas, blocks, or applications that shared a core centralised database of collated information. The Tedds application produced in this research was timber frame racking design, which will be part of an integrated whole house engineering (WHE) mechanism to produce full structural reports for timber design. A flow chart of the timber frame racking application processes is illustrated in Figure 5-9.

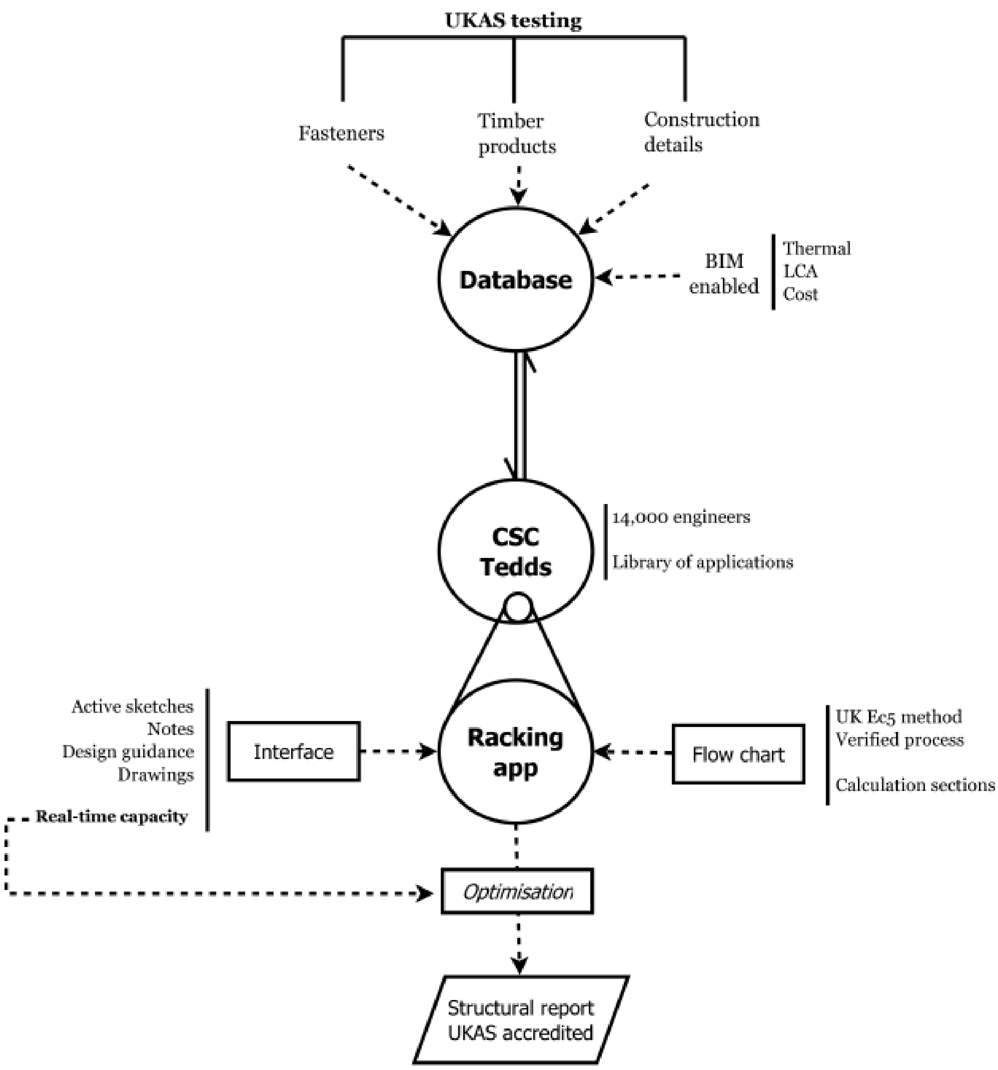


Figure 5-9 Tedds racking application optimisation flow chart

The database retains the material, component or system performance information and the application processes this according to the function to be designed for and, when combined, provide a structural design values. The database, created as spreadsheet and embedded within the Tedds application, can therefore be product specific, product generic or research specific in order to facilitate the subsequent parametric analysis procedure. This allows for final structural design optimisation. The database also has the potential to be BIM enabled as information may be able to be imported or exported from other software in the future.

Language code and programming

The programming language used to generate Tedds pro-formas is a composition of Visual Basic for Applications (VBA) and C++. Furthermore, CSC (UK) Ltd, as a result of a collaborative agreement, provided COCIS with its professional developer software package (Zhang & Leiss, 2001).

This package includes two additional tools, application designer and interface designer, which facilitates the creation of professional sets of calculations with refined graphic user-friendly interfaces (GUI) simplifying the need of advance programming knowledge. A brief description of their capabilities is provided below:

Application Calc Designer

The methodology to develop a new Tedds application begins by defining a unique operational flow chart. This is generated from the *Calc Designer* tool which facilitates access to the Tedds programming code source. The flow chart created follows a sequence managed by the GUI that streamlines possible amendment, checking or verification processes.

Also, for simplification reasons, the main page of the flow chart is a simple linear sequence, composed by sub-modules that describe in detail the whole calculation process. These sub-modules evolve into a more complex secondary flow chart with variable expressions. A screenshot of a part of the Racking application Calc Designer is shown in Figure 5-10.

Internally, every process of the flow chart ultimately relates to a *mini-block* or simplified calculation section, previously documented in Trimble Tedds and saved as section component in the application library.

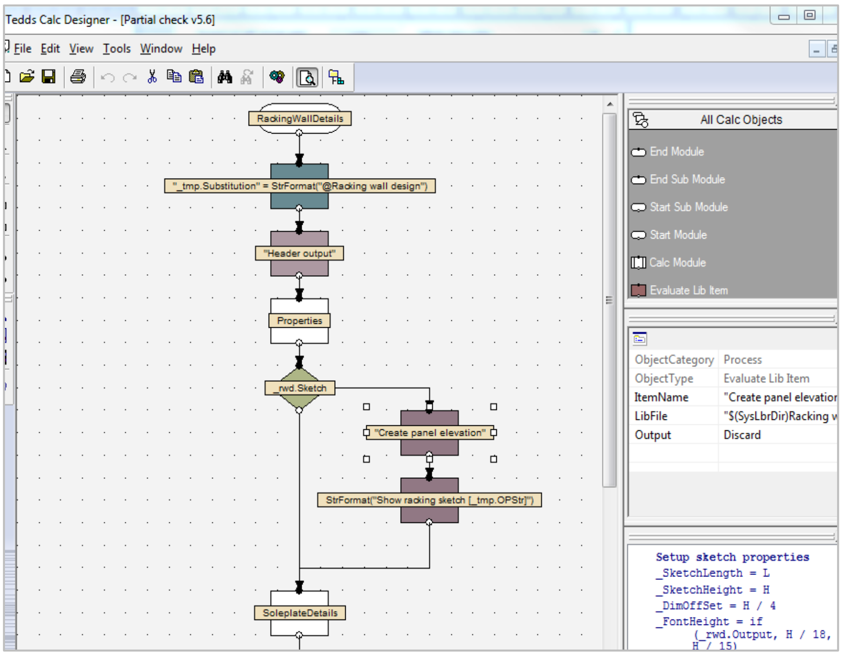


Figure 5-10 Calculation flow chart for Tedds Racking App

These sections or blocks can be structural calculations, active 2-D drawings, static 2-D and 3-D sketches, interfaces, guidance notes or supplementary language code information. Furthermore, these section components can be organised and structured in the user Tedds library as *Calc sets* by a folder-like system as shown Figure 5-11.

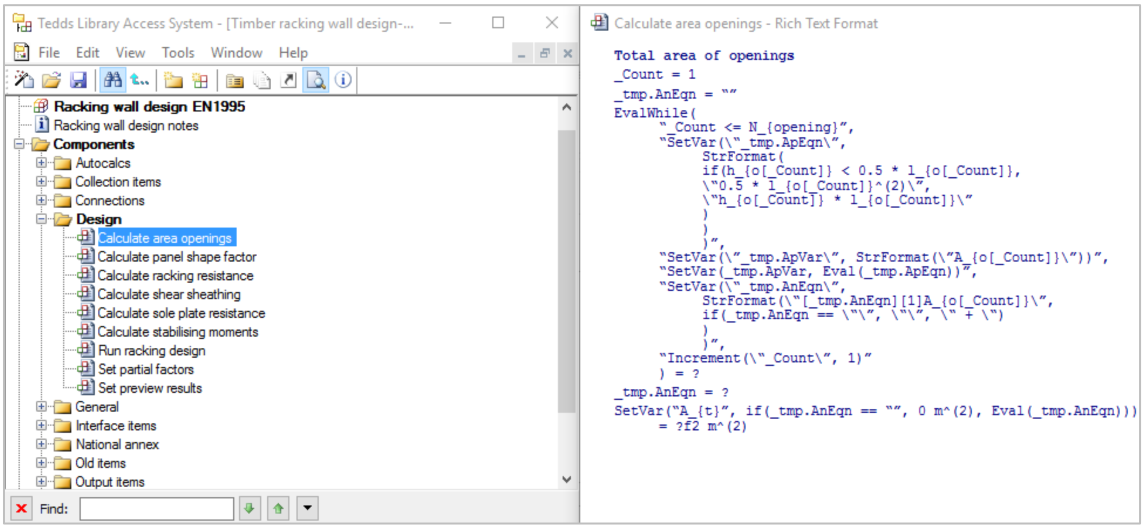


Figure 5-11 Racking wall design Tedds library

The complete library of this Timber Frame Racking Panel application contains the executable file, an informative document explaining the design procedure, a components folder with the calc sub-sections, flow charts, interface, sketches and other descriptions.

Finally, an additional folder containing three different examples of timber frame racking design is also provided.

Interface designer

The second developer tool is an application to create calculation interfaces that governs the Calc Designer flow chart. This tool facilitates C++ programming code via already pre-defined action buttons and input boxes. The main interface of the Timber Racking Wall Design is divided into six groups:

- Racking wall details

The parameters related to the construction and dimensions of the wall are defined in this section. The button *construction…* opens up a new interface containing the input details for the timber frame, the sheathing and fasteners and the type of the sole plate base fixing. Currently, three sole plate details are activated. However, it is possible to incorporate any other additional sole plate detail. The user must determine an equivalent spacing by dividing 1.5 kN/m by the characteristic shear capacity of that particular sole plate detail, in kN/m.

- Panel openings.

The button *Opening…* is activated if the check box in the Panel openings' heading is ticked. The new interface allows for the inclusion of up to five openings. To add a new opening the user can either click on the *Add…* button to create an opening after the last one defined or *Insert…* to create an opening right after the opening selected in the previous drop list menu.

- Loading detail

The loading button enables the introduction of the permanent load acting on top of the panel in kN/m, the self-weight of the panel in kN/m^2 and finally, any uplift forces, such as wind suction on the roof, also in kN/m. The self-weight input presents a drop list control that automatically calculates the standard weight of the panel depending on the current wall configuration. However, insulation or cladding is not included. Also in this section, the in-plane wind load is entered. This information may be needed in certain cases in order to determine the K_w factor for the racking strength.

- Design options

Here, the user can determine if the final calculation is given as design or as characteristic value. Characteristic values are given if the *Unfactored design check* control is ticked. In this section, service classes for the timber frame and the sole plate are also defined as described in Eurocode 5 cl. 2.3.1.3. At the moment, the racking capacity of a timber frame wall is calculated according to PD 6693-1.

- Results

Three columns are shown on the right middle section indicating the structural capacity and the applied stresses, and the factor of utilisation for every design check (Figure 5-12). In the timber racking design those checks are sliding, overturning and racking strength. A fourth check indicates the factor of utilisation of the deflection criteria according to the PD 6693-1. A green tick shows that the factor of utilisation for that design check is less or equal to unity.

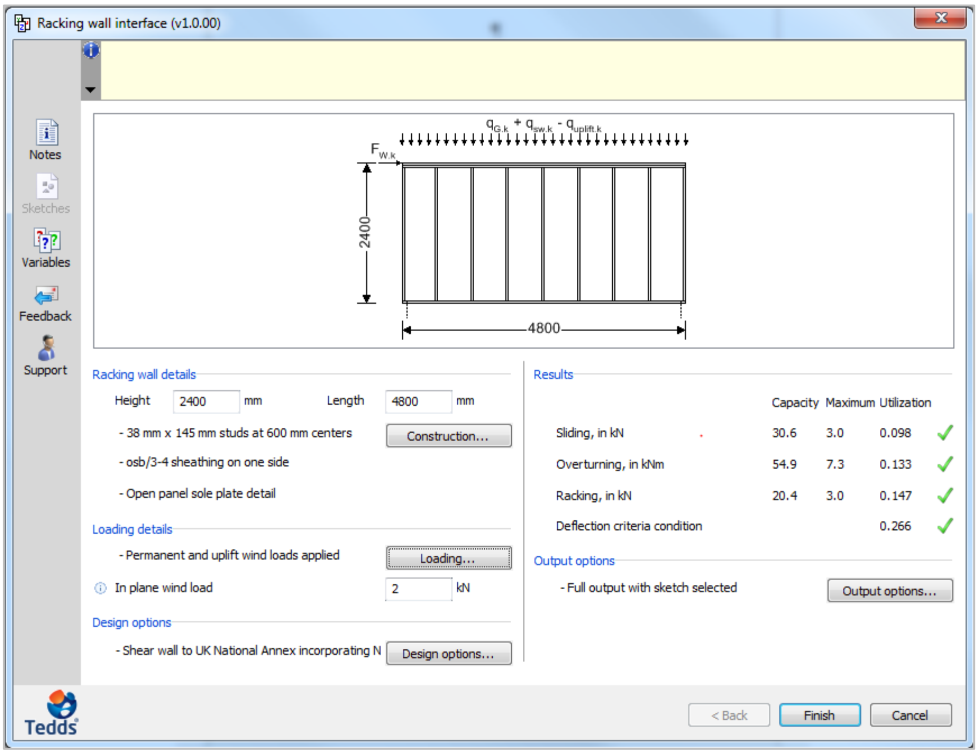


Figure 5-12 User interface for Tedds Racking App

- Output options

Finally, in this section the user can give a title to the current calculation. Also, this section allows the user to select the output format of the calculation. The different details are Full (comprehensive full report) or Summary (brief report). In general, three pages of a full report equals to one page of a summary report. A sketch of the racking wall elevation and cross section will be included in the final calculation report, regardless its output detail, if the relevant check box control is ticked.

Figure 5-12 shows the main GUI of the Tedds racking calculation. It is important to note how all the information relevant to the design, including the results, is displayed in just one window. This provides a fast way to perform structural optimisation and a parametric analysis.

An example of a full and summary output report from the Trimble Tedds Timber Frame Racking Design calculation from a practical timber frame racking exercise is included in the Appendix XI.

5.3.3 Centralised database information system

A Tedds DataList or Database allows the user to access stored data for a wide range of applications. A simple button embedded in the Tedds interface connects these stored and updated Databases with the Trimble Tedds applications.

The pop-up database window allows you to select specific items where all of the associated information is dragged onto the calculation pad. Databases are not only an intuitive way of selecting data to speed up calculations but also are a powerful tool to use for research purposes. This information can, almost immediately, be used in the final pro-forma calculations.

Although there is a wide-ranging list of different databases embedded within the standard Tedds library, Databases can simply be created by the user as required from a MS Excel spreadsheet. The Trimble Tedds application, Data List Designer, is able to convert the data populated in MS Excel to a Tedds format accordingly. The sequence to create a Tedds DataList is shown in Figure 5-13.

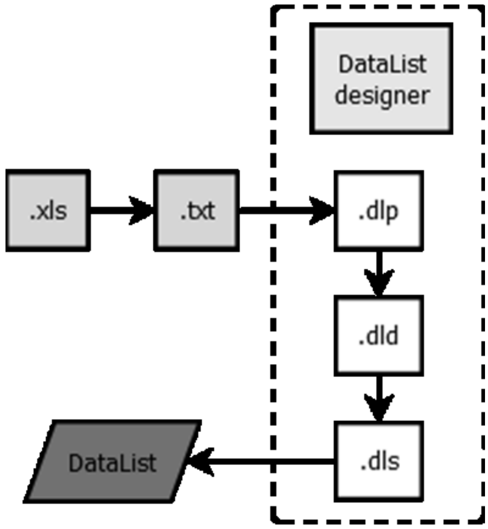


Figure 5-13 Tedds DataList creation sequence

5.4 Racking Software Validation

In terms of the accuracy of the calculation engine, a preliminary racking analytical calculation of 20 wall panels was conducted which validated the Tedds Timber Frame Racking application.

The standard PD 6693-1: Recommendations for the design of structures to BS EN 1995 Eurocode 5 for timber structures was the applied design methodology. The racking results, given by the Tedds application, were then compared with parallel calculations in PTC Mathcad® and MS Excel. These 20 wall panels were also previously tested by Edinburgh Napier University (Leitch, 2013). The satisfactory results determined by the Racking Application were within a 5 % deviation of the 0.7 and 0.4 ultimate load test results for strength and stiffness as proposed by Porteous & Kermani (2013).

Figure 5-14 provides a flow chart of the racking validation procedure. In this figure, the SLS comparison process referred to the compliance of the stiffness clause provided by equation 4.19.

The racking deviation achieved in comparison to the other two methods was mostly due to rounding errors and it fitted within a pre-established deviation limit of ± 2%. This exercise successfully validated the analytical Tedds racking application process defined in the developer tool Calc Designer and governed by the developer tool User Interface.

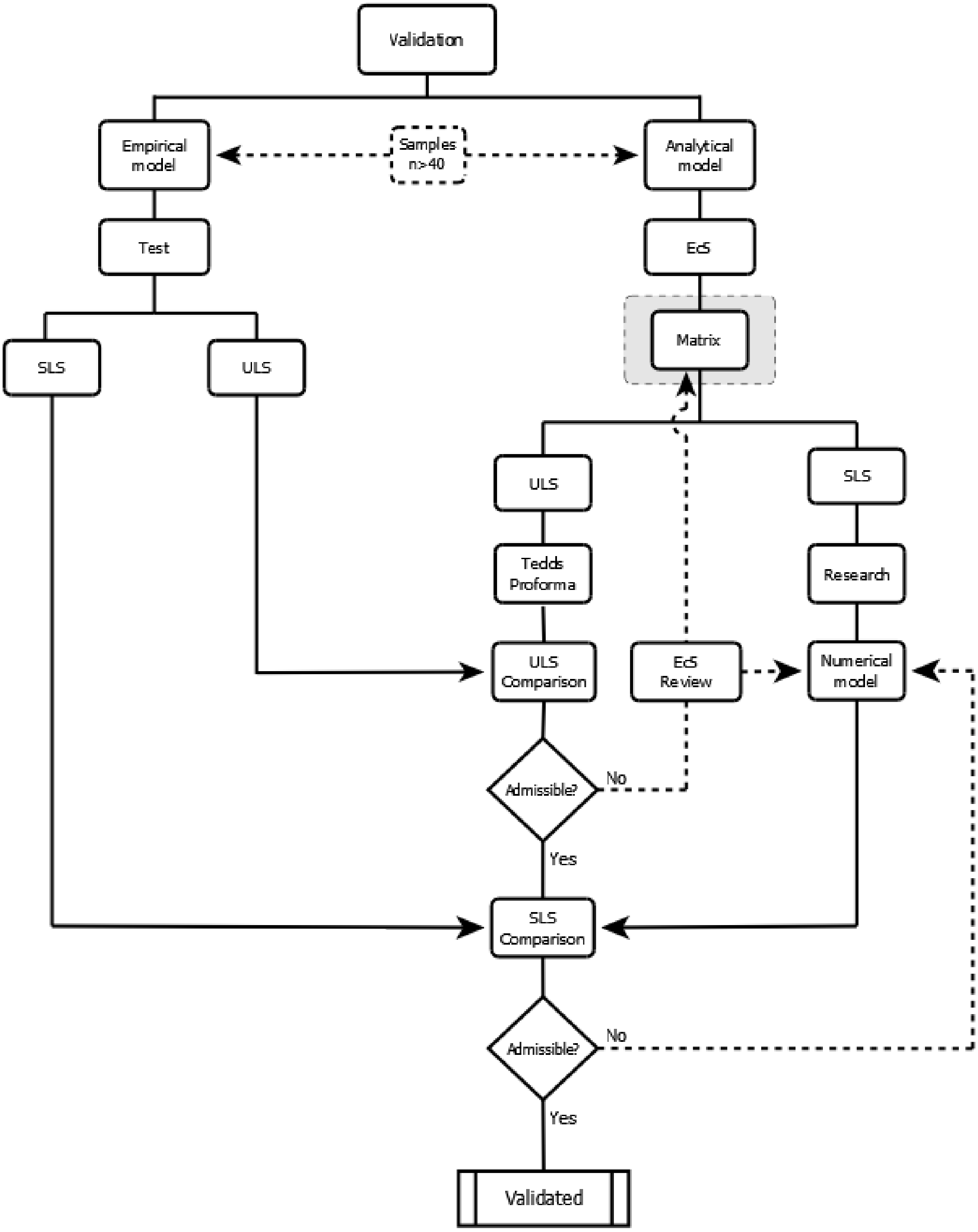


Figure 5-14 Tedds validation flow chart

It must be noted that the racking resistance of one panel (no. 17) was discarded for statistical purposes as the Mathcad result was out of consideration. Results of the racking application validation are additionally presented in Figure 5-15. The Mathcad results were in general slightly lower values than in Tedds and Excel. As there were no difference in the programme analysis, this could be the result of rounding errors.

(kN)	Tedds	Excel	Mathcad
1	9.17	9.17	9.11
2	18.33	18.33	18.23
3	26.94	26.95	26.78
4	47.14	47.17	46.87
5	18.85	18.87	18.80
6	9.72	9.73	9.67
7	27.34	27.54	27.42
8	14.46	14.42	14.37
9	9.72	9.73	9.67
10	11.55	11.54	11.49
11	9.72	9.73	9.67
12	4.82	4.82	4.80
13	4.82	4.82	4.80
14	8.77	8.79	8.75
15	6.08	6.08	6.06
16	12.33	12.30	12.24
17	9.70	9.64	2.23
18	9.99	9.91	9.87
19	3.20	3.18	3.17
20	3.10	3.10	3.10

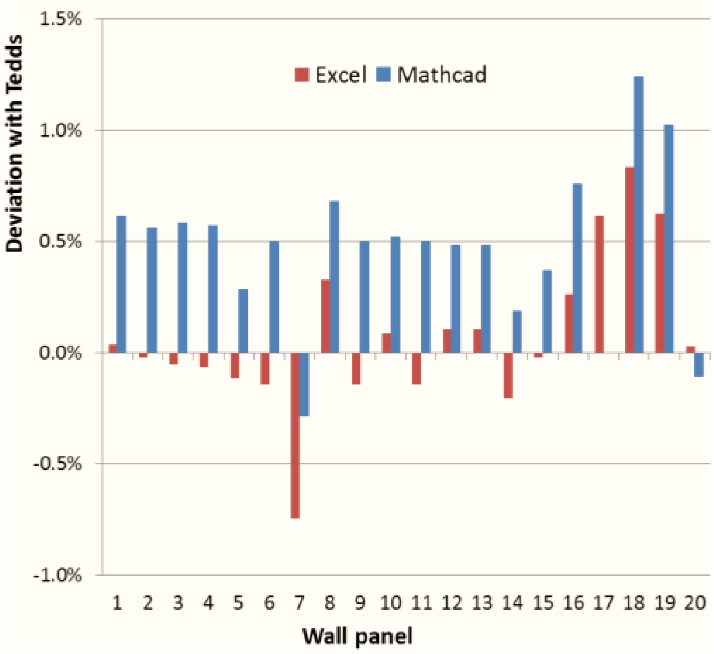


Figure 5-15 Validation of Tedds racking application in correlation with Excel and Mathcad analysis of 20 timber frame panels.

5.5 Timber Frame Racking Design User Statistics

Since the first release of the application in January 2014, Trimble Tedds has been monitored the interaction between their registered users and the Timber Frame Racking Design application developed in this research.

According to internal Trimble Tedds data, Figure 5-16 shows the number of times registered users run the racking application according to EC5 method C and BS 5268. The absolute number of times the racking application, according to EC5, was run for all of the registered Tedds users is 280, 600 and 700 times for year 2014, 2015 and 2016 respectively. According to Trimble Tekla Tedds, the estimation is that there are three unregistered Tedds users per every registered user.

Throughout the course of the PhD, specific training CPD was provided to structural and civil engineers and to Edinburgh Napier University students using the software application which is commercially available. The training manual, including a practical example, is provided in the Appendix XII.

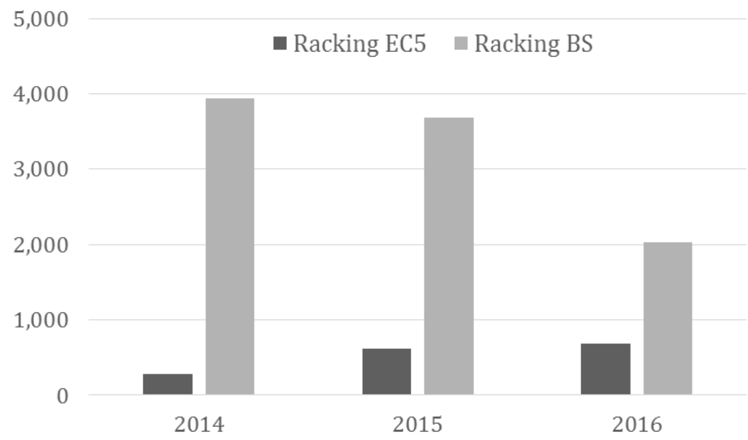


Figure 5-16 Tedds racking application statistics

5.6 Parametric Analysis Based on Eurocode 5

Once the Trimble Tedds application was validated, the second part of the research framework defined in this chapter was carried out. The timber frame racking design tool was used to perform a parametric analysis of a series of timber frame wall panel configurations and to demonstrate the capabilities of the software in terms of speed of calculation.

However, the commercial version of Trimble Tedds does not allow for running simultaneous timber frame racking applications, i.e. more than one panel at a time. Nonetheless, the output of different racking calculations can be determined directly in the user interface (GUI methodology).

Another more complex data management system could have been created to perform multi-variate parametric calculations. As illustrated by Figure 5-17, programming loop techniques in language C++ could have been used for the Tedds Applications in order to set temporal reference variables with a series of multiple iterations (Tahbildar & Kalita, 2010).

However, the additional complexity of the looping technique development for a single parametric study and the no need to produce a full calculation report for each of the parametrical combinations justified the used of GUI parametric methodology instead.

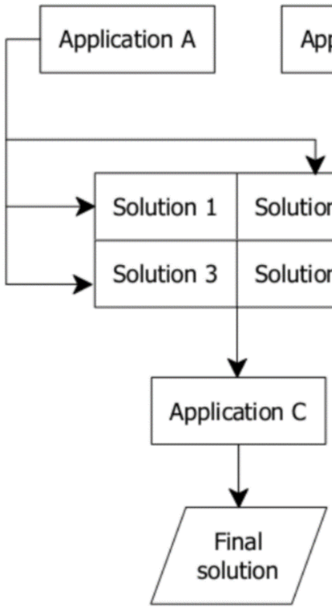


Figure 5-17 Programming loop techniques. Adapted from Tahbildar & Kalita (2010)

The parametric analysis presented two group of variables: timber frame materials and timber frame design (Figure 5-18). The variable *materials* refers to the properties of the elements constituting the timber frame wall panel whereas the variable *design* denotes to the configuration, design and geometry of the wall panel.

The different variables used in the parametric analysis are listed in Table 5-2 for C24 timber frame systems. Other variables such as timber frame height (2.40 m), sheathing fastener type (nail) and sole plate configuration did not change in this exercise.

Table 5-2 List of variables used in the parametric analysis matrix

Element	Component	Attribute	Values
Wall	Frame	Length (m)	2.40 / 4.20
		Openings (%)	0 / 20 / 40
	Sheathing	Material	OSB3 / P5
		Thickness (mm)	9 / 12 / 15
		Number of layers (sides)	1 / 2
Connections	Sheathing Nail Fasteners	Length (mm)	50 / 75 /90
		Spacing (mm)	75 / 100 /150
	Base fixing	Hold-down (kN)	0 / 22
		Withdrawal (kN/m)	2.47
Actions	UDL	Permanent load (kN/m)	0 / 10 / 20

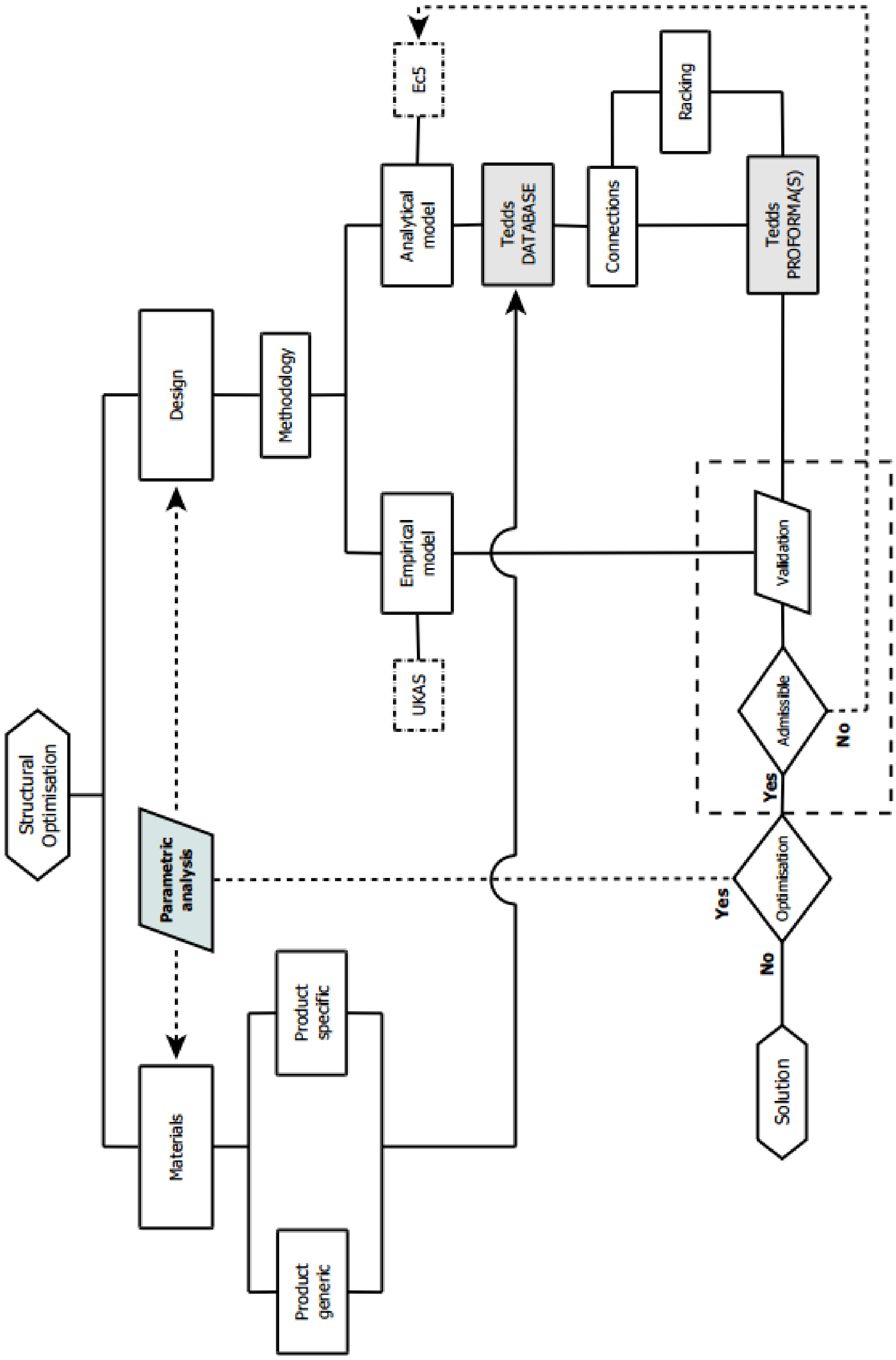
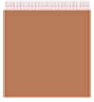
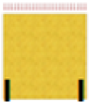
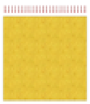
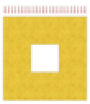
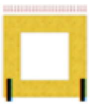
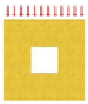
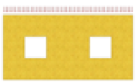
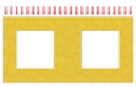


Figure 5-18 Parametric structural optimisation flow chart

From all of the different variables and in order to investigate the effect of each variable independently, nine different possible racking scenarios were proposed as shown in Table 5-3. The parametric study includes different wall lengths, percentage of openings, UDL applied to top runners, type of sheathing material, number of sheathing layers and inclusion of holding down straps.

Table 5-3 Proposed parametric racking panel scenarios

	Length (m)	Opening (%)	UDL (kN/m)	Sheathing	Layers	Hold-down (kN)
	2.4	0	10	P5	1	No
	2.4	0	10	OSB/3	2	No
	2.4	0	10	OSB/3	1	22 kN
	2.4	0	10	OSB/3	1	No
	2.4	20	10	OSB/3	1	No
	2.4	30	10	OSB/3	1	22 kN
	2.4	20	20	OSB/3	1	No
	4.2	20	10	OSB/3	1	No
	4.2	30	20	OSB/3	1	No

Furthermore, each plotted graph in the parametric wall analysis contained structural information about five different sheathing fastener schedules for each of the sheathing thicknesses (9 mm, 15 mm and 18 mm):

1. nail dia3.1x50 mm long and 150 mm perimeter spacing (n3.1-50-150)
2. nail dia3.1x50 mm long and 100 mm perimeter spacing (n3.1-50-100)
3. nail dia3.1x50 mm long and 75 mm perimeter spacing (n3.1-50-75)
4. nail dia3.1x75 mm long and 150 mm perimeter spacing (n3.1-75-150)
5. nail dia3.1x90 mm long and 150 mm perimeter spacing (n3.1-90-150)

The result of the parametric analysis provides a general overview of the influence of sheathing board and sheathing fasteners on the racking strength and stiffness values for the different case scenarios analysed.

The results of the parametric analysis are illustrated by groups of panels with similar characteristics: Figure 5-19 for imperforated wall panels, Figure 5-20 for perforated short wall panels and Figure 5-21 for perforated long wall panels.

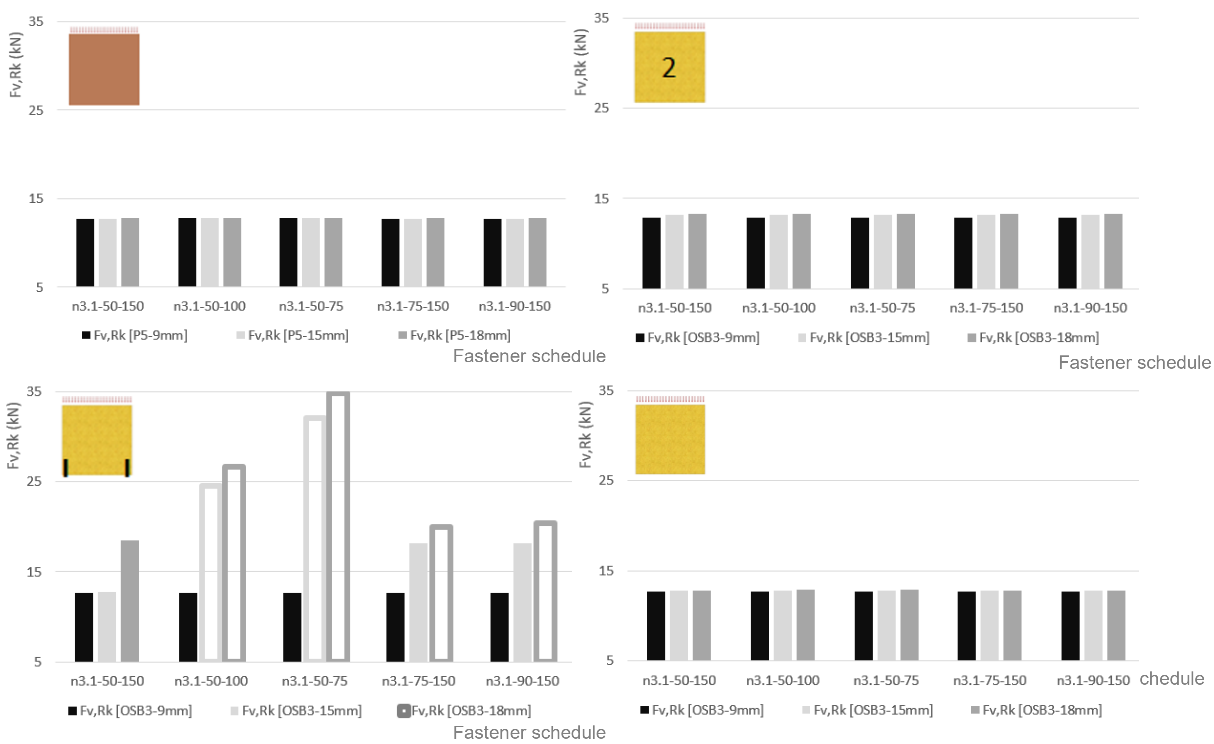


Figure 5-19 Parametric racking analysis for imperforated panels.

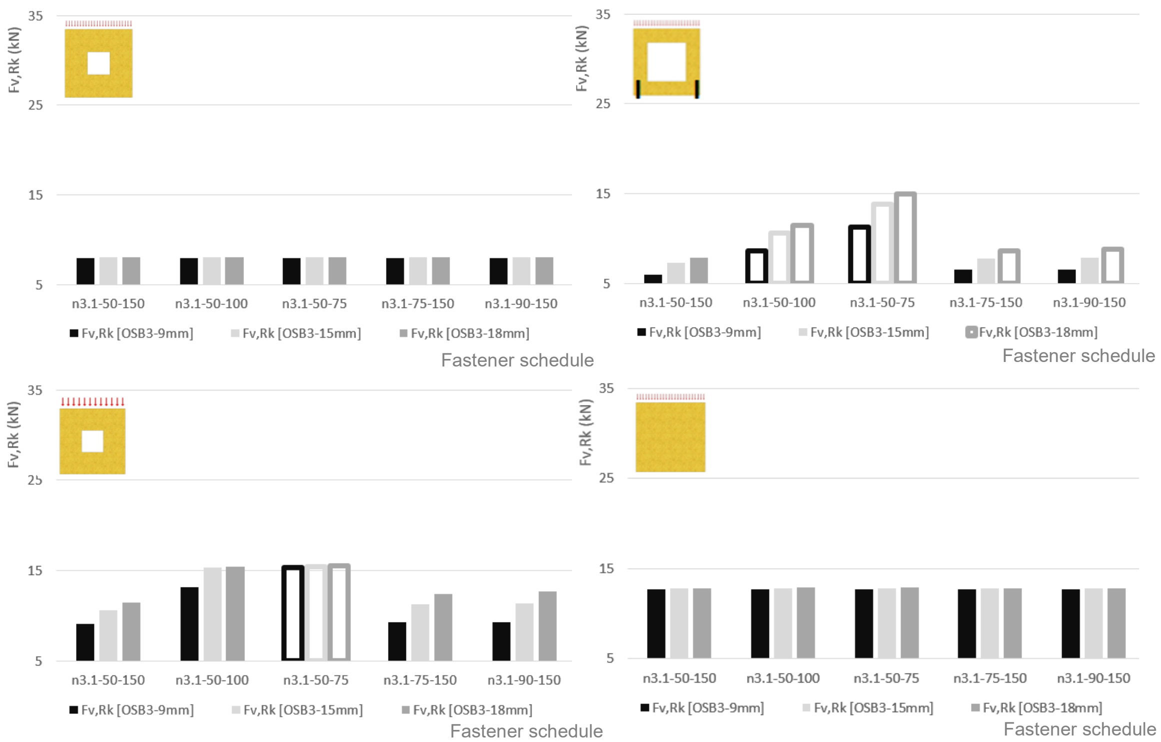


Figure 5-20 Parametric racking analysis for short perforated panels.

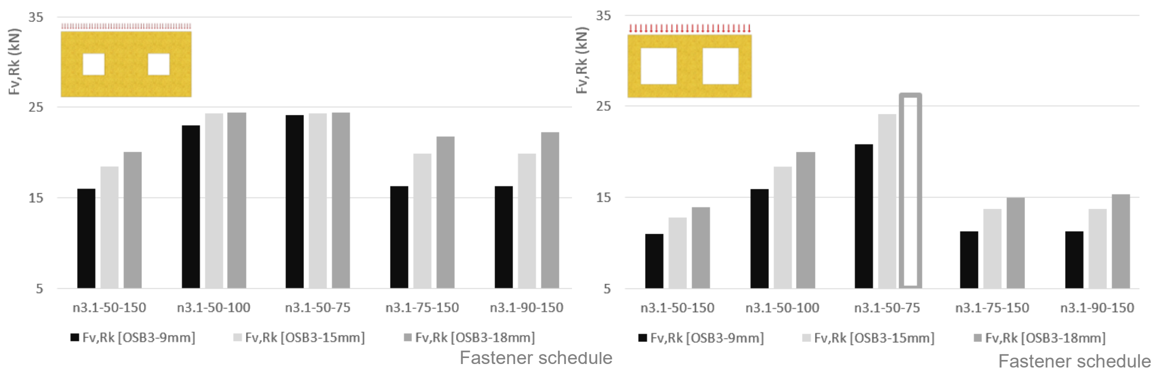


Figure 5-21 Parametric racking analysis for long perforated panels.

In solid black fill, light-grey and dark grey colour are represented the sheathing boards of thickness 9 mm, 15 mm and 18 mm respectively. In case the analytical racking performance does not satisfy with the serviceability criteria given by equation 4.19, the values are represented by a thicker border line with no solid filled rectangle.

In addition, a parametric analysis considering dia2.8 mm and dia3.1 mm smooth and ring-shanked nails for OSB/3 sheathing boards of 9 mm and 15 mm was performed.

The nail schedule presented two different spacing of 100 mm and 150 mm. The results show the characteristic racking capacity of a single sheathed and fully restrained 2.4 m x 3.6 m wall made of C24 timber (Figure 5-22).

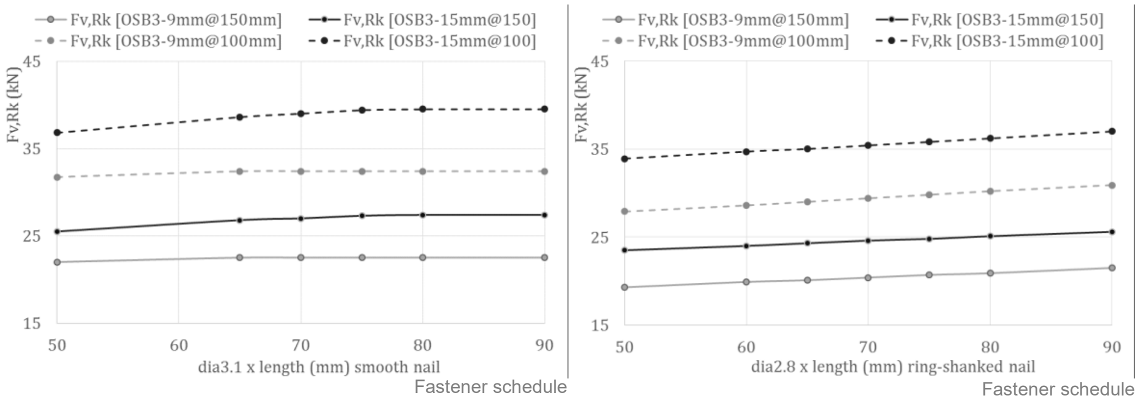


Figure 5-22 Parametric racking analysis for smooth nails (left) and ring-shanked nails (right)

The results of this parametric study show that for dia3.1 mm smooth nails, an optimal nail length, dependent of sheathing thickness and independent of nail spacing, can be specified as 5 times the sheathing thickness. This conclusion was not observed for dia2.8 mm ring-shanked nails which showed a more linear response.

5.7 Summary

There is a modest use made of advanced computer aided tools in the timber industry in comparison with the IT resources available in the steel and concrete construction sector (Palmer, 2000). The introduction of Eurocode 5, a more analytical approach to timber design, facilitates innovation and enables building systems evolution.

An internal survey carried out by Edinburgh Napier University on seventy-seven structural engineers concluded that software solutions and continuous professional development seminars were the prefer actions to increase timber engineering.

A commercial software application for the design and optimisation of timber frame racking walls, according to the Eurocode 5, was developed. This design tool included not only standardised materials, components and sole plate details but outcomes obtained from the research activities carried out in the previous chapter. This provides the engineering community with design flexibility as the tool is not restricted to any specific product in comparison to other related proprietary “black-box” software. The software application is also part of a Whole House Engineering concept and on-going design platform with a centralised database of collated and shared information.

C++ language was the programming code of the design tool developed in collaboration with the software company CSC (UK) Ltd, now Trimble Navigation Limited and integrated within their Tekla Tedds portfolio of design applications. The estimated number of engineers using this software is around 14,000 in UK and around 25,000 over the world.

Prior to public release, the timber frame racking application was validated by a preliminary analytical racking calculation of 20 wall panels. The analytical calculation was then compared with the identical calculations in PTC MathCad and MS Excel. The results obtained by the Tekla Tedds Racking Application was satisfactory within less than a pre-established deviation limit of ± 2 %.

Once the software was validated, a parametric analysis with different wall panel configurations was performed. Fully restrained timber frame walls with holding-down devices were exposed to a larger improvement on the racking resistance by either increasing the number of sheathing layers and its thickness or by improving the nail specification.

It was noted that the optimal racking strength, when fixing the OSB/3 sheathing board with smooth nails, was achieved for nails five times longer than the sheathing thickness. However, this conclusion was only observed for smooth nails.

This chapter has presented the development of a software application for racking design. The application allows for the inclusion in of robust sole plate details within a comprehensive database system, including holding down straps. Furthermore, the understanding of the EC5 methodology implemented in the software, in terms of strength and stiffness, as the deflection criteria condition, for racking design can assist to develop optimised timber frame closed panel systems for low-energy buildings.

6 Outcome of the Research, Conclusions and Future Work

The research carried out in this thesis has addressed relevant thermal and structural issues for closed panel timber frame wall systems in order to provide optimised solutions for low-energy buildings. A "gap in knowledge" was identified regarding the relationship between thermal and structural performance in combination for closed timber frame panels.

The direct specification of two proposed optimised closed timber frame panels for affordable low energy buildings (Chapter 2) is now possible. In isolation, the investigations included, for the closed timber frame and the sole plate details, a thermal performance optimisation (Chapter 3) a structural behaviour investigation (Chapter 4) and a software application for timber frame racking design (Chapter 5).

This chapter collates all the outcomes generated throughout the course of the thesis and proposes an integrated set of solutions templates combining thermal and structural information. This can aid decision making in early design stages explicitly for low energy timber frame projects.

The two core aims of the thesis and presented in the introduction chapter were successfully accomplished:

- **To develop two closed timber frame panel wall systems with thermal and structural optimisation.**

The thermal optimisation was achieved by providing different wall panel build-ups and insulation types condensation free. The structural optimisation was achieved by determining adequate sole plate shear plane fastener schedules free from thermal bridging.

- **To develop a racking software application to provide structural engineers with a platform for flexible design and closed panel optimisation.**

A design tool was created based on the analytical methodology PD 6693-1 and published by Trimble Tekla. The tool was corroborated against MathCAD and Excel. The software application was then utilised to perform a parametric analysis for different timber frame panels.

In order to achieve these two main objectives, the following set of secondary objectives presented in the introductory chapter have been also achieved:

i. A data gathering of timber frame shear walls and sole plate connection tests from open timber frame panels was carried out.

ii. Two different closed panel timber frame configurations suitable for low energy building design were proposed.

iii. The hygrothermal performance of these closed panel systems based on different materials and sole plate details was investigated.

iv. 2-D Thermal Finite Element Analysis (FEA) for different sole plate fixing details and for thermal optimisation was undertaken.

v. The impact of current timber fraction calculations on the overall thermal performance of Passivhaus timber frame buildings was evaluated.

vi. A simplified theory, weakest link theory, for the analytical optimisation of closed panel timber frame sole plate details was developed.

vii. The analytical and experimental racking results for the two proposed closed panel systems and under partially and fully restrained sole plate base fixing conditions was compared.

viii. The software application was validated by comparing the analytical output obtained with the results achieved from other calculation tools under the same analytical methodology.

ix. A series of multi-parametric analyses for shear wall optimisation was performed.

x. The output of the optimised racking walls with the results from the thermal analyses and providing technical data-sheets for direct Passivhaus timber frame wall specification was integrated in a template format.

The final conclusions from the research study and its contribution to knowledge are provided in this chapter. Other suggestions and comments regarding potential future work on low-energy timber frame buildings are also presented.

6.1 Integrated Set of Solutions

Very few accredited closed panel timber frame construction details have been standardised for conformity with regards to low-energy building requirements and in terms of thermal and structural performance combined.

As a direct outcome of this research, robust details including thermal and structural data in combination, for the proposed K2 and RTC closed panel timber frame panel systems, can now be delivered in a visual and informative manner. These enhanced construction details can include assembly, structural performance and thermal related information as shown in Table 6-1.

Table 6-1 Information available for integrated set of solutions

Assembly	Structural	Thermal
Timber frame materials	Racking resistance	U-Value
Dimensions	Holding down straps	Sole plate thermal bridge
Type of fixings	Foundation type	Temperature factor
Service cavity	Sheathing nail spacing	Phase shift
External insulation	Applied permanent load	Heat storage capacity
	Sole plate installation	Permeability
		Phase shift
		Condensation risk

Climates: cold, cold-temperate and warm-temperate according to Passivhaus

An example of enhanced construction details for the RTC and the K2 closed timber panels is included in the Appendix XIIII. The proposed set of solution included in the examples is a potential visual representation of a particular closed panel system detail which can contained, if required, other specific thermal or structural information provided within this thesis.

6.2 Main Conclusions and Contribution to Knowledge

The conclusions of this research and the critical evaluation of the evidence and results presented are summarised in line with each chapter as they chronologically appear within this thesis. Explicit contribution to knowledge resulted throughout the course of this research is also highlighted in bold.

6.2.1 Requirements for future affordable housing

Building Regulations are being updated to reduce housing CO2 emissions by improving the thermal envelop performance. The Passivhaus standard has been adopted as building regulation in several European local authorities. **Passivhaus standard may fulfil the EU directive 2010/31 where these buildings are regarded as NZEB**.

The current UK housing scenario is economically driven by a housing shortage whilst 25 % of the households are considered to be in fuel poverty. These and other production related factors, such as the lack of skilled workforce, were the drivers of the

Construction 2025 report (DBIS, 2013). **Advanced timber frame closed panels are a potential mainstream construction system for low-energy and Passivhaus design.**

Timber frame walls, considering also the integration of windows, account for almost half of the energy losses within a dwelling. The improvement of standard open panel timber frame systems into advanced closed panel solutions provides a practical challenge to achieve integrated and innovative low-energy building design in a cost-effective manner.

Robust construction details with tight interlocking connections and minimal tolerances between components, the practical implementation of these details both off-site and on-site and an adequate stiffness of the timber frame around high stress concentration areas were considered in this research. **To avoid the occurrence of air leakage as a consequence of construction gaps during service life, tight interlocking connections with 5 mm tolerances and minimum shear planes are recommended.**

Prefabricated timber frame panel systems require a high level of detailing which is not always produced causing difficulties in the erection phase. A common example was found on the sole plate base fixing detail for closed panel timber frame construction.

Very few timber frame construction details have been standardised for conformity with regards future regulatory requirements. This research provides a compilation of informative data sheets containing thermal and structural details for advanced timber frame panelised systems.

6.2.2 Thermal performance of timber frame walls

The conclusions of the airtight measurements of two projects accounting for 56 Passivhaus and Minergie-P buildings after completion and two years after showed that 32 out of the 56 buildings showed a certain degree of airtightness degradation whilst 13 out of these 56 buildings presented 50% or greater air change rate.

Two closed panel systems, dual frame and I-joist frame, were investigated with various stud depths and insulation configurations in order to obtain different tabulated U-values and hence, better thermal transmittance. **Low-energy closed timber frame panels**

can moisture safely accommodate several levels of insulation for cold and temperate climates.

Additionally**,** two different sole plate base fixing details were proposed for timber frame on top of a concrete raft and timber frame on top of suspended timber floor cassettes where the foundation was thermally optimised with XPS rigid insulation on the edge. The sole plates proposed tried to provide robust details considering the conclusions from the literature review in terms of resilient airtight constructability.

Three different software packages, for 2-D steady-state thermal simulation, were evaluated to assess the accuracy of their engines with regard of the ISO 2011 method. A fourth 2-D method on-line tool was also evaluated. These values were compared to the analytical simplify method EN 6946. **Simplified U-value analysis methods can be considered effective for timber frame wall panel thermal transmittance.**

The literature review undertaken on timber fraction showed not only a high discrepancy between published standardised timber fraction values but on the methodology employed to measure this percentage. A new approach to account for the timber fraction on low-energy timber frame buildings, where thermal bridges are determined, was provided. Another interesting conclusion was that timber fraction has a greater impact on less insulated timber frame walls, especially on the benchmark scenario. The timber fraction impact when insulation is placed externally was almost negligible. **A new methodology to determine timber fraction values for low-energy timber frame buildings was presented. The mean timber fraction factor observed in four different projects was 10 %.**

One of the main conclusions from the fifty-six U-Value calculations performed is that the benchmark timber frame configuration was not suitable for Passivhaus construction even with external rigid wood fibre board insulation and internal insulated service cavity. Furthermore, the benchmark wall panel without additional insulation and with wood fibre insulation in the core layer did not even comply with current Building Regulations. **The benchmark standard 6-inches timber frame kit, even when a service cavity is insulated, are not suitable for Passivhaus construction.**

In total, over one hundred and fifty thermal bridge were calculated for four different panel configurations and two different sole plate details for each of the closed panel systems proposed. One of the conclusions obtained was that no additional heat loss due to thermal bridging occurred at the sole plate detail for closed timber frame panels. The study also concluded that in general, the RTC wall panel performed slightly better for all timber frame widths except the 300 mm thick. **Also, the slab on grade foundation type thermally performed better than the suspended floor system.** The insulated service cavity in the thermal bridge simulations was modelled with horizontal and vertical battens spaced every 610 mm centres and mineral wool insulation in between them. **Horizontal battens performed thermally slightly worse than vertical battens as the lowest batten, commonly placed to fix the skirt board, increased the thermal bridge.**

Finally, three climates were considered for condensation risk analysis in relation with the range of U-values resulted for the closed panel systems and its respective Passivhaus U-value recommendations for cold, cold temperate and warm temperate climates. Three different methodologies were used: temperature factor, Glaser method and transient 1-D WUFI analysis.

In terms of moisture management, both closed timber frame panels with 195 mm core, the more unfavourable panel, can be rated as condensation-risk safe for cold-temperate and warm-temperate climates except the suspended floor foundation with no additional insulation. **The benchmark timber frame panel condensation risk analysis showed potential condensation issues for both steady-state and transient calculation methods.**

6.2.3 Structural performance of timber frame walls

A series of twenty timber frame racking tests were carried out in order to compare the experimental results with the analytical approach provided by EC5 – method C. The comparison of both methodologies was in good agreement hence, the analytical methodology for the timber frame racking design application developed was validated.

Partially restrained shear walls presented lower shear capacity, greater deflection at maximum loads, less energy dissipation and greater localised damage than fully restrained walls. This is of vital significance for maintaining a robust airtight envelop over its service life.

The racking strength for fully restrained RTC closed panels was about 40 % greater than the obtained by the partially restrained panels. Furthermore, the holding down straps in the partially restrained panels outperformed the sole plate connection. This type of failure may also be found in real practice where the timber frame panels are secured to the substrate by holding down straps only.

A relative low strength and stiffness was found on the single sheathed K2 closed panel partially restrained walls. This may be caused by the brittle failure observed on the closely spaced sheathing screw fasteners.

A relative higher strength performance was found for the double sheathed K2 closed panels in comparison with the single sheathed K2 panels. The contribution of the secondary layer had a little positive impact on the global strength of the panel but this secondary sheathing board showed no contribution to the stiffness of the dual K2 frame.

It can be concluded that the characteristic racking resistance provided by PD 6693-1 is conservatively lower than the resultant 0.8 F_{max} from test results. However, the SLS criteria failed in some wall panels mainly due to high aspect ratio wall dimensions and close sheathing screw fastener spacing.

The isolation-combination approach, based on the weakest link theory, was validated in order to provide an analytical methodology for the structural optimisation of the sole plate base fixing detail.

Based on the tear capacity of commercially available tape and construction membranes, a maximum instantaneous racking displacement for a building airtight serviceability criterion of 10 mm is proposed.

6.2.4 Timber racking design software application

An internal survey carried out by Edinburgh Napier University on seventy-seven structural engineers concluded that continuous professional development activities and software solutions are the preferred actions to increase the use of timber in structural practices.

A commercial software application for the design and optimisation of timber frame racking walls based on the analytical PD 6693-1 approach was developed. This tool includes standardise materials, components and sole plate details and allows for the inclusion of streamlined research findings.

After software validation, a parametric analysis with different wall panel configurations and sheathing fastener schedule was carried out. **The optimised racking strength performance, when using smooth nails, was achieved for nails five times longer than the sheathing thickness.**

The development of the software application facilitates the structural optimisation of timber frame walls but also can be used in a future as a mechanism to include other BIM related information.

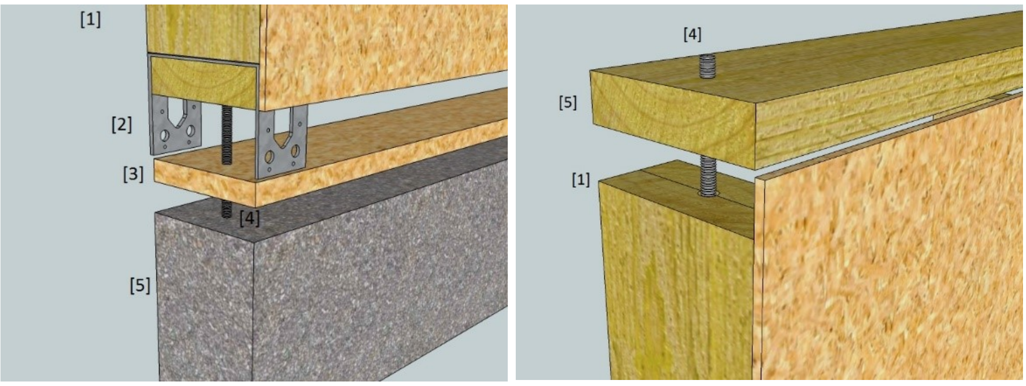
6.3 Recommendations for Future Work

Timber frame closed panel systems for low-energy buildings is a broad research field which offers many areas of study for future work. Although this thesis has met the requirements of the main objectives outlined in section 1.4 and made a contribution to develop potential solutions for low-energy affordable housing, some future directions for additional hygrothermal and structural research related to timber frame systems were provided. Future studies on the topic of this research are recommended on the next sub-sections.

6.3.1 Alternative reinforced closed panel systems

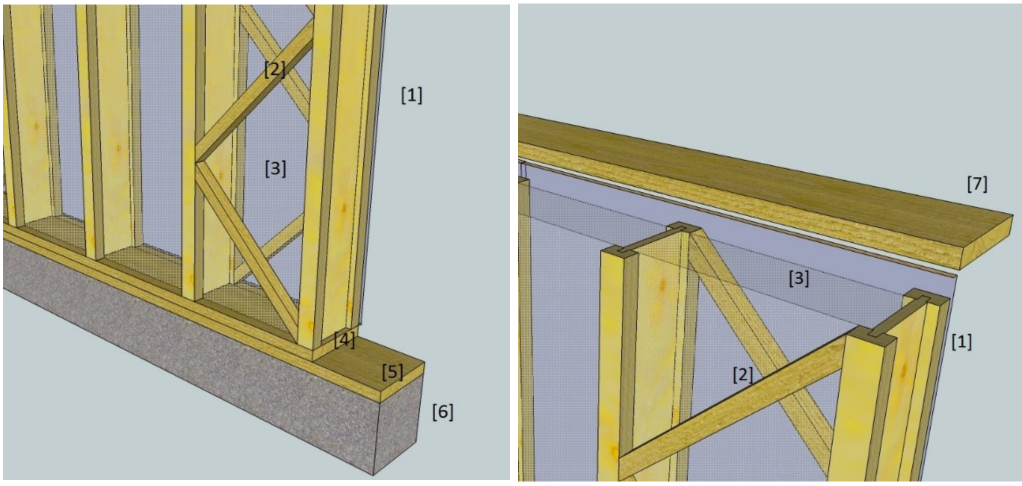
A greater racking stiffness can potentially reduce the deformation and displacements on timber frame shear walls and hence, improve the resilient of highly air-tight, energy efficient buildings. This can be achieved by reinforcing the timber frame system in many different ways.

Two reinforcement techniques were proposed in Figure 6-1 and in Figure 6-2 which could be investigated.



[1] Glued duo stud
[2] Off-site metal cap
[3] High compression strength plate
[4] On-site steel threaded bar
[5] Substrate or foundation
[6] Top rail

Figure 6-1Post-tensioned wall on-site threaded-bar installation



[1] I-Joist 300 mm
[2] Truss web
[3] Insulation (hidden)
[4] Bottom runner
[5] Sole plate
[6] Substrate
[7] Top runner

Figure 6-2 Reinforced truss box stud panel

In the first reinforcement suggested, the innovative solution is particularly focused on the sole plate base fixing details and on the inclusion of post-tensioned steel or FRP bars within the timber studs.

In the second reinforcement suggested, the innovative solution is focused on the strengthening of the leading and trailing studs of the I-joist panels by fabricating a structural truss within the first and last pair of studs. Diagonal timbers are placed only in the flanges of the I-joists hence reducing thermal bridging.

6.3.2 Three-dimensional full-house stiffness investigation

A three-dimensional FEA model could be developed to predict the serviceability and displacement response of a building under full loading scenarios. This investigation could improve the understanding of the global stiffness of the model in relation to a thermal airtightness criterion.

The model may evaluate the full diaphragm action of the building and the contribution to stiffness of non-structural materials such as internal and external finishing. In future investigations, it might be possible to compare the values predicted by this 3-D FEA model with an in-situ displacement monitoring. Other structural issues such as shear torsional moments generated by an asymmetrical racking building stiffness could also be investigated.

6.3.3 Post-occupancy evaluation correlation

Continuous monitoring and data collection for closed panel low-energy buildings may be relevant to investigate potential changes in the performance of the building and to help to mitigate the building "performance gap" problem. Placing long-term displacement gauges on critical places of the building envelope, i.e. sole plate fixing, at corner junctions or at window connections can monitor internal movements. Furthermore, recurring blower door tests may correlate these displacements with air-leakage.

Other post-occupancy evaluation of timber frame buildings could assess the U-value of the building envelope over time in order to correlate design and as-built heat losses through the fabric and throughout the year.

6.3.4 Platform for innovation and BIM

The work carried out in this research delivered the first set of calculations for a whole house engineering platform. This mechanism is a valuable and capable tool of transferring research findings into structural engineering practices. However, the structural design platform created can, through its generic database, also be integrated with additional information where a holistic approach is required including for example cost and other building performance i.e. thermal and acoustic.

As a result, the platform has the potential to provide architects, engineers, quantity surveyors, building planners and the timber industry in general with a reliable valuable information of timber built systems. The long-term aim is for the software to become BIM enabled by the inclusion of relevant information, in a compatible file format; i.e. IFC, to the product library. This could facilitate linking between different applications, i.e. a connection calculation linked directly to the sole plate base fixing definition within the racking application in order to optimise design under certain defined criteria.

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8 APPENDICES

APPENDIX I - TIMBER FRAME WALL FULL STRUCTURAL DESIGN EXAMPLE

APPENDIX II – RTC, K2 AND BENCHMARK PANEL BUILD-UPS AND HYGROTHERMAL ANALYSIS

APPENDIX III – FEA U-VALUE RESULTS

APPENDIX IV – TIMBER FRACTION MANUFACTURING LAYOUTS

APPENDIX V – RTC CLOSED PANEL FULL THERMAL BRIDGE SIMULATION

APPENDIX VI – K2 CLOSED PANEL FULL THERMAL BRIDGE SIMULATION

APPENDIX VII – K2 AND RTC CLOSED PANEL WUFI REPORTS

APPENDIX VIII – PREVIOUS EXPERIMENTS ON TIMBER FRAME RACKING WALLS CARRIED OUT AT EDINBURGH NAPIER UNIVERSITY

APPENDIX IX – ISOLATION-COMBINATION SOLE PLATE TESTING DETAILS

APPENDIX X – TIMBER FRAME QUALITY ASSURANCE PROCEDURES

APPENDIX XI – TIMBER RACKING WALL DESIGN MANUAL

APPENDIX XII – INTEGRATED SET OF SOLUTIONS: SIMPLIFIED DETAILS

I. Timber frame wall full structural design example

Design of RTC Wall Panel

The impermeable damp-proof course (dpc) must be positioned according to the building regulations. For durability purposes, it is recommended to erect the PassiveWall™ panels 250 mm above finished ground or paving level. Nevertheless, for structural purposes, the wall design does not consider this diminution of effective length.

Height of the ground wall		h_{n1} = 2500 mm
Elevation above the finished ground		l_{dpc} = 250 mm
Thickness of the PassivePlate		t_{plate} = 15 mm
Height of the joist binder		h_{binder} = 47 mm
Depth of the floor		$floor_{depth}$ = 318 mm

Selection of studs:	**300 x 47 I_beam**
Selection of binders:	**300 x 47 I_beam**

Stud properties:

Stud length		L_{stud} = 1812 mm
Effective length about y-y axis	$L_{stud.ef.y}$ = 0.85	$L_{stud.y}$ = 1540 mm
Effective length factor about z-z axis	$L_{stud.ef.z}$ = 0.50	$L_{stud.z}$ = 906 mm
Breadth of stud		b_{stud} = 47 mm
Depths of the stud	H_{stud} = 300 mm	h_{stud} = 45 mm
Bearing depth of the stud		$h_{bearing.stud}$ = 90 mm
Lateral spacing of studs		s_w = 600 mm
Area of stud		$A_{stud.strength}$ = 4230 mm^2
Section modulus about y-y axis for stud		$W_{stud.y}$ = 463185 mm^3
Section modulus about z-z axis for stud		$W_{stud.z}$ = 33135 mm^3
Slenderness ratio about y-y axis		$\lambda_{stud.y}$ = 12.02
Slenderness ratio about z-z axis		$\lambda_{stud.z}$ = 66.78

Binder properties:

Width of the binder beam	b_{binder} = 90 mm
Bearing area on binder	$A_{bearing.binder}$ = 4230 mm^2
Effective bearing area on binder - plate	$A_{bearing.effective.plate}$ =6930 mm^2
(Clause 6.1.5(1) of BS EN 1995-1-1:2004 (EC5) apply)	
Length of internal inter-stud bay	$L_{binder.z}$ = 553 mm
Section modulus about y-y axis for binder	$W_{binder.y}$ = 33135 mm^3
Section modulus about z-z axis for binder	$W_{binder.z}$ = 63450 mm^3

Timber strength properties:

Timber properties are according to the manufacturer. In case of not availability of data, refer to BS EN 338:2009.

Stud

Characteristic compression strength parallel to the grain	$f_{stud.c.0.k}$ = 21.0 N/mm^2
Characteristic bending strength	$f_{stud.m.y.k}$ = 24.0 N/mm^2
Charactersitic bearing strength perpendicular to the grain	$f_{stud.c.90.k}$ = 2.5 N/mm^2
5%tile MOE parallel to the grain	$E_{stud.0.05}$ = 7.4 kN/mm^2
5%tile MOE parallel to the grain as composite	$E_{stud.comp.0.05}$ = 7.4 kN/mm^2

Binder beam

Characteristic compression strength parallel to the grain	$f_{binder.c.0.k}$ = **21.0** N/mm^2
Characteristic bending strength	$f_{binder.m.y.k}$ = **24.0** N/mm^2
Charactersitic bearing strength perpendicular to the grain	$f_{binder.c.90.k}$ = **2.5** N/mm^2
Characteristic shear strength	$f_{binder.v.k}$ = **4.0** N/mm^2
5%tile MOE parallel to the grain	$E_{binder.0.05}$ = **7.4** kN/mm^2

<u>Actions on the wall:</u>

Vertical loading - Combination 1	$N1w_d$ = **6.52** kN
Vertical loading - Combination 2	$N2w_d$ = **12.09** kN
Vertical loading - Combination 3	$N3w_d$ = **7.20** kN
Vertical loading - Combination 4	$N4w_d$ = **9.73** kN
Vertical loading - Combination 5	$N5w_d$ = **13.19** kN
Vertical loading - Combination 6	$N6w_d$ = **12.11** kN
Vertical loading - Combination 7	$N7w_d$ = **13.37** kN
Lateral bending loading - Combination 1	$M1w_d$ = **1.32** kNm
Lateral bending loading - Combination 2	$M2w_d$ = **0.44** kNm

<u>Modification factors:</u>

(Recommended partial factors γ_M from Table 2.3 of BS EN 1995-1-1:2004)

Material factor for timber in stud	$\gamma_{stud.M}$ = 1.3
Material factor for timber in binder beam	$\gamma_{binder.M}$ = 1.3

According to Table 3.1of BS EN 1995-1-1:2004, for a service class 2 where the maximum equilibrium moisture content for the timbers must be inferior to 20%, the k_{mod} values for different loading duration are:

$k_{mod.perm.SC2}$ = 0.60 $k_{mod.long.SC2}$ = 0.70 $k_{mod.med.SC2}$ = 0.80 $k_{mod.short.SC2}$ = 0.90 $k_{mod.inst.SC2}$ = 1.10

The wall assembly is considered to be composed of regularly equally spaced studs connected by a continuous load distribution system capable to transfer the load from one stud to the neighbouring members. The binder is acting as two single members (both flanges) and therefore, a unity k_{sys} factor applies (Clause 6.6 of BS EN 1995-1-1:2004).

System strength for studs	$k_{stud.sys}$ = **1.1**
System strength for binder	$k_{binder.sys}$ = **1.0**

For the bearing factor $k_{c.90}$, compression perpendicular to the binder beam direction, the clause 6.1.5(3) of BS EN 1995-1-1:2004 applies at any wall configuration where the spacing of the studs is greater than twice the height of the binder.

Bearing factor	$k_{binder.c.90}$ = **1.25**

The structural performance of the wall for bending and tension considers, for binders and studs components, two independently rectangular solid timber sections (C24 flanges) joined by OSB (web). Therefore, the depth factor from the clause 3.2(3) of BS EN 1995-1-1:2004 applies to the design. In case of the flange of either the stud or the binder is made of finger jointed timber, the component must comply with EN 385.

Depth factor for binder	$k_{binder.h}$ = **1.27**
Depth factor for stud	$k_{stud.h}$ = **1.27**

Although the end of the studs are considered to be restrained against rotation and the compression edge line is assumed to be hold in line by the direct connection of the OSB sheathing, the most favourable case scenario provides a depth-to-breath ratio of almost 5:1 where lateral torsional buckling can not be longer neglected (k_{crit} = 1). In line with the lateral stability performance theory, the I shape of the stud, functioning in this case as a beam, influences the critical bending stress ($\sigma_{m.crit}$) and therefore, the relative slenderness ratio for bending ($\lambda_{rel.m}$).

BS EN 1995-1-1:2004 (EC5) does not give an expression to calculate the critical bending stress of non rectangular sections.The theoretical method to determine the critical bending stress for I studs follows the Timoschenko's theory of elastic stability[1]. Value of γ_4 is given by Figure a.

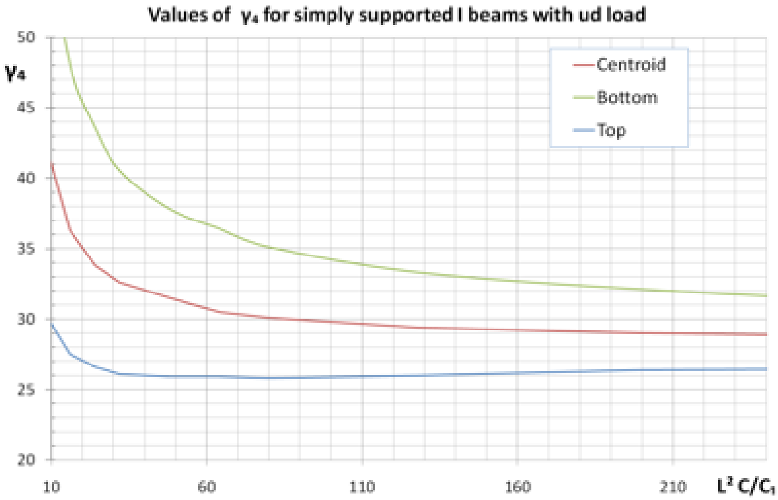

Figure a

Factor L^2 C/C$_1$ = 8.6	$\gamma_{torsion.4}$ = 30.0
Moment of Inertia in normal buckling plane	$I_{stud.n.crit}$ = 778672 mm^4
Moment of Inertia in vertical bucling plane	$I_{stud.e.crit}$ = 75846375 mm^4
Critical bending stress	;$\sigma_{stud.m.crit}$ = 22.87 N/mm^2
Relative slenderness for bending (EC5, Equation 6.30)	$\lambda_{stud.rel.m}$ = 1.02
Factor for lateral buckling (EC5, Equation 6.34)	$k_{stud.crit}$ = 0.79

<u>Analysis of the stud.</u>

(a) Compression strength of the stud.

Axial stress conditions only:

Axial compression combination 1, permanent duration	$\sigma 1_{stud.c.0.d}$ = 1.54 N/mm^2
Axial compression combination 2, medium-term duration	$\sigma 2_{stud.c.0.d}$ = 2.86 N/mm^2
Axial compression combination 3, short-term duration	$\sigma 3_{stud.c.0.d}$ = 1.70 N/mm^2
Axial compression combination 4, instantaneous duration	$\sigma 4_{stud.c.0.d}$ = 2.30 N/mm^2

1. **Timoshenko, Stephen P. and Gere, James M.** *Theory of Elastic Stability - 2nd Edition.* New York : McGraw-Hill Book, 1961.

Axial compression combination 5, instantaneous duration	$\sigma5_{stud.c.0.d}$ = **3.12** N/mm^2
Axial compression combination 6, instantaneous duration	$\sigma6_{stud.c.0.d}$ = **2.86** N/mm^2
Axial compression combination 7, instantaneous duration	$\sigma7_{stud.c.0.d}$ = **3.16** N/mm^2

Design compression strength of stud for combination 1	$f1_{stud.c.0.d}$ = **10.66** N/mm^2
Design compression strength of stud for combination 2	$f2_{stud.c.0.d}$ = **14.22** N/mm^2
Design compression strength of stud for combination 3	$f3_{stud.c.0.d}$ = **15.99** N/mm^2
Design compression strength of stud for combination 4	$f4_{stud.c.0.d}$ = **17.77** N/mm^2
Design compression strength of stud for combination 5	$f5_{stud.c.0.d}$ = **17.77** N/mm^2
Design compression strength of stud for combination 6	$f6_{stud.c.0.d}$ = **17.77** N/mm^2
Design compression strength of stud for combination 7	$f7_{stud.c.0.d}$ = **17.77** N/mm^2

NOTE: According to Clause 6.6(1)(2) of BS EN 1995-1-1:2004, k_{sys} is considered to be 1.1 for all term duration actions but for instantaneous duration where k_{sys} is taken as unity (EC5, Clause 6.6(3)).

(b) Buckling resistance of the stud

(EC5, Clause 6.3.2)

Relative slenderness about y-y axis,
(EC5, Equation 6.21) $\lambda_{stud.rel.y}$ = **0.20**

Relative slenderness about z-z axis,
(EC5, Equation 6.22) $\lambda_{stud.rel.z}$ = **1.13**

As $\lambda_{stud.rel.z}$ is greater than 0.3, condition 6.3.2(3) of BS EN 1995-1-1:2004 applies. According to clause 6.1.6(2), the value of k_m is taken as 1.0. The value of $\beta_{stud.c}$ for Engineered I-beams is not defined in equation 6.29 of BS EN 1995-1-1:2004. The value adopted by the engineer is **0.15**

Re-distribution of stresses factor	$k_{stud.m}$ = **1.0**	$\beta_{stud.c}$ = **0.15**
Instability factor	k_y = **0.51**	k_z = **1.20**
Instability factor about the y and x axis	$k_{stud.c.y}$ = **1.02**	$k_{stud.c.z}$ = **0.62**

(c) Bending strength of the stud.
Bending stress conditions about y-y axis:

Bending stress combination 1, instantaneous duration	$\sigma1_{stud.m.y.d}$ = **2.85** N/mm^2
Bending stress combination 2, instantaneous duration	$\sigma2_{stud.m.y.d}$ = **0.95** N/mm^2
Design bendind strength about y-y axis for instantaneous duration	$f1_{stud.m.y.d}$ = **22.50** N/mm^2

(d) Stress conditions for the studs.
Axial stress and buckling about the y-y axis (EC5 Clause 6.3.2(3):

Combination 1:	$\sigma1_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f1_{stud.c.0.d}$) = **0.14**	PASS
Combination 2:	$\sigma2_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f2_{stud.c.0.d}$) = **0.20**	PASS
Combination 3:	$\sigma3_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f3_{stud.c.0.d}$) = **0.10**	PASS
Combination 4:	$\sigma4_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f4_{stud.c.0.d}$) = **0.13**	PASS
Combination 5:	$\sigma5_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f5_{stud.c.0.d}$) = **0.17**	PASS
Combination 6:	$\sigma6_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f6_{stud.c.0.d}$) = **0.16**	PASS
Combination 7:	$\sigma7_{stud.c.0.d}$ / ($k_{stud.c.y}$ × $f7_{stud.c.0.d}$) = **0.18**	PASS

Combined axial permanent and variable actions and lateral wind only (EC5, Clauses 6.3.2(3) and 6.3.3(6)):

Axial + bending in y-y axis acting as a column	$stud_{lateral.combination.1}$ = **0.21**	PASS
Compression stress about the z-z axis	$stud_{lateral.combination.2}$ = **0.27**	PASS
Bending and axial stress as a beam	$stud_{lateral.combination.3}$ = **0.17**	PASS

Combined axial permanent and combined variable actions, with imposed load acting as leading variable and lateral wind acting as accompanying variable (EC5, Clauses 6.3.2(3) and 6.3.3(6)):

Axial+bending in y-y axis acting as a column	$stud_{lateral.combination.4}$ = **0.22**	PASS
Compression stress about the z-z axis	$stud_{lateral.combination.5}$ = **0.32**	PASS
Bending and axial stress as a beam	$stud_{lateral.combination.6}$ = **0.29**	PASS

Combined axial permanent and combined variable actions, with imposed load acting as leading variable and lateral wind acting as leading variable (EC5, Clauses 6.3.2(3) and 6.3.3(6)):

Axia l+ bending int y-y axis acting as a column	$stud_{lateral.combination.7}$ = **0.30**	PASS
Compression stress about the z-z axis	$stud_{lateral.combination.8}$ = **0.41**	PASS
Bending and axial stress as a beam	$stud_{lateral.combination.9}$ = **0.31**	PASS

The critical condition is due to combined axial stress and buckling about the z-z axis and functioning as a beam with the lateral wind acting as the leading variable. The I-studs are OK as the combined compression and bending ratios are less than unity.

In case of buckling, this would occur about the axis with the highest slenderness ratio. The high depth to breadth ratio for I-beams defines the z-z axis as the weakest axis. The connection of the studs to the both sheathing panels must be fixed according to Section 10.8.1.2 of the Manual for the Design of Timber Building Structures to Eurocode 5[2], where buckling in the y-y axis is ignored.

<u>Analysis of the head-sole binder plates.</u>

(a) Bearing strength of the binder plates

For this analysis, the greatest stress to strength ratio arises from the combination 5 scenario : permanent action, imposed load acting as leading variable and snow and wind acting as accompanying variables.

Design bearing load per stud		$N1b_d$ = **13.37** kN
Design bearing stress on plate		$\sigma1_{binder.c.90.d}$ = **1.93** N/mm^2
Design bearing strength		$f1_{binder.c.90.d}$ = **2.12** N/mm^2
Bearing strength check	$\sigma1_{binder.c.90.d}$ / $f1_{binder.c.90.d}$ = **0.91**	PASS $k_{c,90}$ value of 1.25, although permitted in EC5, is not needed.

[2] **IStructE.** *Manual for the design of timber building structures to Eurocode 5.* London : The Institution of Structural Engineers, 2007.

(b) Bending strength of binder plate acting as a beam.

For this analysis, the greatest bending stress arises from the load taken by a stud acting over the mid span of a bay in a continuous head-sole plate. A three-span continuous beam with a pointed load at both edge spans scenario is considered due to the unfavourable bending condition (Figure b shows in green the greatest deflection).

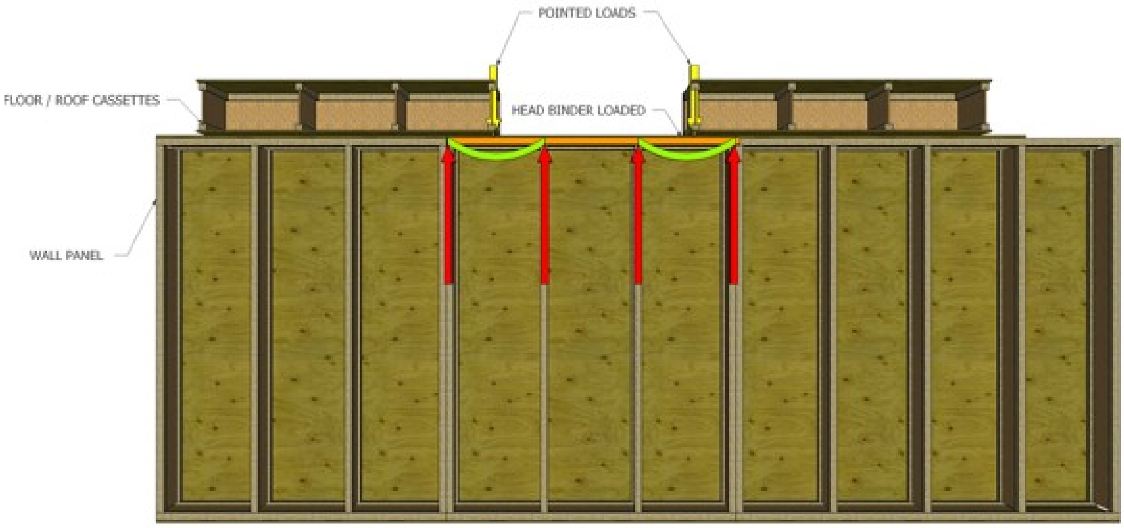

Figure b

Coeficient for bending moment	$k_{binder.bending}$ = 0.213
Depth factor	$k_{binder.h}$ = 1.27
System strength factor	$k_{binder.sys}$ = 1.00
Critical bending stress	$\sigma_{binder.m.crit}$ 1798.82 N/mm^2
Relative slenderness for bending	$\lambda_{binder.rel.m}$ = 0.12
Lateral stability factor	$k_{binder.crit}$ = 1.00
Design bending strength on plate	$f_{binder.y.m.d}$ =25.84 N/mm^2
Bending stress for worst case condition	$\sigma_{binder.m.y.d}$ = 47.53 N/mm^2

Bending strength check $\sigma_{binder.m.y.d} / f_{binder.y.m.d}$ = 1.84

Intermediate studs for pointed loads are needed.

Maximum pointed load allowable for bending $Nw_{m.max}$ =7.27 kN

(c) Shear strength of binder plate acting as a beam.

For this analysis, the greatest shear stress arises from the load taken by two pointed loads acting at mid span of two bays. A three-span continuous beam with a pointed load on a edge bay and a second equal pointed load on the intermediate bay is considered as the worst case scenario (Figure c shows in green the greatest shear stress).

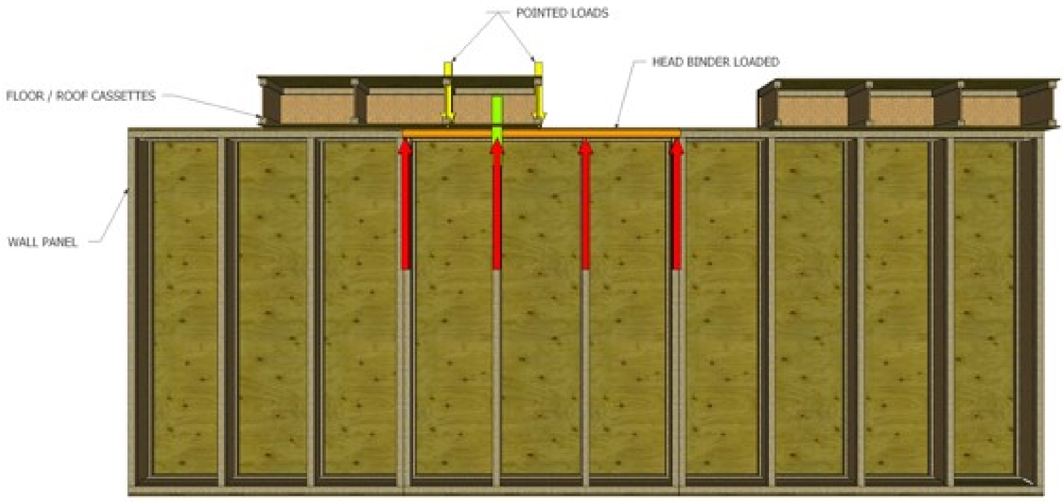

Figure c

Coeficient for vertical reaction	$k_{binder.shear}$ = **1.3**
Crack factor for shear resistance (EC5, Clause 6.1.7(2))	$k_{binder.cr}$ = **0.67**
System strength factor	$k_{binder.sys}$ = **1.00**
Height of notch (if applicable)	$h_{binder.notch}$ = **0** mm
Effective height of binder	$h_{binder.effective}$ = **47** mm
Design shear strength on plate	$f_{binder.v.d}$ = **2.06** N/mm^2
Shear stress for worst case condition	$\sigma_{binder.v.d}$ = **6.16** N/mm^2

Shear strength check $\sigma_{binder.v.d}$ / $f_{binder.v.d}$ = **2.99**

Intermediate studs for pointed loads are needed.

Maximum pointed load allowable for shear $Nw_{v.max}$ = **4.47** kN

Therefore, the maximum pointed load not aligned with the studs line of the inmediate contiguous wall, with a tolerance of 150 mm in both sides, for any load scenario is **4.47** kN.

<u>Lateral deflection of the wall stud.</u>

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Lateral deflection of the wall stud.

According to the manufacturer, the initial I stud deviation due to the out of straightness displacement of the member is considered as XXXX. In addition, as opposed to strength calculations, deflection is determined by considering the I stud as a single engineered wood component, not as the sum of two C24 studs.

Initial out of straightness of the stud $\delta_{stud.initial}$ = **1.81** mm

Characteristic permanent action $G_{k.wall}$ = **4.83** kN
Characteristic imposed variable medium-term $Q_{k.imp1.wall}$ = **3.71** kN
Characteristic imposed variable (if snow-wind) medium-term $Q_{k.imp2.wall}$ = **3.15** kNCharacteristic
snow variable short-term $Q_{k.snow.wall}$ = **0.45** kN
Characteristic axial wind variable (instantaneous) $Q_{k.wind.a.wall}$ = **2.14** kN
Characteristic lateral wind variable (instantaneous) $Q_{k.wind.l.wall}$ = **1.13** kN/m

Combination factors γ_{Gsls} = 1.00 γ_{Gsls} = 1.00
$\psi_{0.AH}$ = 0.70 $\psi_{0.w.sn}$ = 0.50

Design load combination only $G_{k.wall}$ $N1w_{d.sls}$ = **4.83** kN
Characteristic combination $G_{k.wall} + Q_{k.imp.wall}$ $N2w_{d.sls}$ = **8.54** kN
Design load combination $G_{k.wall} + Q_{k.snow.wall}$ $N3w_{d.sls}$ = **5.28** kN
Design load combination $G_{k.wall} + Q_{k.wind.a.wall}$ $N4w_{d.sls}$ = **6.97** kN
Design load combination permanent action, imposed acting
as leading variable and snow and wind as companion variables $N5w_{d.sls}$ = **9.84** kN
Design load combination permanent action, snow acting
as leading variable and imposed and wind as companion variables $N6w_{d.sls}$ = **8.95** kN
Design load combination permanent action, wind acting
as leading variable and imposed and snow as companion variables $N7w_{d.sls}$ = **9.79** kN

Design axial load $Nw_{d.sls}$ = **9.84** kN

Design load combination for lateral wind loading with wind
as dominant action $q_{wind.stud}$ = **1.13** kN/m

Euler load for I studs about y-y axis. $P_{E.stud} = (\pi^2 \times E_{stud.comp.0.05} \times I_{stud.y} /$
$L_{stud.y}^2)$ = **2139.06** kN

Amplification factor check to assure that classical elastic instability theory applies: $Nw_{d.sls}/ P_{E.stud}$ = **0.00** OK
(This approximation is accurate to within 2% for N / PE values less than 0.60)[3]

Lateral displacement due to wind
(shear deflection taken as 15% of bending deflection according to UK NA to EC5) $w_{inst.q.wind}$ = **0.35** mm

Maximum out of straightness of the stud $\delta_{stud.max} = L_{stud}/500$ = **3.62 mm**

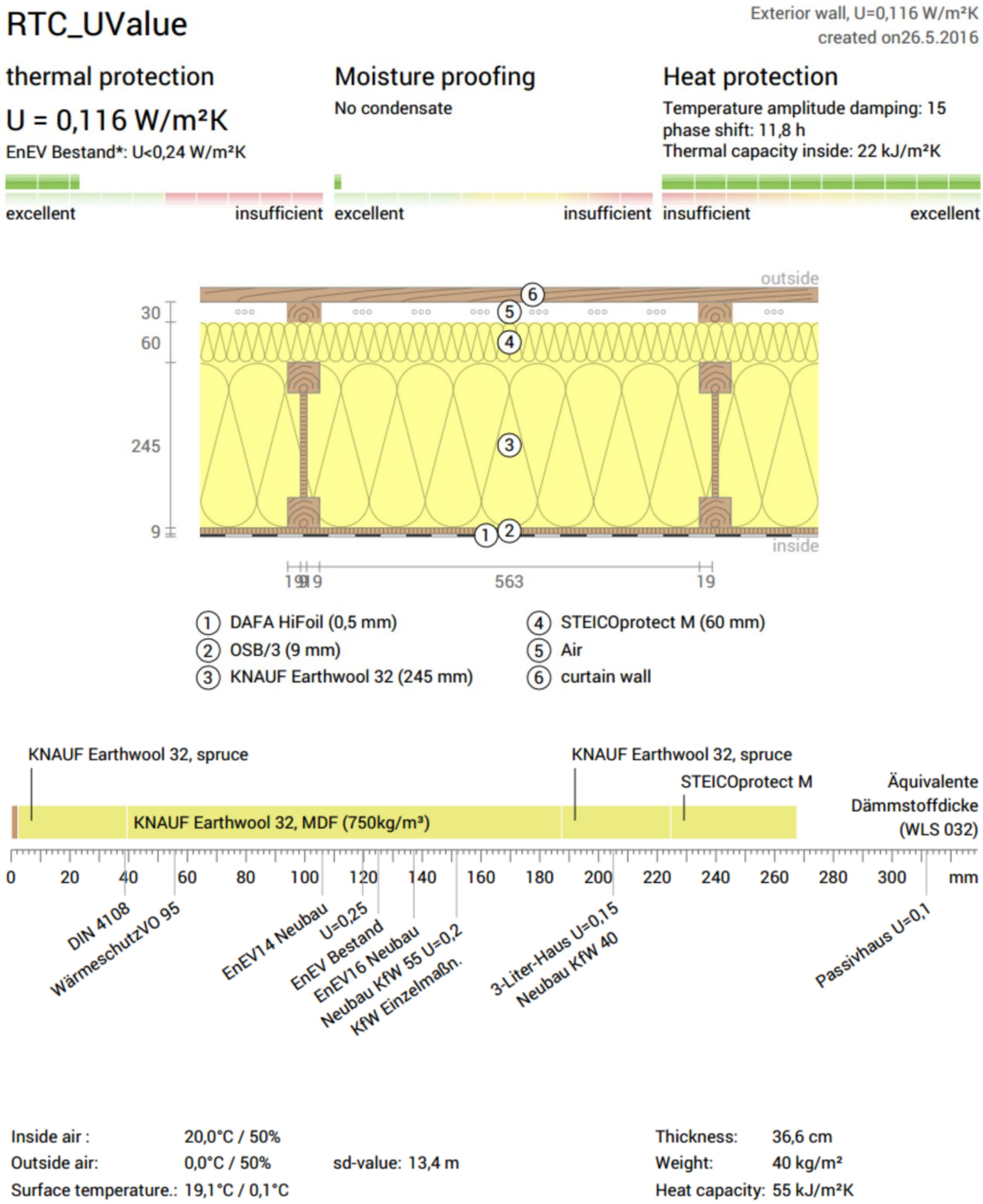
(Clause 10.2 of BS EN 1995-1-1:2004)

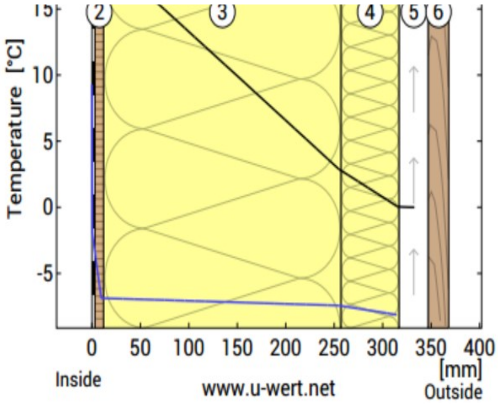
Increase in lateral deflection of the wall at SLS under the
design combination loading $w_{d.inst.wind}$ = **0.37** mm

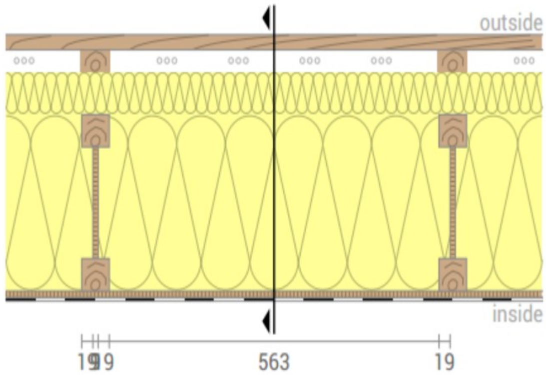
Lateral deflection of I stud $w_{stud.final}$ = **2.19** mm **PASS**

[3] **Porteous J. and Kermani A.** *Structural timber design to Eurocode 5.* Oxford : Wiley-Blackwell, 2007.

II. RTC, K2 and Benchmark panel build-ups and hygrothermal analysis.







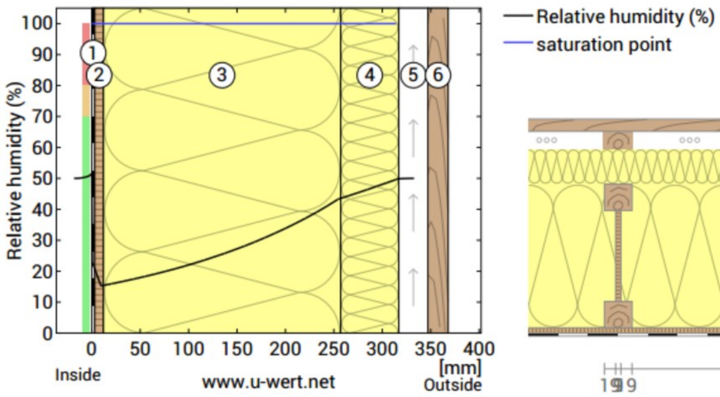
① DAFA HiFoil (0,5 mm) ③ KNAUF Earthwool 32 (245 mm) ⑤ Air
② OSB/3 (9 mm) ④ STEICOprotect M (60 mm) ⑥ curtain wall

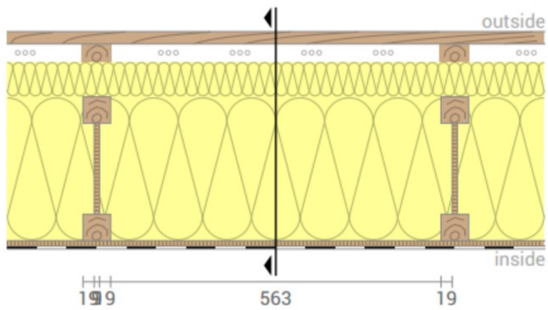
			[W/mK]	[m²K/W]	min	max	[kg/m²]
		Thermal contact resistance*		0,130	19,1	20,0	
1	0,05 cm	DAFA HiFoil	0,220	0,002	19,1	19,5	0,1
2	0,9 cm	OSB/3	0,130	0,069	18,8	19,5	5,6
3	24,5 cm	KNAUF Earthwool 32	0,032	7,656	2,9	19,3	7,1
	4,5 cm	spruce (Width: 4.7 cm)	0,130	0,346	4,0	5,9	1,6
	15,5 cm	MDF (750kg/m³) (Width: 0.9 cm)	0,180	0,861	5,9	16,9	1,7
	4,5 cm	spruce (Width: 4.7 cm)	0,130	0,346	16,9	18,9	1,6
4	6 cm	STEICOprotect M	0,046	1,304	0,1	4,2	13,8
		Thermal contact resistance*		0,130	0,0	0,1	
5		Air (ventilated layer)			0,0	0,0	0,0

#		Material	sd-value [m]	Condensate [kg/m²]	[Gew.-%]	Weight [kg/m²]
1	0,05 cm	DAFA HiFoil	10,00	-		0,1
2	0,9 cm	OSB/3	2,70	-	-	5,6
3	24,5 cm	KNAUF Earthwool 32	0,25	-		7,1
	4,5 cm	spruce (Width: 4.7 cm)	2,25	-	-	1,6
	15,5 cm	MDF (750kg/m³) (Width: 0.9 cm)	7,75	-	-	1,7
	4,5 cm	spruce (Width: 4.7 cm)	0,90	-	-	1,6
4	6 cm	STEICOprotect M	0,30	-		13,8
	36,55 cm	Whole component	13,41			40,9

Humidity

Die Oberflächentemperatur der Wandinnenseite beträgt 19,1 °C was zu einer relativen Luftfeuchtigkeit an der Oberfläche von 53% führt. Unter diesen Bedingungen sollte nicht mit Schimmelbildung zu rechnen sein.
Das folgende Diagramm zeigt die relative Luftfeuchtigkeit innerhalb des Bauteils.





① DAFA HiFoil (0,5 mm) ③ KNAUF Earthwool 32 (245 mm) ⑤ Air
② OSB/3 (9 mm) ④ STEICOprotect M (60 mm) ⑥ curtain wall

K2_Uvalue

Exterior wall, U=0,134 W/m²K
created on26.5.2016

thermal protection

U = 0,134 W/m²K

EnEV Bestand*: U<0,24 W/m²K

excellent insufficient

Moisture proofing

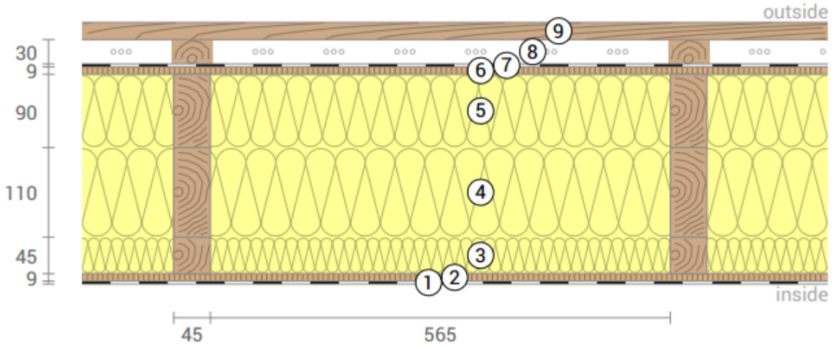
No condensate

excellent insufficient

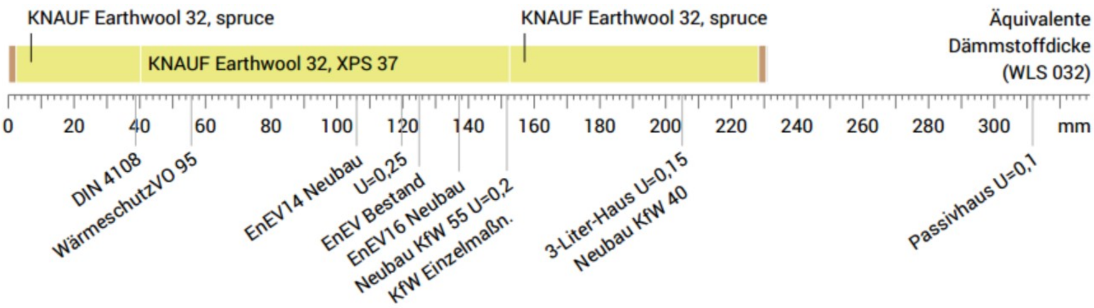
Heat protection

Temperature amplitude damping: 8,4
phase shift: 7,7 h
Thermal capacity inside: 17,8 kJ/m²K

insufficient excellent



1. DAFA HiFoil (0,5 mm)
2. OSB/3 (9 mm)
3. KNAUF Earthwool 32 (45 mm)
4. KNAUF Earthwool 32 (110 mm)
5. KNAUF Earthwool 32 (90 mm)
6. OSB/3 (9 mm)
7. DAFA WindFoil (0,72 mm)
8. Air
9. curtain wall



Inside air : 20,0°C / 50%
Outside air: 0,0°C / 50%
Surface temperature.: 19,0°C / 0,1°C

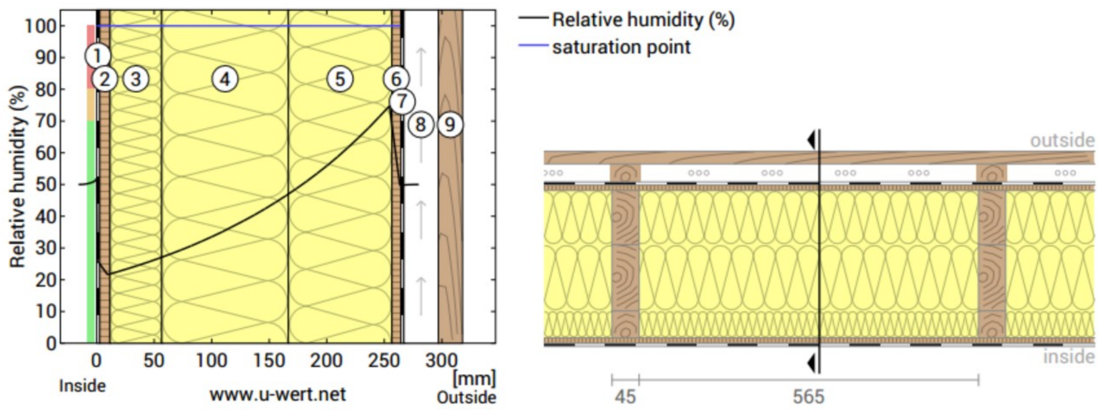
sd-value: 15,2 m

Thickness: 31,5 cm
Weight: 32 kg/m²
Heat capacity: 37 kJ/m²K

#		Material	λ [W/mK]	R [m²K/W]	Temperatur [°C] min	max	Weight [kg/m²]
		Thermal contact resistance*		0,130	19,0	20,0	
1	0,05 cm	DAFA HiFoil	0,220	0,002	19,0	19,4	0,1
2	0,9 cm	OSB/3	0,130	0,069	18,7	19,4	5,6
3	4,5 cm	KNAUF Earthwool 32	0,032	1,406	15,7	19,2	1,3
	4,5 cm	spruce (7,4%)	0,130	0,346	16,7	18,8	1,5
4	11 cm	KNAUF Earthwool 32	0,032	3,438	5,3	16,7	3,1
	11 cm	XPS 37 (7,4%)	0,037	2,973	5,1	16,9	0,3
5	9 cm	KNAUF Earthwool 32	0,032	2,813	0,3	7,2	2,5
	9 cm	spruce (7,4%)	0,130	0,692	0,5	5,3	3,0
6	0,9 cm	OSB/3	0,130	0,069	0,1	0,7	5,6
7	0,072 cm	DAFA WindFoil	0,220	0,003	0,1	0,2	0,2
		Thermal contact resistance*		0,130	0,0	0,2	

Humidity

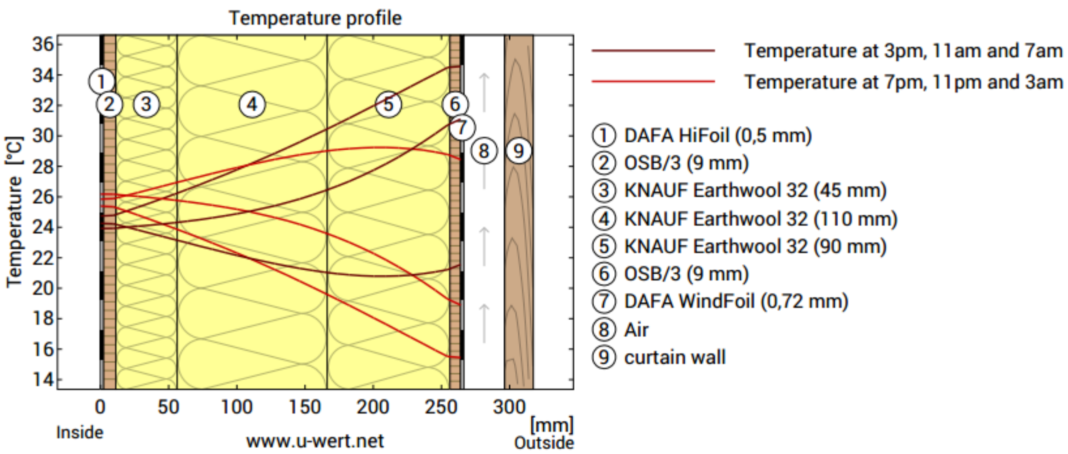
Die Oberflächentemperatur der Wandinnenseite beträgt 19,0 °C was zu einer relativen Luftfeuchtigkeit an der Oberfläche von 53% führt. Unter diesen Bedingungen sollte nicht mit Schimmelbildung zu rechnen sein.
Das folgende Diagramm zeigt die relative Luftfeuchtigkeit innerhalb des Bauteils.

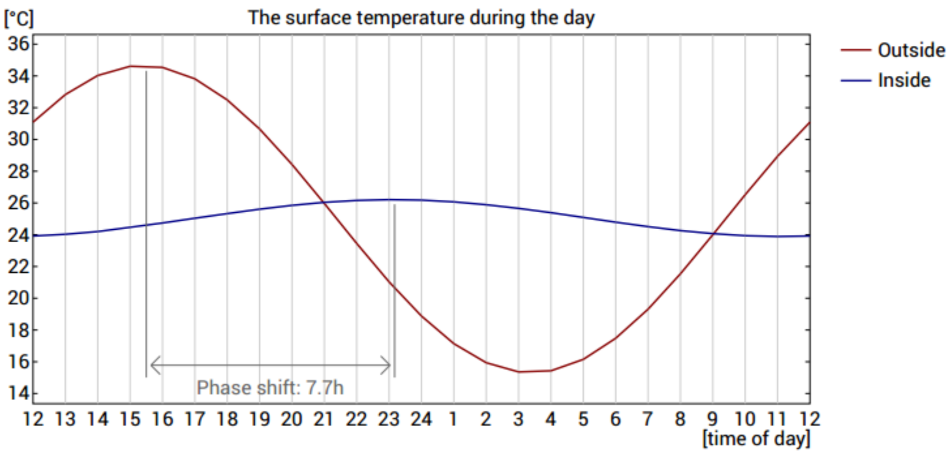


① DAFA HiFoil (0,5 mm)
② OSB/3 (9 mm)
③ KNAUF Earthwool 32 (45 mm)
④ KNAUF Earthwool 32 (110 mm)
⑤ KNAUF Earthwool 32 (90 mm)
⑥ OSB/3 (9 mm)
⑦ DAFA WindFoil (0,72 mm)
⑧ Air
⑨ curtain wall

Heat protection

Für die Analyse des sommerlichen Hitzeschutzes wurden die Temperaturänderungen innerhalb des Bauteils im Verlauf eines heißen Sommertages simuliert:





Benchmark

Exterior wall, U=0,280 W/m²K
created on26.5.2016

thermal protection

U = 0,28 W/m²K

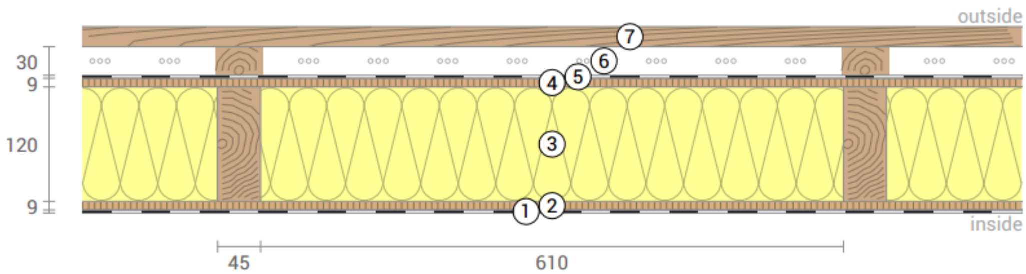
EnEV Bestand*: U<0,24 W/m²K

Moisture proofing

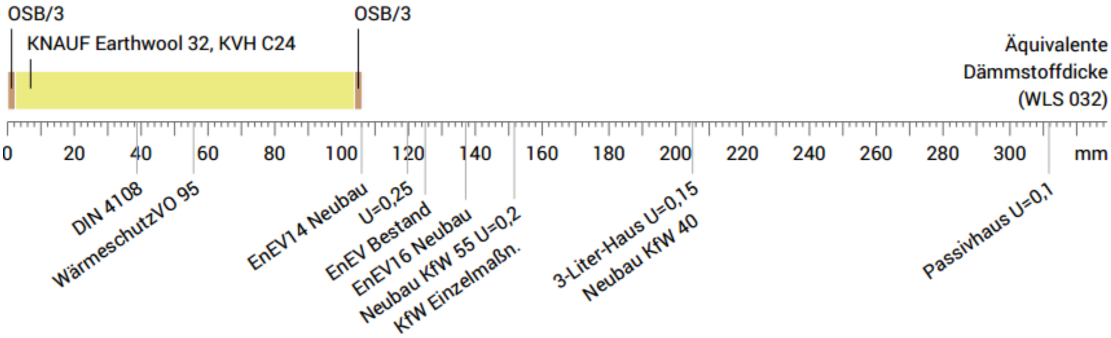
No condensate

Heat protection

Temperature amplitude damping: 3,8
phase shift: 5,6 h
Thermal capacity inside: 15,6 kJ/m²K



1. DAFA HiFoil (0,5 mm)
2. OSB/3 (9 mm)
3. KNAUF Earthwool 32 (120 mm)
4. OSB/3 (9 mm)
5. Breather membrane sd=0,05m (0,5 mm)
6. Air
7. curtain wall

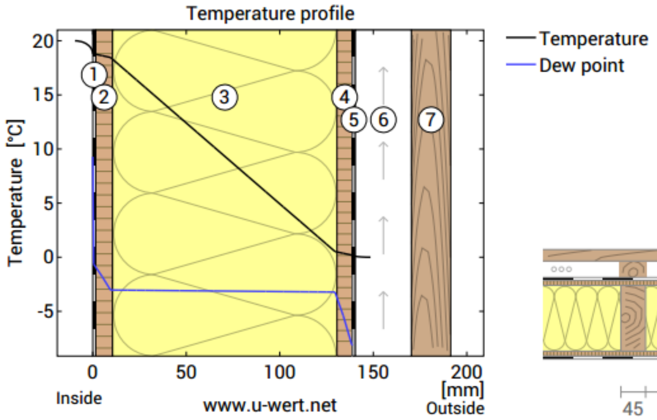


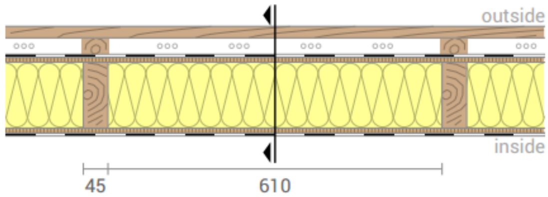
Inside air :	20,0°C / 50%	Thickness:	19,0 cm
Outside air:	0,0°C / 50% sd-value: 14,7 m	Weight:	28 kg/m²
Surface temperature.:	17,3°C / 0,2°C	Heat capacity:	31 kJ/m²K

Layers (from inside to outside)

#		Material	λ [W/mK]	R [m²K/W]	Temperatur [°C] min	max	Weight [kg/m²]
		Thermal contact resistance*		0,130	17,3	20,0	
1	0,05 cm	DAFA HiFoil	0,220	0,002	17,3	18,8	0,1
2	0,9 cm	OSB/3	0,130	0,069	16,4	18,8	5,6
3	12 cm	KNAUF Earthwool 32	0,032	3,750	0,5	18,5	3,4
	12 cm	KVH C24 (6,9%)	0,130	0,923	1,2	16,9	3,7
4	0,9 cm	OSB/3	0,130	0,069	0,2	1,5	5,9
5	0,05 cm	Breather membrane sd=0,05m	0,500	0,001	0,2	0,5	0,4
		Thermal contact resistance*		0,130	0,0	0,5	
6		Air (ventilated layer)			0,0	0,0	0,0
7		curtain wall			0,0	0,0	9,5
	19 cm	Whole component		3,577			28,4

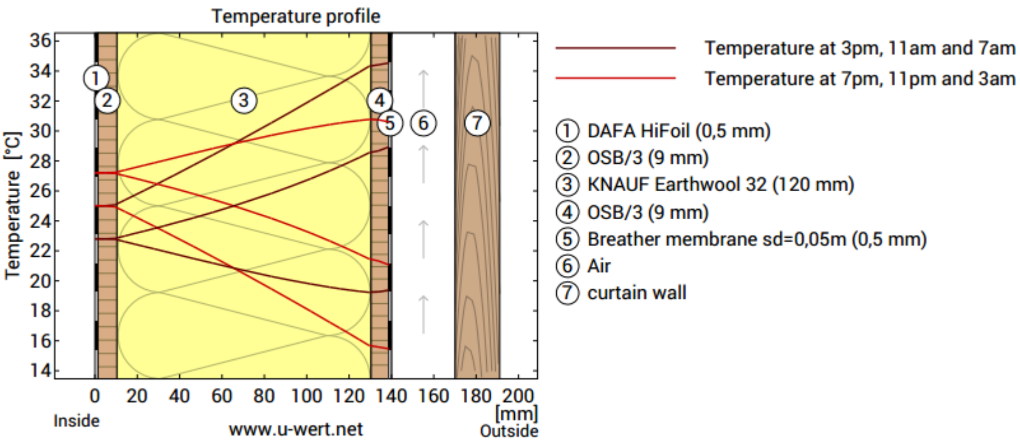
Temperature profile

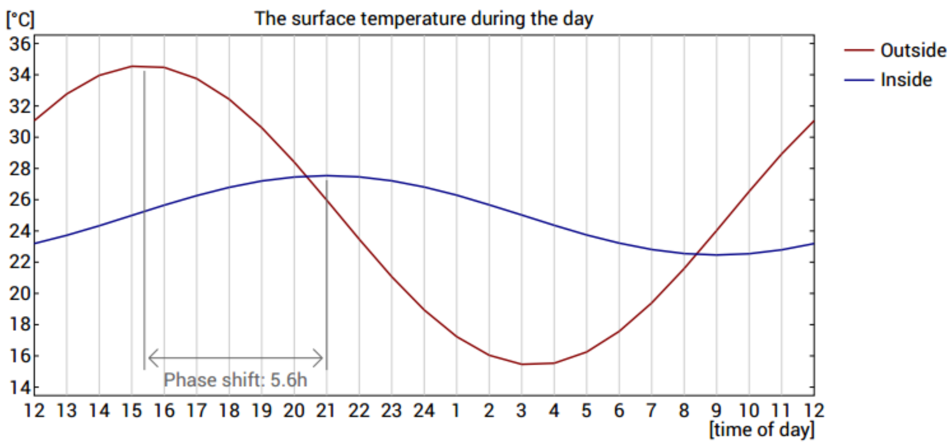




① DAFA HiFoil (0,5 mm)
② OSB/3 (9 mm)
③ KNAUF Earthwool 32 (120 mm)
④ OSB/3 (9 mm)
⑤ Breather membrane sd=0,05m (0,5 mm)
⑥ Air
⑦ curtain wall

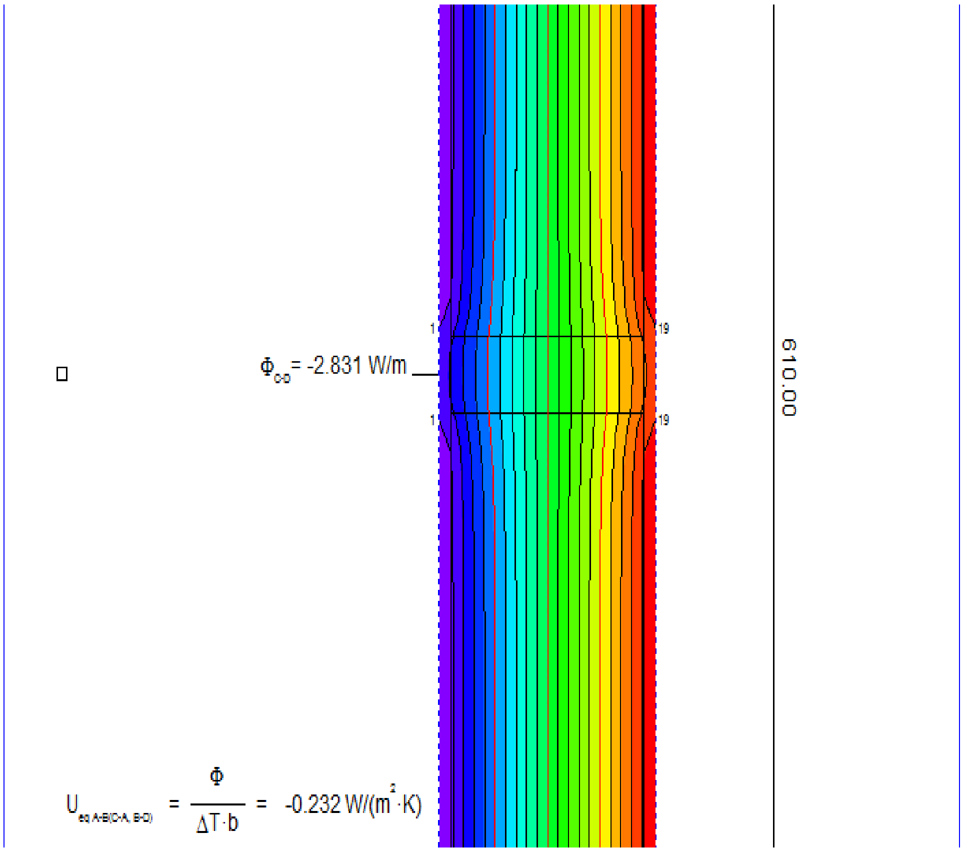
#		Material	sd-value [m]	Condensate [kg/m²]	[Gew.-%]	Weight [kg/m²]
1	0,05 cm	DAFA HiFoil	10,00	-		0,1
2	0,9 cm	OSB/3	1,80	-	-	5,6
3	12 cm	KNAUF Earthwool 32	0,12	-		3,4
	12 cm	KVH C24 (6,9%)	2,40	-	-	3,7
4	0,9 cm	OSB/3	2,70	-	-	5,9
5	0,05 cm	Breather membrane sd=0,05m	0,02	-		0,4
	19 cm	Whole component	14,75			28,4

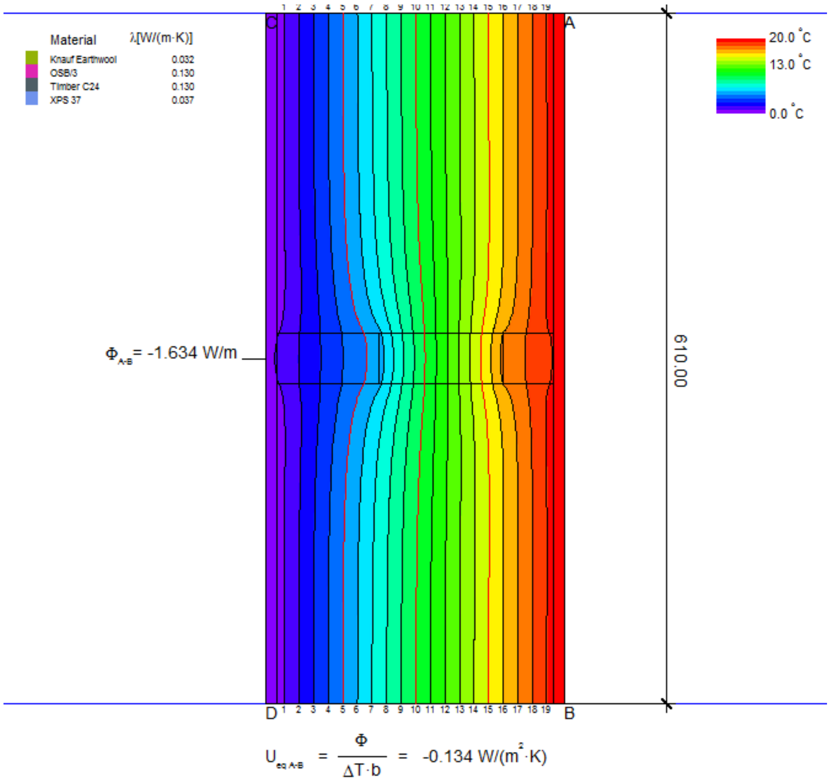


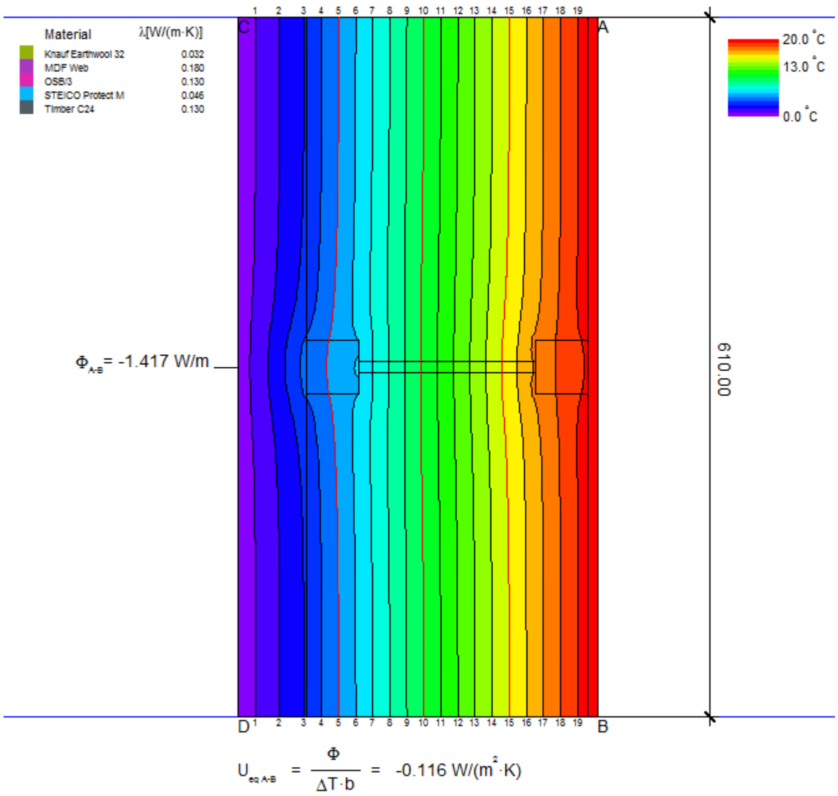


III. U-value Calculation Results

Calculation results from FLIXO

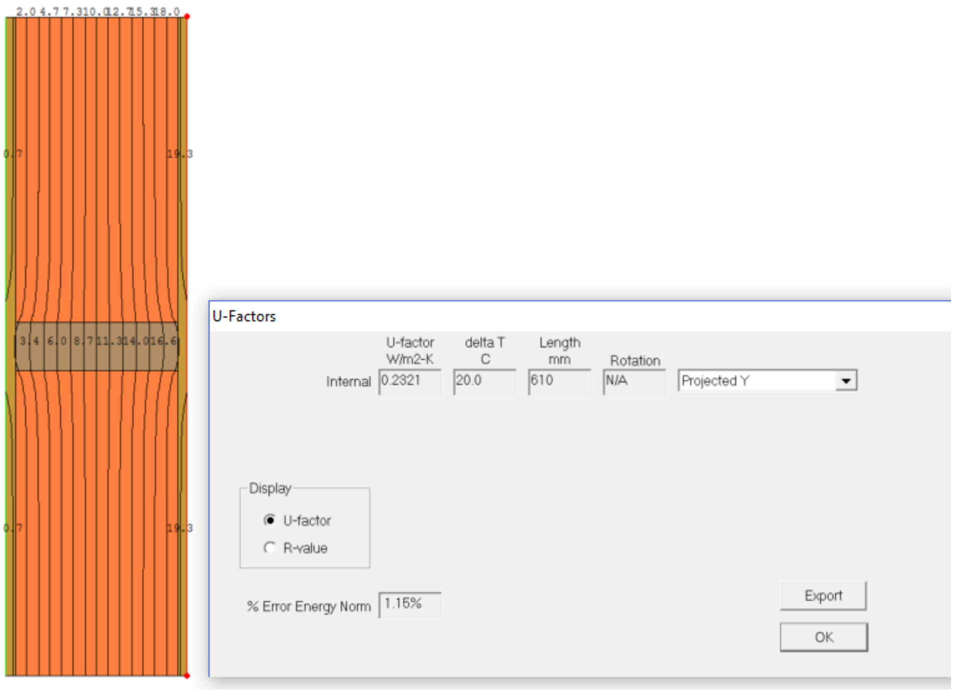



Material
λ[W/(m·K)]
Knauf Earthwool 0.032
OSB/3 0.130
Timber C24 0.130
XPS 37 0.037
20.0 °C
13.0 °C
0.0 °C
C
A
D
B
610.00
Φ A-B = -1.634 W/m
U eq A-B = Φ / ΔT·b = -0.134 W/(m²·K)

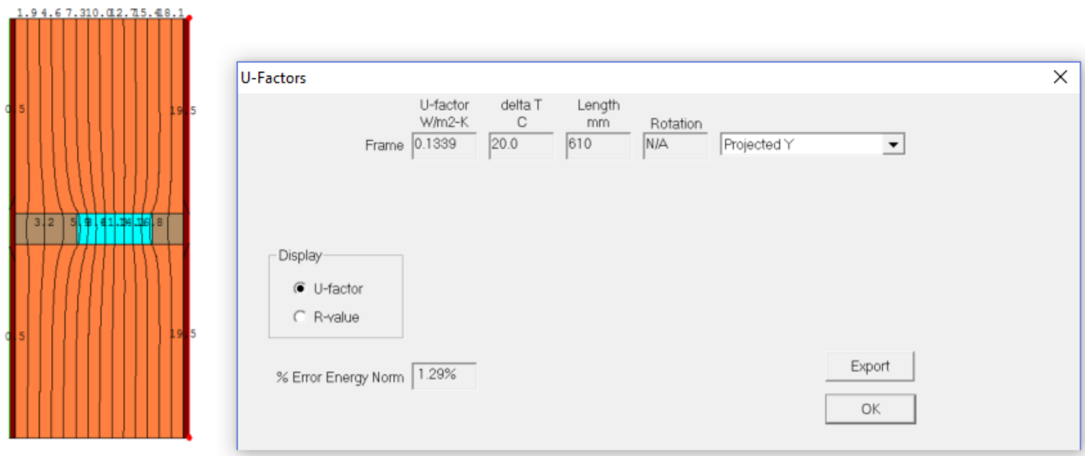

Material
λ[W/(m·K)]
Knauf Earthwool 32 0.032
MDF Web 0.180
OSB/3 0.130
STEICO Protect M 0.046
Timber C24 0.130
20.0 °C
13.0 °C
0.0 °C
C
A
D
B
610.00
Φ A-B = -1.417 W/m
U eq A-B = Φ / ΔT·b = -0.116 W/(m²·K)

Calculation results from THERM

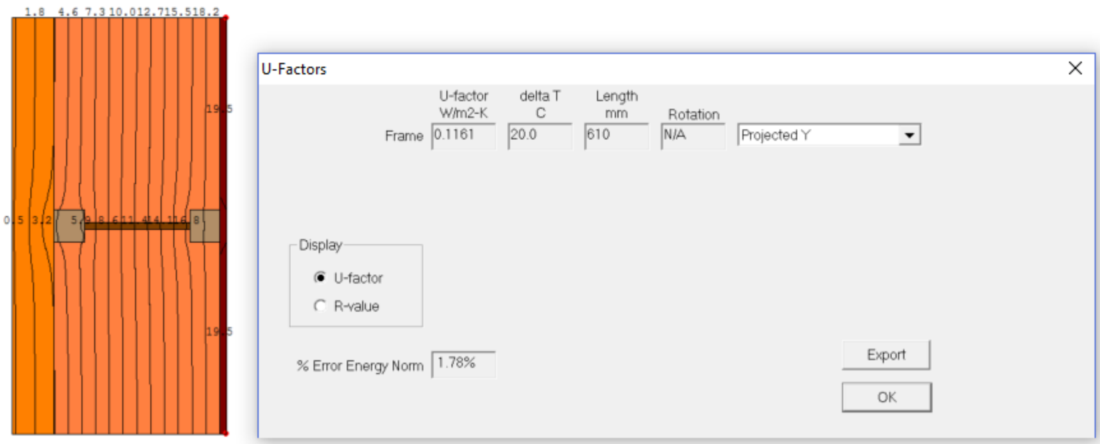
```
Frame    Total Length=   778     U-factor=       0.181947
Frame    Projected X=    168     U-factor=       0.842589
Frame    Projected Y=    610     U-factor=       0.232057
wall panel       Total Length=   610     U-factor=       0.232057
wall panel       Projected X=    0       U-factor=       -1
wall panel       Projected Y=    610     U-factor=       0.232057
```



```
Frame    Total Length=   610     U-factor=       0.133926
Frame    Projected X=    0       U-factor=       -1
Frame    Projected Y=    610     U-factor=       0.133926
```

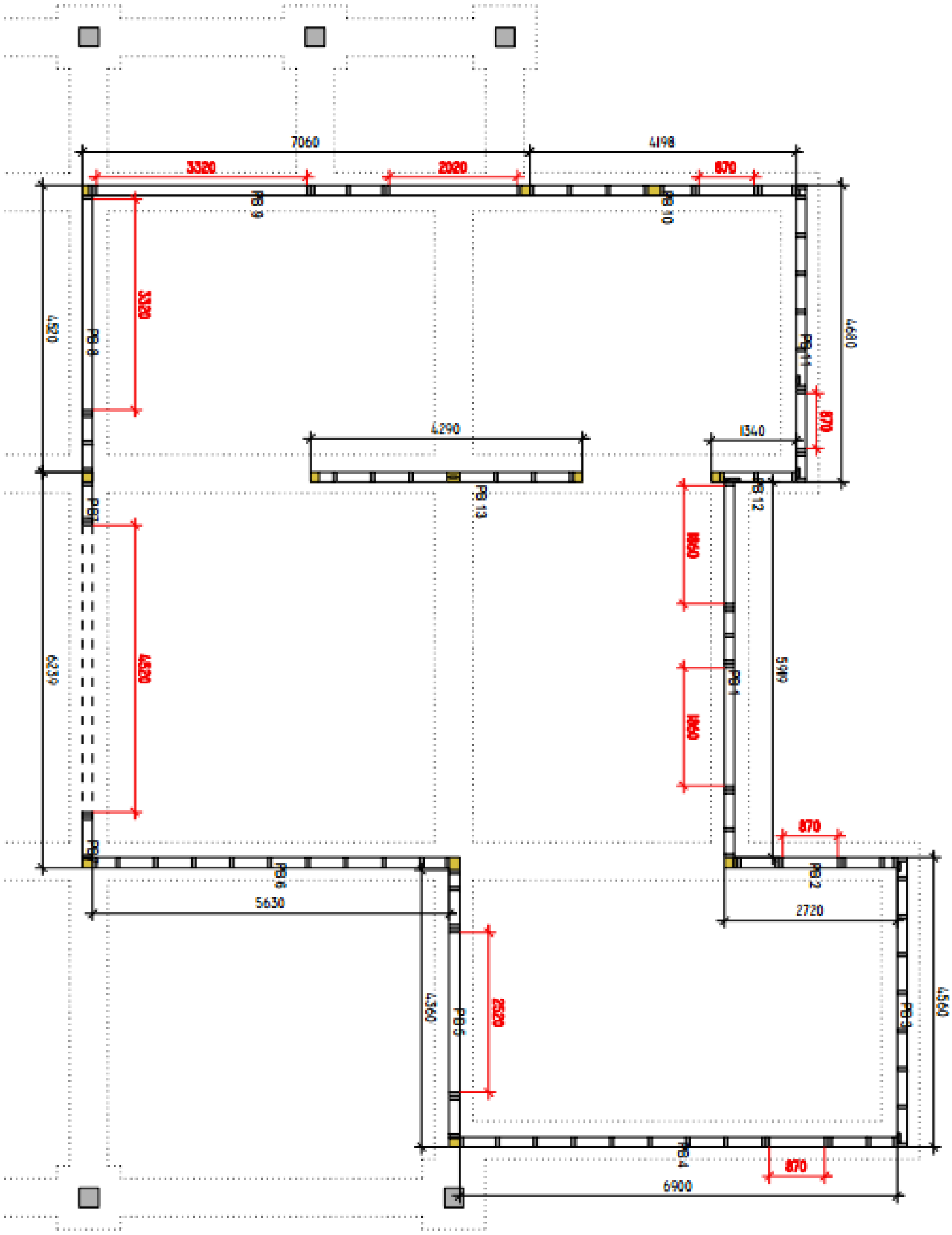



```
Frame    Total Length=   610     U-factor=       0.116115
Frame    Projected X=    0       U-factor=       -1
Frame    Projected Y=    610     U-factor=       0.116115
```

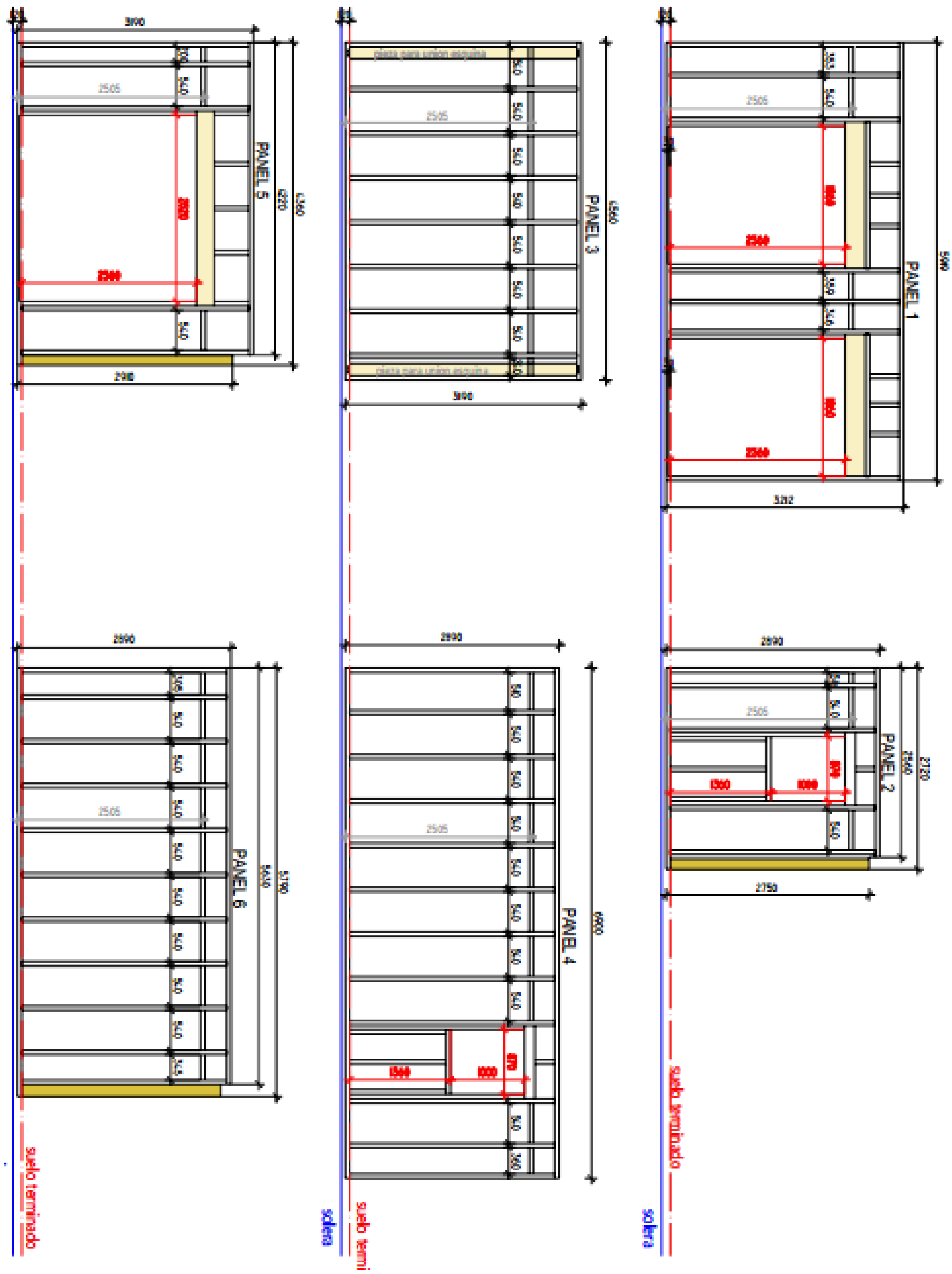


IV. Timber Fraction manufacturing layouts

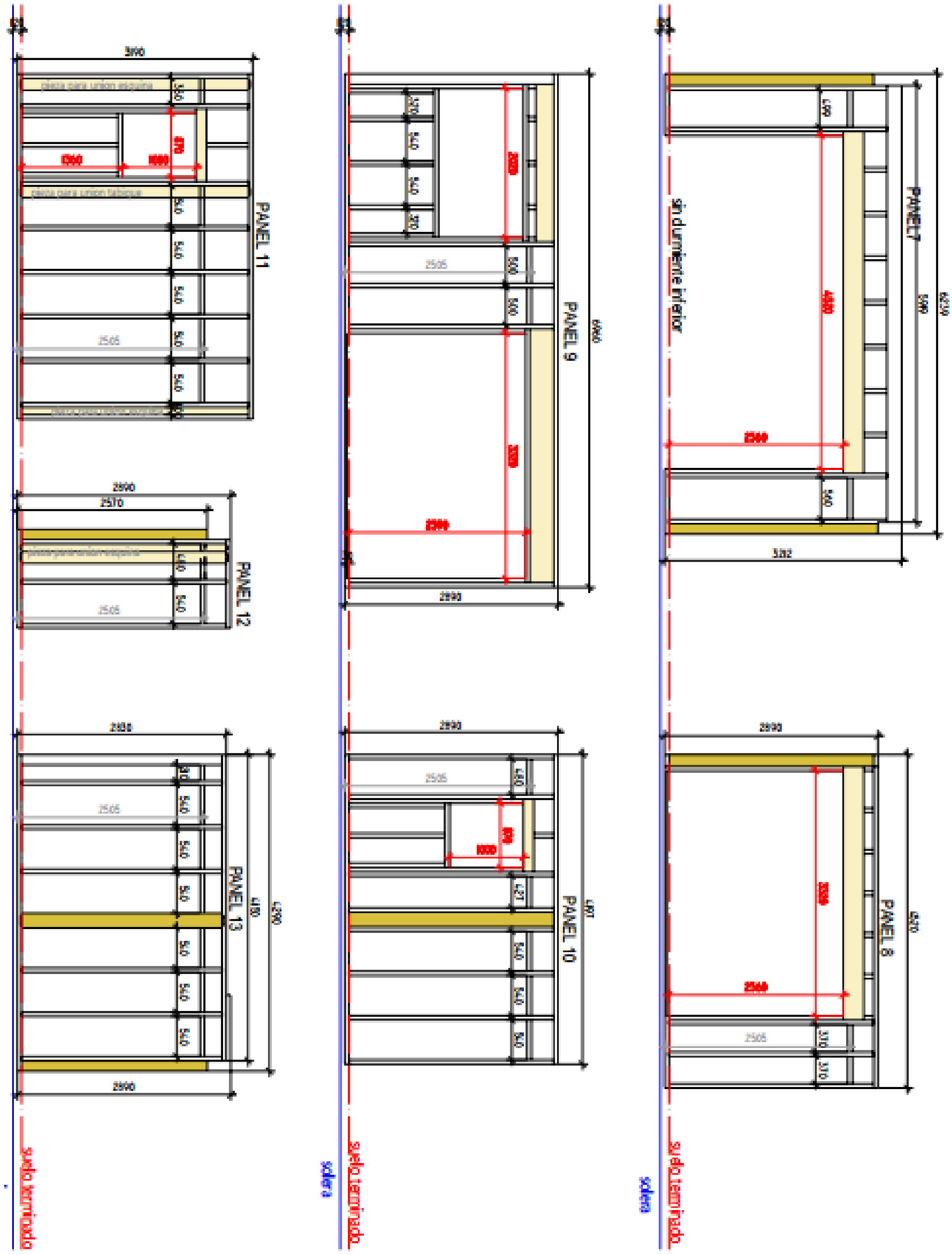
Layout of Project no. 4: Two storeys: Stud 45mmx120 mm at 625 mm c/c



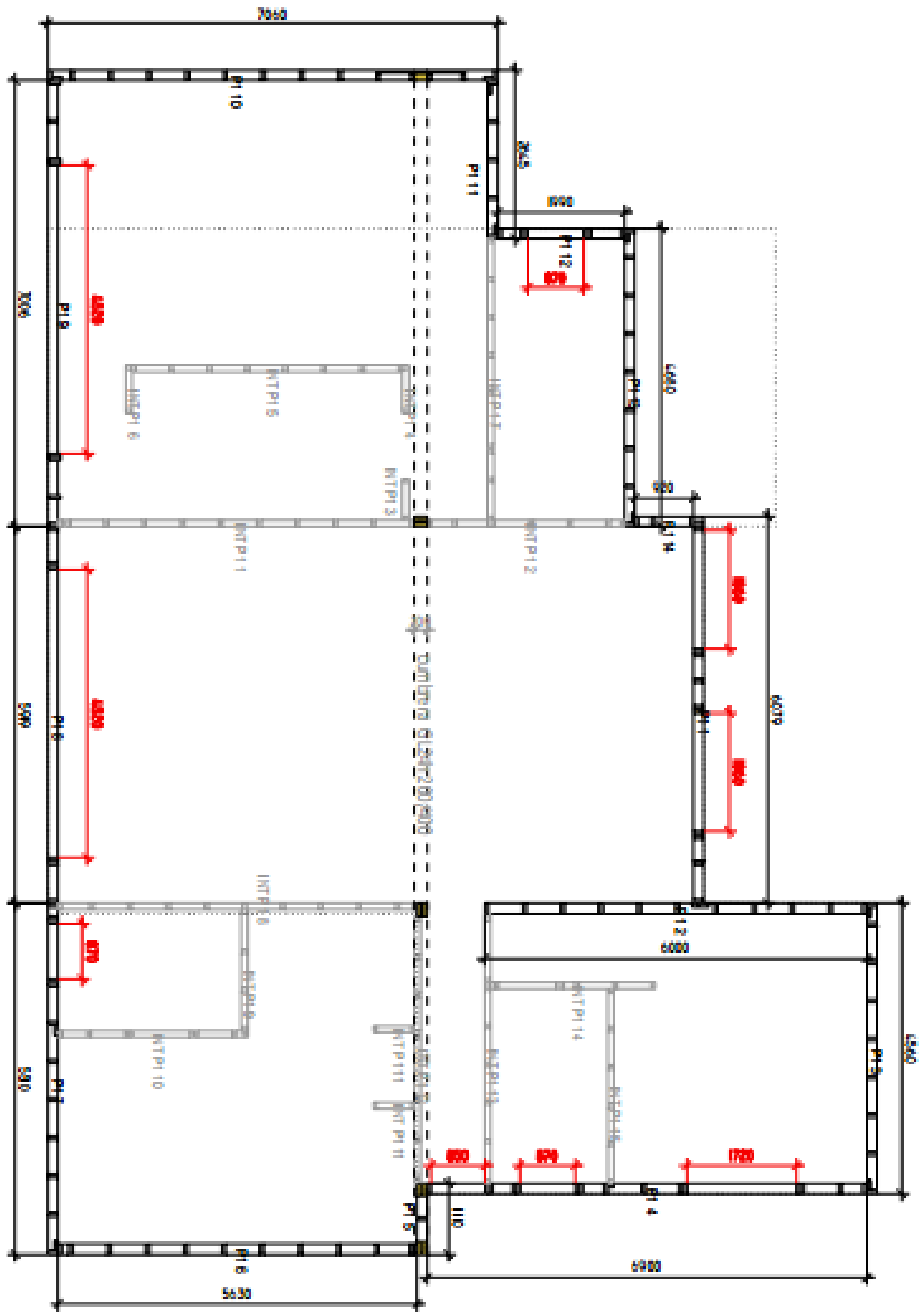
Floor plan of ground floor



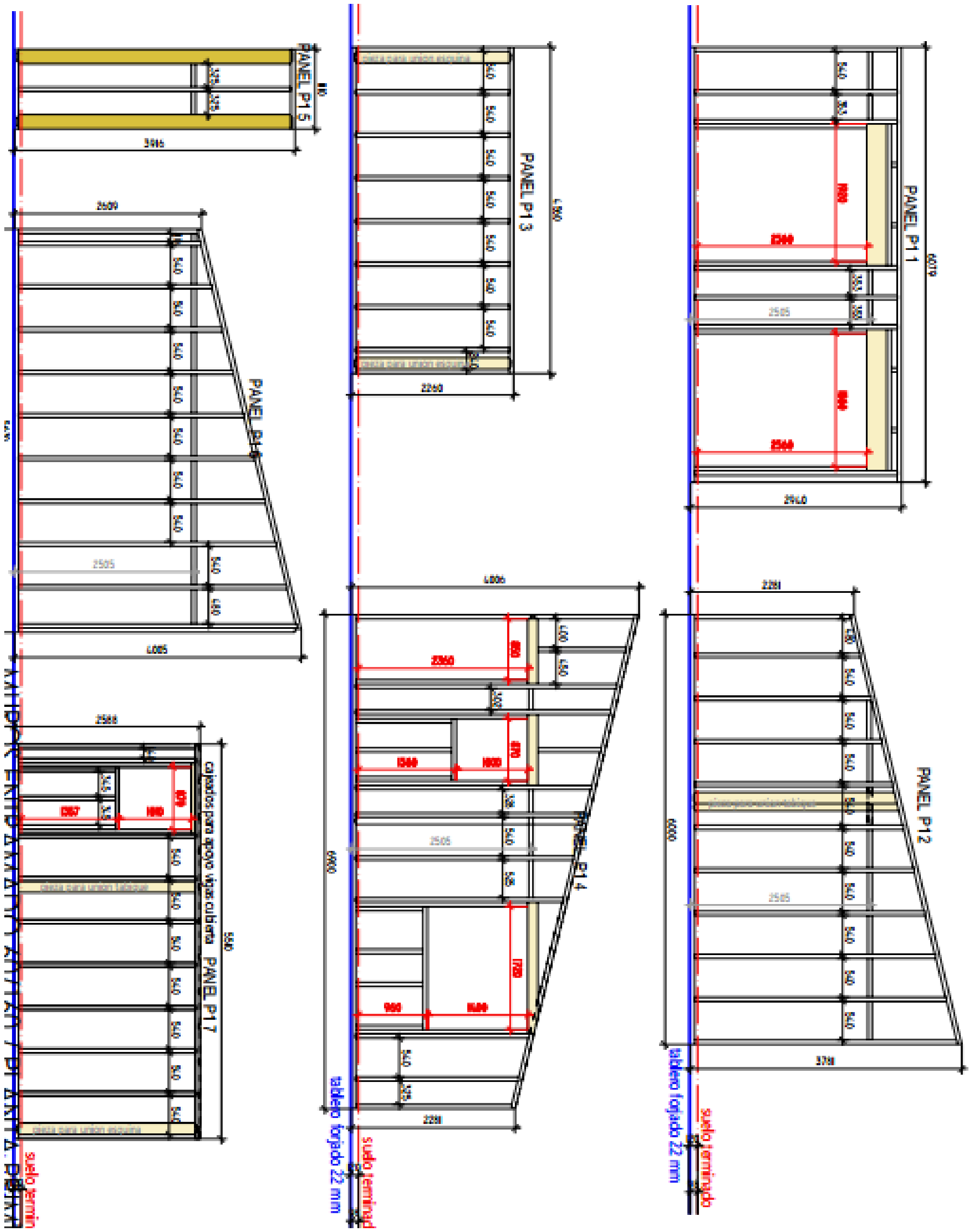
Panelised timber frame walls. Ground floor Part I.



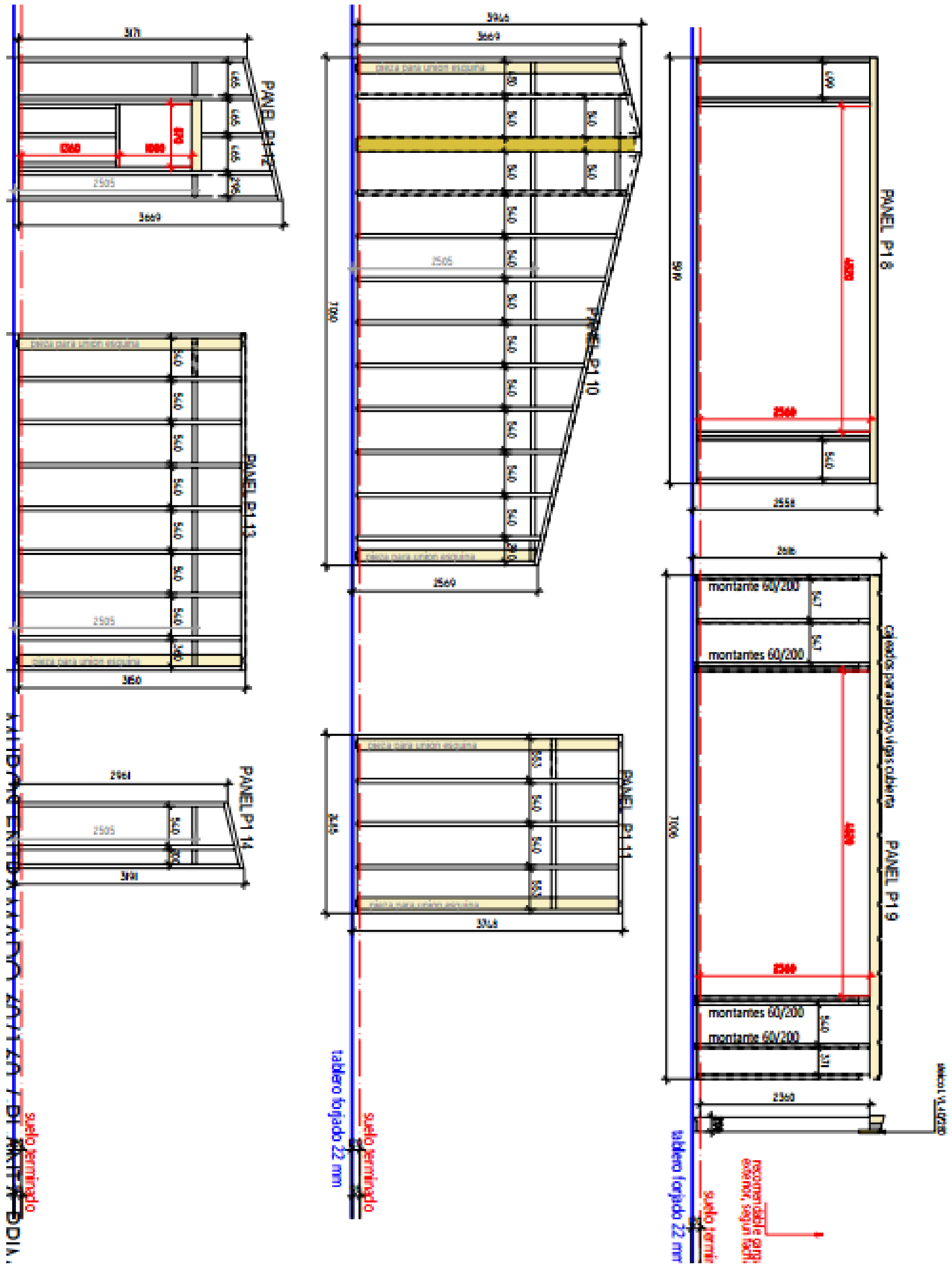
Panelised timber frame walls. Ground floor Part II.



Floor plan of first floor.



Panelised timber frame walls. First floor I.



Panelised timber frame walls. First floor II.

Panel by Panel TF calculation. Overall TF = 10 %

Ground floor	**Panel 1**	5919	3212	0	TF	7.4%
		9276	60	19011828		
		3920	60	0		
			60			
			60			
			60			
			60			
		23360	1401600	19011828		
	Panel 2	2720	2890	0	TF	16.7%
			60	7860800		
			60	0		
			60			
			60			
			60			
			60			
		21905	1314300	7860800		
	Panel 3	4560	3190		TF	15.6%
			60	14546400		
			60	0		
			60			
			60			
			60			
			60			
		37940	2276400	14546400		
	Panel 4	6900	2890	0	TF	12.0%
			60	19941000		
			60	0		
			60			
			60			
			60			
			60			
			60			
		39845	2390700	19941000		
	Panel 5	4360	3190		TF	6.8%

		60	13908400		
		60	0		
		60			
		60			
		60			
		60			
		60			
	15705	942300	13908400		
Panel 6	5790	2890		TF	13.7%
		60	16733100		
		60	0		
		60			
		60			
		60			
		60			
		60			
	38220	2293200	16733100		
Panel 7	6239	3212		TF	2.2%
		60	20039668		
		60	0		
		60			
		60			
		60			
		60			
		60			
	7450	447000	20039668		
Panel 8	5790	2890		TF	4.0%
	8670	60	16733100		
	2420	60	0		
		60			
		60			
		60			
		60			
		60			
	11090	665400	16733100		

Panel 9	6960	2890		TF	7.7%
		60	20114400		
		60	0		
		60			
		60			
		60			
		60			
		60			
	25900	1554000	20114400		
Panel 10	4197	2890		TF	15.7%
		60	12129330		
		60	0		
		60			
		60			
		60			
		60			
		60			
	31697	1901820	12129330		
Panel 11	4680	3190		TF	14.6%
		60	14929200		
		60	0		
		60			
		60			
		60			
		60			
		60			
	36380	2182800	14929200		
Panel 12	1340	2890		TF	22.7%
		60	3872600		
		60	0		
		60			
		60			
		60			
		60			
		60			
	14660	879600	3872600		

	Panel 13	5790	2890		TF	10.8%
			60	16733100		
			60	0		
			60			
			60			
			60			
			60			
			60			
		30040	1802400	16733100		
First floor	**Panel 1**	6079	2940	0	TF	4.6%
			60	17872260		
			60	0		
			60			
			60			
			60			
			60			
		13688	821280	17872260		
	Panel 2	6000	2281	1500	TF	12.9%
			60	13686000		
			60	4500000		
			60			
			60			
			60			
			60			
		39024	2341440	18186000		
	Panel 3	4560	2260		TF	11.3%
			60	10305600		
			60	0		
			60			
			60			
			60			
			60			
		19332	1159920	10305600		
	Panel 4	6900	2281	1725	TF	4.0%

		60	15738900		
		60	5951250		
		60			
		60			
		60			
		60			
		60			
	14346	860760	21690150		
Panel 5	1110	3916		TF	27.9%
		60	4346760		
		60	0		
		60			
		60			
		60			
		60			
		60			
	20230	1213800	4346760		
Panel 6	5630	2609	1396	TF	9.3%
		60	14688670		
		60	3929740		
		60			
		60			
		60			
		60			
		60			
	28720	1723200	18618410		
Panel 7	5510	2588		TF	13.2%
		60	14259880		
		60	0		
		60			
		60			
		60			
		60			
		60			
	31308	1878480	14259880		

V. RTC closed panel full thermal bridge simulation

Thermal bridge calculation for 195 mm thick RTC timber frame wall panel with both internal and external insulation.

HTflux J Menendez - RTC PassiveWall | Thermal Bridge Calculation

PROJEKTINFORMATION

Project name:	Thermal Bridge Calculation
Project ID:	Chapter 3.6
Project Date:	07/06/16

Creator data

Company	Edinburgh Napier University
Creator	J. Menendez

MATERIALIEN & RANDBEDINGUNGEN

USED MATERIALS

Material	λ W/(mK)	Obj.	Comment
Wood flex Flange	0.047	1	EN 13171
Wood flex Web	0.041	2	EN 13171
Glass wool	0.032	2	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
XPS-R (rough surf., cell gas:air, d < 130 mm)	0.037	3	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Wood fibre insulation board	0.045	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Timber |500 kg/m³)	0.13	7	EN 12524
Oriented strand board (OSB/3)	0.13	2	EN 12524
MDF board	0.18	1	EN 12524
Gypsum plasterboard	0.25	1	EN 12524
Cement screed	1.33	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Reinforced concrete (2%)	2.50	1	EN 12524
Reinforced concrete (1%)	2.30	1	EN 12524
Soil	2.00	1	ISO 13770

BOUNDARIES

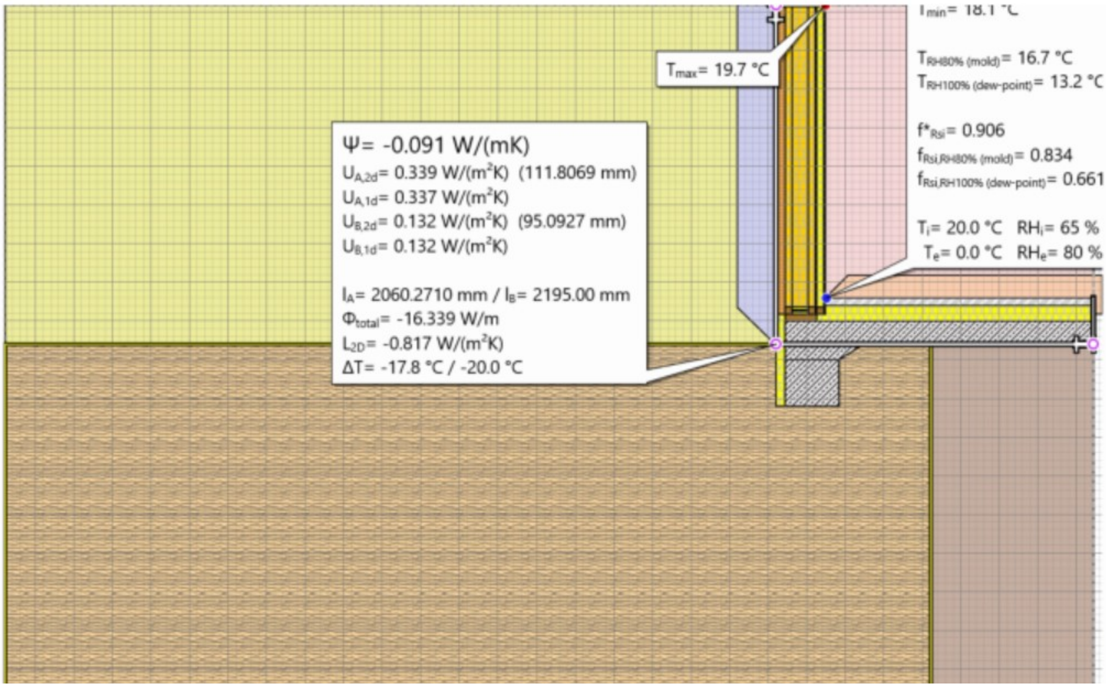
Boundary	T °C	RH (%)	Objects	Comment
PH internal wall	20.0	65	1	
PH rainscreen wall	0.0	80	1	
PH external wall	0.0	80	1	
PH internal down	20.0	65	1	
PH soil conditions	2.2	80	1	

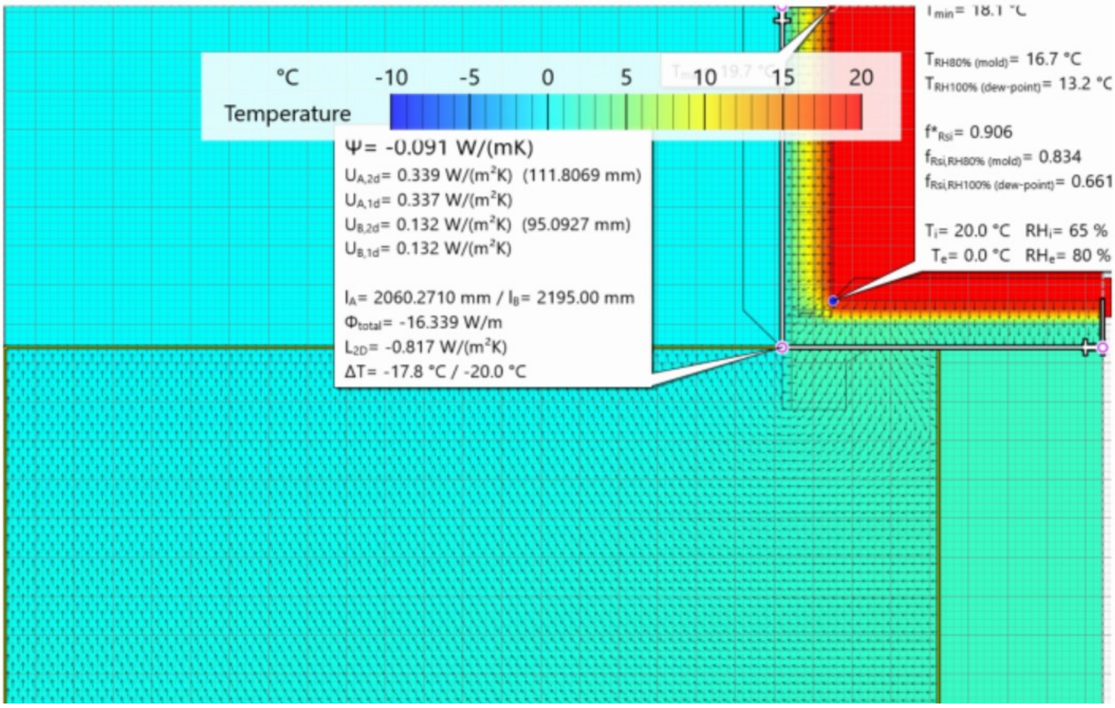
HEAT TRANSFER RESISTANCE

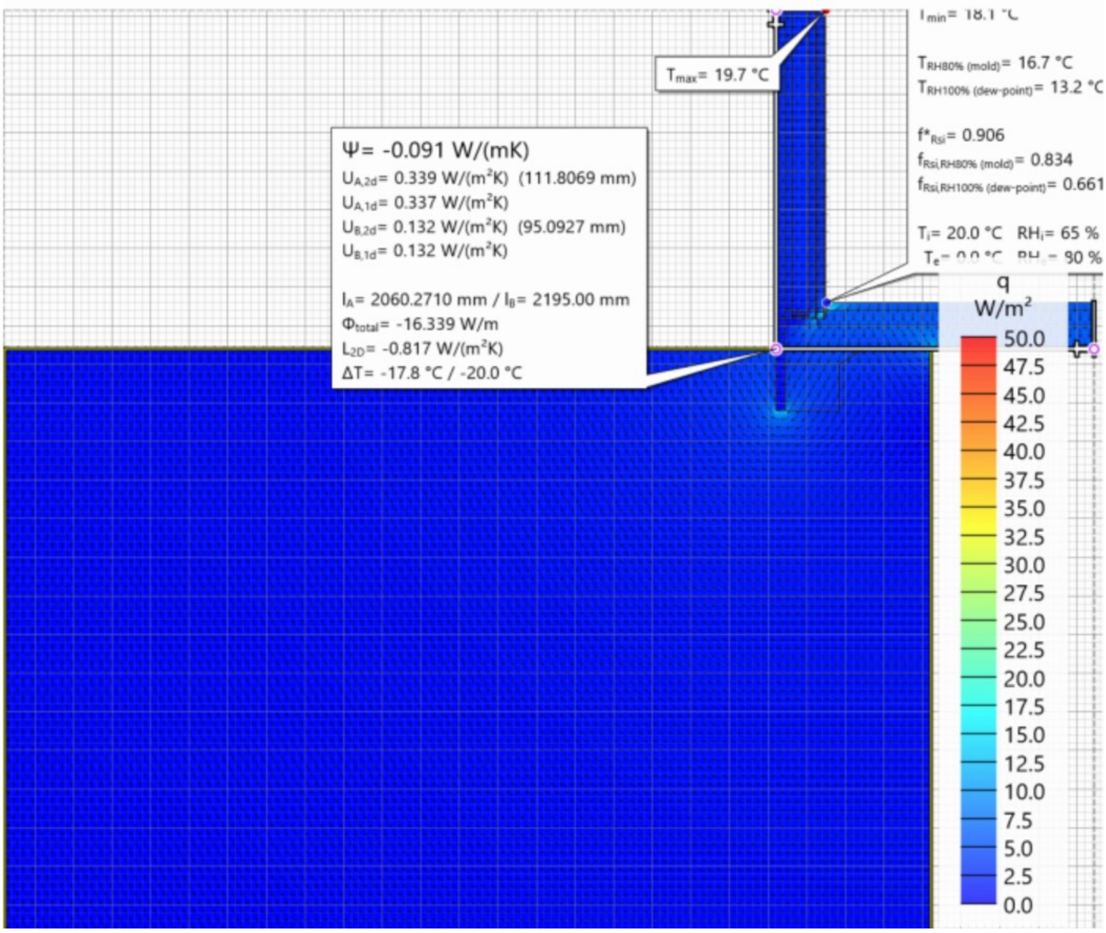
Name	R m²K/W	from material	to material
dyn1	0.13	ALL	PH internal wall
dyn2	0.25	ALL	PH internal increased wall
dyn3	0.13	ALL	PH rainscreen wall
dyn4	0.04	ALL	PH external wall
dyn5	0.17	ALL	PH floor void
dyn6	0.17	ALL	PH internal down
dyn7	0.10	ALL	PH internal roof

MATERIALANSICHT

- PH internal wall
- PH rainscreen wall
- PH external wall
- PH internal down
- PH soil conditions
- Wood flex Flange
- Wood flex Web
- Glass wool
- XPS-R (rough surf., cell gas:air, d 130 mm)
- Wood fibre insulation board
- Timber |500 kg/m³)
- Oriented strand board (OSB/3)
- MDF board
- Gypsum plasterboard
- Cement screed
- Reinforced concrete (2%)
- Reinforced concrete (1%)
- Soil




T_{min}= 18.1 °C
$T_{RH80\% (mold)}$= 16.7 °C
$T_{RH100\% (dew-point)}$= 13.2 °C
°C -10 -5 0 5 10 15 20
Temperature
f^*_{Rsi}= 0.906
$f_{Rsi,RH80\% (mold)}$= 0.834
$f_{Rsi,RH100\% (dew-point)}$= 0.661
T_i= 20.0 °C RH_i= 65 %
T_e= 0.0 °C RH_e= 80 %
Ψ= -0.091 W/(mK)
$U_{A,2d}$= 0.339 W/(m²K) (111.8069 mm)
$U_{A,1d}$= 0.337 W/(m²K)
$U_{B,2d}$= 0.132 W/(m²K) (95.0927 mm)
$U_{B,1d}$= 0.132 W/(m²K)
l_A= 2060.2710 mm / l_B= 2195.00 mm
Φ_{total}= -16.339 W/m
L_{2D}= -0.817 W/(m²K)
ΔT= -17.8 °C / -20.0 °C


T_{min}= 18.1 °C
T_{max}= 19.7 °C
$T_{RH80\% (mold)}$= 16.7 °C
$T_{RH100\% (dew-point)}$= 13.2 °C
f^*_{Rsi}= 0.906
$f_{Rsi,RH80\% (mold)}$= 0.834
$f_{Rsi,RH100\% (dew-point)}$= 0.661
T_i= 20.0 °C RH_i= 65 %
Ψ= -0.091 W/(mK)
$U_{A,2d}$= 0.339 W/(m²K) (111.8069 mm)
$U_{A,1d}$= 0.337 W/(m²K)
$U_{B,2d}$= 0.132 W/(m²K) (95.0927 mm)
$U_{B,1d}$= 0.132 W/(m²K)
l_A= 2060.2710 mm / l_B= 2195.00 mm
Φ_{total}= -16.339 W/m
L_{2D}= -0.817 W/(m²K)
ΔT= -17.8 °C / -20.0 °C
q
W/m²
50.0
47.5
45.0
42.5
40.0
37.5
35.0
32.5
30.0
27.5
25.0
22.5
20.0
17.5
15.0
12.5
10.0
7.5
5.0
2.5
0.0

VI. K2 closed panel temperature factor reports

USED MATERIALS

Material	λ W/(mK)	Obj.	Comment
Wood wool flex	0.038	1	EN 13171
Wood flex Flange	0.047	2	EN 13171
Glass wool	0.032	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
GW 0.9 / Stud 0.1	0.0418	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
XPS-R (rough surf., cell gas:air, d < 130 mm)	0.037	2	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Wood fibre insulation board	0.045	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Timber |500 kg/m³)	0.13	7	EN 12524
Oriented strand board (OSB/3)	0.13	2	EN 12524
Plywood Exterior	0.13	2	EN 12524
Gypsum plasterboard	0.25	1	EN 12524
Cement screed	1.33	1	HTflux Standard Materials, no warranty as to the completeness or accuracy of the data
Reinforced concrete (1%)	2.30	1	EN 12524
Soil	2.00	1	ISO 13770

BOUNDARIES

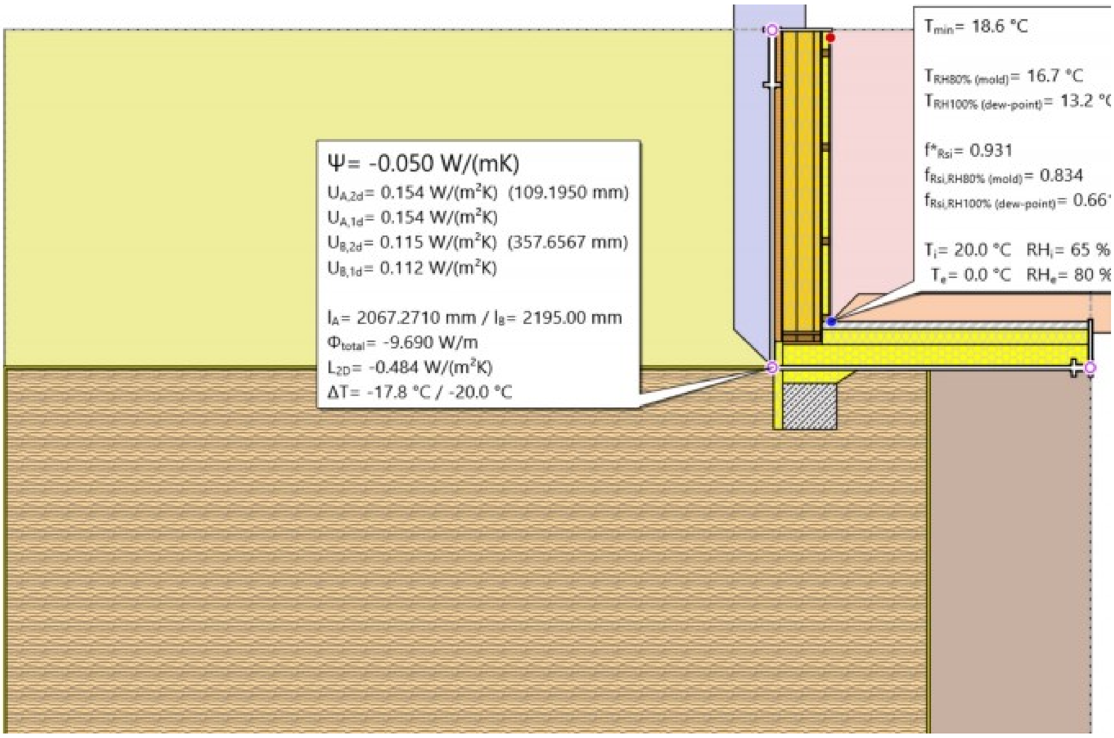
Boundary	T °C	RH (%)	Objects	Comment
PH internal wall	20.0	65	1	
PH rainscreen wall	0.0	80	1	
PH external wall	0.0	80	1	
PH internal down	20.0	65	1	
PH soil conditions	2.2	80	1	

HEAT TRANSFER RESISTANCE

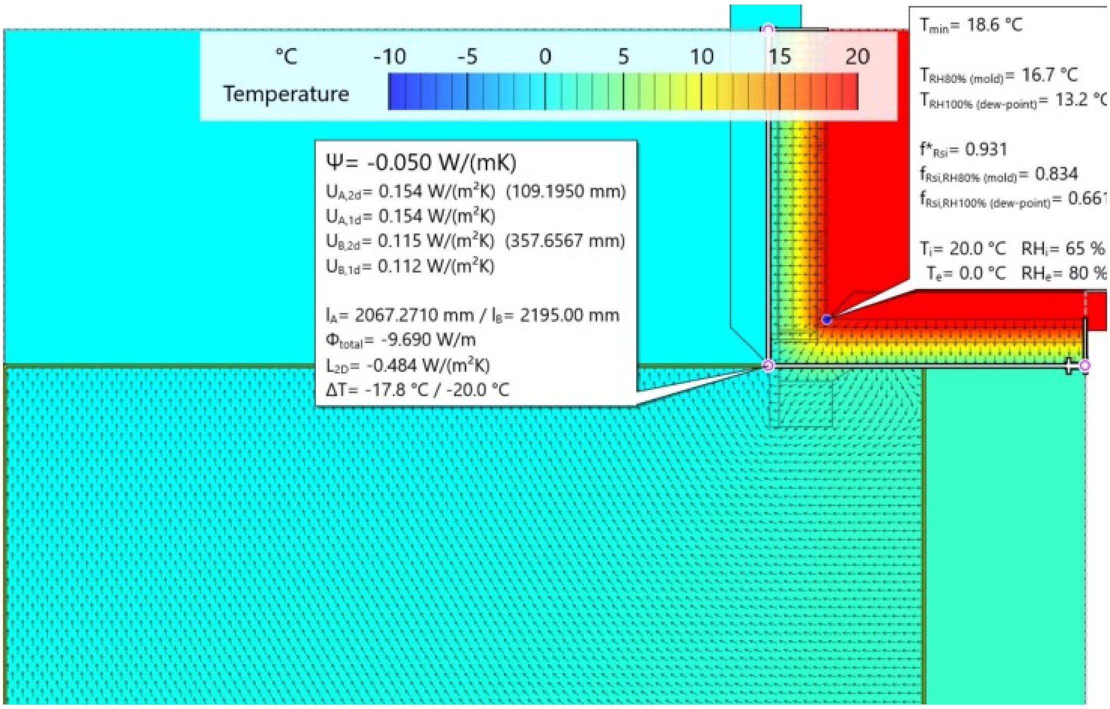
Name	R m²K/W	from material	to material
dyn1	0.13	ALL	PH internal wall
dyn2	0.25	ALL	PH internal increased wall
dyn3	0.13	ALL	PH rainscreen wall
dyn4	0.04	ALL	PH external wall
dyn5	0.17	ALL	PH floor void
dyn6	0.17	ALL	PH internal down
dyn7	0.10	ALL	PH internal roof
dyn8	0.001	ALL	PH soil conditions

PH internal wall	PH rainscreen wall	PH external wall
PH internal down	PH soil conditions	Wood wool flex
Wood flex Flange	Glass wool	GW 0.9 / Stud 0.1
XPS-R (rough surf., cell gas:air, 130 mm)	Wood fibre insulation board	Timber |500 kg/m³)
Oriented strand board (OSB/3)	Plywood Exterior	Gypsum plasterboard
Cement screed	Reinforced concrete (1%)	Soil

Thermal bridge simulation

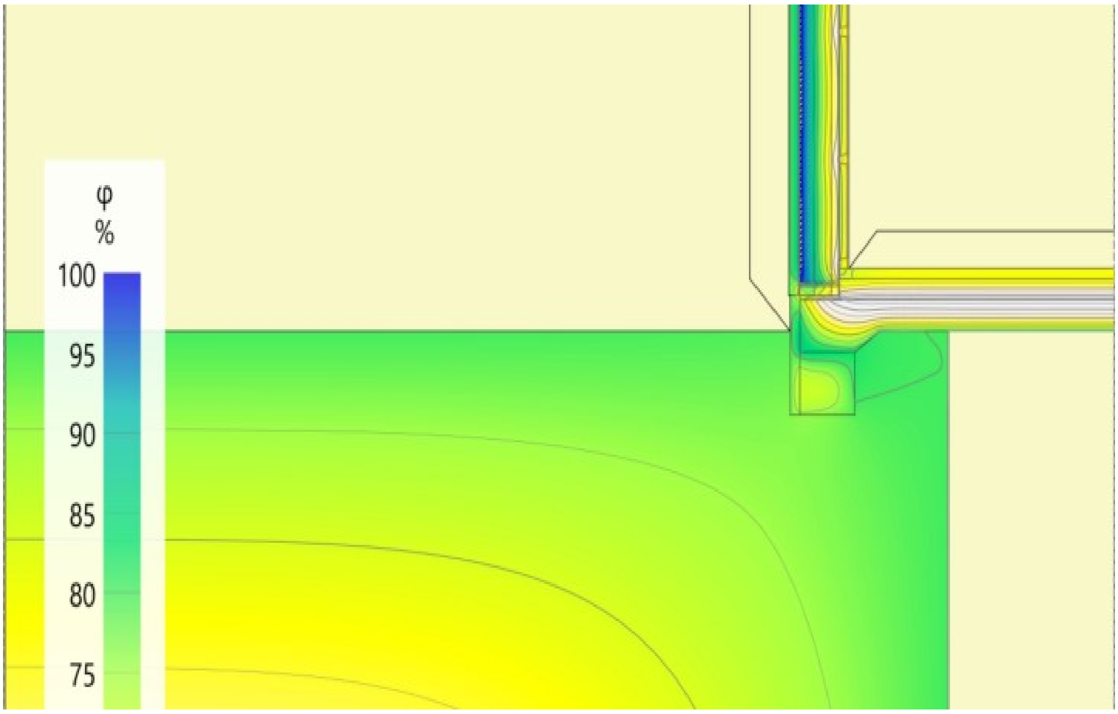


Isotherms

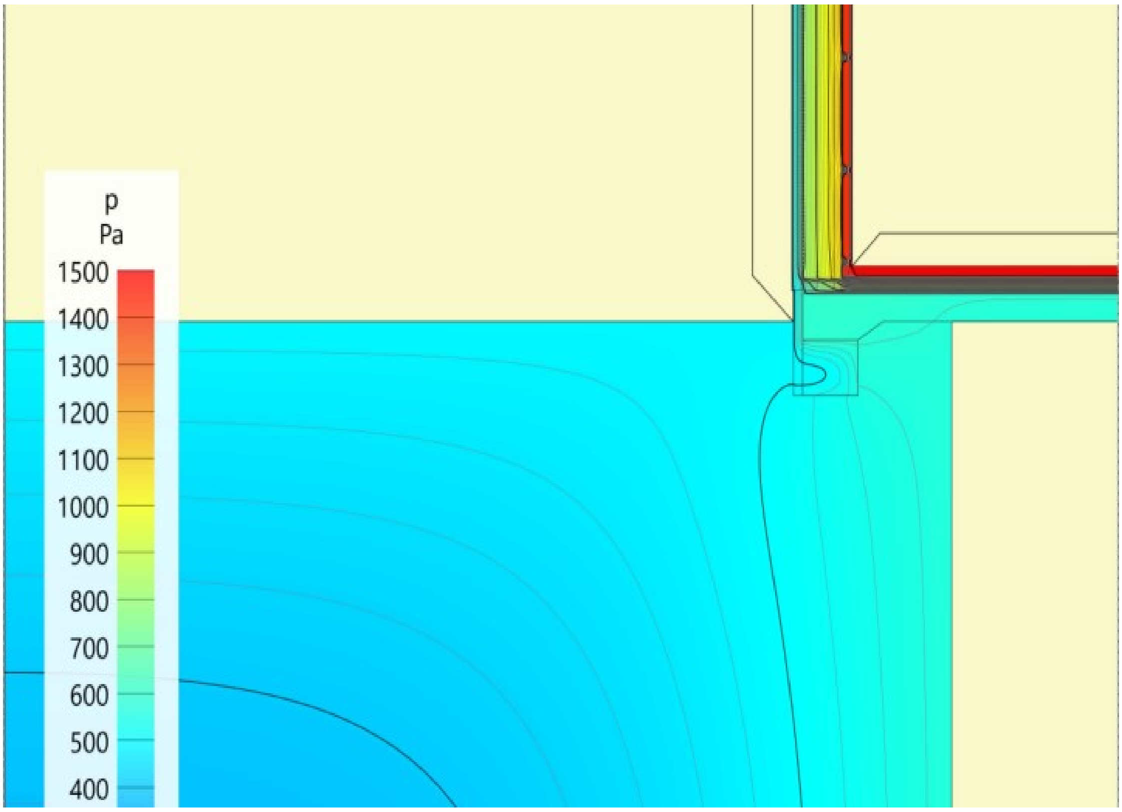


Simulation resolution: 3.5 mm; Cell count: 2,690,932

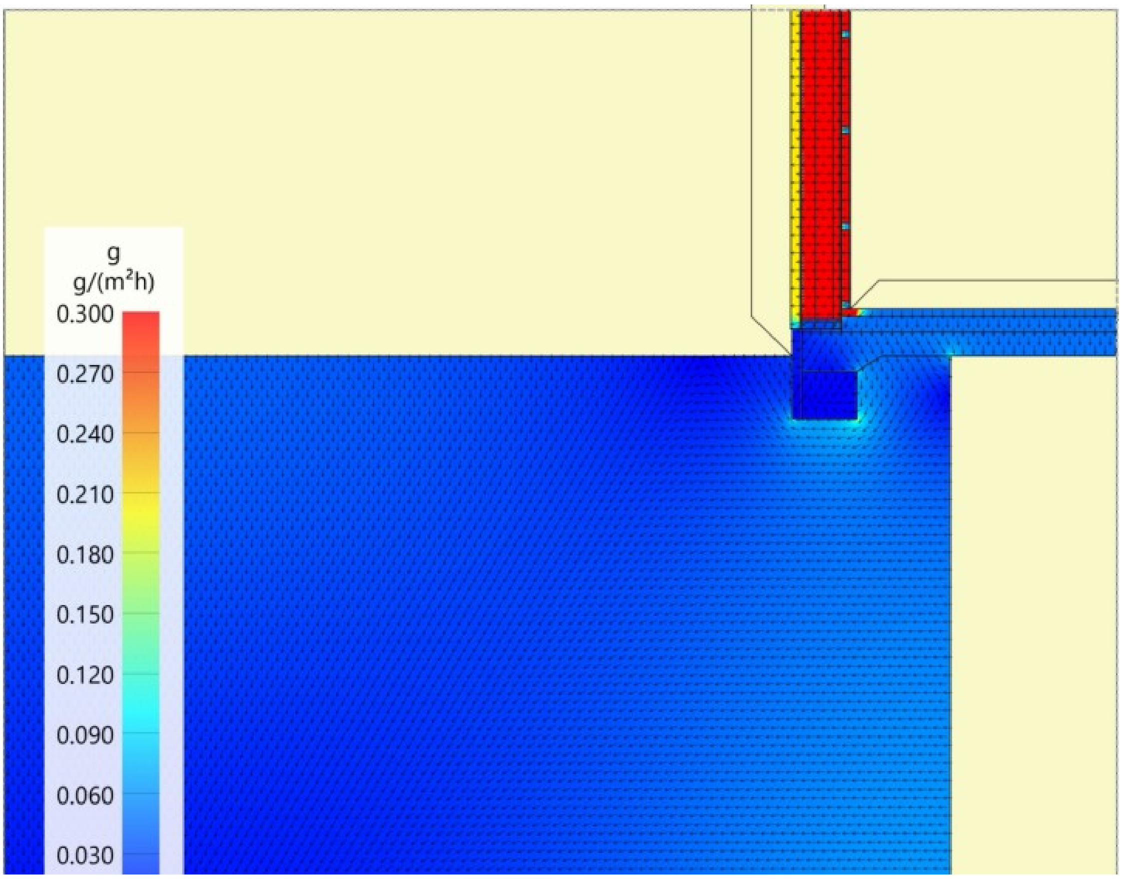
Relative humidity view



Partial pressure view



Vapour flux view

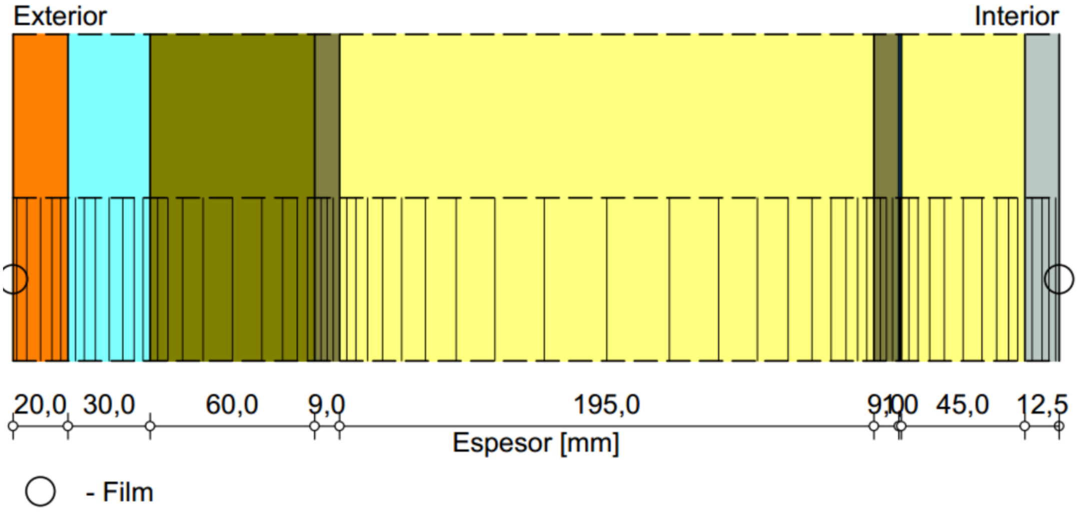


VII. K2 and RTC closed panel WUFI® reports for Warsaw climate

WUFI® Pro 5.2 — Fraunhofer IBP

Componentes del Ensamblaje

Variante: #15 195K2_MW_1.1.1_cold



Materiales :

- Maderas Blandas_Coníferas
- *Capa de Aire 30 mm
- AGEPAN THD
- Tablero OSB - Tablero de Fibra Orientada (densidad 595 kg/m³)
- ISOVER GW Integra ZSF - 032
- Tablero OSB - Tablero de Fibra Orientada (densidad 595 kg/m³)
- retardante de Vapor (sd=20m)
- ISOVER GW Integra ZSF - 032
- FERMACELL Yeso-Tablero de Fibra

Espesor Total: 0,38 m
Resist. Térmica R: 9,2 m²K/W
U-Valor: 0,107 W/m²K

Condiciones para Límites

Exterior (Lado Izquierdo)

Ubicación: Warsaw; Technical University Lodz
Orientación / Inclinación: Oeste / 90 °

Interior (Lado Derecho)

Clima Interior: WTA Pauta 6-2-01/E
Condiciones Interiores, Carga de Humedad Baja

Coef. de Transferencia en Superficie

Exterior (Lado Izquierdo)

Nombre	Descripción	Unidad	Valor
Resistencia Térmica - Incluye Radiación Onda-Corta	Pared Externa	[m²K/W]	0.0588 Si
Sd-Valor	Sin Recubrimiento	[m]	----
Absorción (Radiación de Onda Corta)	Sin Absorción/Emisión	[-]	----
Emisión (Radiación de Onda Larga)	Sin Absorción/Emisión	[-]	----
Coef. Penetración del Agua (Lluvia)	De acuerdo a la Inclinació	[-]	0,7
Balance de Radiación Explícita			No

Interior (Lado Derecho)

Nombre	Descripción	Unidad	Valor
Resistencia Térmica	Pared Externa	[m²K/W]	0.125
Sd-Valor	Sin Recubrimiento	[m]	----

Resultados de último Cálculo

Desarrollo del Cálculo

Calculación: Hora y Fecha	28/06/2016 18:33:41
Duración del Cálculo	1 mín,25 seg.
Begin / End of calculation	01/10/2016 / 01/10/2019
No. de Errores Convergentes	6

Chequeo de Calidad Numérica

Intergal de flujos, lado Izquierdo (kl,dl)	[kg/m²]	0,0 -0,56
Intergal de flujos, lado Derecho (kr,dr)	[kg/m²]	0,0 -0,04
Balance 1	[kg/m²]	-0,52
Balance 2	[kg/m²]	-0,52

Cont. de Agua [kg/m²]

	Inicio	Fin	Mín.	Máx.
Contenido Total de Agua	5,06	4,56	4,17	5,25

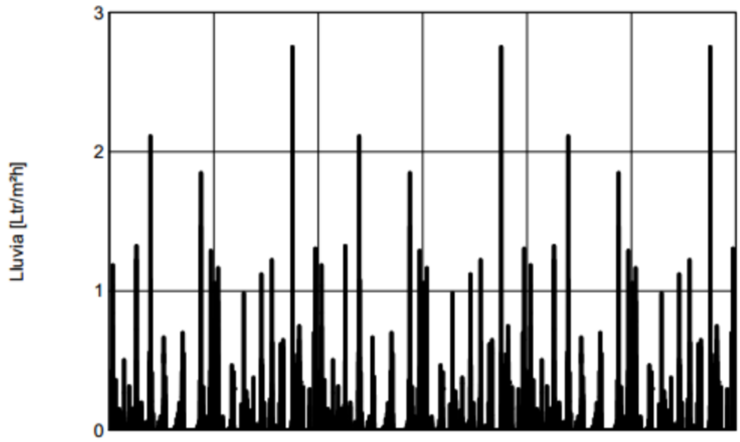
Cont. de Agua [kg/m³]

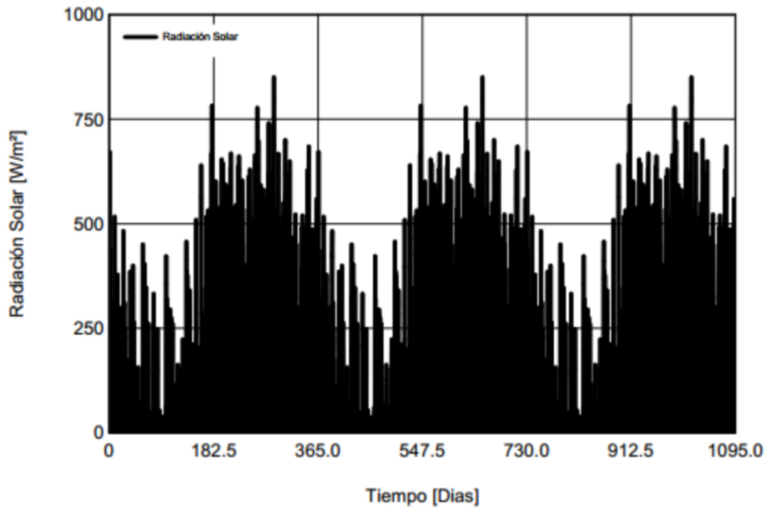
Capa/Material	Inicio	Fin	Mín.	Máx.
Maderas Blandas_Coníferas	60,00	61,43	46,29	85,88
*Capa de Aire 30 mm	1,88	1,77	1,11	3,81
AGEPAN THD	30,00	26,76	25,30	31,17
Tablero OSB - Tablero de Fibra Orie	95,00	88,26	82,74	104,38
ISOVER GW Integra ZSF - 032	0,40	0,29	0,17	0,44
Tablero OSB - Tablero de Fibra Orie	95,00	73,68	59,18	95,00
retardante de Vapor (sd=20m)	0,00	0,00	0,00	0,00
ISOVER GW Integra ZSF - 032	0,40	0,27	0,10	0,40
FERMACELL Yeso-Tablero de Fibra	15,80	11,48	6,30	15,80

Integral de Flujos respecto al Tiempo

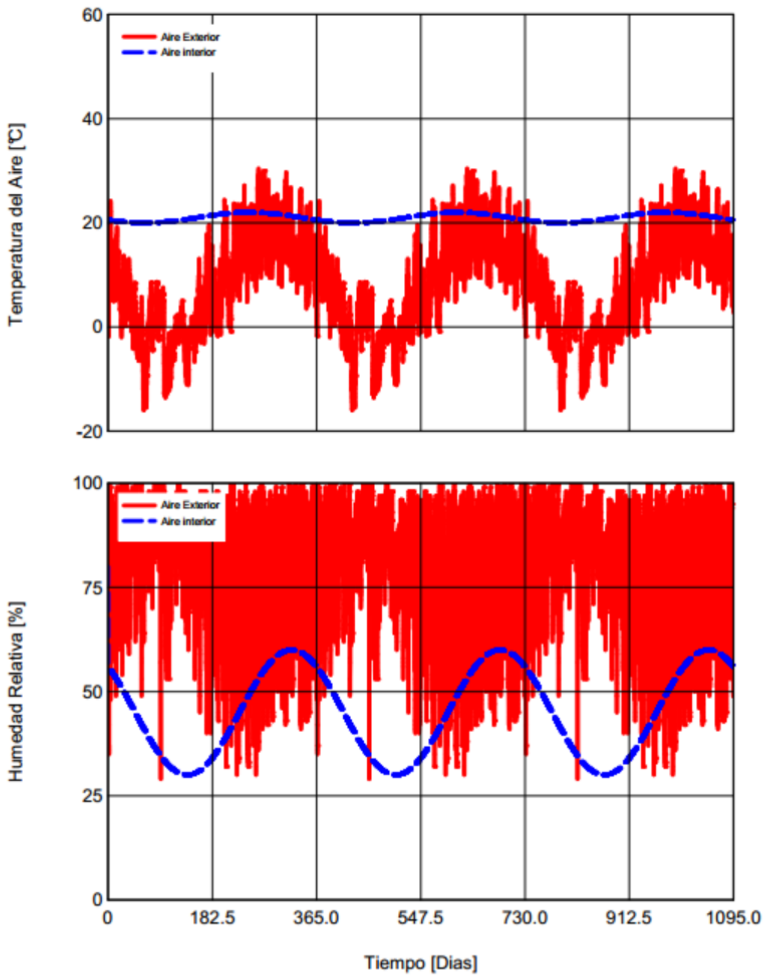
Flujo de calor, Lado Izquierdo	[MJ/m²]	-133,22
Flujo de calor, Lado Derecho	[MJ/m²]	-132,68
Fuentes de Calor	[MJ/m²]	0,0
Flujos de Humedad, Lado Izquierdo	[kg/m²]	-0,56
Flujos de Humedad, Lado Derecho	[kg/m²]	-0,04
Fuentres de Humedad	[kg/m²]	0,0
Clipped Moisture Sources	[kg/m²]	0,0

Lluvia, Radiación (Clima Exterior)

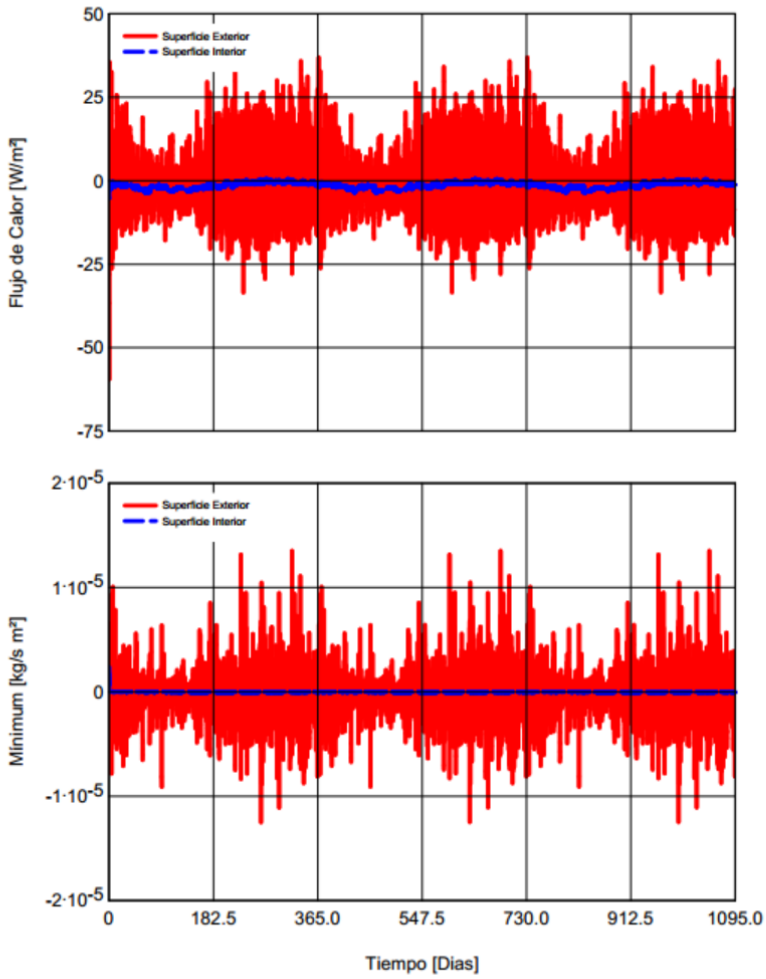




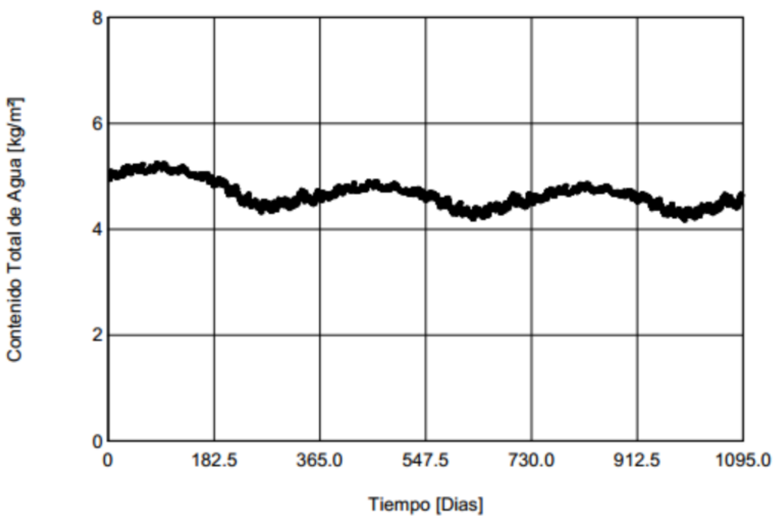
Temp. Aire, H.Rel (Exterior, Interior)



Flujos de Calor, Humedad



Contenido Total de Agua en la Construcción



VIII. Previous Experiments on Timber Frame Racking Walls Carried Out at Edinburgh Napier

The specification of the walls was carried out with the objective of replicating standardised procedure adapted by the UK timber frame industry and already reported in Table 3-8. Other literature reviewed in chapter 4 showed the timber frame materials and fasteners commonly specified:

- 38x140 mm timber classified as C16
- 9 mm OSB structural sheathing board
- 2.50 x 50 mm smooth wire nails to fix sheathing to frame
- 3.25 x 90 mm helical or twisted fasteners to assembly the frame

The fasteners fixed around the perimeter of the frame were fixed in the centre stud line whilst two sheathing sheets were fixed to the same stud the end and edge distance was approximately 6 mm. In all of the cases, fasteners fixed on internal studs were spaced twice the distance of those around the perimeter.

The results provided by these experiments were considered pertinent for the benchmark open timber frame scenario. Furthermore, the empirical data was compared with the analytical approach incorporated in the software racking design application. According to the naming convention explained in Table 4-18, the racking strength and stiffness average values, according to Equation 4.6, of sixteen pairs of imperforated walls tested are presented in Table 8-1.

The racking strength and stiffness values resulted from these previous tests were used to compare the benchmark open timber frame with both closed timber frame panels. Furthermore, the characteristic $F_{v,Rk}$, racking strength of these frame references together with other perforated open panels and the closed panels specified in next section was determined by three different calculation tools to validate the software developed in this thesis.

Table 8-1 Values from previous racking tests (Leitch, 2013)

Series name	Reference	F_{max} (kN)	$R_{SLS,594}$ (N/mm)
Imperforate fully restrained	F[L1.2].75	17.02	414
	F.75	36.14	2,012
	F.50	49.37	1,916
	F.50.DS	82.48	4,500
Imperforate subjected to vertical load	V10.50.BR	34.86	1,954
	V10.150.BR	13.89	1,723
	V10.50.BR.DS	41.94	2,805
	V10.150.BR.DS	35.60	2,088
Imperforate partially restrained	P.150.BR	8.06	868
	P.50.BR	9.98	724
	P.50	24.79	1,753
	P.50.DS.BR	16.84	1,174
	P.150.DS.BR	16.84	1,172
	P.75.BR	12.57	943
	P.150	12.25	1,949
	P[L1.2].75	4.72	287
Perforate subjected to vertical load	V5.150.900x128W	15.61	660
	V5.150.900x128L	11.43	441
	3.6.V5.150.1380x1250L	20.16	1,051
	3.6.V5.150.1900x1365L	14.54	936
	3.6.V5.50.1900x1365L	23.83	1,221
	3.6.V5.150.2x1380x1250WL	28.47	1,426
	V10.50.1200x1200C	18.53	1,305
	V10.50.1200x2100C	14.58	1,889

IX. Isolation-Combination Sole Plate Testing Details from Previous Research

In this Appendix, the complete test results with images of the apparatus and equipment utilised and the load versus displacement diagrams from previous published research is provided (Menendez et al., 2013; Leitch, 2013).

The results of the isolated connection tests are given in Table 1. Due to the ductile nature of the connections, the ultimate strength is based upon the measured force resistance at 15mm of displacement.

Figure 1 shows the connection types a) b) and type d) being tested with their associated load slip curve results. In order to confirm the performance of the SPFD in isolation with the overall performance of the complete detail, full sole plate fixing open and closed panel details were also tested.

These tests were performed according to a heavily modified version of the BS EN 1380 test set up so as to replicate the shear load being transferred from the wall panel to the substrate.

In addition to the lateral shear resistance for each connection type, the experiment also includes testing of fasteners for tensile strength, yield moment and axial withdrawal capacity. On the other hand, nominal wire and root diameter were considered to analytically determine the mechanical properties of nails and screws respectively. Test results, as mean values, are shown in Table 2.

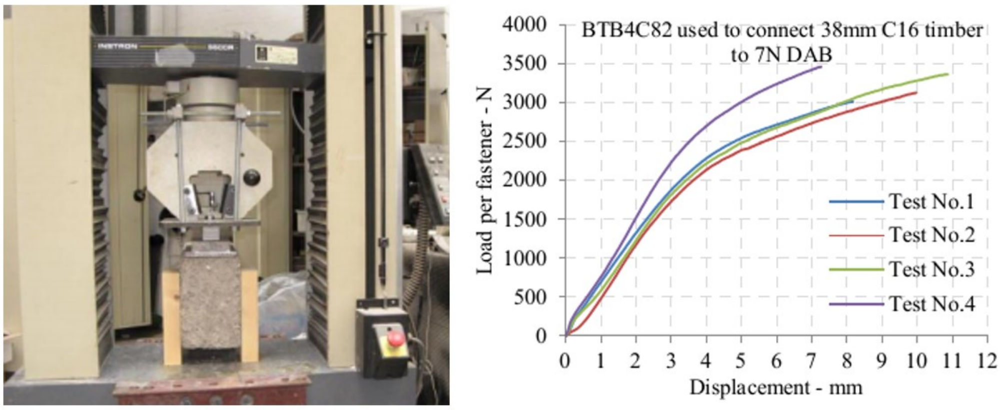
Figure 2 illustrates the sole plate fixing detail test set up and the load – slip curves for the sole plate tested and for the open (OP) and the closed panel (CP) systems. Furthermore, Table 3 presents the strength and stiffness test results of the full sole plate base fixing detail.

Table 3 presents the strength and stiffness test results of the full SPFD. The maximum strength value, f_{max}, is determined when 15mm displacement of the bottom rail relative to the substrate occurs. Similarly, stiffness, K_{ser}, is based upon the displacement of the bottom rail at 40% of its ultimate strength.

Table 1: Strength and stiffness results of the isolated sole plate components

Connection	Ultimate strength f_{max} *(N)*	Slip modulus[(1)] K_{ser} *(N/mm)*
a) c)	4087	1594
b)	3159	1500[(2)]
d)	5727	1297
e)	11470	1542
[(1)]Slip modulus taken as linear stiffness between 0.1-0.4Fmax [(2)]Slip modulus in accordance with BS EN 26891		

a) b)



d)

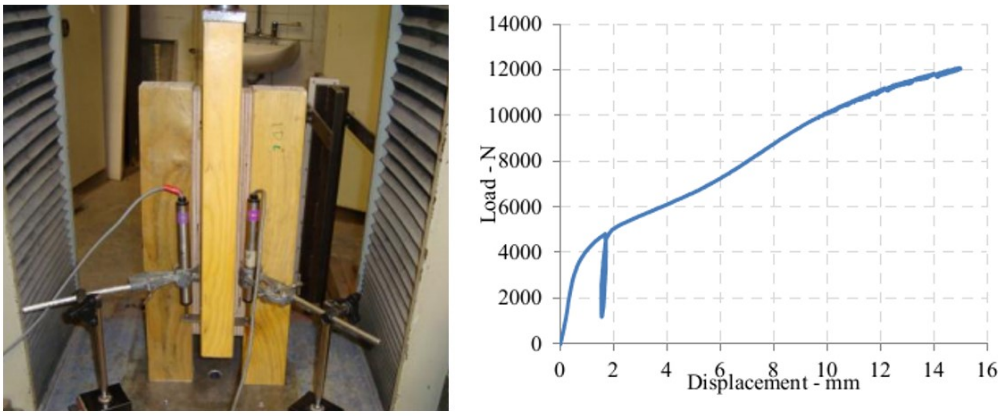


Figure 1 Test execution and displacement results of isolated sole plate components

Table 2. Mechanical properties for sole plate fasteners

Fastener type	Tensile strength f_u *(N/mm²)*	Yield moment M_y *(Nmm)*	Withdrawal strength[(1)] *(N/mm²)*	Pull-through strength[(1)] *(N/mm²)*
3.0x90mm SWN	894	5126	6.4	50.1
4.4x115mm STS	1079	21562	18.2	30.5
7.5x100mm EXN	1290	44845	3.58(2)	50.0
[(1)]Values given refer to the performance of the fastener when installed in C16 timber [(2)]Withdrawal strength given when installed in DAB 7N				

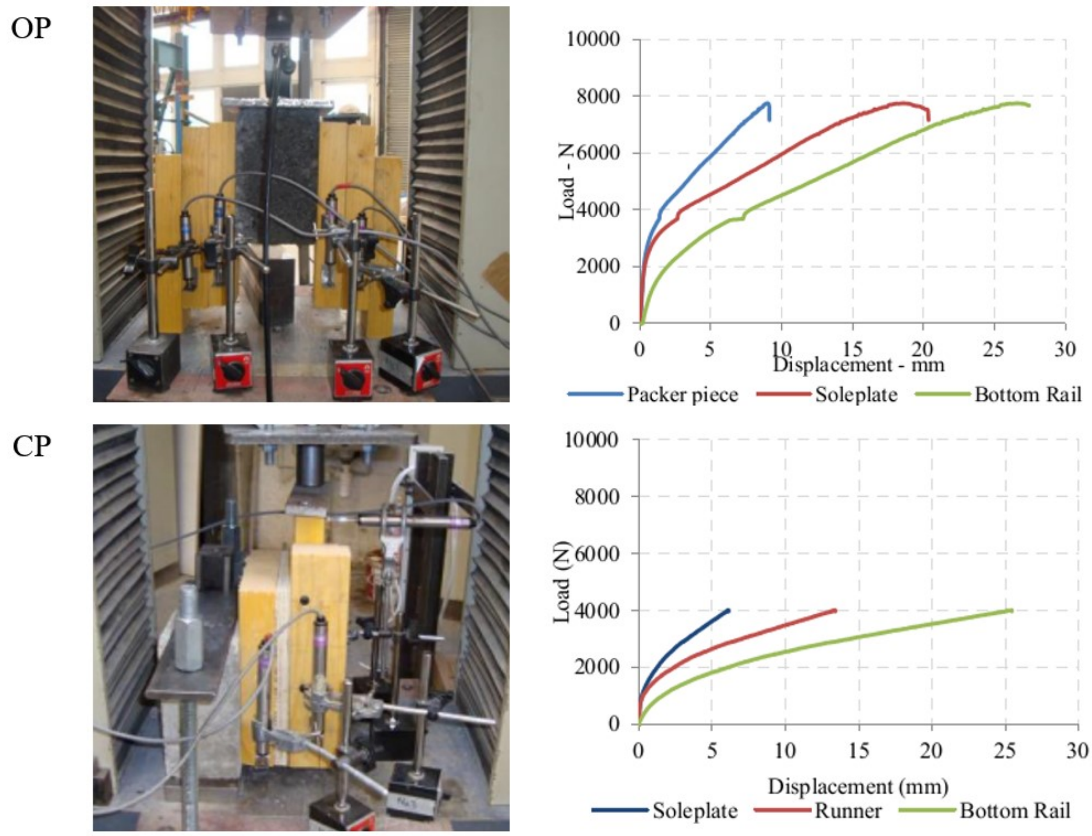


Fig. 2. Test execution and displacement results for full open and closed panel sole plate details.

Table 3. Strength and stiffness results for both open and closed panel SPFD

Sole plate	Ultimate strength f_{max} *(N)*	Slip modulus K_{ser} *(N/mm)*
OP	2841	1364
CP	4036	745

X. Timber Frame Quality Assurance Procedure

Institute for Sustainable Construction

Edinburgh Napier UNIVERSITY

Quality Assurance Procedure

1. - Nailing schedule

Nails: 2.9x50mm smooth wire
Spacing: 75/150mm c/c

2. - Timber frame

Timbers:	T1: 95x45mm [Q1, Q2, Q3]	T2: 45x45mm [Q3]
Frame fixings:	T1: 2no 3.1x90mm ring-shanked	
	T2: 1no 3.1x90mm ring-shanked	
Studs:	600mm c/c	
Frame binders:	Plywood 19x200mm [Q2, Q3]	
Sheathing:	OSB/3 9mm 2no 1.2x2.4m	[Vertically laid]

3. - Check List

- ☐ Wall panel dimensions 2.4x 2.4m
- ☐ Wall panel **squareness** (both diagonals = 3.4m)
- ☐ Sheathing fasteners **evenly** spaced in the perimeter at 75mm c/c and internally at 150mm
- ☐ Minimum edge distance nail to OSB of **10mm** for **all edges**
- ☐ Ensure OSB faces are **flush**
- ☐ Ensure right airgun pressure to **avoid overdriven** nails

XI. Timber Racking Wall Design Manual

1.1 Notes to the calculation

1.1.1 Scope

This calculation determines the structural shear capacity of a sheathed timber frame wall panel in platform timber frame buildings acting as elements of a lateral wind force resisting system in accordance with the design provision contained within the Published Document 6693-1:2012, UK Non-Contradictory Complementary Information to Eurocode 5: Design of timber structures.

The calculation determines the resistance of a single wall panel with no storeys above. In order to determine the total racking resistance of a wall assembly as shown in the below figure, several single wall panel calculations must be manually added.

1.1.2 References

- Published Document PD 6693-1 as UK Non-Contradictory Complementary Information to Eurocode 5: Design of timber structures (2012 Publication).
- Eurocode 5: Design of timber structures (BS EN 1995-1-1:2004, Edition 2008) and UK National Annex.
- Eurocode 1: Actions on structures – General actions (BS EN 1991-1-1, Edition 2009) and UK National Annex.
- Eurocode 1: Actions on structures – Wind actions (BS EN 1991-1-4, Edition 2010) and UK National Annex.
- Eurocode 0: Basis of structural design (BS EN 1990, Edition 2005)
- Structural timber – Strength classes (BS EN 338, 2009).
- Wood-based panels for use in construction (BS EN 13986, 2004).
- Timber structures – requirements for dowel type fasteners (BS EN 14592, Edition 2012).
- Code of practice for dry lining and partitioning using gypsum plasterboard (BS 8212).
- Gypsum plasterboards – Definitions, requirements and test methods (BS EN 520).
- Lancashire, R. and Taylor, L., - Timber frame construction. 5th Edition, High Wycombe, BM Trada Technology, 2011.

1.1.3 General notes

- Simplified method of analysis for shear wall in platform timber frame buildings. The panels consist of timber framing connected on one or both sides to a wood-based sheathing material or solely of plasterboard. A racking wall panel may comprise of a single wall diaphragm, if the panel contains any discontinuity, for example a door or a large opening then the panel is considered to have multiple diaphragms.
- The full length of the building wall is referred in this document as "building side wall". The building side wall may comprise of one or more racking wall panels. A racking wall panel with discontinuities is formed by two or more shear wall diaphragms. Normally, a building side wall corresponds to a racking wall panel.
- For overturning and racking calculations, additional permanent load can be added from both returning walls and holding-down straps from the bottom rail of the shear wall diaphragm.
- The user can select three different sole plate fixing detail. Shear resistance per metre run can be altered by selecting different fastener spacings and the number of fasteners in the direction perpendicular to the sole plate. In case that closed panel sole plate detail is selected, this detail shall comply with BS EN 12436:2002 – Adhesives for load-bearing timber structures.
- For racking wall subjected to service class 3, a fibre saturation factor of 2/3 is applied to the characteristic pull-through and withdrawal strength values.

1.1.4 Assumptions and limitations

- The timber frame wall must consist of studs, not exceeding 610 mm centre to centre, between horizontal top and bottom timber rails. All of the timber in the frame shall have a minimum strength class of C16, in accordance with BS EN 338. General sheathing arrangements shall comply with document PD 6693-1.
- Small unframed openings with every side less than 150 mm or framed openings with every side less than 300 mm are ignored in the racking design.
- There must be less than two sheathing sheets of a length less than 600 mm used consecutively along the full length of any shear wall diaphragm.

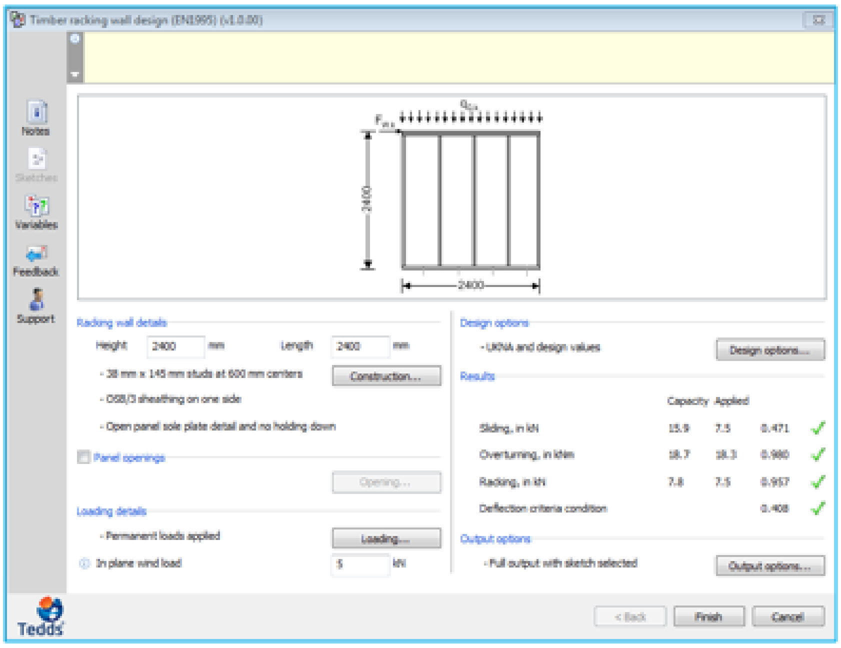
- The total distributed wind load applied to each racking wall panel forming the full building length wall shall be proportional to the racking strength of that racking wall panel for the full building length.
- The load duration of the applied wind action is considered to be "instantaneous " as defined in the Eurocode standard BS EN 1995-1-1:2004+A1
- This document does not consider forces applied to orthogonal racking walls as a result from torsional moment caused by eccentricity between the centroids of the wind load and the summary of the wall racking resistances.
- If a holding down strap is specified, it is assumed that the strap is fixed to the primary sheathing side of the wall panel.
- The default coefficient of friction between the timber members and the damp-proof course is taken as 0.4 when calculating the design sliding resistance.
- The overturning resistance of each shear wall diaphragm is provided by the design withdrawal capacity of the bottom rail to floor connection. This withdrawal resistance must be mobilised by the underlying construction including at foundation level. This calculation ensures overturning stability of the shear wall diaphragm.
- Although in this calculation the racking contribution from masonry cladding systems is ignored, a reduction of the wind applied to the racking wall panel may be considered from masonry shielding.

1.1.5 Revision History

Version	Date	Description
1.0.00	01/12/2013	Original version

1.2 User interface

The main interface of the Timber Racking Wall Design is divided into six groups:



<u>Racking wall details</u>

The parameters related to the construction and dimensions of the wall are defined in this section. The button *construction...* opens up a new interface containing the input details for the timber frame, the sheathing and fasteners and the type of the sole plate base fixing. Currently, three sole plate details are activated. Nevertheless, you can manually introduce any other detail: select open panel construction, enter 1 mm for spacing of C1 and amend the C2 spacing according to your own specifications.

[Note: The equivalent spacing according to your sole plate detail is 1.5kN/m divided by the characteristic shear capacity of your sole plate detail, in kN/m. Please note that the withdrawal capacity of the sole plate will not be valid requiring the inclusion of a holding down strap.]

Panel openings.
The button *Opening…* is activated if the check box in the Panel openings' heading is ticked. The new interface allows for the inclusion of up to five openings. To add a new opening you can either click on the *Add…* button to create an opening after the last one defined or *Insert…* to create an opening right after the opening selected in the previous drop list.

Loading detail
The loading button enables the introduction of the permanent load acting on top of the panel in kN/m, the self-weight of the panel in kN/m^2 and finally, any uplift forces, such as wind suction on the roof, in kN/m. The self-weight input presents a drop list control that automatically calculates the standard weight of the panel depending on the current wall configuration. Insulation or cladding is not added and typical increment on weight largely varies depending on wall thickness and type of materials.

Also in this section, the in-plane wind load is set. This information may be needed in certain cases in order to determine the Kw factor for the racking strength. Nevertheless, most of the cases you can input a very low load (0.001 kN) in order to learn the racking capacity of that particular diaphragm.

Design options
You can determine if the final calculation is given as design or as characteristic value. Characteristic values are given if the *Unfactored design check* control is ticked. In this section, service classes for the timber frame and the sole plate are also defined as described in Eurocode 5 cl. 2.3.1.3.

At the moment, the racking capacity of a timber frame wall is calculated according to the United Kingdom (PD 6693-1).

Results
Commonly, Tedds applications show the results in the same fashion. Three columns are shown on the right middle section indicating the structural capacity and the applied stresses, and the factor of utilisation for every design check. In the timber racking design those checks are sliding, overturning and racking. A fourth check indicates the factor of utilisation of the deflection criteria according to the PD 6693-1.

A green tick shows that the factor of utilisation for that design check is less or equal to unity.

Output options
Finally, in this section you can give a title to the current calculation. Also, this section allows you to select the output format of the calculation. The different details are Full (comprehensive report) or Summary (brief report). In general, three pages of a full report equals to one page of a summary report.

A sketch of the racking wall elevation and cross section will be included in the final calculation report, regardless its output detail, if the relevant check box control is ticked.

1.3 Practical example

In this section, you will determine the total racking capacity of a wall assembly. The complete racking wall (North facing side) is comprised by four wall diaphragms and a door discontinuity. Run Tedds for Word and add the design of the four shear walls into one Tedds document. The plan and elevation of the modular dwelling is given in Figure 1.

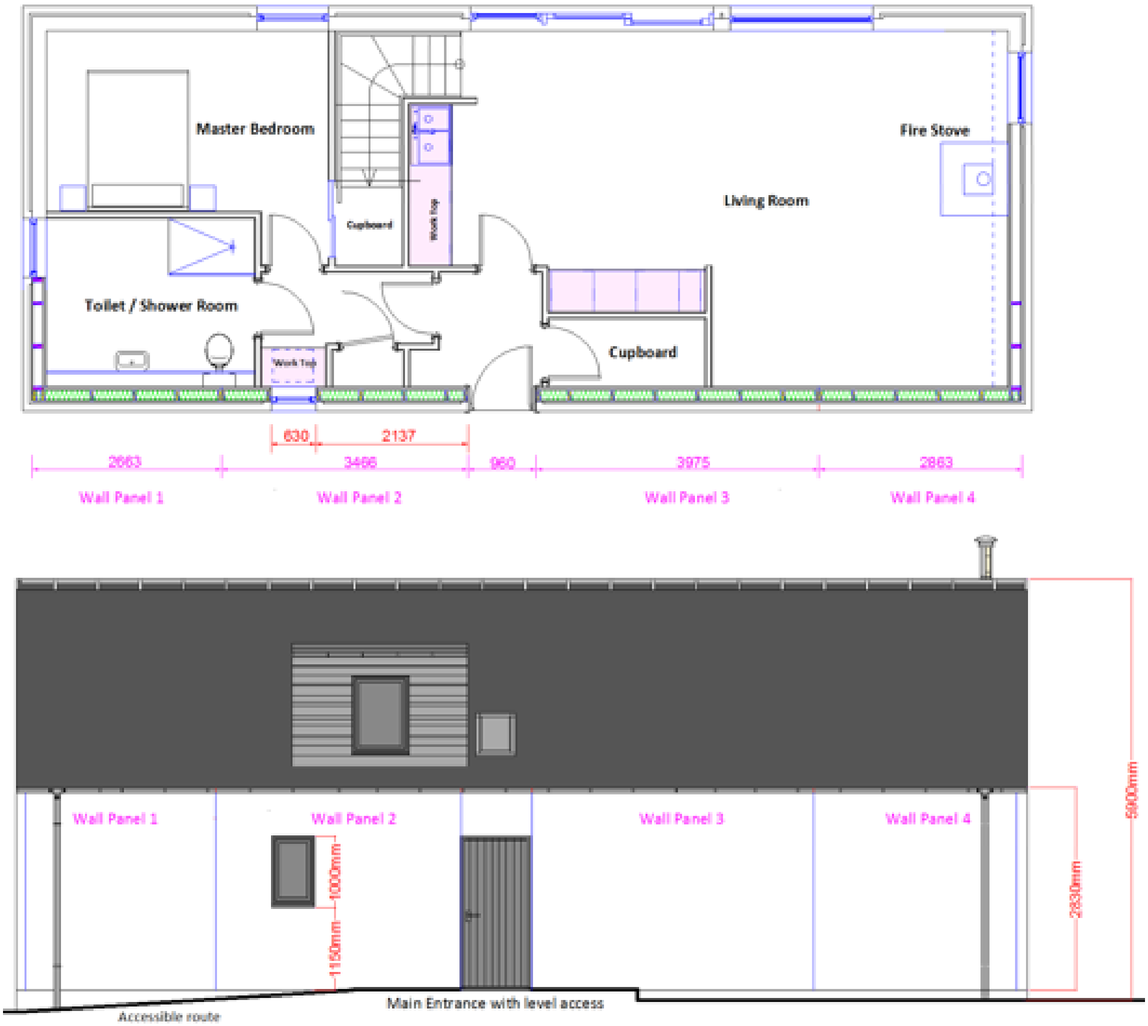


Figure 1 Plan and elevation of dwelling

1.3.1 General data

<u>Wall panel specification</u>

Section size:	38 x 145 mm, strength class C16, @ 600 mm cc
Sheathing:	1No 9mm OSB/3
	Fasteners, 2.8 x 50 mm ring shanked BRT nail @150 mm cc
Soleplate:	Open panel construction (insulation on-site)
	Sole plate to foundation: 1No 3.5 x 70 mm smooth round wire express nail @150 mm cc
	Bottom rail to sole plate: 1No 3.1 x 90 mm ring shanked BRT nail @300 mm cc

<u>Opening:</u>

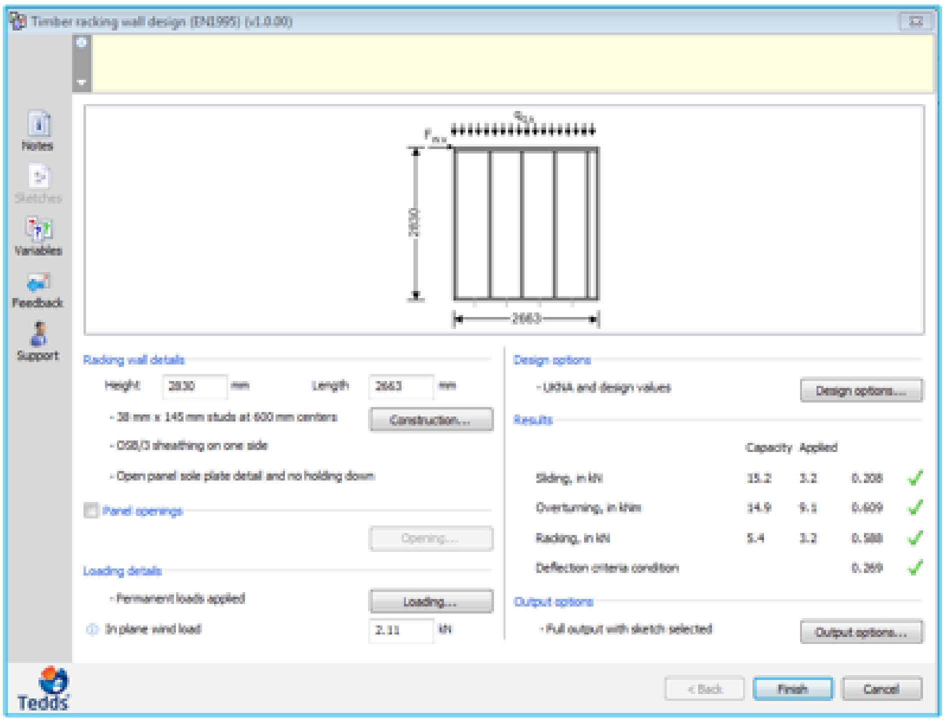
Dimension:	1000 x 630 mm
Distance to sole plate:	1150 mm
Distance to lead stud:	699 mm

<u>Loading:</u>

Variable (w):	11.25 kN (on top corner of the wall assembly), no wind uplift
Permanent:	3.65 kN/m
Service class:	SC 2

1.3.2 Racking wall 1

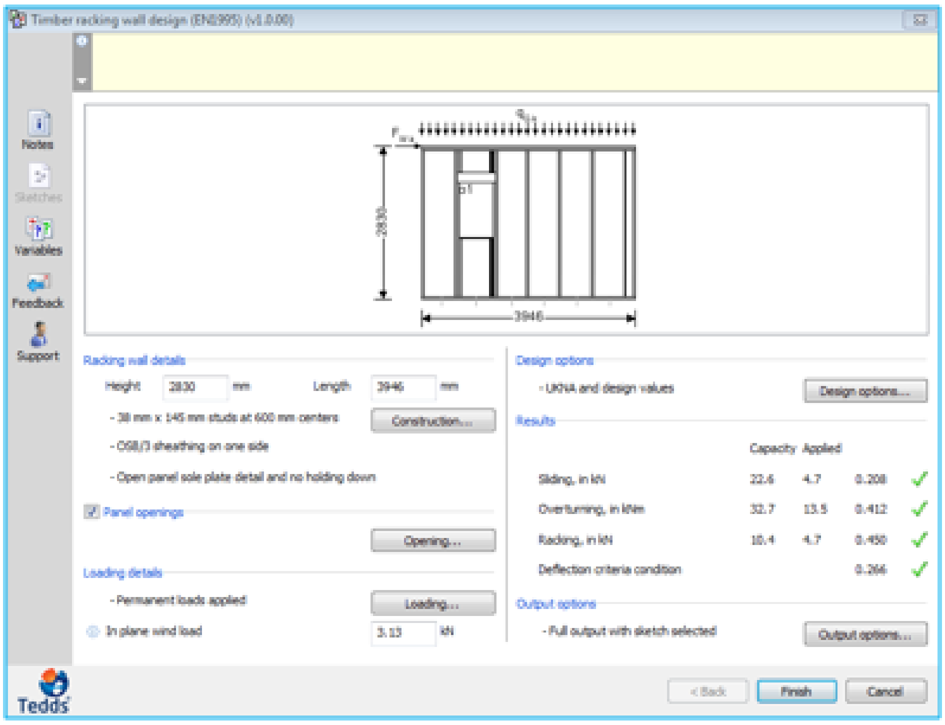
Assumed wind load: 11.25 kN × 2.663 m / 14.20 m = 2.11 kN



Racking design strength: 5.38 kN

1.3.3 Racking Wall 2

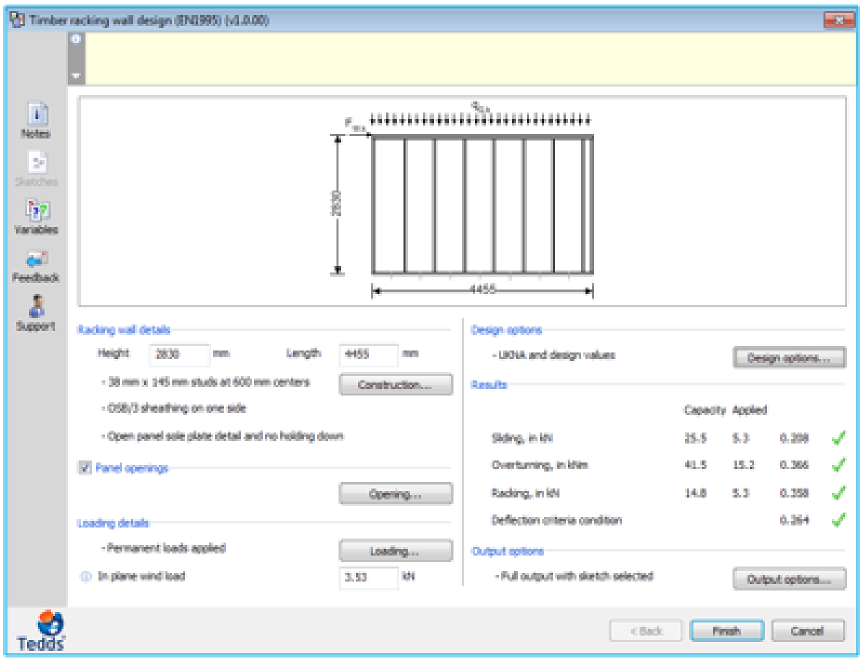
Assumed wind load: 11.25 kN × 3.946 m / 14.20 m = 3.13 kN



Racking design strength: 10.44 kN

1.3.4 Racking Wall 3

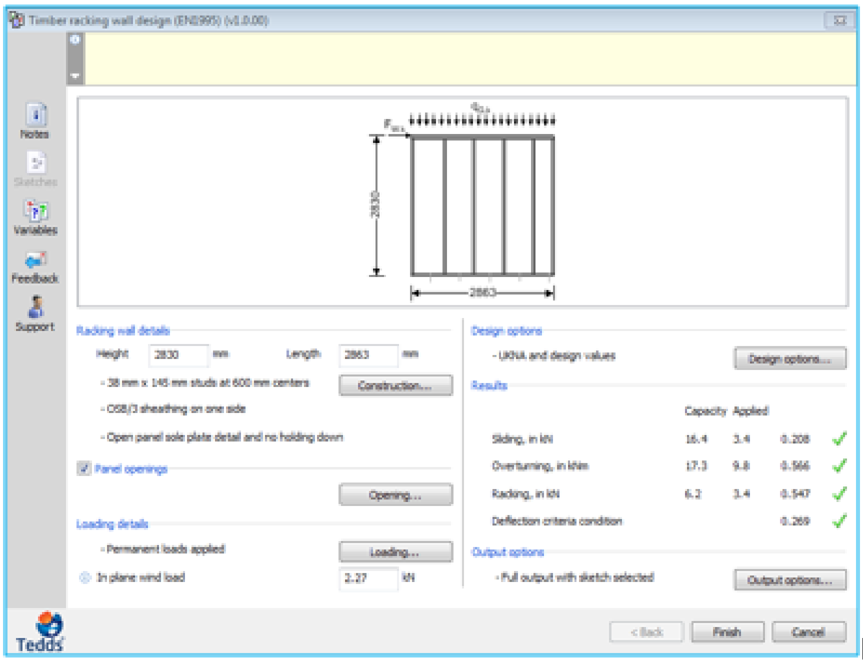
Assumed wind load: 11.25 kN × 4.455 m / 14.20 m = 3.53 kN



Racking design strength: 14.79 kN

1.3.5 Racking Wall 4

Assumed wind load: 11.25 kN × 2.863 m / 14.20 m = 2.27 kN



Design racking strength: 6.21 kN

1.3.6 Total racking resistance of wall assembly

For this example, we have assumed that the lateral load capacity of each wall panel is proportional to its length. Other approach, especially when the rigidity of the wall panels significantly varies, assumes instead that the shear resistance taken by each wall is proportional to its stiffness.

Timber frame racking panel design (EN1995)

Tedds calculation version 1.0.01

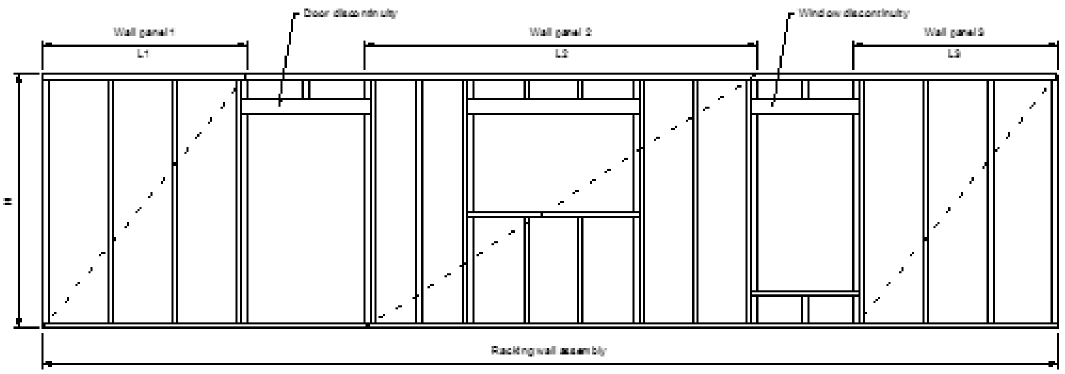
This calculation was developed in partnership with Edinburgh Napier University and funded by:

EPSRC Engineering and Physical Sciences Research Council

Scope

Calculation which determines the structural shear capacity of a sheathed timber frame wall panel in platform timber frame buildings acting as elements of a lateral wind force resisting system in accordance with the design provision contained within the Published Document 6693-1:2012, UK Non-Contradictory Complementary Information to Eurocode 5: Design of timber structures.

The calculation determines the resistance of a single wall panel with no storeys above. In order to determine the total racking resistance of a wall assembly as shown in the below figure, several single wall panel calculations must be manually added.



References

- Published Document PD 6693-1 as UK Non-Contradictory Complementary Information to Eurocode 5: Design of timber structures (2012 Publication).
- Eurocode 5: Design of timber structures (BS EN 1995-1-1:2004, Edition 2008) and UK National Annex incorporating Corrigendum No. 2.
- Eurocode 1: Actions on structures – General actions (BS EN 1991-1-1, Edition 2010) and UK National Annex.
- Eurocode 1: Actions on structures – Wind actions (BS EN 1991-1-4, Edition 2011) and UK National Annex incorporating Corrigendum No. 1.
- Eurocode 0: Basis of structural design (BS EN 1990, Edition 2010)
- Structural timber – Strength classes (BS EN 338, 2009).
- Wood-based panels for use in construction (BS EN 13986, 2004).
- Timber structures – requirements for dowel type fasteners (BS EN 14592, Edition 2012).
- Code of practice for dry lining and partitioning using gypsum plasterboard (BS 8212).
- Gypsum plasterboards – Definitions, requirements and test methods (BS EN 520).

- (Lancashire, R. and Taylor, L., 2011) Timber frame construction. 5th Edition, High Wycombe, TRADA Tech. Ltd, October 2011.

General notes

Simplified method of analysis for shear wall in platform timber frame buildings. The panels consist of timber framing connected on one or both sides to a wood-based sheathing material or solely of plasterboard. A racking wall panel may comprise of a single wall diaphragm, if the panel contains any discontinuity, for example a door or a large opening then the panel is considered to have multiple diaphragms.

The full length of the building wall is referred in this document as "building side wall". The building side wall may comprise of one or more racking wall panels. A racking wall panel with discontinuities is formed by two or more shear wall diaphragms. Normally, a building side wall corresponds to a racking wall panel.

For overturning and racking calculations, additional permanent load can be added from both returning walls and holding-down straps from the bottom rail of the shear wall diaphragm.

The user can select three different sole plate fixing detail. Shear resistance per metre run can be altered by selecting different fastener spacings and the number of fasteners in the direction perpendicular to the sole plate. In case that closed panel sole plate detail is selected, this detail shall comply with BS EN 12436:2002 – Adhesives for load-bearing timber structures.

For racking wall subjected to service class 3, a fibre saturation factor of 2/3 is applied to the characteristic pull-through and withdrawal strength values.

Assumptions and limitations

The timber frame wall must consist of studs, not exceeding 610 mm centre to centre, between horizontal top and bottom timber rails. All of the timber in the frame shall have a minimum strength class of C16, in accordance with BS EN 338. General sheathing arrangements shall comply with document PD 6693-1. If solid timber is used for the frame, material shall be indiviudally graded and marked.

Small unframed openings with every side less than 150 mm or framed openings with every side less than 300 mm are ignored in the racking design.

There must be less than two sheathing sheets of a length less than 600 mm used consecutively along the full length of any shear wall diaphragm.

The total distributed wind load applied to each racking wall panel forming the full building length wall shall be proportional to the racking strength of that racking wall panel for the full building length.

The load duration of the applied wind action is considered to be "instantaneous " as defined in the Eurocode standard BS EN 1995-1-1:2004+A1

This document does not consider forces applied to orthogonal racking walls as a result from torsional moment caused by eccentricity between the centroids of the wind load and the summary of the wall racking resistances.

If a holding down strap is specified, it is assumed that the strap is fixed to the primary sheathing side of the wall panel.

The default coefficient of friction between the timber members and the damp-proof course is taken as 0.4 when calculating the design sliding resistance.

The overturning resistance of each shear wall diaphragm is provided by the design withdrawal capacity of the bottom rail to floor connection. This withdrawal resistance must be mobilised by the underlying construction including at foundation level. This calculation ensures overturning stability of the shear wall diaphragm. It is assumed that the full length of the sole plate provides shear transfer and friction.

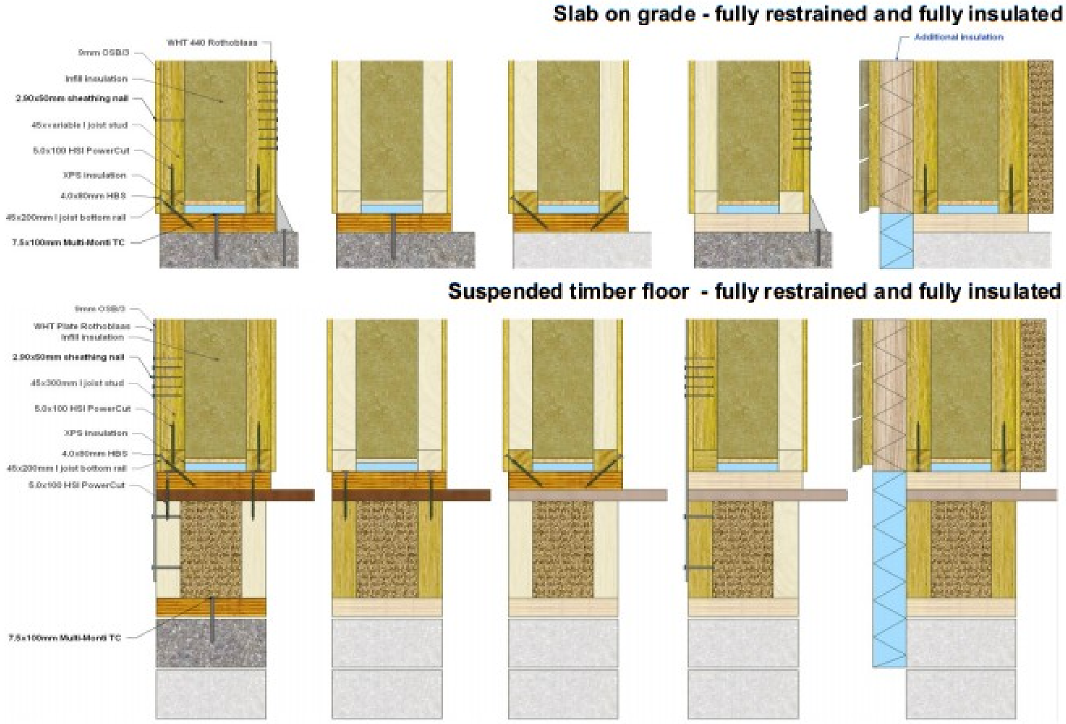
Although in this calculation the racking contribution from masonry cladding systems is ignored, a reduction of the wind applied to the racking wall panel may be considered from masonry shielding.

Revision History

Version	Date	Description
1.0.01	26th March 2014	Minor correction to calculation of destabilising moments on the wall. Addition of destabilising moment on top of the head binder when no dead load is applied. Minor correction to sheathing data list for secondary member. Plasterboard sheathing is now not allowed if primary member is not also plasterboard. UK National Annex to EN 1995-1-1 updated to Corrigendum No. 2 with assumption added to notes for solid timber material. Minor correction to clarify holding down metalwork and sole plate fastener specification.
	27th February 2014	Minor correction to sheating data list for OSB/3 thickness 11mm which included a variable related to OSB/2. This did not affect the calculation.

XII. Integrated Set of Solutions: Simplified Details

RTC Closed Panel Details



Structural

RTC Racking (k)
1x OSB/3 - 9mm

wall length (m)

nail s (mm) / kN	1.2	1.8	2.4	3.0	3.6	4.2	4.8
75	4.7	8.6	14.8	20.8	39.3	45.8	52.4
100	4.7	8.5	13.7	25.0	30.0	35.1	40.1
150	4.7	10.4	13.9	17.4	20.8	24.3	27.8

RTC Racking (k)
2x OSB/3 - 9mm

nail s (mm) / kN	1.2	1.8	2.4	3.0	3.6	4.2	4.8
75	4.8	8.7	13.7	19.6	26.6	34.5	74.8
100	4.8	8.9	13.9	19.9	26.9	60.5	70.1
150	4.8	8.6	13.5	19.2	36.5	42.5	48.6

Common considerations		
5 kN/m permanent load		
Self-weight included		
C24 flanges		
No openings considered		
3.1x50mm smooth nails		
Fully restrained straps	22kN	5.4kN

Thermal

RTC fully insulated
Slab on grade

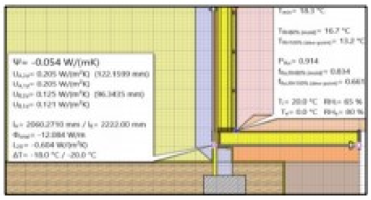
timber frame thickness (mm)

	195	245	300
U-Value	0.123	0.104	0.090
ψ sole plate	-0.097	-0.086	-0.105
fRsi_warsaw	0.90	0.92	0.94
fRsi_edin	0.91	0.93	0.95
fRsi_bilbao	0.93	0.95	0.96
Sd value	13	13	13
Heat storage	21	22	24

Common considerations
Mineral wool 032 in frame
VCL Sd= 10m internally
60 mm wood fibre 046
60 mm XPS 037 at edge
Ventilated timber cladding
Insulated I-joist
EPDM dpc around concrete

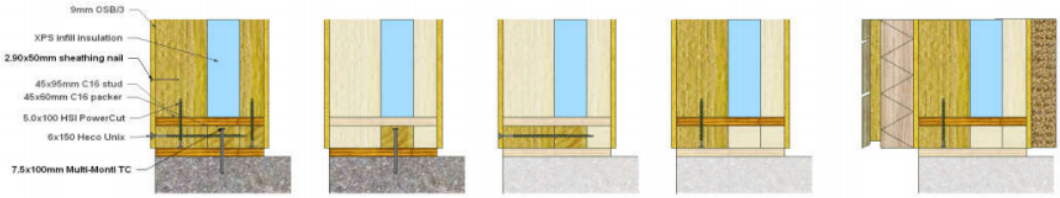
RTC fully insulated
Suspended floor

	195	245	300
U-Value	0.123	0.104	0.090
ψ sole plate	-0.054	-0.058	-0.063
fRsi_warsaw	0.87	0.89	0.91
fRsi_edin	0.89	0.91	0.92
fRsi_bilbao	0.91	0.92	0.93
Sd value	13	13	13
Heat storage	21	22	24

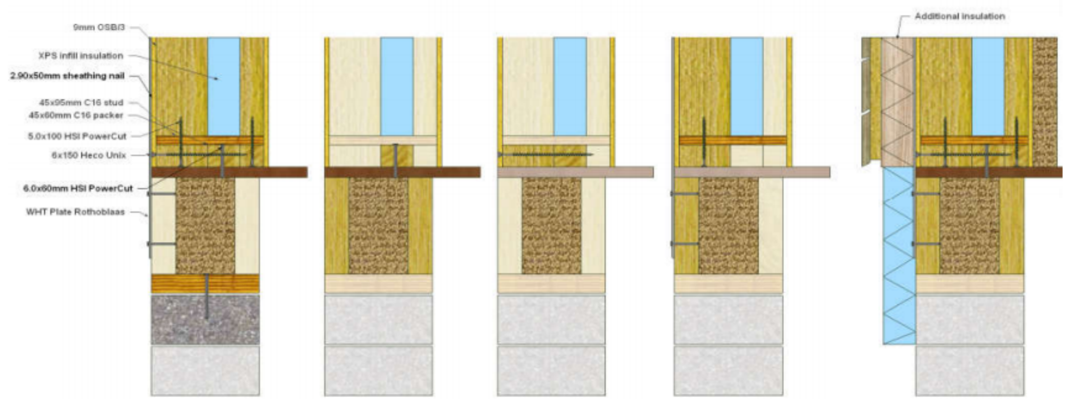


K2 Closed Panel Details

Slab on grade - fully restrained and fully insulated



Suspended timber floor - fully restrained and fully insulated



Structural

K2 Racking (k)
1x OSB/3 - 9mm

wall length (m)

nail s (mm) / kN	1.2	1.8	2.4	3.0	3.6	4.2	4.8
75	4.4	7.8	12.0	17.1	39.3	45.8	52.4
100	4.4	7.8	12.0	25.0	30.0	35.1	40.1
150	4.3	10.4	13.9	17.4	20.8	24.3	27.8

K2 Racking (k)
1x OSB/3 - 15mm

nail s (mm) / kN	1.2	1.8	2.4	3.0	3.6	4.2	4.8
75	4.4	7.9	12.1	17.3	23.2	54.2	61.9
100	4.4	7.9	12.1	29.6	35.5	41.5	47.4
150	4.4	7.8	16.4	20.5	24.6	28.7	32.9

Common considerations		
5 kN/m permanent load		
Self-weight included		
C24 flanges		
No openings considered		
3.1x50mm smooth nails		
Fully restrained straps	22kN	5.4kN

Thermal

K2 fully insulated
Slab on grade

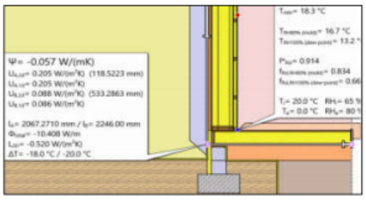
timber frame thickness (mm)

	195	245	300
U-Value	0.121	0.102	0.086
ψ sole plate	-0.085	-0.048	-0.115
fRsi_warsaw	0.90	0.91	0.92
fRsi_edin	0.91	0.92	0.93
fRsi_bilbao	0.93	0.94	0.95
Sd value	15	15	15
Heat storage	23	23	23

Common considerations
Mineral wool 032 in frame
VCL Sd= 10m internally
60 mm wood fibre 046
60 mm XPS 037 at edge
Ventilated timber cladding
Insulated I-joist TF= 10%
EPDM dpc around concrete

K2 fully insulated
Suspended floor

	195	245	300
U-Value	0.121	0.102	0.086
ψ sole plate	-0.046	-0.051	-0.057
fRsi_warsaw	0.87	0.88	0.89
fRsi_edin	0.89	0.90	0.91
fRsi_bilbao	0.91	0.92	0.93
Sd value	15	15	15
Heat storage	23	23	23



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