Racking Performance of Platform Timber Framed Walls

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A thesis submitted in partial fulfilment of the requirements of Edinburgh Napier University, for the award of Doctor of Philosophy

November 2018

Declaration

I hereby declare that this research was entirely conducted by me at the Edinburgh Napier University. No contents of this research have been used for the award of any other degree in any other University. To best of my knowledge, this research contains no materials previously contributed by any other authors, except where explicit references are provided.

(Roshan Lal Dhonju)

Signature ______________________________

Date __________________________________

Dedication

This Thesis is dedicated to my late father Karma Lal Dhonkaji and my late eldest brother Bhushan Lal Dhonju.

Abstract

Platform timber frame construction is considered an efficient building method for multi-storey dwellings where timber walls and diaphragms provide the overall stability for the structure to resist lateral forces such as those generated by wind action. Although, so far, many research studies have been conducted on the racking performance of platform timber frame walls, there remain some gaps in knowledge in a number of key areas which this research has aimed to address.

A quantitative assessment of the racking performance of partially anchored timber framed walls has been carried out via experimental test campaign. Timber framed walls, sheathed with oriented strand board (OSB) panels and/or British gypsum plasterboards (PB) were constructed from a combination of material types under different loading configurations and tested according to standardized procedure. The experimental study was designed to examine the influence of a range of geometrical parameters, such as (panel-to-frame) fastener size and spacing, wall length, arrangement of studs and horizontal members, and the effect of vertical loading on the racking strength and stiffness of the walls.

When subjected to a vertical load, the wall's racking strength has been found to be more sensitive to variations in the fastener spacings, compared to the racking strength of similar walls without applied vertical loads. Conversely, it is racking stiffness to be more sensitive to variations in fastener spacings when no vertical load is applied to the wall. In such a case, the stiffness increase was up to three folds when the fastener spacing was reduced from 150 to 50 mm. However, such gain in stiffness did not occur in similar walls when they were subjected to a vertical loading of 25 kN, with stiffness increasing by only 24%.

The comparison of experimental results, with the results from the UK design code formulae, showed that, on average, the design code underestimated the racking strength by 25% for walls under vertical loading of 25 kN and by 54% for walls without any vertical loading.

The influence of test procedure on the racking performance of timber framed walls was also examined in an extensive experimental and analytical programme which investigated the compatibility and suitability of the test method in BS EN 594:2011 with the racking design method of BS 5268-6.1:1996. The research findings led to appropriate recommendations for determination of the design racking values from the test results.

The effects of openings/discontinuities caused by windows and doors on racking performance of OSB walls with and without the use of trimmers, as well as spreaders were also examined. The results led to determination of a relationship between the size of the opening for a window or a door and the percentage reduction in the racking performance of the wall.

Finally, this research examined the racking strength and stiffness of a recently developed shear wall referred to as “Mid-ply wall”. Comparing the performance characteristics of the Mid-ply walls with the “standard walls”, the Mid-ply walls performed significantly better in both strength and stiffness terms, therefore providing a considerable potential for use in the UK and European timber frame construction.

Acknowledgement

First and foremost, I am indebted to my supervisor, Prof. Abdy Kermani, for offering me the opportunity to carry out this research and for his constant support, expert advice, guidance, constructive criticism, and encouragement throughout this research. I would also like to give my sincere gratitude to my supervisors Dr. Bernardino D'Amico and Dr Jack Porteous, for their helpful advice in publication of the research results and for discussions and feedback. I would also like to thank my other co-supervisor Dr. Ben Zhang for his support and feedback.

Special thanks are also due to Dr. Guillaume Coste and his company for supplying the materials for experimental tests. I would also like to thank all the Technical staff team of Edinburgh Napier University who directly and indirectly helped and supported me in the laboratory during experimental tests. And in particular special thanks are due to my past line manager Jim Goodlet who always supported and helped me in organizing the funding for this research work.

Thanks are also due to all the under graduate and post graduate students who assisted me in setting up the racking walls for experimental tests as a part of fulfilment of the requirements for their dissertation in Heavy Structure Lab at Merchiston Campus.

I would also like to acknowledge the Centre for Timber Engineering and School of Engineering and the Built Environment (SEBE) of Edinburgh Napier University for providing me a fee waiver scholarship for conducting this research. I am indebted to the administration staff for their co-operation in all kinds of administrative works.

Finally, my deepest gratitude to my parents (Late Mr. Karma Lal Dhonkaji and Mrs. Krishna Kumari Dhonkaji) and my brothers Late Bhushan Lal Dhonju, Saileswor Lal Dhonju, Dr. Sudarshan Lal Dhonju and Er. Digdarshan Lal Dhonju and all my family for their support and encouragement without whom, it would not have been possible to accomplish this study. Last but not the least I would like to thank my wife Dr. Pooja Shrestha (Dhonju) for her unrelenting encouragement and support during this study. Also, thank you my cute little daughter Prisha Dhonju for bringing happiness when I was writing my thesis.

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Abbreviations

AV/B OSB	Air Vapour Barrier oriented strand board
BR	Bottom rail
BS EN	British Standard European Norm
BSI	British Standards Institution
Charc.V.	Characteristic value
C.V.	Coefficient of variation
CLT	Cross Laminated Timber
EC5	Euro code 5
EYM	European Yield Model
FR OSB	Fire resistant oriented strand board
GFB	Gypsum fiberboard
GWB	Gypsum wall board
IS	Intermediate stud
LS	Leading stud
LVDT	Linear variable differential transformer
MOE	Modulus of elasticity
MPSW	Mid-ply shear wall
MS	Middle stud
OSB	Oriented strand board
PB	Plaster board
RS	Rear stud
SLS	Serviceability Limit State
TR	Top rail
UDL	Uniformly distributed loading
UKNA	UK National Annex
ULS	Ultimate Limit State

Definitions of terminologies

Point load: Point load is a force applied at a single infinitesimal point at a set distance from the ends of the beam. In this research the point load are at each stud position from a total vertical load of 25kN providing a proportion of that load to each stud.

Uniformly distributed load: Uniformly distribute load is a force applied over a length, i.e. force per unit length. In this research, UDL is 25kN over 2.4m wall length hence 10.42kN/m.

Racking load: Racking load in timber frame wall is the strength of the wall that can resist the lateral forces such as those generated by wind actions.

Racking resistance: In the timber frame construction, walls provide resistance to the racking, lateral forces such as those generated by wind actions. The amount of racking resistance a wall provides entirely depends on its type of construction. It is a measure of a building system's ability to resist wind loads which is also termed as Racking strength.

Shear wall: A wall intended to resist the racking loads such as those generated by wind actions.

Ultimate racking load: In timber frame wall, ultimate racking load is the maximum load the wall can resist when applied the lateral forces such as those generated by wind actions.

Shear forces: In timber frame walls, the shear forces are produced when two surfaces are move over each other due to the forces such as those generated by wind actions.

Shearing resistance: Load per unit length

Maximum load: It is the greatest load the wall can support.

Notations

Δh	racking deformation of the wall
b_l	wall length in m
b_i	the wall panel width
d	nominal nail diameter
d	nail diameter
F_{stiff}	racking stiffness
f_{ax}	withdrawal capacity of the connections fixing the wall to the underlying structure per unit length
$F_{ax,Rd}$	design value of axial withdrawal capacity of the fastener
$F_{ax,Rk}$	characteristic axial withdrawal capacity of the fastener
F_{est}	estimated maximum loads
$F_{f,Rd}$	design load-carrying capacity per fastener in wall diagram
F_{fail}	maximum racking strength load at failure condition
f_h	average compressive strength
$f_{h,1,k}$	characteristic embedment strength of the timber frame members
$f_{h,2,k}$	characteristic embedment strength of the sheathing panel
$f_{h,k}$	characteristic embedment strength which is the product of fastener penetration length and diameter
$F_{i,v,Rd}$	design load-carrying capacity per shear plane per fastener, Design racking load capacity
$F_{i,v,Rd,max}$	design racking strength of the stronger sheathing
$F_{i,v,Rd,min}$	design racking strength of the weaker sheathing
F_{max}	maximum racking load, in newtons
$F_{max,est}$	estimated maximum racking load
$F_{max,min}$	lowest failure (or maximum) racking load
FoS	factor of safety for the type of sheathing or sheathing combination
$f_{p,d,2}$	design shear capacity per unit length of perimeter sheathing fasteners of the second sheathing layer in kN/m
$f_{p,d,t}$	summation of the design shear capacities per unit length of the perimeter sheathing fastener in kN/m
f_{pd}	the panel-to-frame fastener strength per unit length
f_u	tensile strength of wire (minimum of 700 N/mm^2 from manufacturer)
F_v	vertical load
$F_{v,mean}$	mean strength value of the panel-to-frame fasteners

$F_{v,RK}$	characteristic load-carrying capacity per shear plane per fastener
$f_{w,d}$	design withdrawal capacity of bottom rail-to-floor conncection per unit length in kN/m
H	height of the wall
h	height of the wall
H_{wp}	wall panel height (in mm)
K_{109}	modification factor
K_{comb}	sheathing combination factor
k_d	dimension factor for panel
$k_{i,q}$	uniformly distributed load factor for wall *i*
$K_{i,w}$	modification factor taking into account wall length, vertical load and holding –down arrangements
k_n	sheathing material factor
$K_{opening}$	opening modification factor
k_s	slip modulus
k_s	fastener spacing factor
L	length of the wall diaphragm
l_{eff}	effective length
L_i	total length without discontinuities
M	stabilising moment at the leeward side of the wall
$M_{d,stb}$	design stabilizing moment in kNm, about the leeward end of the wall diaphragm from design permanent load, reduced by any vertical component of design wind load
m_i	test value
m_k	characteristic value
$M_{y,Rk}$	characteristic fastener yield moment
n	number of test values
$P_{h,max}$	racking strength of the wall
P_{ULS},	design racking load
Q	total load
R	average racking stiffness loads
R_b	basic racking resistance
s	fastener spacings
s_0	basic fastener spacing in m
S_y	standard deviation

t_1	head-side thickness in a single shear and the minimum of the head side timber thickness and the pointside penetration in a double shear connection panel
t_2	point-side penetration in a single shear connection or the central member thickness in a double shear connection
t_{pen}	point-side penetration length
υ	panel deformation, in millimetres
V_{01}	slip at 10% of estimated maximum load
V_{04}	slip at 40% of estimated maximum load
v_{imod}	modified initial slip
y	mean value
α	the angle between the direction of nail force and the grain
β	ratio between the embedment strength of the members
γ_M	partial coefficient for material properties
μ	ratio between the withdrawal capacity of the connections fixing the wall to the underlying structure per unit length (f_{ax}) and the panel-to-frame fastener strength per unit length (f_{pd})
ρ_k	the characteristics timber density in kg/m^3
LS	leading stud
IS	intermediate stud
MS	middle stud
RS	rear stud
TR	top rail
BR	bottom rail

List of Publications

B. D'Amico; R. Dhonju; Kermani, A.; Porteous, J. and Zhang, B. (2018) "Influence of parameters affecting the racking strength of partially anchored timber framed walls", *WCTE 2018, August 20-23, 2018*.

R. Dhonju; B. D'Amico; Kermani, A.; Porteous, J. and Zhang, B. (2017) 'Parametric Evaluation of Racking Performance of Platform Timber Framed Walls', *Construction and Building Materials-Elsevier,* 12, 75-87.

Coste, G.; Kermani, A.; Porteous, J.; Murray, D. and Dhonju, R. (2016) 'Influence of test procedure on timber wall racking performance', *Structure and Buildings, ICE/ Proceedings-,* 169(12), 925-934.

B. D'Amico; Kermani, A.; Proteous, J.; Dhonju, R. and Zhang, B. (2016) 'Racking performance of platform timber framed walls assessed by rigid body relaxation technique', *Construction and Building Material, Elsevier,* 129, 148-158.

Contributions

Supervision of MSc students during their dissertation:

Contribution to the student's research and experimental works during dissertation.

O'connor, P. (2015) *Optimisation of Mid-Ply Timber Frame Shear Walls with Partial Anchorage.* Master of Science in Timber Engineering, Unpublished thesis, Edinburgh Napier University.

Lopez, M. C. (2013) *Racking Resistance-Timber Frame Walls.* Master of Science in Timber Engineering, Unpublished thesis, Edinburgh Napier University.

Chapter 1 Introduction

1.1 Timber Platform frame

Timber Platform frame construction has been recognized as one of the most effective building methods by building engineers, developers as well as the occupants due to its various advantages such as speed of construction, low cost and better quality, safe, good thermal insulation, use of sustainable materials, lightweight and ease of transportation. The term 'platform frame' derives from the method of construction where floor structures bear onto load bearing wall panels, thereby creating a 'platform' for construction of the next level of wall panels as shown in Figure 1.1. This type of the construction is suited to both low-rise and medium rise buildings. In recent years, buildings of up to six and seven storeys in height have been constructed for residential, institutional and hotel purposes (Structural Timber Association, 2013).

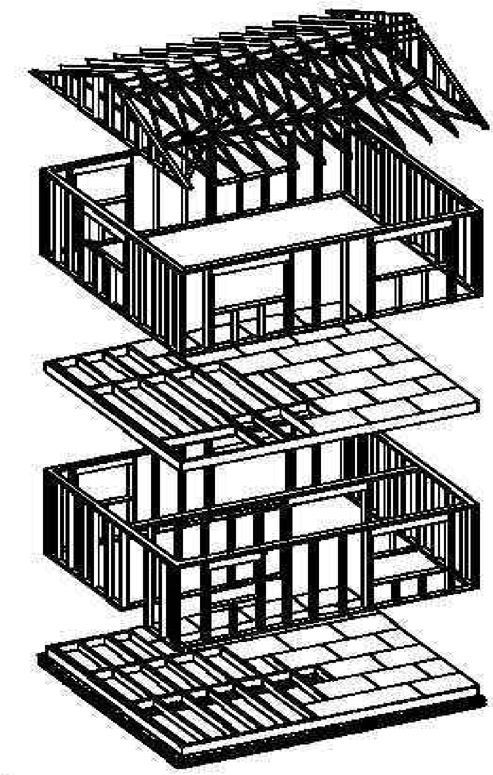


Figure 1.1 Platform frame construction

Source: www.trada.co.uk

Generally, a building method that depends on a timber frame as a basic means of structural support is called timber frame construction. Timber frame buildings are often constructed using prefabricated wall panels, made up of softwood studs at regular centers (typically not more than 600 mm centers) which act as vertical columns, wood-based panel sheathing and a plasterboard (PB) lining jointed together by means of nails and/or screws to use as load bearing elements as shown in Figure 1.2. These buildings are usually subjected to the vertical loads as well as horizontal forces due to wind actions or other lateral forces, e.g. seismic forces (earthquake). In order to resist these loads, in-plane shear resistance is required from the walls, which is mainly provided by sheathings connected to a bare timber frame to form as single wall diaphragms as shown in Figure 1. **3**. The sheathing material is fixed to the frame via mechanical fasteners, such as nails or screws, whereas the wall diaphragms are fixed to the underlying floor or foundation using tie-down anchors and/or shear bolts, depending on the design/construction method used to transfer the load from the top of the building to the foundation.

Figure 1.2 Sheathed timber frame
Source: Structural Timber Association (2017)

1.2 Components of timber frame wall

A timber frame wall diaphragm is comprised of the elements shown in Figure 1. 3 Namely:

- Vertical timber members called "studs" which carry the vertical loads coming from the above floors.

Horizontal timber beams called "rails" both at top and bottom connected to the studs so as to fix the sheathing on it to form as a panel and to support the floors.

- Sheathing are the board materials such as oriented strand board (OSB) which are nailed/screwed to the studs and beams thus enabling the wall panel to resist in-plane horizontal forces (known as racking resistance).
- Soleplates, which are connected to the foundation or sub-deck and transfer the load to the foundation. They also help to locate the position of wall panel.
- Head binders which help to connect the adjacent walls as well as to spread the vertical load from floor joists to the wall studs.
- Cripple studs, header and opening stud helps to transfer the vertical and horizontal loads around openings in the wall panels.

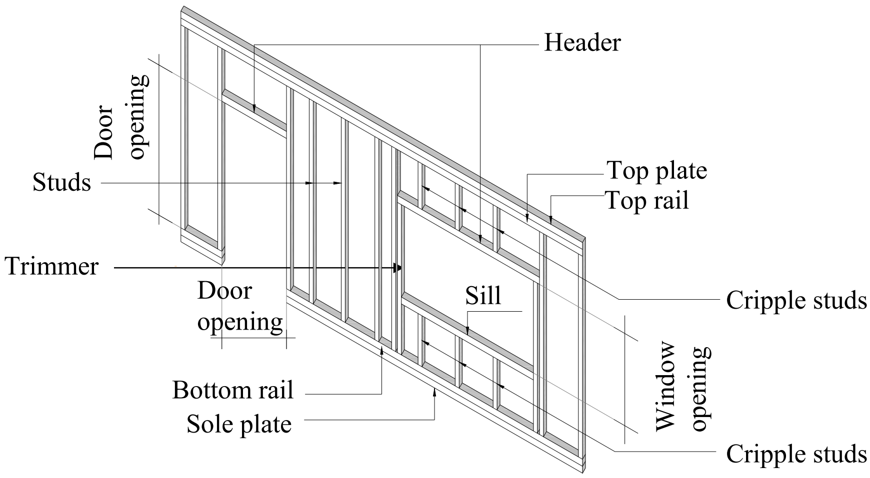


Figure 1. 3 Timber frame wall panels with its components

1.3 Research justification

Most of the European countries, encouraged by EU policies, have set targets to reduce carbon-dioxide emissions and are adopting legislative methods to ensure buildings and materials achieve individual country targets. This has steered the use of timber-based construction materials as an alternative to steel and concrete (Jonsson, 2009). In many developed countries across the world, 70% of new houses are made from timber frame. Because of cost effective and energy efficient method of production with rapid construction, timber framed houses have become the mainstream construction method in many courtiers such as in Scotland, Canada, Sweden, USA, Germany, Austria, and Japan (Holbrook, n.d.). For instance, in Canada and the USA, over 90% of low-rise buildings have used timber frame technology (www.heritagedesigns.co.uk/why-timber-frames). In Scotland, 75% of new houses are timber framed, whereas in the UK, this figure is around 25% (UKEssays, 2013). Due to several benefits for builders, developers and occupants, this type of the building has been widely used, not just for dwellings, but also in the construction of schools, sports halls, hotels, offices, and health centers (TRADA, 2008).

The structural performance of timber framed walls has been subject of investigation since 1940 and considerable research has been carried out to understand and improve the performance of timber shear walls under seismic or wind load. A comprehensive literature review about the evolution of timber shear wall experiments, modelling, reliability analysis from 1983 to 2001 was carried out by Van de Lindt (2004). Many structural analysis programs for timber shear walls were introduced on the basis of lateral behaviour of sheathings (Dinehart and Shenton III, 2000; Gupta and Kuo, 1985; Richard et al., 2002). The experiments done by McCutcheon (1985) shown that racking behaviour of a sheathed wall depends mostly on the lateral load-slip characteristics of the nails that fasten the sheathing to the frame. Dolan and Madsen (1992) conducted full scale shear wall tests; their results shown that sheathing material has an insignificant effect in the working stress range of the shear walls as well as the ultimate strength.

Many experiments have been conducted with the use of different sheathing materials on shear walls under different loading conditions. Lyon and Barnes (1979) studied the racking resistance of wall components using particleboard sheathing; their test showed that panels oriented parallel to the studs were stiffer than those oriented perpendicular. Wolfe (1982) investigated the racking resistance of gypsum wallboard considering the effect of panel orientation, wall length, and the openings. The results showed that horizontal panel orientation provides greater racking strength than vertical orientation and wall strength is linearly proportional to uninterrupted wall length. Patton-Mallory et al (1984) performed tests on plywood and gypsum sheathing to study aspect ratio (length effects), additive nature of individual sheathings in double sided shear walls, and the contribution of gypsum sheathing to shear wall behaviour. Their results show that racking resistance of plywood-sheathed walls is directly proportional to wall length whereas for gypsum-sheathed walls, it was not directly proportional to wall length. Dorwick and Smith (1986) have discussed the principles of timber sheathed shear walls, modelling, analysis and researched their behaviour under cyclic loading. However, their study is limited to walls sheathed with plywood, particle or fibre board sheets, nailed to the framing only. Lam et al. (1997) conducted comparative evaluations of the racking performance of wood-based shear walls built with regular and over-sized Oriented

Strand Board (OSB) panels under monotonic and cyclic loading conditions. Their result showed that the oversized OSB panels caused the significant improvements in shear wall strength and stiffness. Durham et al. (2001) conducted study on the seismic response of shear walls with standard and oversize OSB panels under monotonic, cyclic as well as shaking table tests. The result showed that the oversized walls improved shear capacity generally under monotonic loading due to redistribution of the nail locations, but questionable under cyclic loading since the failure modes were different under monotonic (nail withdrawal) and cyclic (nail fatigue) cases, the walls sheathed with the oversized panels drifted less than the standard walls under shaking table test.

The size of openings also affects the racking stiffness of timber frame walls. There are cases where large openings weaken the walls and result in the failure of the structures. Collins (1977) conducted racking tests on gypsum plasterboard lined, metal angle braced wall panel with opening size of 1150 x 1130 mm; the test revealed that the opening caused a loss of 30 % strength over similar walls without opening. Yasumara and Sugiyama (1984) investigated the influence of openings in shear walls on stiffness and strength capacity under static monotonic tests and developed a design method based on “shear strength ratio”. Hayashi (1988) investigated the effect of the wall opening ratio of the shear walls; the result indicated that the strength and stiffness of the wall decreases as the ratio of wall opening increases. Ge et al. (1991) used a model that examined the effects of openings on the racking stiffness and resistance of walls. Johnson (1997) also studied the effects of opening on shear walls under both monotonic and cyclic loading conditions with large openings. He et al.(1999) investigated the influence of openings on the lateral resistance of wood-based shear walls, built with both standard and oversized oriented strand board panels. The result showed that the door and window openings accounted for a significant decrease in the strength and stiffness of the walls and accelerated a change in failure mode, especially for oversize panel walls. Silih and Premrov (2010) studied the influence of the openings on the wall’s racking load-carrying capacity; they found that ultimate resistances of the wall panels with openings amounted up to 50% of the ultimate resistances of the panels without openings. Their study also confirmed that no-opening wall panels have a relatively higher horizontal stiffness and load-bearing capacity than wall panels with openings. Hence, their study concluded that timber-framed wall elements containing a door or window opening contributed to the racking load-carrying capacity, especially when a considerable part of the structure is made of such panels. Yasumura (2010) conducted research on the racking resistance of wood-framed shear walls with various opening configurations and boundary conditions. The result showed that racking resistance of a wall with opening increases as the opening area decreases. Similarly, Steensels et al. (2017) in their study also concluded that the size of openings in wall panels decrease the total racking resistance of the wall. Their study also found that the location of the opening had negligible influence on the racking resistance and the presence of vertical load increases the racking resistance of the wall. However, the study of Muthukumar and Kumar (2014) on the influence of opening location on the maximum displacement response of slender and squat shear walls, shows that the slender shear walls have higher displacement than squat shear walls. From the response of shear walls with different opening locations and with various damping ratios, it was concluded that the larger number of small openings resulted better displacement response. The influence of strengthening also considered massive in the case of staggered openings; the strengthening resulted in better behaviour of the shear wall.

Various models have also been used to analyse the performance of timber shear walls. Dolan and Foschi (1990) considered three methods as the prime methods for predicting racking performance of shear walls – first is the empirical relations derived from the test data, second is the simplified mathematical derivations, and the third is the finite elements to model the wall. The finite element model has become more popular and been increasingly used in the recent years. To name a few, researchers who have used finite element analysis include Foschi (1977), Easley et al. (1982), Itani and Cheung (1984), and Dolan (1989). It is worth noting here that the model used by Foschi was improved by Dolan and developed two finite element models (one for monotonic and one for time-step dynamic loading) to predict the behaviour of timber shear walls. The finite element programs also considered the wall configurations that are to be modelled such as walls with openings (Dolan and Foschi, 1990). Other methods include mathematical model developed by Gutkowski and Castillo (1988) to analyse shear walls, methods developed by Sugiyama and Matsumoto (1993a; 1993b; 1994) to calculate racking strength of shear walls. Richard et al. (2002) also used a numerical model based on finite element analysis to predict the cyclic response of shear walls with large openings.

Different design codes are used in different countries. For examples, Canadian national timber design code CSA Standard 086-01 in Canada (Canadian Standards Association,

2005) and BRANZ P21 in New Zealand (Cooney and Collins, 1979). In UK, Griffiths (1987) studied the performance of shear walls based on UK methods of construction which contributed the empirical basis for British Standard BS 5268-6. The racking design of EC5 (European timber design code) contains two methods: Method A and Method B, of which at present UK National Annex to EC5 specifies the use of Method B, a conversion of BS 5268. Since, in the conversion process to limit state methodology, the EC5 codifiers have incorrectly interpreted some important factors in the UK procedure and the method will not give an accurate result. Recognizing the deficiencies in the methodology and also that neither Method A nor Method B fully covers all design issues, these methods are to be replaced in EC5 by a unified method (Porteous and Kermani, 2013). Recently, a unified method is published as the UK's Non-Contradictory Complementary Information to Eurocode 5, PD 6693-1:2012 that has wider design criteria for global structural issues than the previous codes of practice (Porteous and Kermani, 2013). Although, there are many research studies that have been conducted on the racking performance of timber frame walls, these studies have had limited objectives and have not addressed several client and architectural requirements or design configurations such as effects of wall length, fixing types and details, size and positions of openings for doors and windows or the effects of the interaction between the adjoining walls or other components of the building. Hence, this research aims to improve our understanding of the real structural behaviour of the shear walls and to examine the accuracy of the existing methods in addressing the above issues for the analysis and design of shear walls that reflect their performance characteristics more effectively.

1.4 Research objectives

The research focuses on better and in depth understanding of the racking performance of timber frame walls, in particular adopting the method of construction typically used in the UK. Although many research have already been conducted, they were limited on their objective and have not collective information regarding the test procedures, geometrical parameters, effect of openings with and without trimmers as well as the spreader, effect of vertical loads, and on enhanced shear walls called Mid-ply walls. Hence, to address these issues collectively and to improve our understanding of real structural behavior of the shear walls, the main objectives aimed on this research are:

i. To examine the compatibility and suitability of the two different test procedures detailed in BS EN 594:1996 and BS EN 594:2011 versions on the racking performance of timber framed walls.

ii. To examine the influence of a range of geometrical parameters such as fastener size and spacing, wall length, arrangement of studs and horizontal members, and the effect of vertical loading on the racking strength and stiffness of walls assembled with OSB and PB sheathings.

iii. To determine the effects of openings/discontinuities for windows and doors on racking performance of OSB walls with and without using the trimmers, as well as the spreader.

iv. To examine the effect of a range of geometrical parameters such as fastener size and spacing, wall length, sheathing thickness, size and position of studs, as well as the effect of vertical loading on the racking strength and stiffness of the Mid-ply walls.

1.5 Research Methodology

The investigation on racking performance of shear walls was conducted based on the objectives as described in section 1.4. The entire experimental programme was undertaken at the Centre for Timber Engineering (CTE) at Edinburgh Napier University. For the test programme, the loads were applied using two separate loading systems. The racking load was applied via a horizontal jack connected to a data-acquisition system which followed a pre-programmed loading procedure based on BS EN 594:1996 and BS EN 594:2011 standards, depending on the test requirements. The materials required for each component of the walls were selected in accordance with the British Standards as provided in Appendix 1.1. All timber framed walls were tested in accordance with the procedure described in BS EN 594:1996 or BS EN 594:2011 to examine the effect on racking performance under these test methods and to investigate the compatibility and suitability of the test procedures. In Figure 1.4, the research methodology is summarized.

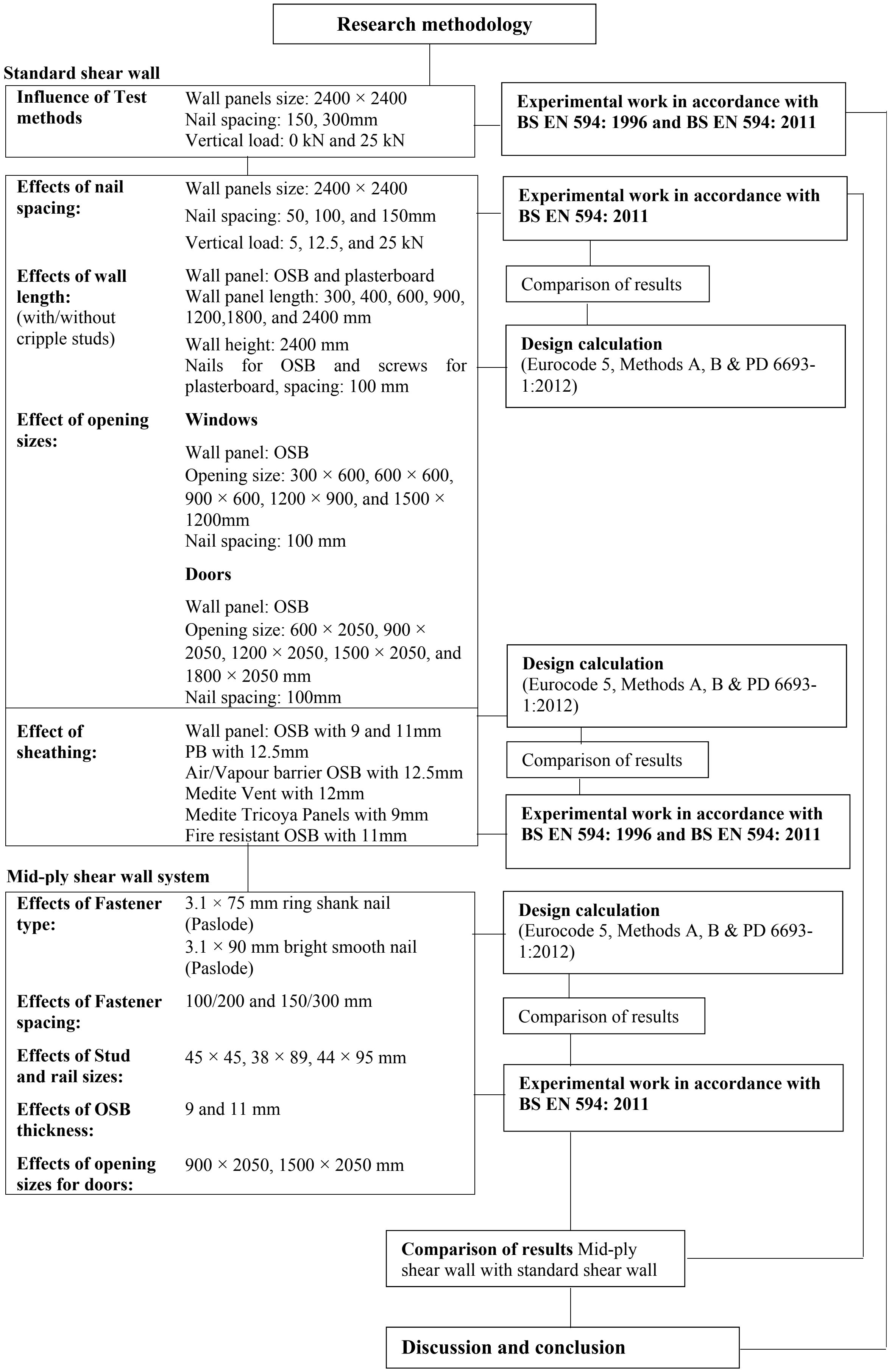


Figure 1.4 Research Methodology

1.6 Structure of thesis

The structure of this Thesis is outlined as follows.

Chapter 1 provides an introduction to the research, briefly reviews the existing literature in timber frame constructions. The chapter also establishes the research aims, objectives, and questions.

Chapter 2 reviews the literature on performance of the shear walls. It includes a review on the racking performance of shear walls affected by aspects ratio (ratio of height to length) of walls; size, types, and orientation of sheathing; openings (size and layout of doors and windows); fastener's types (nails and screws), spacing, and the failure modes (such as fasteners withdrawal from the main member, fasteners-head pull-through in the side member, splitting of either the main or side member, bearing failure of the wood, or shear failure of the fasteners); and anchorage conditions. It further included study on different loading protocols for shear wall test done by different researchers in different country contexts.

Chapter 3 examines the compatibility and suitability of the test procedures detailed in BS EN 594:1996 and BS EN 594:2011 versions on the racking performance of a series of timber framed walls. A comparison of the strength at failure and stiffness at the serviceability condition between identical wall panels tested in accordance with these procedures were shown in this chapter. Using the two test methods, the experimental programme was conducted on 2.4 m long by 2.4 m high walls comprising a range of OSB/3 panels, Air/Vapour barrier OSB, Medite Vent panel, Medite Tricoya panel, and Fire resistant OSB boards fixed to one side only of the timber frame. This chapter also examines influence of vertical load on the strength and stiffness of the walls.

Chapter 4 studies the racking performance of partially anchored timber framed walls with OSB and PB sheathings according to BS EN 594:2011 requirements. It determines the effects of parameters such as: panel-to-frame fastener spacing; wall length; arrangement and composition of studs and bottom rail members (e.g. use of double studs and double bottom rail); magnitude of vertical loading on the racking performance of OSB and Plasterboard (PB) sheathed walls. The chapter also assessed the differences between the experimental results and the design racking values obtained from the relevant European standards, in particular, the requirement of the UK National Annex to

Eurocode 5 (EC5), regarding the design for racking strength of timber framed walls using the procedure described in the PD 6693-1 document.

Chapter 5 shows the effects of openings/discontinuities of windows and doors on racking performance of OSB walls. Different opening sizes and the openings with the spreader on the top rail and trimmers were assessed in accordance with BS EN 594:2011. The experimental results were then compared with the existing design methods: EC5 (Method B) and PD 6693-1:2012.

Chapter 6 compared the racking strength and stiffness of the Enhanced Mid-ply and with the standard shear walls, constructed using OSB/3 sheathing boards. The experimental study examined the effect of a range of geometrical parameters, such as fastener size and spacing, wall length, sheathing thickness, size and position of studs as well as the effect of vertical loading on the racking strength and stiffness of the walls. The experimental results conducted in accordance with BS EN 594:2011 were then compared with the results obtained from design rules, as given in the relevant European standards.

In **Chapter 7,** the performance of timber frame walls obtained from the experimental works and the existing design methods were discussed and concluded.

1.7 Limitations

The research is aimed at the analysis of timber frame walls, specifically the shear walls. For the test specimen, the thickness of OSB/3 and Plasterboard, the diameter and spacing of nails and screws were selected based on their readily availability in the market (for the specification of the specimens used in the experiments, refer Appendix 1.1).

The 5 kN and 25 kN vertical loads assigned in the experimental works for examining the compatibility and suitability of the two different test procedures detailed in BS EN 594:1996 and BS EN 594:2011 (**Objective i**); examining the influence of a range of geometrical parameters such as fastener size and spacing, wall length, arrangement of studs and horizontal members, and the effect of vertical loading on the racking strength and stiffness of walls assembled with OSB and PB sheathings (**Objective ii**); and examining the effect of a range of geometrical parameters such as fastener size and spacing, wall length, sheathing thickness, size and position of studs, as well as the effect of vertical loading on the racking strength and stiffness of the Mid-ply walls (**Objective iv**). The 5 kN vertical load was only used for stabilising the walls.

To examine the effect of size of opening for windows and doors on racking performance of timber frame walls (**objective iii**), only limited sizes of the openings for windows and doors were randomly selected. For windows, sizes of 300 × 600, 600 × 600, 900 × 600, 1200 × 900, and 1500 × 1200 mm and for doors, sizes of 600 × 2050, 900 × 2050, 1200 × 2050, 1500 × 2050, and 1800 × 2050 mm were selected. Also, for this objective, the tests were performed in the walls with either openings for windows or doors. The combination of openings for windows and doors were not considered in the single wall panel. The sizes of openings for windows and doors in the wall were obtained by cutting the required size in the single/both sheathing of wall panel rather than joining the separate pieces of sheathing boards.

The numbers of similar wall panels test for the test procedures of the **Objective i** were conducted on three similar wall panels, whereas, for all other tests of the **Objectives i, ii, iii, and iv,** only one type of the walls were tested. However, for only one tests, the values have been assumed to make three number of test specimens including the experimental result (F_{max}) for determining the Factor $K_{s,}$ for calculation of characteristic 5-percentile values in accordance with BS EN 14358:2006. Then, design racking resistance was calculated on the basis of 5-percentile values to compare with existing design methods in accordance with EC5 (Method A, Method B and PD6693-1:2012).

To quantify the influence of fastener spacing on racking performance of timber frame wall (**Objective ii**) typically built in the UK, the fastener spacing were considered from the range of 50 mm to 150 mm for OSB sheathing and for sheathing with PB, an extra spacing of 300 mm in addition to that of spacing in OSB was considered as specified by EC5, PD 6693-1:2012. Though, the fastener spacing of 100 mm and 150 mm are practised in Scotland, extra spacings of 50 mm was used in order to examine the influence of dense number of fasteners on the racking performance of the walls.

Chapter 2 Literature Review

2.1 Introduction

This chapter focuses on the general background of the platform timber frame walls, shear wall and diaphragm action, effects of aspect ratio, openings, sheathings and anchorage on racking performance of timber frame walls.

Timber frame construction in the UK uses prefabricated wall panels, timber studs and rails, together with a wood-based sheathing, typically OSB or plywood, to form a structural frame which transmits all vertical loads as well as horizontal loads due to wind actions or other lateral forces to the foundation. Timber frame can offer many aesthetic and structural benefits, for example effective insulation for energy efficiency, sustainable design and ease and speed of construction (Munir et al., 2012). As a result, timber has become a popular construction material around the world. The platform frame construction is one of the methods of construction where floor structures bear onto load bearing wall panels, thereby creating a "platform" for construction of the next level of wall panels. Platform frame construction is particularly suited to buildings that have a cellular plan form. Internal walls may be used to contribute to this cellular layout and are used as load bearing elements for resistance to both vertical and horizontal loads (Structural Timber Association, 2013).

2.2 Shear wall and diaphragm action

A shear wall or diaphragm is a plate-type structural element designed to transmit forces in its own plane. McCormick P.T (2005, p.17-28) has defined shear walls as vertical elements (resisting horizontal forces) which are typically wood frame stud walls covered with a structural sheathing material like plywood or OSB. The system of load path works in such a way to transmit horizontal loads, acting perpendicular to the walls, to the side walls (in-plane loading), which in turn carry the loads to the foundations. The distribution of load is shown in Figure 2.1 where the side walls, considered to be simply supported at roof and foundation, transfer one half of the total wind loads to the roof level. The roof diaphragm acting as a deep horizontal beam transmits the load to the end shear walls, which in turn transfer the load to the foundations. Here, the dissimilarity between shear walls and diaphragm elements arises because of different load and support conditions at their boundaries. The roof diaphragm is subjected to the normal forces from the wind pressure on the side walls, and is supported by shear forces from the end shear walls. The end shear walls are subjected to shear forces at roof level from the roof diaphragm, and are supported by shear and normal reactions at the foundations. Shear walls must be fixed to the foundation to resist uplift forces (Prion and Lam, 2003). There is another wall type in the timber framing system called stud wall, which is quite different from shear wall. A shear wall in timber construction is effectively a load-bearing wall that is designed to carry vertical loads as well as racking loads in the plane of the wall (in addition to wind pressure loads acting perpendicular to its plane).

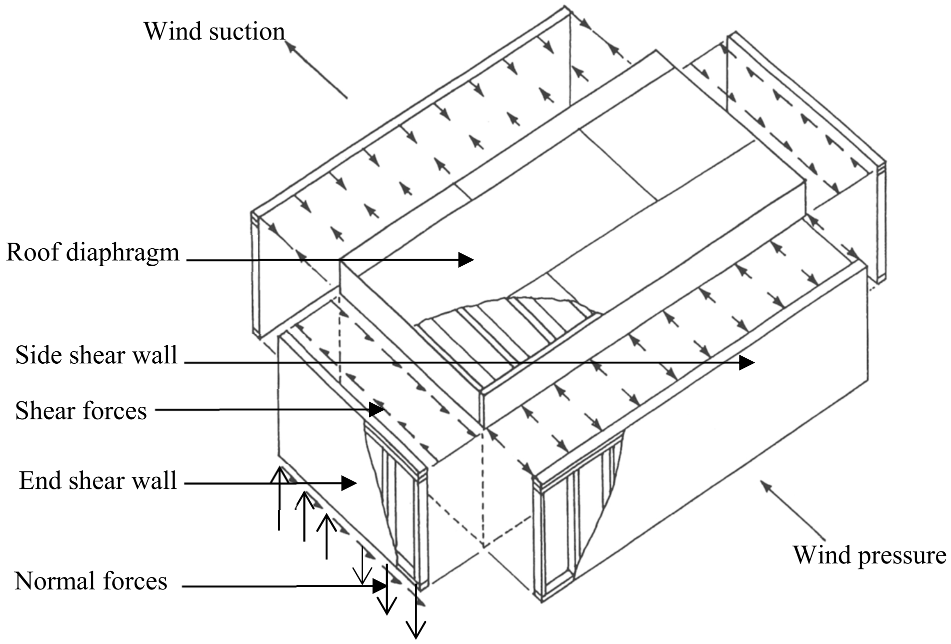


Figure 2.1 Transmission of applied shear to foundation
Source: Prion and Lam (2003) and Salenikovich (2000)

A floor or roof diaphragm is oriented in a horizontal or inclined direction, and carries loads perpendicular to its surface, while also providing racking resistance through in-plane shear. Racking loads are transferred to the framing through the connections with other plate elements such as shear walls or diaphragms. Clearly, the connections design is of primary importance in order to properly transfer the racking loads to the sheathing. The sheathing essentially fulfills the purpose of preventing the framing to deform into a parallelogram, and it provides the shear stiffness and strength to the wall.

2.3 Loading protocols

Monotonic loading has been the standard testing method to assess the strength and stiffness of shear walls for many years (Toothman, 2003). Toothman studied the monotonic and cyclic performance of light frame shear walls with various sheathing materials such as OSB, hardboard, fiberboard, and gypsum wall board. The tests were conducted on each of the sheathing materials subjected to each type of loading: monotonic, cyclic with hold-downs and cyclic without hold-downs. His study showed that the OSB and hardboard indicate similar performance and they were the strongest among all other sheathing materials. The performance of shear walls decreased when the walls were subjected to cyclic loading. The gypsum contributed significantly to the walls with hold-downs, however was not linearly additive. The use of hold-downs had a large effect on the performance of the walls; the shear wall performance decreased when hold-downs were excluded. However, the vertical load and fastener spacings that affect the stiffness and the strength of the walls were not considered during the testing of the walls.

The first standard for testing wall panels for monotonic racking resistance ASTM E72 was published by the American Society of Testing and Materials (ASTM). In this test, the wall is assumed to be fixed to the underlying floor or foundation via steel hold-down rods in order to resist the arising overturning moment. Because of the debate over its use in the certification of wall performance, another standard ASTM E 564 was developed to evaluate the wall's racking performance "as a whole" rather than the single performance of the sheathing, thus allowing variations in the hold down mechanism and wall configuration (Sherwood and Moody, 1989). This test standard, however, is not applicable for the UK standard. In the UK context, Griffiths (1987) mentioned that the proposal for racking test to replace the ASTM holding down strap with a system of vertical loads was prepared by Lantos (1967) who considered cyclic loading only and safety factors. Works on monotonic and cyclic tests of shear walls includes studies by Dolan and Johnson (1996), Lam et al. (1997), Salenikovich and Dolan (2003), Seader et al. (2009), Memari and Solnosky (2014). Works on pseudo-dynamic tests include the studies by Kamiya et al. (1996), Yasumura and Yasui (2006), Richard et al. (1998). Works involving shaking table test include the studies by Martinelli and Filippou (2009), Varoglu et al. (2007), Christovasilis et al. (2008)

In the UK, the racking performance of shear walls is determined based on the guidelines established in Eurocode 5 and provided in PD 6693-1:2012. The timber frame walls are tested in accordance with BS EN 594:2011, which superseded BS EN 594:1996 by introducing significant changes in the test procedure such as the removal of the stiffness cycle procedure and reduction in the test duration.

2.4 Effects on racking performance of shear walls

Timber shear walls consist of two main components: a timber frame and a sheathing, (Figure 2.2). The racking stiffness of shear walls is influenced by its aspect-ratio, size and orientation of sheathing; presence/absence of openings; fasteners size and spacing; and anchorage conditions.

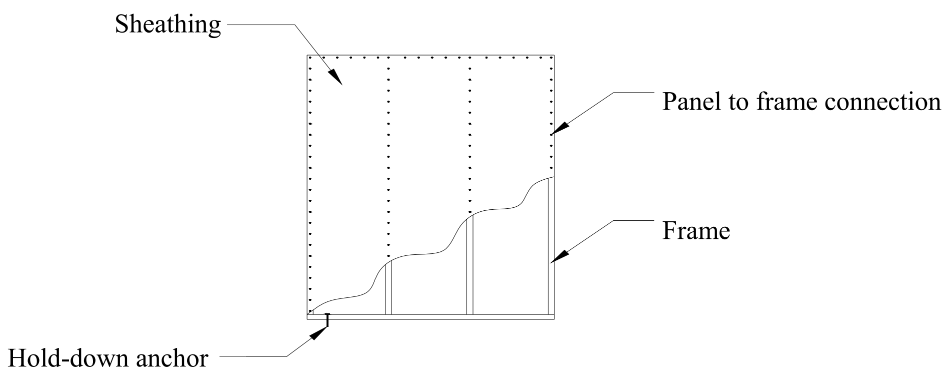


Figure 2.2 A representation of shear wall

2.4.1 Aspects-ratio

The aspects-ratio of shear walls is defined as the ratio of height to length of the wall. Kamiya et al. (1981) studied the effects of wall lengths on racking resistance and concluded that racking resistance is indeed proportional to the wall length. Patton-Mallory et al. (1984) also confirmed that racking resistance in plywood-sheathed walls was directly proportional to wall length, but in case of gypsum-sheathed walls proportionality relation was different compared to walls sheathed with plywood. Patton-Mallory et al. (1985) compared the shear resistance of “small” walls (consisting of 22 in. high and lengths ranging from: 2 to 8ft.) sheathed with gypsum to that of full-size walls (consisting of 8ft high and three length: 8, 16, 24ft.). The aspect ratios of small scale walls ranged from 1 to 4 and for full-scale wall it ranged from 1 to 3. Results of their tests indicated that racking strength was linearly proportional to wall length. Stiffness of small walls increased linearly with length while the stiffness of full-size walls increased nonlinearly. On the other hand Salenikovich and Dolan (2003) conducted monotonic and cyclic tests of full-size wall with aspect ratios of 4:1, 2:1, 1:1, and 2:3 and found that walls with aspect ratios ≤ 2:1 were equally stiff while narrow (4:1) walls were approximately half as stiff relatively to the longer walls.

2.4.2 Sheathing

Different sheathing materials have different strength capacities in the racking resistance and stiffness of the timber wall frames. The strength capacities also differ, depending on whether it is fixed at one side only or both sides of the frame.

Sheathing types

The following section defines some of the common sheathing materials used to provide racking capacity to the timber frame.

Plaster board (PB)

In the construction of racking wall, the most popular material for sheathing of internal walls is gypsum board, often combined with exterior plywood sheathing (Patton-Mallory et al., 1985).The materials in the board wall panels consist of a gypsum plaster core which is non-combustible and covered on both surfaces with paper veneer often referred to as plasterboard. However, the plaster core is brittle in nature whereas the paper veneer provides strength and stiffness to resist racking forces. Due to the brittle nature of its core material and low stiffness and strength relative to that of wood-base panel materials, gypsum boards are rarely recognized for any structural contribution to the integrity of light-frame buildings. However, Wolfe (1983) has shown that gypsum wallboards can indeed provide a contribution to racking capacity, which varies with panel orientation and wall length. Wolfe also asserts that the relationship between ultimate shear strength and wall length was approximately linear, but for low shear deformations, a power function was found to better approximate the relationship with the wall length.

Oriented Strand board (OSB)

Oriented strand board have high dimensional stability in the presence of high humidity or water; however, the durability and usage of the boards largely depends on the types of adhesive being used (Mirski et al., 2015). Fakhri et al. (2006) reported that the density and fines content and their interaction significantly influence the permeability of core layer of OSB. BS EN 300:2006 has defined four grades of OSB in terms of their mechanical performance and resistance to moisture, which are listed below (BSI, 2006).

OSB/1 - General purpose boards and boards for interior fitments (including furniture) for use in dry condition,
OSB/2 - Load-bearing boards for use in dry conditions,
OSB/3 - Load-bearing boards for use in humid conditions
OSB/4 - Heavy-duty load-bearing boards for use in humid conditions.

Air/Vapour barrier OSB

Different materials are available in the market to use as sheathing in the construction of timber frame structure. During the application of these sheathing materials, some aspects need to be considered as control of the migration of moisture. According to the UK National Building Code, any material that allows less than 60 NG (nanograms) of moisture to pass through under specific condition is considered as type 9 residential vapour barrier; example includes Smartply Propassiv OSB, which was used for this research. SMARTPLY (2016) defines Propassiv as a structural OSB panel with integrated vapour control and air barrier properties that are used as a structural sheathing, applicable for both new build and renovation projects. The coated surface of the panel provides a smooth durable surface and superior bonding of airtight tape at the panel joints. The advantageous features of the panels are accounted for their airtightness, ease to cut and fix, rigidity, high vapour resistance, durability, and high racking strength.

Medite Vent

MEDITE (2015) defined Medite Vent MDF is a good choice for the outer layer in “diffusion open” wall and roofing applications because of its high racking strength with excellent vapour permeability and high weather resistance. It has the features of a very low water vapour diffusion factor to prevent condensation (tested by Fraunhofer Institute for Building physics) and high performance - Category 1 (tested by UKAS accredited laboratory). Although, it is a high performance breathable external sheathing panel that could be used in all types of timber frame structures, the boards must be protected from direct contact with water with a suitable weather-proof breathable membrane during and after installation. It is therefore principally adopted as sheathing where drying of the structure is required, otherwise membrane with a vapour diffusion factor equivalent or lower to that of Medite Vent is recommended to use. Medite Vent and classified as Service Class 2 conditions to EC 5 (EN 1995-1-1) and is suitable to use in humid conditions. The boards may be installed with nails, staples, and woodscrews fixings.

Extreme Medite Tricoya

Extreme Medite Tricoya (2016) is a high performance wood-based panel product that has outstanding durability and dimensional stability in the most extreme and challenging environment, both in exterior and interior, wet and high moisture applications. This product uses proprietary acetylated wood technology (reducing the ability of the wood to absorb water, which is the most fundamental reason for wood swelling) and a modified fibreboard manufacturing process to create a wood panel with outstanding performance. The test on Extreme Medite Tricoya conducted by the Fraunhofer Institute for Wood Research in Germany confirmed its outstanding performance and the one conducted by Building Research Establishment (BRE) in the UK confirmed its durability class (*very durable*) according to EN350-2 standard. The moisture content should not exceed 8 %; otherwise, it should be allowed to dry. The fire rating of this material has achieved a fire class of Euro Class D within the Euro classification system. The feature and benefits of Extreme Medite Tricoya includes durable, design freedom, sustainably sourced, 50 years guarantee, lower maintenance cost, resistant to fungal decay, enhanced stability, perfect for coating and desired service life of 60 years.

Fire resistive FR OSB

According to SmartPly (2015), FR OSB is a flame retardant structural OSB/3 panel. The panels are manufactured in accordance with EN300 and EN 13986. It has high shear strength and is used in roofing, flooring and wall sheathing where strength, moisture resistance and flame retardance are of primarily importance. It is manufactured using Zero ignition solution – a water-based, eco-friendly, fire retardant (The Building Centre, 2017).

Racking performance of sheathing types

Many studies have been conducted on the effect of sheathing on shear wall behaviour. The experiments conducted by Dolan and Madsen (1992) showed that sheathing played an insignificant role in the working stress range of the shear walls as well as the ultimate strength. Iizuka (1975) investigated the sheathing effect on timber frames. The investigations were conducted on 47 walls tested monotonically that included seven different types of sheathing; plywood, particleboard, wood fibre hardboard, insulation board wood fibre cement board, gypsum board, and asbestos cement sheets. The test results showed that the strength and stiffness of double-sided wall panels was less than the sum of two single sided wall panels. The result also showed that the shear stiffness of the material has an influence on the racking stiffness of the wall; plywood seemed to be the most effective sheathing for shear resistance. Patton-Mallory et al. (1984) investigated the nature of one-sided walls and double-sided shear walls with different sheathing using small-scale shear wall tests. Their tests included 20 wall types that comprised plywood on one side and two sides, gypsum on one side and two sides, and mixed wall with plywood one side and gypsum one side. Results obtained from their tests showed wall panels with a single side of gypsum sheathing have a racking strength that is about 38 to 64% of the racking resistance of walls sheathed with single-sided plywood panels, and 30 to 39% the racking resistance of walls sheathed with double-sided plywood-gypsum panels. Walls sheathed with gypsum on two sides have a racking resistance which is 57 to 67% the resistance of walls sheathed with plywood (on one side) and gypsum (on the other side). This result seems to be in contrast to the study conducted by Iizuka (1975) as discussed earlier. Uang and Gatto (2003) studied the effect of non-structural finish materials (gypsum wallboard and stucco) with structural sheathing (plywood and OSB). Their study concluded that non-structural finish materials have significant influence on the performance of wood frame shear walls, thus increasing their strength and stiffness. Although, the addition of wall finish materials increased strength and stiffness, there was reduction in deformation capacity. The addition of gypsum wallboard seemed significant as it resulted in a 12% increase in strength and a 31% reduction in deformation capacity whereas there was 34% increase in strength and about 31% reduction in deformation capacity in the case of stucco. Because of the increased strength, brittle failure was observed. The failure of wall panels without finish is due to nail failure that facilitates panel rotations, whereas the walls with sheathing on one side only, the failure resulted the torsion in the walls due to twisted corner studs. However, when the finish materials were added, the twisting in the studs was reduced significantly. Sinha and Gupta (2009) studied the load sharing between the OSB and gypsum wall board (GWB) on 16 shear walls tested monotonically. Out of the 16 shear walls, 11 walls were sheathed on both sides (OSB on one side and GWB on the other) and 5 walls were tested without GWB. They found that as GWB fails at about 60% of ultimate load capacity of the wall, the load shifts to the OSB panel until the failure of the wall. While the test conducted by Sartori (2012) for evaluating the behaviour of panels with three configurations; OSB on both sides, gypsum fiberboard (GFB) on both sides and OSB on one side and GFB on the other side, showed that all three configurations have similar stiffness and strength for the wall sheathed with GFB or with the mixed configuration was 35 % lower than the wall sheathed with OSB. Likewise, the test conducted by Seim et al. (2015) on OSB and GFB panels in terms of maximum load bearing capacity, ultimate deformation, ductility, and equivalent damping demonstrated that same behaviour factor can be used for both types of wall if the basic requirements regarding the minimum thickness and detailing of the connections are considered. The results showed no significant difference in the performance between the single and double-sheathed walls. Branco et al.(2017)'s study to evaluate how the sheathing material and fixation to the base influence the overall response of the wall, concluded that the stiffness almost doubles in relation to one side with OSB board to both sides with OSB boards. Goodall and Gupta (2011) in their research on improving the performance of gypsum wallboard in wood shear walls concluded that increasing the stiffness and strength of a shear wall resulted in less GWB damage for a given loading or displacement.

Sheathing panel orientation

Wolfe (1983) evaluated the contribution of gypsum wallboard to racking resistance of light frame walls considering variables such as wind bracing, wall length, and wallboard orientation. The test result found that the contribution does not seem to be affected by interactions with wind bracing, but varies with panel orientation and wall length. The racking resistance of walls tested with a gypsum diaphragm and a diagonal wind brace was equal to the sum of contributions of these elements tested independently. Walls tested with panels oriented horizontally were more than 40% stronger and stiffer than those with panels oriented vertically. The contribution of gypsum wallboard as an interior surface was also investigated. The results showed that there was significant strength degradation during the cyclic tests when the walls reached a displacement value of 1.6 in. (40 mm) for plywood or OSB sheathed, 0.2 in. (5 mm) for gypsum wallboard, and 0.8 in. (20 mm) for a combination of the two. Chen et al. (2014) conducted tests on 10 shear walls sheathed with OSB and GWB under static monotonic lateral load. The test result showed that vertically oriented shear walls with OSB have higher strength and lower stiffness and ductility ratio, whereas vertically oriented shear walls sheathed with GWB have higher or similar structural performance, compared to parallel shear walls with sheathing panels oriented horizontally. Chen et al. (2016) investigated the racking performance on 12 shear walls sheathed with single layer OSB and GWB, or in combination. They found that horizontally oriented OSB or GWB had similar failure modes to those with vertically oriented sheathing; however, the failure location of sheathing-to-framing joints appeared from the adjoining sheathing panel edges in the middle of the wall, spreading to the top/bottom edges. They also found that the vertically oriented panels have higher strength and energy dissipation, larger ultimate displacement, and lower stiffness and ductility ratio than those of horizontally oriented panel.

2.4.3 Openings

A racking wall may comprise a single wall diaphragm or contain more than one wall diaphragm with discontinuities such as openings for doors or windows. Several studies have been carried out to investigate the effects of an opening on the racking resistance of a wall panel. According to Prion and Lam (2003), the openings in shear walls and diaphragm can have a significant effect on their performance. When large openings, such as doors, divide a shear wall into a number of smaller elements, proportioning of lateral shear among the various elements requires special considerations. Prion and Lam further mentioned that if there is not much difference in length between the wall segments, the load can be shared in proportion to the wall segment. In case of window openings, a series of narrow tall shear walls extending over the full height of the building has to be considered. The load bearing capacity and stiffness of wooden walls is influenced by dimensions and layout of openings (Dujic et al., 2007). The existence of openings in the walls results in decrease in strength and stiffness in comparison with wall without openings (Sartori et al., 2012). A typical example of racking wall with wall diaphragm and opening is shown in Figure 2.3.

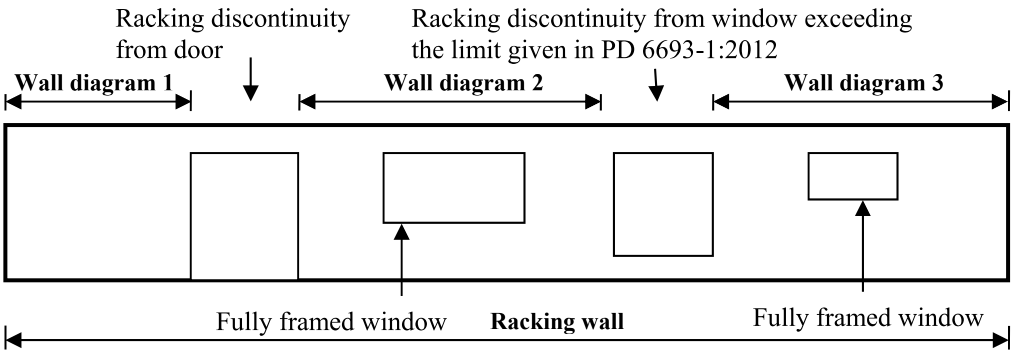


Figure 2.3 Division of racking wall into wall diaphragms

Source: Structural Timber Association (2013)

For calculating shear resistance ratio (F) in relation to the opening coefficient (r), Yasumura and Sugiyama (1984) derived the formula $F=\frac{r}{3-2r}$ based on their experimentation on wall panels with a single opening of various configurations. Kamiya and Itani (1998) also derived a simplified calculation for determining the shear forces in diaphragms with openings. They conducted a horizontal loading test on three floor diaphragms under same loading condition and found that the ultimate loads are almost the same, regardless of presence of the opening. Jang (2000) studied the lumber shear walls with sheathing materials such as plywood, OSB, and gypsum board and concluded that the position of opening does not have effect on racking resistance of shear walls, whereas the racking resistance of shear walls decreases with the increase in size of openings. As pointed out by Dujic et al. (2007), the load-bearing capacity and stiffness of fenestrated wood walls are influenced mostly by the size and layout of the openings. Their experiments on cross-laminated solid wood walls concluded that openings with a total area of up to 30 % of the entire wall surface do not significantly influence the load-bearing capacity of the wall but the shear stiffness is reduced by about 50%. Silih and Premrov (2010) on their experiments on a timber framed wall elements coated with single fibre-plaster boards with different areas of openings, concluded that wall panels with no opening have a relatively higher horizontal stiffness and load-bearing capacity than wall panels with openings.

The monotonic test conducted by He et al. (1999) showed that opening significantly decreased the strength and stiffness of shear walls. Walls with opening had 28% drop in strength and a 15% reduction in stiffness compared with wall without opening. The static test results showed that the stiffness and strength of oversize panel with openings are 124% and 41% respectively that of conventional panels with openings. This indicated that the impact of openings on a shear wall with oversize panel is less significant than that on conventional panel. Though, several research were already conducted on the effects of opening on racking performance of the walls, there are still some gaps in relation to the effects of trimmers and spreaders use on the walls, fasteners spacings, sizes of openings, and partially anchored walls that is typical in the UK context.

2.4.4 Fasteners

In a timber frame wall, the sheathing panels are fixed by means of fasteners. These fasteners, as a connector, play a significant role. Effective use of fasteners in timber buildings contribute to racking strength, stiffness, and ductility. Structural failures are often caused by improper design or defects of these connectors. Fastener types include nails, screws, timber rivets and bolts. Larsen and Jensen (2000) defined the first category of fasteners such as dowels, staples, nails, screws, and bolts where load is transferred along the shank. A second category of fasteners are those, where load is transmitted over a large bearing area at the surface of member such as split-rings, shears plates, and punched metal plates. The Wood Information Sheet in TRADA (2016) describes that with the dowel-type fasteners such as nails, staples, screws, dowels and bolts, the magnitude of load transfer between the connected members, depends on the bending behaviour of the fastener as well as the bearing stresses developed in the timber along the shank of the fastener. It further discusses that the friction within the interface between two connected members and axial pull-out resistances could also contribute to the shear (lateral) capacity depending on the fastener type.

According to reThink Wood (2015), the fasteners help the wood connection to become stronger by distributing the load over them; this builds a degree of redundancy that is useful in high-wind or seismic events. For this reason the designers are advised to use small fasteners (less than1/4 in. diameter,), to use multiple fasteners, and to keep the scale of fasteners relative to the size of the wood members being connected. Though, there are different types of fasteners, only nails and screws will be discussed in the following section.

Fastener types

Nails: Nails are generally used in low loading conditions such as construction of diaphragms and shear walls (Fridley, 1997). Nails transfer the loads from one member to another and nailed connection is dependent on the thickness and density of material, the type and size of the nail used, and the moisture and humidity levels to which the connection will be exposed (Winterbottom, 2000). Based on the types of heads, Winterbottom, (2000), defined two types of nails: common nails that have flat circular head and finish nails that have a very narrow head resembling a slight bulge at the top. These nails are available in a variety of lengths that are proportioned to the shank diameters. Fridley (1997) also mentioned about the deformed shank and coated nails which were developed to provide better withdrawal resistance. According to Paslode (2016), round head nails are considered as conventional type that is used where higher pull-through resistance is required. D-head nails also have similar performance as the round head nails, however, these enables fastener to be collated hard up against each other. Ring shank nails are deformed shank nail in which circular threads are rolled into the shank after the point and head are formed (Skulteti et al., 1997). These nails have a greater withdrawal capacity than smooth shank nails (Sebestyen, 2003) and provide a stronger grip (Matthews, 1991). The performance of smooth shank fasteners are significantly affected by the changes in moisture content of timber, whereas hardened threaded and ring shank nails are not affected as much by such variations (Dolan, 2005). The experiments conducted by Theilen et al. (1998) showed that ring-shank nail connections have roughly twice the strength of smooth shank nail connections.

Screws: Screws offer more withdrawal capacity than is usually assumed. In terms of shear, wood screws behave like nails but their withdrawal capacity is higher than nails (Herzog et al., 2004). The tapping screws also known as sheet metal screws have higher withdrawal resistance than wood screws. Hence, these screws are commonly used to fix particleboards where withdrawal strength is important. Plasterboard screws have a bugle shaped head which countersinks neatly without crushing the core or tearing the face paper (Paslode, 2016).

Failure modes of fasteners

The fasteners such as nails and screws can result in either brittle, ductile or mixed modes of failure at the timber joints (Zarnani and Quenneville, 2013). In the brittle zone, the deflection is in the elastic range and the wood connection capacity is less than the yielding resistance of fasteners. In the mixed failure mode, the wood fails to some deflection of the nails before they reach complete yielding (Zarnani and Quenneville, 2014) (refer Figure 2.4). According to Zarnani and Quenneville (2015), in this failure mode, the effective wood depth is significantly smaller than the one associated with the brittle failure mode and is derived from the governing failure mode of the fastener. The mixed failure mode can happen even if wood strength of the new connection is greater than fastener yielding resistance, as the deflection of the connection progresses if the wood capacity of new connection is less than the ultimate ductile strength. If the wood strength based on the effective wood thickness is greater than ultimate ductile strength, ductile failure develops with no wood rupture.

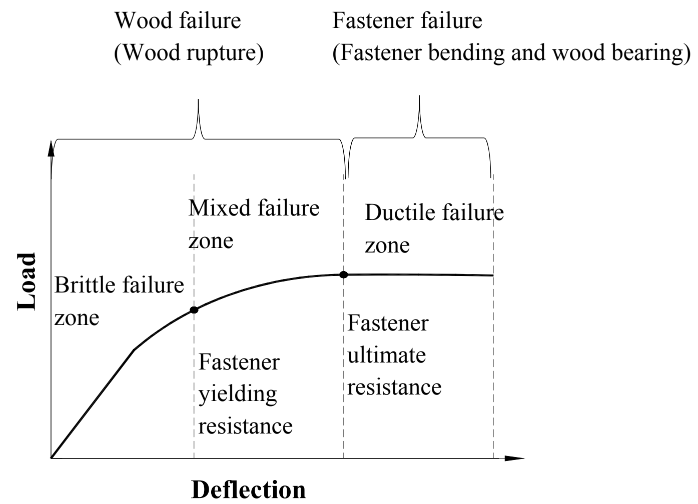


Figure 2.4 Failure modes of nails in timber joints
Source: Zarnani and Quenneville, 2014

According to Theilen et al. (1998), the failure of common nail connections is dominated by nail withdrawal, whereas ring-shank nails experience one of other failure modes such as nail withdrawal from the main member, nail-head pull-through in the side member, splitting of either the main or side member, bearing failure of the wood, or shear failure of the nail.

(a) Sheathing to framing connections

According to Judd and Fonseca (2005), the lateral force that is transferred from the timber frame to the sheathing through nails causes displacement of the nails head (see Figure 2.5) with respect to the nail shank, thus developing shear deformation of the connection. As the load increases, the shear deformation of the connection further increases, eventually causing crushing of wood fibres and yielding of the nail. If the loading continues after yielding of the nail, the strength of the connection decreases with increasing displacement leading to a failure. The angle of the applied lateral load with respect to the timber grain has a negligible effect on the connection behaviour (Dolan and Madsen 1992).

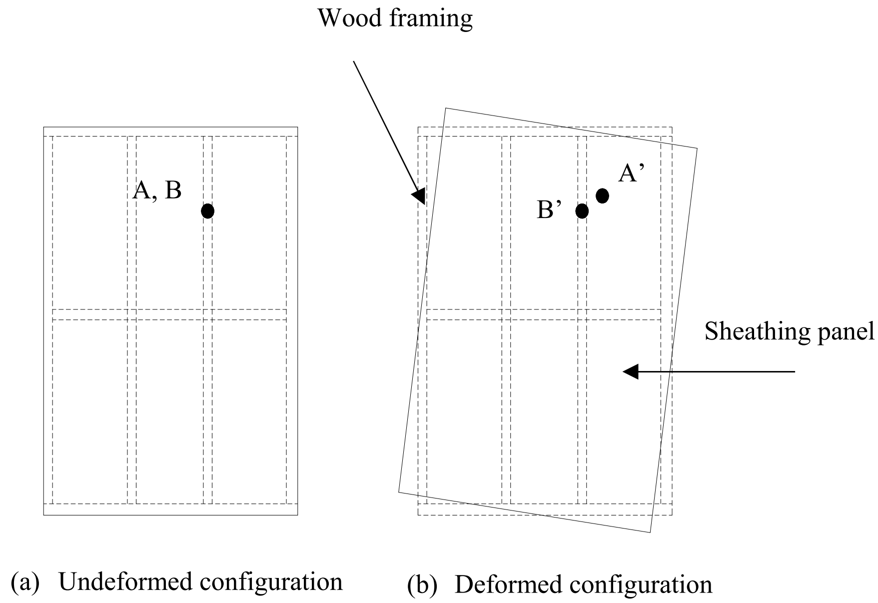


Figure 2.5 Lateral displacement of a panel in wood shear wall
Source: Judd and Fonseca (2005)

Different arrangements of the base connections have significant effect in the distribution of shear among the sheathing nails (Gattesco and Boem, 2016). Gattesco and Boem performed five full-scale experimental tests on shear walls subjected to in-plane horizontal cyclic loads. They argued that the force distribution among the fasteners differs significantly when the base steel devices are assigned with the sheathing interrupted and/or the panels are nailed to a base timber plate. In their experiments, they observed that ring nails at the middle height of the studs deform along the vertical direction, whereas those at the middle length of the joists deform horizontally and those applied on the corners deform along the diagonal direction of the sheathing. The failure is initiated at those nails placed at the ends of the external studs, then extending to the whole length of the joists. They also observed that the shear distribution among the nails differs significantly when hold-downs brackets are fixed to the studs with the panel interposed and the sheathings are nailed to a fixed timber base plate. In the initial phase, the fasteners shear loading was primarily distributed among the base-plate nails. The presence of the hold-down connection and the friction between the timber frame and the base-plate provides a negligible contribution against the horizontal sliding of the shear wall, preventing its abrupt failure if/when the base plate nails collapse. The panel rotation in respect to the timber frame was initially controlled by fixing the hold-down on the sheathing, stiffening the shear wall performances. However, this generated the concentration of forces in correspondence of the hold-down nails, causing significant tensile stresses in the sheathing and its tear-out. The tear-out, which induces shear redistribution among the fasteners, can limit the resistance capacity of the shear wall anticipating the collapse of nailed connections.

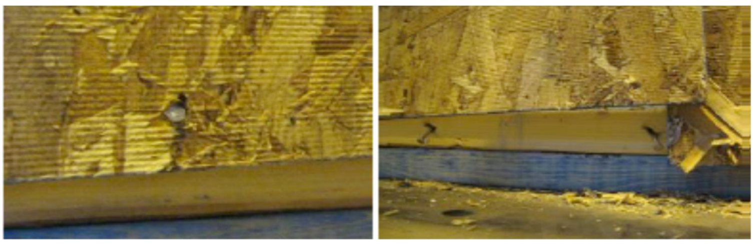
(b) Effect on racking performance

In their contribution for a methodology for evaluating the racking performance of wood sheathed walls with and without openings, Itani et al. (1982) substituted the sheathing panels by a pair of diagonal springs. The stiffness of each spring is calculated based on the stiffness of an individual nail that fastened the sheathing to frame. McCutcheon (1985) argued that the racking behaviour of a timber shear walls primarily depends upon the load-slip behaviour of the fasteners that affix the sheathing to the frame. The wall panel when subjected to racking loads, the nail connectors deform and the stud frame distorts as a parallelogram, while the sheathing retains its original rectangular shape. The corner nails distort most, the directions of which are approximately along the diagonals of the sheathing. The nail's load slip characteristics make it possible to predict the performance of the wall. Wang et al. (2010) developed a model that can predict load-slip response of a nailed joint. The authors conducted the test program consisting of 27 combinations of nail diameter (2.5- 4.1 mm), sheathing thickness (9.5 - 18.5 mm), and lumber density (300 - 525 kg/m3) and confirmed that the nail joint strength can be predicted by knowing sheathing thickness, nail diameter, and lumber density. Heine and Dolan (2001) also developed a new approach that predicts the load-slip interaction of a single shear bolted joint in timber exhibiting two plastic hinges at yield. Kochkin and Loferski (2005) proposed a model that can be used to predict linear and post-linear stiffness of moment-resistant connections, connection capacity, and ductility. Their proposed procedure explicitly included the nonlinear response of the nails and plate bearing to accurately predict the moment-rotation relationship over a wide range of deformations. The proposed procedure is formulated such that the nonlinear response of the nails and plate bearing are explicitly included in the model to accurately predict the moment-rotation relationship over a wide range of deformations. However, other factors such as flexibilities of the sheathing and stud frame can also affect the performance. Tuomi and

Gromala (1977) evaluated the racking strength of walls with let-in corner bracing. They determined the lateral nail resistance values of fiberboard sheathing materials and observed that the ultimate nail load increases with edge distance in a nonlinear manner.

Lee and Hong (2002) found that nail head diameter has considerable effect on the performance of a shear wall. For this, they investigated the effect of nail size on the performance of the shear wall using models constructed with three types of nail: Type A (39.2 mm length, 4.6 mm head diameter, 2.2 mm shank diameter), Type B (37.9 mm length, 3.9 mm head diameter, 2.0 mm shank diameter), and Type C (39.7 mm length, 3.1 mm head diameter, 1.9 mm shank diameter). The failure mode for Type A nails appeared to be panel breaking and nail pull-out, for type B nails it was panel breaking off, nail pull-out, two nail-head-pull-through and for type C it was nail-head pull-through, panel breaking off, nail pull-out. Nail spacing is also an effective factor on the variation of load-resisting capacities. Anil et al. (2017) did the comparative tests on walls with aspect ratio of 0.68 using: (i) 100 mm nail spacing along the sides of OSB plates and 300 mm nail spacing at the mid regions, and (ii) 300 mm nail spacing at the sides of the OSB plate as well as at mid regions. They found that the load-resisting capacity of (i) was 28 % larger than (ii), meaning the increase in nail spacing decreases the load-resisting capacity. They also concluded that increasing nail spacing (i.e. a smaller number of nails) decreases stiffness and increases displacement ductility. Sørensen et al. (2013) on the other hand confirmed that the ultimate capacity of the connection will be each nail's capacity times the number of nails in the row. For which, they conducted test of the capacity of shear connections consisting of nails in a row placed at distances 7, 10, and 14d, (d is the cross-sectional dimension of the nail).

Girhammar et al (2004) described different failure modes of nails of different sheathing materials under different loading directions. For this, they conducted the tests of joints with respect to different sheathing materials such as hardboard, particleboard, and plywood with the load directions parallel (0º) and perpendicular (90º) to the grain direction. The test results for the hardboard with the load parallel to grain direction showed a ductile type of failure (nail yielding followed by withdrawal of nail); the brittle type of failure occurred after ultimate load bearing capacity was reached. The perpendicular tests had a ductile type of failure (nail yielding followed by withdrawal of nail). For the particleboard, both parallel and perpendicular tests showed failures by withdrawal of nail, punching of nail head, and nail failure in timber member. However, the failure due to withdrawal of nail and punching of nail head in parallel test was less in number than the perpendicular, whereas the nail failure in timber member in parallel test was more in number than the perpendicular. For the plywood, the test result showed that the parallel test had failure due to punching of nail and nail failure in timber whereas the perpendicular test had failure due to withdrawal of nail and punching of nail head. They also noted that the characteristics and failure modes differ such as with respect to loading to-grain directions and edge distances of the fasteners. For this, the test was done only for the hardboard. For, the load directions parallel to grain, the edge distance of 1d and 2d (where d = 2.1 mm) were used and for perpendicular to grain, the edge distance of 2d, 3d, 4d, and 5d were used. The result showed that the boundary for edge failure is 2d for parallel and 4d for perpendicular tests. They also argued that the capacity of wood framed shear walls is governed by the characteristics of the sheathing-to-timber joints. Chen et al. (2014) on the other hand observed different failure modes of the shear wall sheathed with OSB affixed by nails (see Figure 2.6 (a) and (b)). In panel-frame nail connections, the failure modes are nail yielding with head embedding in OSB under shear, nail head pull-through, nail withdrawal from the framing member, sheathing edges and framing members torn by nails. In framing connections, the failure mode is nail yielding between end studs. In hold-down connections, the failure modes are washers embedment into studs, and in studs, the failure mode is studs bending loading.



(a) (b)

(c) (d)

Figure 2.6 Failure modes of shear walls sheathed with OSB and GWB

(a) nail yielding with head embedding in OSB (b) nail head pull-through the OSB panel (c) screw yielding in GWB (d) screw head pull-through in GWB

Source: Chen et al. (2014)

For the specimen sheathed with GWB and affixed by screws, Chen et al (2014) observed five failure modes (see Figure 2.6 (c) and (d)). In panel-frame screw connections, the failure modes are screw yielding under shear, screw head pull-through, sheathing edge torn by screw, and screw complete sheared off, and in stud the failure was stud bending. They found that the ductility ratios of shear walls sheathed with GWB was similar or higher than those with OSB. They also compared the load-displacement response of GWB shear walls without and with panel joint taping and found that taping increases the strength but decreases the stiffness and ductility ratio.

Germano et al. (2015) investigated the local behaviour of sheathing-to-frame nailed or stapled connection and cyclic behaviour of shear walls with ring nails, with and without vertical load. They observed that the surface feature of the nail shank affect strength and stiffness of the particleboard sheathing-glulam stud. Their experiment showed that the strength of ring shanked nail connection is 1.75 times the strength of smooth nails. The full scale shear walls with ring nails failed at 2.5% drift due to low cycle fatigue fracture of the nailed connections between the particleboard panel and the frame. The cyclic behaviour of the shear walls with ring nails was not affected by the vertical load because of the over strength factor of hold-downs and angle brackets. Furthermore, at the peak shear load the horizontal displacement provided by the sheathing-to-frame connection contributed 75% of the total displacement, while the hold-down and angle brackets connections contributed 15% and 4% to the total deformation respectively. Sartori and Tomasi (2013) also observed that ring nails perform better than smooth nails, while staples used with gypsum fibre panel perform more or less the same resistance and same stiffness of nail used with OSB panel, but has brittle behaviour. The ductility and the dissipation of the connections done with nails and OSB panel are higher than with staples and gypsum fibre panel. Casagrande et al. (2016) discussed about the sheathing-to-framing connection, the rigid-body rotation, the rigid-body translation, and the sheathing-panels that are responsible to contribute the elastic horizontal displacement of a timber frame wall subjected to a horizontal force. They highlighted two different regimes in a timber wall stiffness – i) when the hold-down is not in tension, since the stabilizing moment is greater than the overturning one; ii) when the hold-down is in tension. Their experiments concluded that wall stiffness is linearly proportional to wall length when the hold-down is not in tension.

The failure modes in shear walls with openings may differ from those without openings. This was shown by the study of He et al. (1999). In the walls without openings, the deformations occurred predominantly in the nails along the panel edges, whereas panel failures occurred in the walls with openings. Panel failure modes occurred in the form of panel crushing and buckling and panel tearing around the corners of openings. Nail withdrawal occurred mainly along the edges of the panels at the mid height of the wall (in conventional panels) and along the bottom edge of the panel (in oversize panel). In the tests conducted on 8 × 8 ft. shear walls with sheathing nails spaced at 4 in. on the perimeter and 12 in. in the field of each panel, Anderson et al. (2007) observed four modes of failure for perimeter sheathing nails: withdrawal, pull-through, fatigue, and tear out. However, they found that the dominant failure mode for the sheathing nails was withdrawal.

2.4.5 Anchorage

The shear wall is influenced by anchorage types that transfer lateral shear forces and prevent overturning of wall (Prion and Lam, 2003). The anchorage could be hold-down device as well as the horizontal nailing between the sheathing and the sill plate together with the anchor bolts. The anchor bolts provide horizontal shear continuity between the bottom rail and the foundation and hold-downs serve as vertical anchorages that connect vertical end studs to the foundation (Prion and Lam, ibid). Prion and Lam (2003) also pointed that anchor bolts are not designed to transmit vertical forces to the foundation, although some capacity can be achieved, if necessary. In such case, the bottom row of nails transmits the vertical forces in the sheathing to the sill plate (instead of the vertical end stud) where the anchor bolts will further transmit the forces into the foundation. Because of the eccentric load transfer, transverse bending is created in the sill plate and splitting often occurs. To prevent such a brittle failure mode, large washers (preferably square or rectangular) need to be provided to affect the eccentric load transfer from the sheathing through the nails, into the sill plate to the anchor and foundation. Hold downs are substantially larger than anchor bolts due to large concentrated forces. Because failure of the hold downs often occurs in a brittle mode, it must be ensured that capacity design principles need to be considered so that wall fails in shear along the nail connectors before any of the hold downs connections fail.

The hold-down devices contribute to the overall stiffness of the shear wall. If the hold-down devices stretch or slip, the top of the shear wall will move horizontally which when added to the movement of the lumber, sheathing, and fasteners, reduce the effective stiffness of the shear wall (Timothy P. McCormick, 2005). Hence, hold-down connections are required at both ends of shear walls to avoid overturning restraint (Ni and Karacabeyli, 2000). The design for shear wall requires the chords to be attached to the lower structures (foundation) through hold-down anchors/anchor bolts to restrain the walls from overturning (Salenikovich and Dolan, 2003). Walls with only anchor bolts installed without hold-downs are partially anchored wall while walls with both anchor bolts and hold-downs installed are fully anchored wall.

Jang (2000) analysed the effect of connectors of shear walls with and without hold-down connectors. For this, the connectors were provided at both ends of shear walls and found that the racking resistance of shear walls increased when hold-down connectors were used. Jang argued that the hold-down connectors are required to simulate the vertical load applied to shear walls from the upper structures. Yasumura (2010) conducted test on single plywood panels using two hold-down bolts at both ends of walls connected to the studs with three bolts of 12 mm. The result indicated that there was a considerable decrease of strength by removing hold-down bolts at the end of opening especially in the case of those with door opening, and some reduction of the shear strength is necessary to remove the hold-down connecting studs at the end of opening. Varoglu et al. (2007) used the steel rods (16mm diameter) as hold down connectors connecting the top and bottom plates at each end of the midply test walls and their test result showed that the walls exhibited good ductility and prevented the premature failure of end stud tension. Ni and Karacabeyli (2000) performed a comparative full-scale shear wall specimens tested under lateral loads with and without hold-down connections. The tests result showed that the ultimate unit lateral load capacities were similar for shear walls with different wall lengths when hold-downs were fitted. A combination of nail withdrawal, nail pull-through, and nail chip-out was observed at the perimeter of the panels. In contrast, the ultimate unit lateral load capacities were different for shear walls without hold-downs and vertical loads. The unit lateral load capacity was strongly influenced by the wall aspect ratio; the load capacity varies inversely with the wall aspect ratio.

Girhammar and Kallsner (2004) studied the racking resistance of fully and partially anchored shear walls with different sheathing materials such as hardboard, particleboard, and plywood. The test showed that the failure mode of the fully anchored hardboard was ductile caused by yielding and withdrawal of nails, where the direction of the nail forces was parallel to the grain direction of the frame members whereas the failure mode of a partially anchored shear wall was semi-ductile caused by yielding and withdrawal of nails, but the direction of nail forces was perpendicular to the bottom rail. For the fully and partially anchored particleboard, they found the failure mode of the walls was of semi-brittle nature. For the fully and partially anchored plywood shear walls, they found the failure mode of the walls was brittle and semi-brittle respectively. Seaders et al. (2009) verified that the addition of hold-downs produced a large increase in load-carrying capacity, deformation capacity, and energy dissipation characteristics of the shear wall specimens. In their experiments conducted on partially and fully anchored wood frame shear walls, they concluded that failure mode of fully anchored walls differs from partially anchored wall due to change of load path of the hold-downs. They also found that partially anchored walls failed only in the sheathing to sill-plate nail connections and in the sill plate itself, irrespective of monotonic and cyclic loading protocols.

2.5 Conclusion

Timber Platform frame construction is widely recognised as an effective and efficient building method for multi-storey buildings and in particular, in residential dwellings. There are many research on the racking performance of timber frame walls however; there remains a gap in the knowledge in several key areas influencing the racking performance of the walls. These studies have limited objectives and have not addressed several client and architectural requirements with regard to effect of increase or decrease in the length of walls and effects of openings for doors and windows or possible design configurations such as effects of wall dimensions and aspects ratios, fixing types and details, size and positions of openings or the effects of the interaction between the adjoining walls or other components of the building which are essential for the racking performance of the walls.

Different countries use different test standards for determining racking performance of shear walls. In the UK, it is based on the guidelines established in Eurocode 5 and provided in PD 6693-1:2012. The modifications introduced in BS EN 594:2011, superseding BS EN 594:1996 may significantly change the input values to be used for determining the racking performance of the timber frame panels. There are not any studies on the compatibility and the suitability of these two test procedures for determining racking performance of shear walls. The OSB and gypsum board are widely used sheathing materials; no other sheathing materials such as Air/Vapour barrier OSB, Medite Vent panel, Medite Tricoya panel, and Fire resistant OSB boards seemed to be considered for determining racking performance of shear walls.

Furthermore, the existing literature included the effect of one-sided walls and double-sided shear walls using different sheathing boards such as plywood on one side and two sides, gypsum on one and two sides, and plywood on one side and gypsum on other side on racking performance. Similarly, the use of OSB on one side and GWB on another side was also considered in a few literature. However, the evaluation of the accuracy of the formulae proposed in the design code to determine the racking strength and stiffness of the OSB on one side and GWB on another side, specifically in the UK that practices the partially anchored walls, are missing in the existing literature. It is to be noted that the failure mode of fully anchored shear walls differs from partially anchored wall due to change of load path of the hold-downs, irrespective of monotonic and cyclic loading protocols.

Moreover, reviewing the literature, no research on racking performance of shear walls had considered the collective parameters such as vertical load, fastener size and spacing, wall length, and arrangement of studs and horizontal members. Either one or two of these parameters were studied in the existing literature. The size, types, length, and spacing of fasteners significantly influence the racking performance of the shear walls; the failure modes vary on different sheathing materials irrespective of loading conditions as well as framing connections and hold-down connections.

Analysis on the effects of aspects ratio (with regular and oversized walls) on racking performance were done with sheathing materials such as gypsum board, plywood, and OSB under different loading conditions. All research show that racking strength of the wall is proportional to the wall length and different sheathing boards have different performances in this regard. However, the racking performance of wall panels with different length using double end-studs and double bottom rail of walls sheathed with

OSB and walls sheathed with PB panels were not studied in any of the research available.

Many research on effect of fasteners were conducted in the standard shear walls only. However, there are no comprehensive studies on the new system of shear wall ‘Mid-ply wall’ that uses the mechanism of double shear system of fastener to prevent from nail failures. While in the UK, no investigation has been conducted to determine the performance of the mid-ply walls till date. No validated study seemed to have been conducted on the practical use of existing calculation models on the mid-ply system. Also, smooth nails were commonly used for analysing their influence on racking performance. However, it is also important to examine the influence of other types of nails such as ring shank nails and their performance on the racking resistance.

The size of openings also affects the racking stiffness of timber frame walls. From the existing research, it was found the door and window openings accounted for a significant decrease in the strength and stiffness of the walls and accelerated a change in the failure mode. Although, several research were conducted on the effects of opening on racking performance of the walls, there still remains some gaps in relation to the effects of using trimmers and spreaders on the walls, fasteners spacings, and sizes of openings in partially anchored walls.

Taking into considerations of the discussion above, this research aims to fill the gaps in determining and better understanding of the racking performance of the standard walls by using different geometrical parameters such as fastener sizes and spacing, wall lengths, arrangement of vertical studs and horizontal members, conducting walls test using two different design procedures with and without vertical load, different size of opening for windows and doors with and without trimmers and spreader, and conducting the test on “Mid-ply wall” to quantify experimental how these series of factors influence on the racking performance of walls which typical built in the United Kingdom that was not done in the past.

Chapter 3 Influence of test methods on racking performance

3.1 Introduction

The design procedure for determining the racking strength of timber framed walls in the UK is based on the guidelines established in Eurocode 5 and provided in PD 6693-1. However, the procedure for calculating racking strength using the wall panel racking test to BS EN 594 is not included in the PD. The BS 5268-6.1:1996, which was superseded by Eurocode 5, is still used and it includes a calculation method using BS EN 594:1996. The BS EN 594:2011 that superseded BS EN 594:1996 introduced significant changes in the test procedure, such as the removal of the stiffness cycle procedure as well as reduction in the test duration. These modifications in BS EN 594:2011 significantly alter the input values to be used in the determination of the racking performance of the timber frame panels. Hence, this chapter aims to examine the compatibility and suitability of the two different test procedures detailed in BS EN 594:1996 and BS EN 594:2011 versions on the racking performance of a series of timber framed walls. It first discusses the experimental and theoretical approaches for calculating racking performances. The effect of sheathing using OSB, Air/Vapour barrier OSB, Medite vent, Medite Tricoya, and fire resistant OSB on racking performance were examined using these two test methods. This chapter also examines the strength and stiffness of the walls affected due to the vertical loads of 0 kN and 25 kN using the two test methods.

3.2 Experimental procedures for calculating racking performance

The initial test method for the determination of the racking strength and stiffness of timber frame wall panel was developed by Griffiths (1987). Based on the Griffiths's research, the British Standard test method and the design method to determine racking resistance was derived. His work was codified and incorporated in BS 5268-6.1:1988, providing recommendations for the design, testing, fabrication, and erection of timber frame walls for dwellings not exceeding three storeys (BSI, 1988). The codified procedure in the BS 5268-6.1:1988 outlined that the racking resistance of timber frame can be derived by using either 'assessment method' or 'load testing'. Both of these methods are based on the use of a 'basic racking resistance' (R_b), which is modified by other factors to derive the racking strength value. In the assessment method, the basic racking resistance values of some materials were derived by Griffiths from test results and incorporated into the code, as shown in Table 3.1. For a timber wall frame constructed using a board material listed in Table 3.1, the associated basic racking resistance value to be used to derive the design racking strength is modified by various factors that include nail diameter, nail spacing, sheathing thickness, wall height, openings in the frame, and wall length. Likewise, the design racking strength for the load testing method is also derived, but using a basic racking resistance obtained from the results of a number of tests performed in accordance with section 5 of BS 5268-6.1:1988 or for the subsequent revision of this standard (BS 5268-6.1:1996), using BS EN 594 (this will be discussed later in this section) test results.

Table 3.1 Basic racking resistances for certain materials and combinations of materials BS 5268-6.1:1988

Primary board material	**Fixing**	**Racking resistance kN/m**	**Additional contribution of secondary board on timber frame wall: kN/m**	
			Category 2 or 3 materials	**Category 1 materials**
Category 1 materials: - 9.5 mm plywood - 9.0 mm medium board - 12.0 mm chipboard (type C3M, C4M or C5) - 6.0 mm tempered hardboard - 9.0 mm OSB/3	3.00 mm diameter wire nails at least 50 mm long, maximum spacing 150 mm on perimeter, 300 mm internal	1.68	0.28	0.84
Category 2 materials: - 12.5 mm bitumen impregnated insulation board	3.00 mm diameter wire nails at least 50 mm long, maximum spacing 75 mm on perimeter, 150 mm internal	0.9	0.45	1.06
- Separating wall of minimum 30 mm plasterboard (in two or more layers)	Each layer should be individually fixed with 2.65 mm diameter plasterboard nails at 150 mm spacing, nails for the outermost layer should be at least 60 mm long	0.9	0.45	1.06
Category 3 materials: - 12.5 mm plasterboard	2.65 mm diameter plasterboard nails at least 40 mm long, maximum spacing 150 mm	0.9	0.45	1.06

This was revised in 1996 to consider buildings up to four storeys and was extensively used by the designers. However; since 1996, designers have gained increasing experience with this form of construction and in the light of further research (notably the TF2000 project at BRE Cardington), this edition of BS 5268-6.1 has extended the scope to cover dwellings up to seven storeys high (BSI, 1996a). According to BS 5268-6.1:1996, the basic racking resistance is calculated in the following steps.

For the racking stiffness, the load should be calculated by averaging the racking stiffness load for similar panel tests. The racking stiffness load for each new panel is calculated as,

$$F_{stiff} = R \times 0.002 \times H_{wp} \times 1.25 \times K_{109} \tag{3.1}$$

Where,

R	average racking stiffness loads of similar panels (in kN/mm) (In the Table 3.2, Racking stiffness was calculated in accordance with equation 3.4 and 3.5 and then average of three similar panels was considered.)
H_{wp}	wall panel height (in mm) and
K_{109}	modification factor to account for the number of similar panels tested under the same conditions: for example, for one test K_{109} = 0.8, for three tests K_{109} = 0.93, and for five tests K_{109} = 1.0.

The racking strength load (F_{fail}, in kN) is calculated as,

$$F_{fail} = \frac{F_{max,min} \times K_{109}}{FoS} \tag{3.2}$$

Where,

$F_{max,min}$	lowest failure (or the maximum) racking load achieved during the tests of similar panels (in kN) (In the Table 3.2, minimum of the maximum racking load (F_{max}) of similar wall panel tests were considered.)
FoS	factor of safety for the type of sheathing or sheathing combination For any material or combination of two materials that includes plasterboard, FoS = 2.4 For any material or combination of two materials that excludes plasterboard, FoS = 1.6.

Now, the basic racking resistance (R_b, in kN/m) is calculated as,

$$R_b = \frac{min\{F_{stiff;}\ F_{fail}\}}{2.4\ K_{111}} \quad (3.3)$$

Where,

K_{111} modification factor to account for vertical loading on the studs

for no vertical load, K_{111}=1.0

for a load of 1 kN/stud, K_{111} = 1.18

for 2.5kN/stud, K_{111} = 1.43

for 5 kN/stud, K_{111} = 1.77

Note: Also refer to Appendix 3.1 for the example on calculation using above equations.

Moreover, the test method that was previously set in section 5 of BS 5268-6.1:1988 (now withdrawn) was superseded by the BS EN 594:1996. The prime amendment from 1988 was that it requires an estimated racking load ($F_{max,est}$) and the stiffness test was reduced effectively to two load cycles as opposed to four in the original test requirement. However, this modification did not warrant any change to the procedure that was used to derive the racking strength of the wall.

On the other hand, the BS EN 594:1996 adhered to the principle of the original test method, but the overall test cycle for a wall panel was reduced in order to shorten the duration of the test. The dimensions of panels as recommended by the BS EN 594:1996 (BSI, 1996b) is shown in Figure 3.1. It also noted that the number, location, and orientation of intermediate studs are not critical to the test panel. If the construction needs the sheets to be arranged with the long edge horizontal, the vertical joint can be replaced by a mid-height horizontal joint (Figure 3.1). The sheathing to one face of the panel will generally comprise of two sheets approximately 1.2 m × 2.4 m. If other sizes of sheet are required by the construction practice, these may be substituted, but must be configured to suit the 2.4 m × 2.4 m size of the timber frame. The test panels may include sheathings on both faces of the panel or more than one layer of sheathing on one face if required.

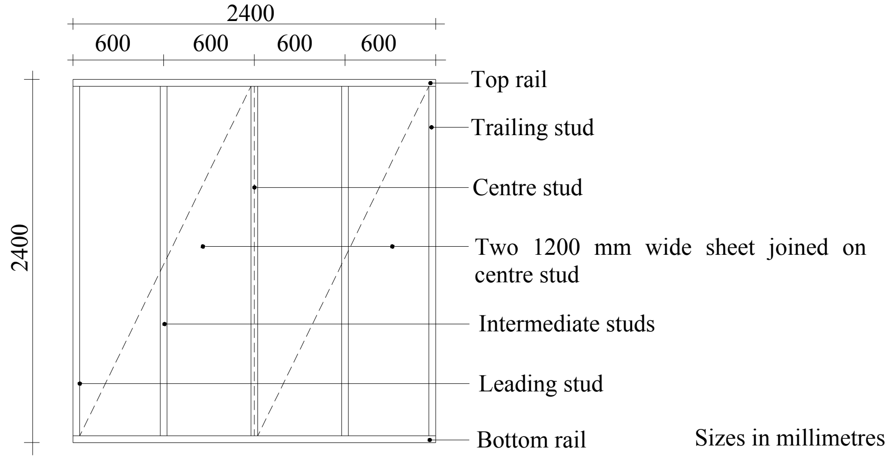


Figure 3.1 Racking test panels

The vertical loads F_v should be applied at the stud positions and the racking load must be applied at a constant rate of movement related to the displacement at point A (Figure 3.2). For loading and unloading up to 0.4 $F_{max,est}$ (estimated maximum racking load) the rate of loading shall be (2 ± 0.5)mm/min. For loading above 0.4 $F_{max,est}$, the rate of loading shall be (4 ± 1) mm/min. If $F_{max,est}$ for a test deviates by more than 20 % from the mean value of F_{max}, obtained for all similar tests, the value of racking stiffness for that test should be rejected. The displacements of the panel shall be monitored at points A, B, C. The deformations υ should be taken as the displacement at A minus the displacement at B. The displacement at C should be reported separately.

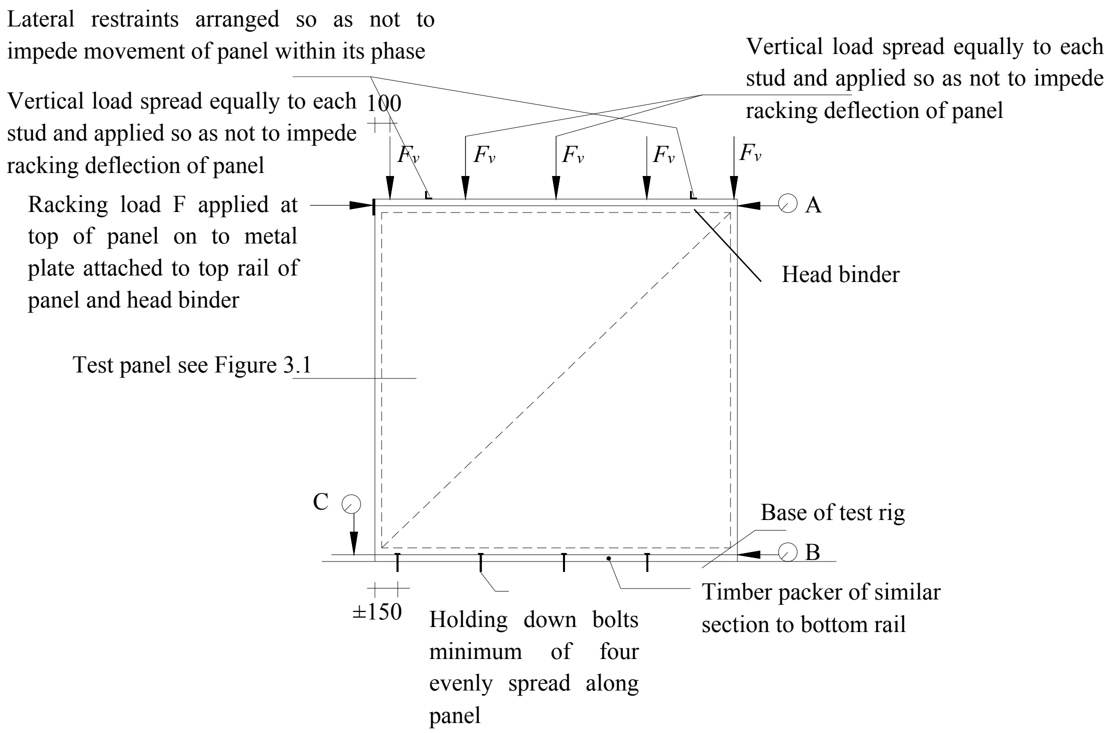


Figure 3.2 Test setup with racking and vertical loads and position of displacement

The procedure for applying racking load that included full test cycles is shown in Figure 3.3. When the vertical load F_v is applied in the stiffness or strength tests are less than 1 kN per stud, a vertical preload cycle is required. The vertical preloads of 1 kN per stud are applied for 120 s, which are then released allowing the panel to recover for a minimum of 300 s before continuing. In the stabilizing load cycle, the vertical load Fv is applied to the head binder at the stud positions, as shown in Figure 3.2 and maintained constant throughout the cycle. The racking load F is then be applied and increased to $0{,}1F_{max,est}$ and maintained for 120 s. It is then removed allowing the panel a recovery period of (600 ± 300) s before continuing with the strength test. The deformations υ_{01} to υ_{10} and the corresponding racking loads F_1 to F_{10} are recorded. In the strength test, the vertical loads applied in the stabilizing load cycle are maintained. The racking load F = 0.4 $F_{max,est}$ is then applied and maintained for 300 s. The racking load is then increased until F_{max} is reached when either the panel collapses or the panel attains a deformation υ of 100 mm, whichever occurs first. It should be ensured that 90 % of the racking load F_{max} is within (300 ± 120) s.

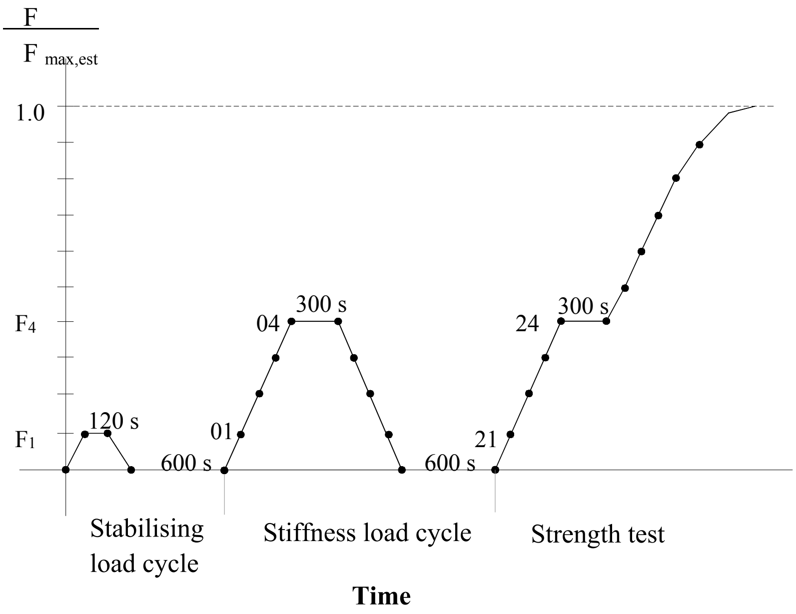


Figure 3.3 Racking load cycle - BS EN 594:1996

The racking stiffness according to BS EN 594:1996 is a calculated stiffness of a panel when it is loaded to approximately 40 % of its racking strength. It is determined as,

$$R = \frac{1}{2}\left[\frac{F_4 - F_1}{\upsilon_{04} - \upsilon_{01}} + \frac{F_{24} - F_{21}}{\upsilon_{24} - \upsilon_{21}}\right] \tag{3.4}$$

Where,

F_1 racking load of $0.1 \times F_{max,est}$ in Newtons, and υ_{01} is the deformation in millimetres

F_4 racking load of $0.4 \times F_{max,est}$ in Newtons, and υ_{04} is the deformation in millimetres

as determined in the stiffness test;

F_{21} racking load of $0.1 \times F_{max,est}$ in Newtons, and υ_{21} is the deformation in millimetres

F_{24} racking load of $0.4 \times F_{max,est}$ in Newtons, and υ_{24} is the deformation in millimetres

as determined in the strength test

The BS EN 594:1996 was superseded by BS EN594:2011; however, the design process is still ongoing based on BS 5268-6.1:1996. The revised standard BS EN594:2011 (BSI, 2011) introduced significant changes in the test procedure. The loading cycle requirement up to 40% of the failure load (which had been introduced to be able to derive stiffness properties of the wall panel at the stage when stability in load-displacement behaviour under this load level would have been considered to have been reached) has been removed and the overall test duration has been greatly reduced. The 2011 version of BS EN 594 attempted to increase the scope for more panel types and to allow a more straightforward comparison between results of different panels. The modified code excluded the stiffness cycle procedure and reduced the duration in the test. The requirement is to undertake a stabilising load cycle, where a vertical load of 1 kN is applied to the studs for a period of 120 s. following a recovery period of 600 ± 300 s. The strength test is conducted as shown in Figure 3.4. Since, the code excluded the stiffness load cycle procedure in the test method; the racking stiffness is derived from the strength test. It is calculated by taking load and deflection results from the test between 20% and 40% of the maximum load while it was previously between 10% and 40% of the maximum load. In addition, the test duration was reduced from about an hour to one requiring that percent of the racking load should be reached within 300 ± 120 s.

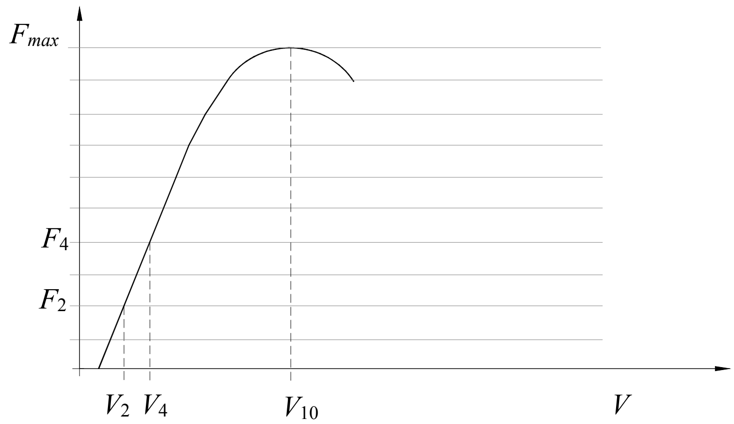


Figure 3.4 Load versus displacement test procedure - BS EN 594:2011

The racking stiffness is determined as,

$$R = \frac{F_4 - F_2}{v_4 - v_2} \tag{3.5}$$

Where,

F_2	racking load of 0.2 F_{max} in Newtons
F_4	racking load of 0.4 F_{max} in Newtons
v_2 and v_4	deformations in millimeters

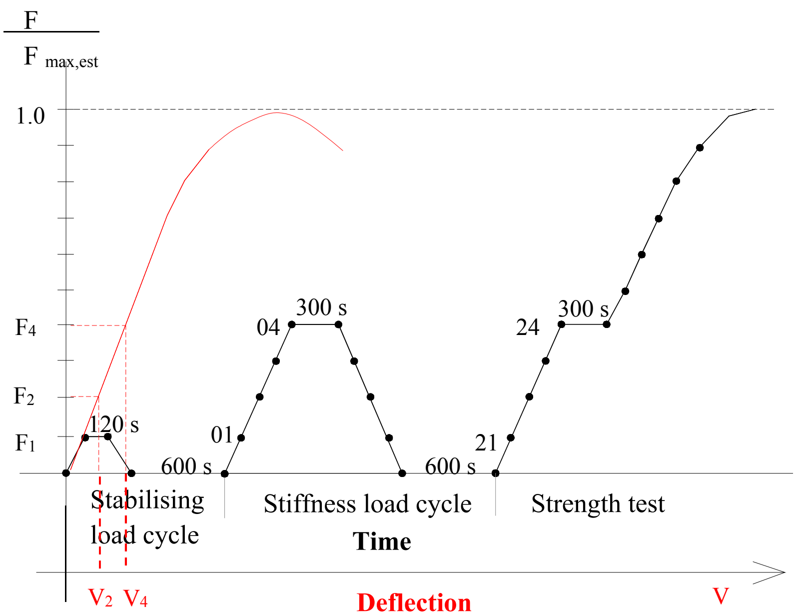


Figure 3.5 Combined Figure showing racking load cycle-BS EN 594:1996 (black coloured Load vs Time curve) and test procedure - BS EN 594:2011 (red coloured Load vs Deflection curve).

It is to be noted that the revisions to this standard took place at the time when BS 5268-6.1:1996 was in the process of being replaced by Eurocode 5 (EC5) and it is to be questioned that the linkage between the test procedure in BS 5268-6.1:1996 was considered in the revision process.

EC5 provides two simplified design methods for the determination of the racking strength of timber-frame wall systems (referred to in the code as wall diaphragms). The first (Method A) was developed to suit the construction procedure where racking walls are fully anchored at their ends, which is a method commonly used in mainland Europe countries, but not in the UK. The second (Method B) is an attempt to amend the UK racking procedure referred to in BS 5268-6.1, in which racking walls are generally connected to support structure along their lengths, to a limit states design procedure. However, some significant aspects of the UK method were excluded or inaccurately interpreted by the codifiers leading to the development of a unified method by the UK and European researchers (Griffiths et al., 2005b). This unified method was not adopted for the UK design; another method was developed instead and included in PD 6693-1 (BSI, 2012b). The UK National Annex to Eurocode 5 (BSI, 2012a) requirement emphasized the use of PD method to derive racking resistance rather than Method B. The racking strength method in PD 6693-1 is a design approach that draws on the design rules in Eurocode 5 and unlike the racking procedure in BS 5268-6.1:1996, there is at present no procedure for being able to use the results from racking wall tests to BS EN 594 in the PD method to derive racking strength. Where there is a requirement to derive the racking strength of timber-framed wall panels from the results of racking tests, the only calculation method that will currently permit this, is the design procedure given in BS 5268-6.1 (BSI, 1988, 1996a).

The adaptation of Eurocodes by the UK timber industry has been slow. The BS 5268-6.1:1996 is still considered as a design standard accepted in England and Wales by the Building Regulations. It is still being used by designers to determine the racking strength of timber frame wall systems. However, the test results obtained from BS EN 594:2011 are consistently lower than those obtained using the test procedure in BS EN 594:1996 leading to panels failing to achieve the basic racking resistance values specified in BS 5268-6.1:1996.

In order to determine the compatibility and suitability of the racking test method given in BS EN 594:2011 with the structural design method still used by engineers in the UK as detailed in BS 5268-6.1, the extensive experiments were conducted in the laboratory. The experimental programme included racking tests on a variety of wood-based panels with different vertical loading using both the 1996 and 2011 versions of BS EN 594 (BSI, 1996b, 2011). The test results are analysed, compared, and discussed in the following sections.

The determination of the test racking stiffness can have a direct effect on the determination of the basic racking strength and consequently on the design racking resistance of a timber frame wall. Therefore, any modifications in the test method or calculation method of the racking stiffness that will influence stiffness behaviour have the potential to affect the racking design strength of panel.

3.3 Experimental programme and results

3.3.1 Test setup and programme

Considering the 1996 and 2011 versions of BS EN 594 (BSI, 1996b, 2011), an extensive experimental programme was conducted on a range of sheathing panels, to evaluate the racking performance of timber frame panels using the above test methods. The timber frame wall panels consisted of a series of predetermined geometrically configured walls of 2.4 × 2.4 m in size comprising a range of OSB/3 panels, Air/Vapour barrier OSB, Medite Vent panel, Medite Tricoya panel and Fire resistant OSB boards, fixed to one side only of the timber framing. The wall frame was fabricated using studs/timber sections of 45 × 90 mm or 38 × 89 mm from C16 timbers (covering the range of section sizes used by construction industry). The panels were fixed using 3.0 mm-diameter × 50 mm-long round wire nails, with nailing density/patterns of 150/300 around the perimeter and at internal studs, as detailed in Table 3.3.

The wall panels were tested in an upright position as shown in Figure 3.6. The bottom rail was connected to the test bed using four M12 × 150 mm long bolts. Lateral restraints (to prevent lateral distortion) were provided by means of two pairs of rollers at the top plate (header level) which permitted free in-plane movement of the wall both in the vertical and horizontal directions. Loads were applied using two separate loadings systems.

- The racking load was applied by horizontal jack connected to an automatic/computerised loading and data acquisition system that followed a pre-programmed loading procedure based on either BS EN 594:1996 or BS EN 594:2011, as appropriate.
- The vertical loading, when used, was applied through an air bag pressurised to provide a constant 25 kN total vertical load, which in turn was transferred to the head binder at stud positions as point loads, through rollers.

For the vertical loading, the amount of required air pressure was calibrated for different increment of total vertical loading as shown in Table 3.2.

Table 3.2 Calibrated air pressure values for the vertical loading air-bag device

Pressure (bar)	Total vertical load (kN)
0.26	5
0.38	10
0.50	15
0.63	20
0.75	25

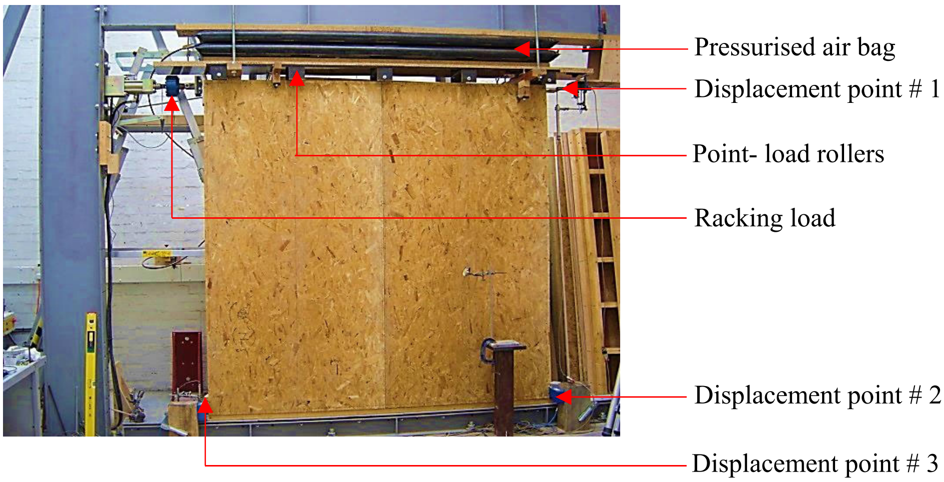


Figure 3.6 Racking test panel with vertical loads

Displacement transducers were used to record the horizontal movement of the walls at the leeward base (point #2) and the header levels (point # 1) and the vertical uplift of the lead stud, including any movement of the sole plate at this position, on the loaded side of the wall (point #3).

For tests to BS EN 594:2011 (BSI, 2011), a stabilising vertical load of 5 kN (in total) was applied through the air bag to the head binder at the stud positions and maintained for 120 s . The load was then removed and the panel was allowed to recover for a period of 600 s before the strength test was carried out. For walls under vertical loading, a constant vertical load of 25 kN was applied through the air bag to the head binder at the stud positions and maintained throughout the racking test which was monitored by using dial gauge as shown in the Figure 3.7 below. The horizontal racking load was then applied at a steady rate in which 90% of the maximum load was reached within 300 ± 120 s.

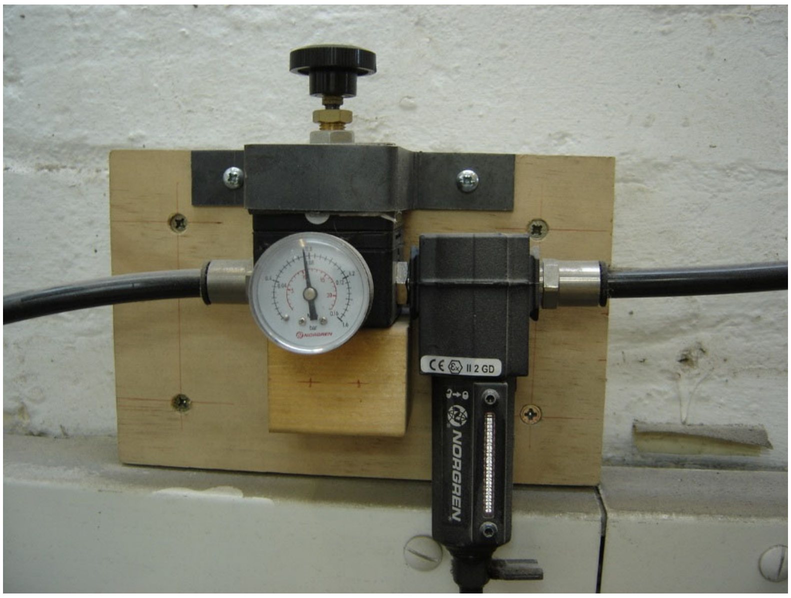
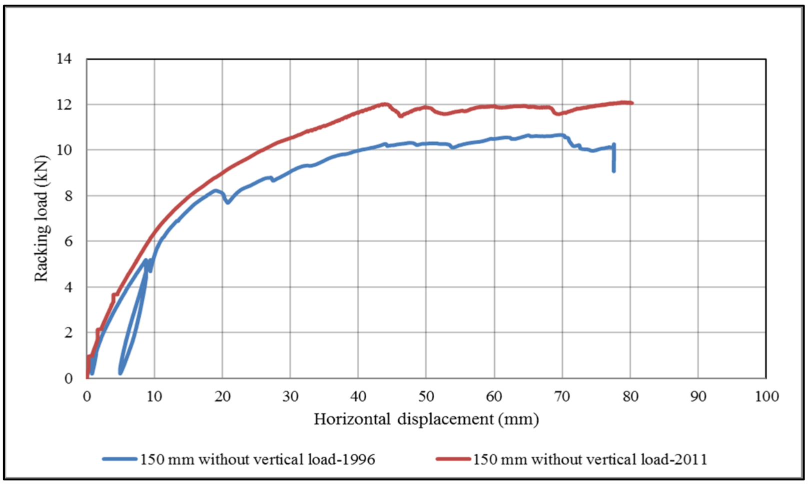


Figure 3.7 Dial pressure gauge used to monitor the air pressure to apply vertical load on the walls where 0.75 bar pressure gives the vertical load of 25kN.

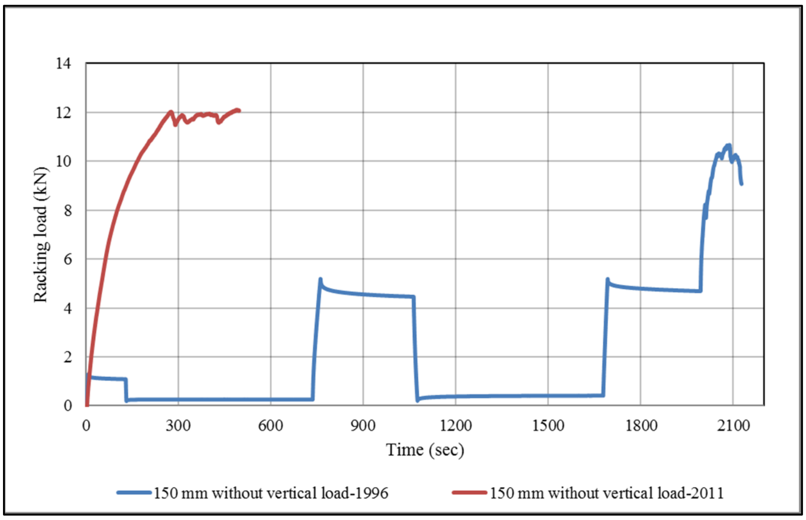
For tests to BS EN 594:1996 (BSI, 1996b), the test procedure was followed as described in section 6.4 of the standard, with the test loading applied as illustrated in Figure 3.3.

3.3.2 Test results

The results of the tests carried out are presented in Table 3.3. The racking stiffness (R) was calculated as recommended for the relevant test standard used. Guidance is given in BS 5268-6. (BSI, 1988, 1996a) on how to calculate the racking stiffness load (F_{stiff}), racking strength load (F_{fail}) and the basic racking resistance (R_b) and the calculated values of these functions are also given in Table 3.3 for the walls tested in accordance with the requirements of BS EN 594:1996 (BSI, 1996b) or BS EN 594:2011 (BSI, 2011) (also refer Figure 3.5 and Figure 3.8 for the typical example) where appropriate.



(a)



(b)

Figure 3.8 Typical examples of load vs. (a) displacement and (b) time in accordance with BS EN 594:1996 (blue coloured) and BS EN 594:2011 (red coloured)

The failure behaviour of all wall panels was recorded as recommended in the test standard. In general, ductile failure behaviour was observed in all instances.

The results show that, for all tests undertaken in accordance with BS EN 594:1996, racking resistance, R_b for the sheathing panel material used exceeds the value of 1.68 given in BS 5268-6.1 (BSI, 1996a) (Table 3.1). However, for the wall panels tested using the same panel material type but in accordance with BS EN 594:2011 and analysed using the method given in BS 5268-6.1, no wall panel achieved the category 1 requirements as defined in BS 5268-6.1 and shown in Table 3.1 except in test 15. It is also to be noted that, apart from test 2, the critical design condition was always due to stiffness rather than strength behaviour.

Table 3.3 Experimental test programme and results

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
Walls with OSB sheathing																	
10		9	45 x 90	3.0 x 5.0	150/300	5	2011		19.85	1298.01							
15	1	9	45 x 90	3.0 x 5.0	150/300	5	2011	3	20.70	1225.00	19.32	1210	11.23	6.75	1.59	8.10	1.91
16		9	45 x 90	3.0 x 5.0	150/300	5	2011		19.32	1105.86							
STDEV									0.57	79.20							
13		9	45 x 90	3.0 x 5.0	150/300	5	1996		18.20	2079.58							
14	2	9	45 x 90	3.0 x 5.0	150/300	5	1996	3	20.97	2286.86	18.20	2246	10.58	12.53	2.49	12.53	2.49
17		9	45 x 90	3.0 x 5.0	150/300	5	1996		21.23	2370.60							
STDEV									1.37	122.32							
1		9	45 x 90	3.0 x 5.0	150/300	0	2011		12.10	572.33							
3	3	9	45 x 90	3.0 x 5.0	150/300	0	2011	3	14.75	492.84	12.10	597	7.04	3.33	1.39	4.00	1.67
5		9	45 x 90	3.0 x 5.0	150/300	0	2011		13.36	726.42							
STDEV									1.08	96.97							

Contd….

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
2		9	45 x 90	3.0 x 5.0	150/300	0	1996		10.67	834.26							
4	4	9	45 x 90	3.0 x 5.0	150/300	0	1996	3	15.57	840.35	10.67	850	6.2	4.74	1.98	4.74	1.98
6		9	45 x 90	3.0 x 5.0	150/300	0	1996		14.57	876.18							
STDEV									2.12	18.49							
19		11	45 x 90	3.0 x 5.0	150/300	5	2011		20.80	1172.83							
20	5	11	45 x 90	3.0 x 5.0	150/300	5	2011	3	20.84	977.35	18.75	1080	10.90	6.02	1.42	7.23	1.70
23		11	45 x 90	3.0 x 5.0	150/300	5	2011		18.75	1088.82							
STDEV									0.98	80.07							
21		11	45 x 90	3.0 x 5.0	150/300	5	1996		20.17	1655.46							
22	6	11	45 x 90	3.0 x 5.0	150/300	5	1996	3	22.60	1964.16	20.17	1813	11.72	10.12	2.38	10.12	2.38
24		11	45 x 90	3.0 x 5.0	150/300	5	1996		22.89	1818.78							
STDEV									1.22	126.10							

Contd

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
11	7	9	45 x 90	3.0 x 5.0	150/300	5	2011	3	20.70	1399.53	20.70	1222	12.03	6.82	1.60	8.18	1.93
28		9	45 x 90	3.0 x 5.0	150/300	5	2011		20.80	1218.25							
29		9	45 x 90	3.0 x 5.0	150/300	5	2011		21.01	1047.34							
STDEV									0.13	143.80							
25	8	9	45 x 90	3.0 x 5.0	150/300	5	1996	3	20.31	2015.28	20.16	2032	11.72	11.34	2.67	11.34	2.67
26		9	45 x 90	3.0 x 5.0	150/300	5	1996		20.16	2194.41							
27		9	45 x 90	3.0 x 5.0	150/300	5	1996		21.81	1886.94							
STDEV									0.75	126.09							
Walls with Air/Vapour barrier OSB sheathing																	
1	9	12.5	38 x 89	2.8 x 5.0	150/300	0	2011	3	11.47	537.48	11.47	645	6.67	3.60	1.50	4.32	1.80
2		12.5	38 x 89	2.8 x 5.0	150/300	0	2011		12.08	599.63							
3		12.5	38 x 89	2.8 x 5.0	150/300	0	2011		12.56	796.39							
STDEV									0.45	110.36							

Contd…

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
4		12.5	38 x 89	2.8 x 5.0	150/300	0	1996		10.47	949.89							
5	10	12.5	38 x 89	2.8 x 5.0	150/300	0	1996	3	10.65	1133.26	10.43	1054	6.06	5.88	2.45	5.88	2.45
6		12.5	38 x 89	2.8 x 5.0	150/300	0	1996		10.43	1077.96							
STDEV									0.10	76.80							
17		12.5	38 x 89	2.8 x 5.0	150/300	5	1996		22.85	2068.07							
18	11	12.5	38 x 89	2.8 x 5.0	150/300	5	1996	3	23.22	1709.96	21.89	1911	12.72	10.66	2.51	10.66	2.51
19		12.5	38 x 89	2.8 x 5.0	150/300	5	1996		21.89	1955.24							
STDEV									0.56	149.49							
Walls with Medite Vent sheathing																	
7	12	12	38 x 89	2.8 x 5.0	150/300	0	2011	3	10.32	697.09	10.32	679	6.00	3.79	1.58	4.55	1.89
11		12	38 x 89	2.8 x 5.0	150/300	0	2011		10.82	694.36							
13		12	38 x 89	2.8 x 5.0	150/300	0	2011		11.29	645.16							
STDEV									0.40	23.86							

Contd

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
8	13	12	38 x 89	2.8 x 5.0	150/300	0	1996	3	11.75	759.46	9.07	758	5.27	4.23	1.76	4.23	1.76
9		12	38 x 89	2.8 x 5.0	150/300	0	1996		9.66	783.08							
10		12	38 x 89	2.8 x 5.0	150/300	0	1996		9.07	731.93							
STDEV									1.15	20.90							
14	14	12	38 x 89	2.8 x 5.0	150/300	5	1996	3	18.84	1483.68	18.41	1534	10.70	8.56	2.01	8.56	2.01
15		12	38 x 89	2.8 x 5.0	150/300	5	1996		18.41	1587.06							
16		12	38 x 89	2.8 x 5.0	150/300	5	1996		18.9	1530.62							
STDEV									0.22	42.26							
Walls with Medite Tricoya sheathing																	
1	15	9	38 x 89	2.8 x 5.0	150/300	0	2011	3	10.96	718.25	10.17	947	5.91	5.28	2.20	6.34	2.64
2		9	38 x 89	2.8 x 5.0	150/300	0	2011		10.39	894.63							
3		9	38 x 89	2.8 x 5.0	150/300	0	2011		10.17	1227.89							
STDEV									0.33	211.32							

Contd…

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
4	16	9	38 x 89	2.8 x 5.0	150/300	0	1996	3	9.51	1191.25	9.51	1152	5.53	6.43	2.30	6.43	2.30
5		9	38 x 89	2.8 x 5.0	150/300	0	1996		12.31	1042.66							
6		9	38 x 89	2.8 x 5.0	150/300	0	1996		11.37	1222.52							
STDEV									1.16	78.46							
7	17	9	38 x 89	2.8 x 5.0	150/300	5	1996	3	23.06	1947.98	23.06	1974	13.40	11.01	2.59	11.01	2.59
8		9	38 x 89	2.8 x 5.0	150/300	5	1996		24.71	2092.57							
9		9	38 x 89	2.8 x 5.0	150/300	5	1996		25.44	1880.27							
STDEV									1.00	88.55							
Walls with Fire resistance OSB sheathing																	
7	18	11	38 x 89	2.8 x 5.0	150/300	0	1996	3	12.66	954.03	12.23	902	7.11	5.04	2.10	5.04	2.10
8		11	38 x 89	2.8 x 5.0	150/300	0	1996		13.3	999.13							
9		11	38 x 89	2.8 x 5.0	150/300	0	1996		12.23	753.95							
STDEV									0.44	106.55							

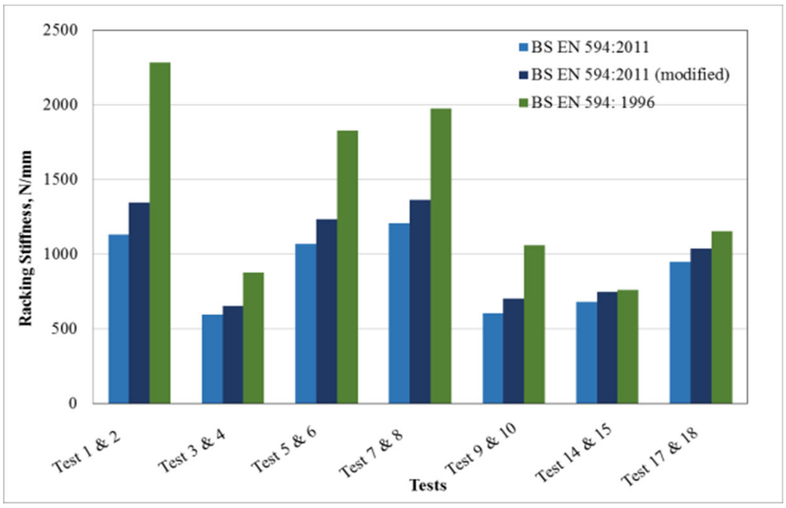
Contd…

Wall sample	Test	Thickness	Stud section	Nails		Vertical load	BS EN594	Number of wall tests	Racking test results								
				Φ x L	Spacing				F_{max}	*Stiffness*	$F_{max, min}$	R	F_{fail}	F_{stiff}	R_b	F_{1stiff}	R_{1b}
		mm	mm	mm	mm	kN/m	version		kN	N/mm	kN	N/mm	kN	kN	kN/mm	kN	kN/mm
10	19	11	38 x 89	2.8 x 5.0	150/300	5	1996	3	23.29	1963.41	20.43	1910	11.87	10.66	2.51	10.66	2.51
11		11	38 x 89	2.8 x 5.0	150/300	5	1996		20.43	1835.54							
12		11	38 x 89	2.8 x 5.0	150/300	5	1996		22.5	1931.18							
STDEV									1.21	54.30							

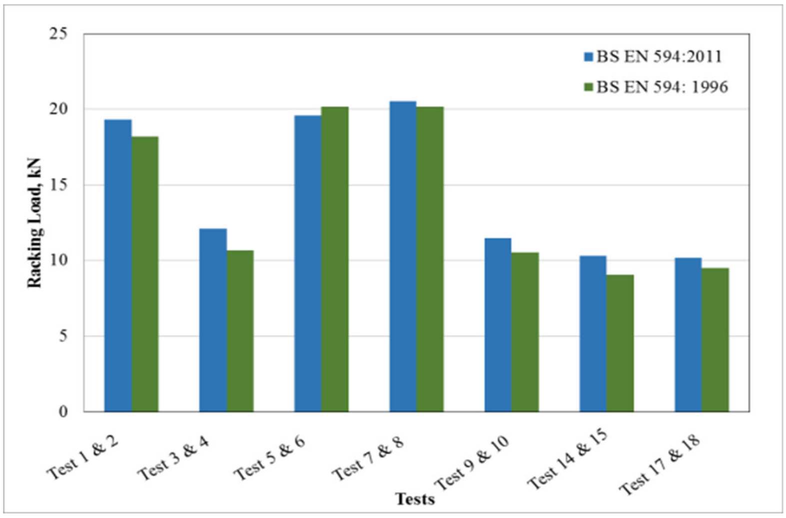
3.3.3 Analysis and discussion

i. Test methods

The results of the experimental programme indicate that there is a clear difference in the basic racking resistance for the same wood-based panels when tested to the two different versions of the European standard BS EN 594 (BSI, 1996b and 2011). As discussed in Section 3.2, with the reduction in the test duration from about an hour to requiring the test to be completed within 300 ± 120 s, as well as the exclusion of the stiffness load cycle and the changes in the stiffness calculation method, when using BS EN 594:2011 there are likely to be consequences on panel behaviour and this was shown to be the case by the test results.



(a)



(b)

Figure 3.9 Comparison of racking stiffness (a) and load (b) for identical wall panels tested under different test procedures

From the test results, a comparison of the strength at failure and stiffness at the serviceability condition between identical wall panels tested to BS EN 594:1996 and to BS EN 594:2011 is shown in Figure 3.9. For strength behaviour, as shown in Figure 3.9 (b), the racking strength load is similar for both test procedures and for the walls tested there was an average variation between results of just -2.7% for the walls with OSB/3 panels. Similarly, for AV/B OSB, Medite, Vent and Medite Tricoya Extreme the average variation between the results of racking strength load were 10.4%, 6.4%, and 5.2% recorded respectively The revised test procedure appears to have no significant effect on the failure strength of a wall panel.

For stiffness behaviour, however, shown in Figure 3.9 (a), there is a clear difference between the results of the wall panels tested under the two procedures. Tests to BS EN 594:1996 consistently resulted in stiffness values greater than those derived from BS EN 594:2011; on average, the stiffness of say OSB/3 walls were over 46.6% and for AV/B OSB walls, Medite Vent walls and Extreme Medite Vent walls were in average over 37.6%, 11.7% and 21.7% greater than the stiffness of identical wall panels to BS EN 594:2011.

The results of wall panels tested under similar loading but to the 1996 and the 2011 test procedures of BS EN 594 are compared in Table 3.3. The variation, in percentage terms, between the maximum racking loads (F_{max}) and between the racking stiffness values (R) for the respective test procedures have been calculated. In addition, for those panels tested in accordance with the BS EN 594:1996 procedures, Table 3.3 also include the stiffness results for each of the two load cycles defined in Figure 3.3.

The two load cycles are referred to as R_1 and R_2, which were calculated using,

$$R_1 = \left[\frac{F_{04}-F_{01}}{v_{04}-v_{01}}\right] \tag{3.6}$$

$$R_2 = \left[\frac{F_{24}-F_{21}}{v_{24}-v_{21}}\right] \tag{3.7}$$

Table 3.3 compares the strength and stiffness values of the tests illustrated in Figure 3.8 and, in addition, also show that the stiffness calculated as part of the strength test described in Figure 3.3 (R_2) is consistently higher than the stiffness calculated as part of the stiffness cycle (R_1). The stiffness increase ranges from 21% to 57% with an average value 39 %. This demonstrates that wall panels subjected to cyclic loading up to 40 % of the failure load (i.e. within the serviceability limit state) will stiffen up under repeated racking loading and this was the behaviour that Griffiths (1987) took into account when developing the original test procedure for racking walls incorporated into BS 5268-6.1:1996 (BSI, 1996a). By deleting the load cycling procedure, the stiffness will be reduced and this is clearly demonstrated from the test results.

Table 3.4 Result variations between panels tested under different test procedures

	BS EN 594:2011	**BS EN 594:1996**	**Variation %**
	Test 1	**Test 2**	
Fmax: kN	20.22	20.13	- 0.4
R: N/mm	1252.4	2281.15	82.1
R1: N/mm	-	1899.51	51.7
R2: N/mm	-	2662.78	112.6
	Test 3	**Test 4**	
Fmax: kN	13.41	13.6	1.5
R: N/mm	597.15	876.36	31.9
R1: N/mm	-	706.02	18.2
R2: N/mm	-	1046.7	75.3
	Test 5	**Test 6**	
Fmax: kN	20.4	21.89	6.8
R: N/mm	1068.17	1828.16	41.6
R1: N/mm	-	1556.68	45.7
R2: N/mm	-	2002.02	87.4
	Test 7	**Test 8**	
Fmax: kN	20.78	20.76	- 0.1
R: N/mm	1207.24	1975.96	38.9
R1: N/mm	-	1539.96	27.6
R2: N/mm	-	2411.95	99.8
	Test 9	**Test 10**	
Fmax: kN	12.76	10.59	- 21.4
R: N/mm	650.6	1059.5	38.6
R1: N/mm	-	959.68	47.5
R2: N/mm	-	1159.33	78.2
	Test 12	**Test 13**	
Fmax: kN	10.81	10.16	-6.0
R: N/mm	678.87	758.16	11.7
R1: N/mm	-	709.09	4.5
R2: N/mm	-	807.23	18.9
	Test 15	**Test 16**	
Fmax: kN	10.51	11.06	5.3
R: N/mm	946.92	1152.14	21.7
R1: N/mm	-	1013.33	7.0
R2: N/mm	-	1290.96	36.3

However, the variation between stiffness results to BS EN 594:1996 (BSI, 1996b) and BS EN 594:2011 (BSI, 2011) may not be fully attributed to the difference in the number of test cycles. It is anticipated that the initial settling vertical load of 1 kN/stud does not eliminated any slack in the plane of the wall panel and therefore has only limited effect on the initial stiffness result. It is therefore considered that the rate of loading and stiffness calculation method between 20% and 40% of the maximum load play a part in reducing the stiffness values compared with tests to BS EN 594:1996. As the test programme did not include any tests with varying loading rates, only the stiffness calculation method is evaluated here.

To further compare the effect of the different procedures, a stiffness value was calculated between 10% and 40% of the maximum load for all tests to BS EN 594:2011 and, from the initial stiffness, the new stiffness calculated were on average 11.5% greater. This is shown on Figure 3.9 (a) as the "modified" stiffness values. However, this method of calculation of the stiffness according to BS EN 594:2011 still does not compare with the results obtained when using BS EN 594:1996. The comparison between modified 10% & 40% with 20% and 40% of BS EN 594:2011 is shown in the Figure 3.10.

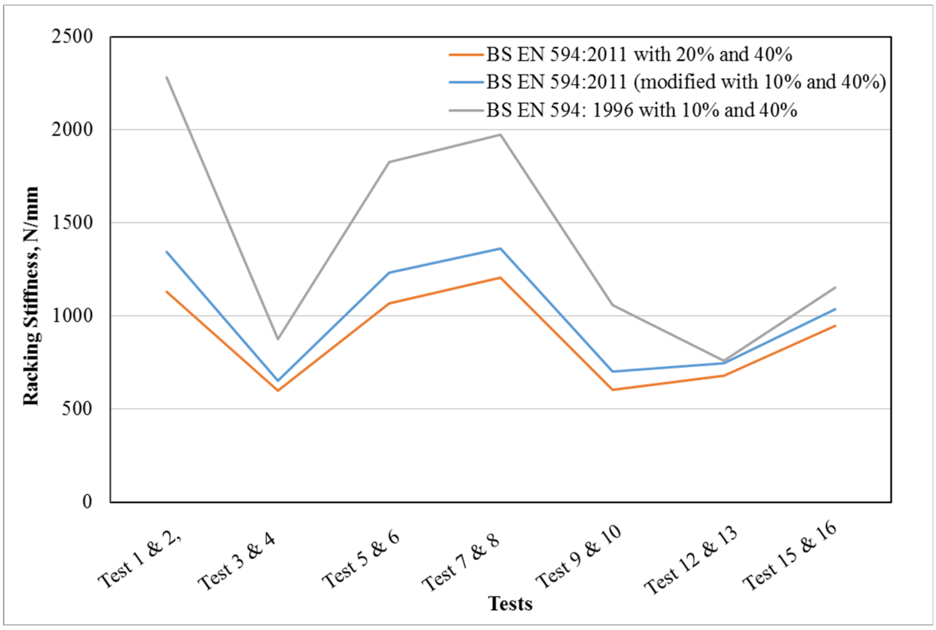


Figure 3.10 Comparison of racking stiffness for identical wall panels tested under different test procedures. (The comparison between modified 10% & 40% with 20% and 40% of BS EN 594:2011 also shown in the figure).

A significant issue affecting these results is how the value of the racking stiffness load (F_{stiff}) is calculated. The stiffness test procedure in BS EN 594:1996 involves more than one load cycle and to convert this frequency of loading to an equivalent once in 50 year wind return period single cycle condition, required for the derivation of the racking stiffness load, a factor of 1.25 is applied. The 1.25 factor is shown in Equation 3.1 (Griffiths, 1987).

In the BS EN 594:2011 test procedures there is no stiffness load cycling and so for tests to this standard the use of this factor is not appropriate. The racking stiffness load at a deformation of 0.003 x the wall panel height (F_{stiff}) will be derived from R × 0.003 × wall panel height and this relationship was used to derive this load for the test results from the BS EN 594:2011 test procedure. The racking stiffness load calculated on this basis for these tests is given in the column headed F_{1stiff} in Table 3.3, together with the racking stiffness loads for those tests undertaken using the BS EN 594:1996 test procedure and calculated in accordance with the requirements of BS 5268-6.1 (BSI, 1996a). The value of the basic racking resistance derived from the F_{fail} and F_{1stiff} values calculated in accordance with the requirements of Equation 3.3 are given in the column headed R_{1b} in Table 3.3. A comparison of the basic racking resistance values for the tests carried out using the BS EN 594:2011 test procedure and derived using the above approach with those derived using the BS 5268-6.1 (BSI, 1996a) requirement is given in Table 3.5 and this shows that the average increase in the value is 20.1%.

Table 3.5 Comparison of R_b and R_{1b} values from Table 3.3

Test	R_b	R_{1b}	Increase: %
1	1.49	1.78	19.6
3	1.39	1.67	19.9
5	1.4	1.68	20.3
7	1.58	1.9	20.4
9	1.4	1.68	20.1
12	1.58	1.89	19.6
15	2.20	2.64	20

To confirm the revised value derived for the racking stiffness load would not exceed the deflection limit of 7.2 mm (i.e. 0.003 × 2.4 m), the deflection behaviour of each wall panel at its racking stiffness load was checked and shown to be less than this value. As an example and further check, the racking strength stiffness used in the derivation of R_{1b} for each OSB wall test reference was compared with the equivalent test stiffness derived on a conservative basis by using the test load at a deformation of 7.2 mm; in all instances the stiffness value was less than the equivalent value obtained from the test.

Table 3.6 Comparison of ratio of R_{1b} values for similar tests

Test	Respective R_{1b} values	Ratio of respective R_{1b} values
2, 1	2.49, 1.78	1.4
4, 3	2.04, 1.67	1.22
6, 5	2.40, 1.68	1.43
8, 7	2.60, 1.90	1.37
10, 9	2.46, 1.68	1.46
13,12	1.76, 1.89	0.93
16,15	2.30, 2.64	0.87

Comparing the value of the basic racking resistance (R_{1b}) for wall panel tests having the same configuration but tested under BS EN 594:2011 and 1996 regimes, the ratios of the respective test values are listed in Table 3.5. From these results, the average increase in value is approximately 37%, which gives an indication of the possible effect of the removal of the cyclic loading regime and the overall test period reduction associated with the 2011 procedure.

ii. Effect of vertical loading

When the racking walls were tested in accordance with the requirement of BS EN 594:2011 and BS EN 594:1996 under 0 kN and 25 kN, these walls behaved differently. Both the strength and stiffness of the walls increased significantly when these walls were tested using different sheathing materials in accordance with BS EN 594:1996 under the vertical load of 25 kN compared to the walls tested without vertical load (refer Table 3.3). As a result, the basic racking resistance tested under 25 kN vertical load was higher compared to the walls tested under 0 kN vertical load. However, the basic racking resistance of all tested walls for all sheathing materials were higher than the requirement stated in BS 5268-6.1 (category 1), which is 1.68 kN/m for both with and without vertical loads. In case of walls tested accordance with BS EN 594:2011, the basic racking resistance were lower than that stated in BS 5268-6.1 (category 1) for both with and without vertical loads, except Test 15 that consists of wall sheathed with Medite Tricoya Panel that was recorded as 2.2 kN/m as shown in Table 3.3.

3.4 Conclusions

In the UK, the current design procedure for determining the racking strength of timber-framed wall panels is subjected to the design rules in Eurocode 5 and is presented in PD

6693-1. However, there is currently no procedure that uses the results from the racking strength and stiffness test standard BS EN 594 to calculate racking strength by the PD method.

BS 5268-6.1:1996, which was superseded by Eurocode 5 but is still permitted for use under Building Regulations (England and Wales) is directly linked to BS EN 594, enabling calculation of racking strength from the results of wall panel tests. The BS EN 594:1996 was fully compatible with the design procedure given in BS 5268-6.1 for calculation of the racking strength of wall panels. The 2011 version, a revision of the test standard, BS EN 594:2011 was undertaken to increase the scope for more types of panels and to allow a more straightforward comparison between results of different panels. However, the 2011 revision included significant changes in the test procedure. The loading cycle requirement up to 40% of the failure load, which had been introduced to be able to derive stiffness properties of the wall panel at the stage when stability in load-displacement behaviour under this load level would have been considered to have been reached, had been removed and the overall test duration had been greatly reduced.
It has been found by the industry that the basic racking resistance values given in BS 5268-6.1, which was derived from tests in accordance with the 1996 version of BS EN 594 (BSI, 1996b), could not be achieved when the same types of sheathing panel were tested in accordance with the requirements of the 2011 revision. This was investigated by conducting a number of wall tests to each version of the BS EN 594 standard.

From a programme of wall panel tests conducted on panels formed using OSB/3 sheathing incorporating variations in panel thickness, vertical loadings, and wall framing sections, the results showed that the failure strength of walls tested to BS EN 594:2011 and BS EN 594:1996 are comparable. However, the stiffness values calculated for similar wall panels showed that results to BS EN 594:1996 were over 46% greater than the stiffness of panels tested to BS EN 594:2011. Because of this difference, when applying the procedure given in BS 5268-6.1 to calculate the value of the basic racking resistance, it was always significantly lower under the procedure in BS EN 594:2011 than in the procedure in BS EN 594: 1996. Furthermore, the basic racking resistance values derived from the tests to BS EN 594:2011 were always less than the basic resistance value given in Table 3.3 in BS EN 5268-6.1, confirming the views expressed by the industry.

The stiffness procedure in BS EN 594:1996 was based on the application of four load cycles and, to convert to the equivalent single annual load cycle load condition, a factor of 1.25 is incorporated in the BS 5268-6.1:1996 calculation procedure. As the BS EN 594:2011 test procedure for stiffness behaviour only uses the equivalent of one load cycle, use of this factor is inappropriate. When the factor 1.25 is not used, the value of the basic raking resistance is increased and with the exception of one result (i.e. 1.67 for test 3) lower than the code value (1.68), the results from all other test is equal or exceed the design value Table 3.3.

Although the approach provides a method for calculating, the basic racking resistance that removes the effects of the stiffness cycle procedure to account for the changed loading requirement in the 2011 revision of BS EN 594, the results of the test programme show there will still be a significant underestimation of the value of the basic racking resistance from that which would be achievable had testing to BS EN 594:1996 been used.

Whilst the Building Regulation permit the use of BS 5268-6.1:1996 to derive the racking strength of timber frame walls, it is recommended that the test procedure used to derive the basic racking resistance value should remain as that given in BS EN 594:1996.

Under the vertical load of 25 kN, the strength and stiffness of the walls increased significantly compared to the walls tested without vertical load in accordance with BS EN 594:1996. The basic racking resistance of all tested walls for all sheathing materials were higher than the requirement stated in BS 5268-6.1 (category 1) for both with and without vertical loads.

When the materials A/V barrier OSB, Medite Vent, and Medite Tricoya were tested in accordance with BS EN 594:2011 under 0 kN vertical load, the strength were 15%, 23% and 26% respectively lower than OSB. However, A/V barrier performed better in stiffness with 62% higher, then followed by Medite Vent 31% and Medite Tricoya 46% when compared to the stiffness of OSB. Medite Tricoya due to enhanced material properties, performed exceptionally well with basic racking resistance of 2.2kN/m when tested to BS EN 594:2011 procedure under 0 kN vertical load. Medite Tricoya also performed well when tested in accordance of BS EN 594:1996 under vertical load of 25 kN which was 11 % and 3% higher both in strength and stiffness respectively when compared with OSB except the stiffness of FR OSB which was 8% higher than OSB.

Chapter 4 Parametric evaluation of racking performance

4.1 Introduction

This chapter provides a quantitative assessment of the racking performance of partially anchored timber framed walls, based on experimental tests conducted on walls with Oriented Stranded Board (OSB) and British Gypsum Plaster Board (PB) sheathings. A total of 17 OSB sheathed timber framed wall specimens and 15 PB sheathed timber framed wall specimens, constructed from a combination of materials under different load configurations, were tested. The experimental study was designed to examine the influence of a range of geometrical parameters, such as fastener size and spacing, wall length, arrangement of studs and horizontal members, and the effect of vertical loading on the racking strength and stiffness of the walls. The experimental results were then compared with results obtained from design rules, as given in the relevant European standards, to determine the racking performance of the walls. The chapter has also assessed the differences between the experimental results and the design racking values obtained from the relevant European standards, in particular, the requirement of the UK National Annex to Eurocode 5 (EC5), on design for racking strength of timber framed walls using the procedure described in the PD 6693-1 document. Double sided walls with OSB sheathing on one side and PB sheathing on other were also tested to examine the combined effects of sheathings on the racking performance of timber framed walls.

4.2 Background

In timber frame construction, racking walls are often classified in two categories: fully anchored and partially anchored walls. Fully anchored walls are walls which are prevented from lifting, when subjected to a lateral load, by the use of anchors (such as steel brackets) secured to underlying support structure or by the weight/actions the wall supports. For partially anchored walls, resistance against lifting is provided solely by the fixings between the sheathing and the bottom rail and fixings between the bottom rail connection to the support structure. Because of the absence of holding down ties in partially anchored walls, the studs experience a moderately high amount of uplift when the wall is subjected to in-plane racking loads. In the UK, the most common form of racking wall used in Platform timber construction is the partially anchored wall.

4.2.1 Wall specimens

For OSB sheathed walls, all wall specimens were assembled using C16 (BSI, 2009a) white spruce timber with a cross-section of 44 mm × 95 mm, for the frame members, whilst for sheathing, 9 mm thick Oriented Strand Boards (OSB/3) (BSI, 2006) were used. As reported in Table 4.2, two sizes of bright smooth wire nails were used for OSB panel-to-frame connections: 2.8 mm diameter × 49 mm long and 3.0 mm diameter × 52 mm long. Header beam and bottom rail were fixed to the studs by using 75 mm long screws with a smooth shank diameter of 3.2 mm (see Figure 4.1). For each specimen, the nail spacing of the sheathing panels along the intermediate studs was set at twice the perimeter nail spacing. The effects of use of additional studs and bottom rails were examined by doubling studs at the leeward and windward sides of the wall specimens by screwing together two (44 mm wide × 95 mm deep) timber members at 345 mm centres. The panel-to-frame fixings along the double studs and double bottom rail were spaced at 100 mm on two staggered rows, effectively providing pairs of fasteners spaced at 100 mm (see Figure 4.2 b).

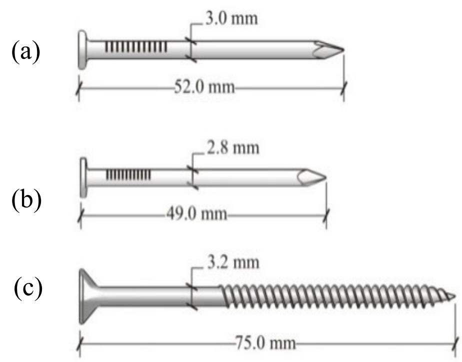
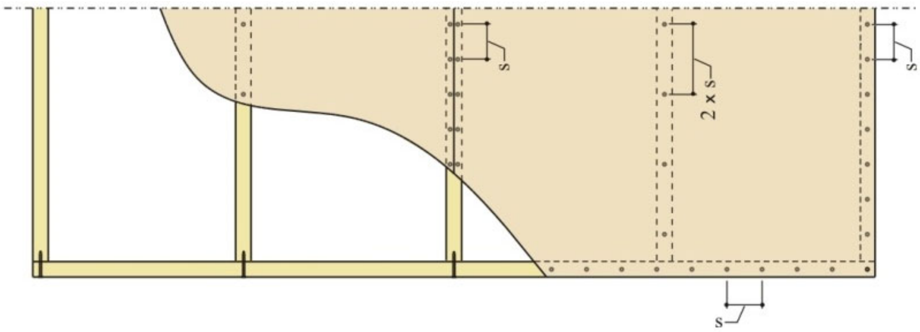


Figure 4.1 Fastener sizes and type.

(a) and (b) bright wire nails, used for the OSB panel-to - frame fixing; (c) screws used for the stud-to-beam connections

For PB sheathed walls, all wall specimens were assembled using C16 white spruce timber with a cross-section of 44 mm × 95 mm for the frame members and 12.5 mm thick British gypsum plasterboard. The wall panels were fixed using 3.5 mm diameter × 40 mm length drywall screws at 100 mm centers along the perimeter of the walls. To compare the stiffness and racking strength of plasterboard with OSB, the dimensions and configurations of plasterboard wall panels were kept similar to that of OSB wall panel set-up, except the fasteners, which were replaced by the drywall screws (also refer Figure 4.2 b).

(a)



(b)

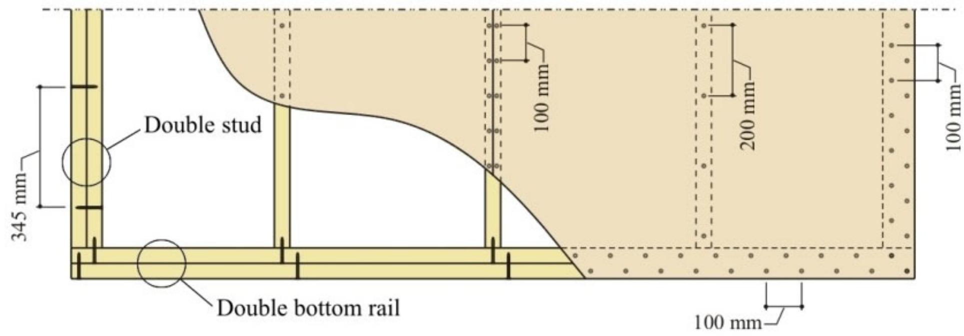


Figure 4.2 Wall specimen

(a) standard frame, (b) frame with double end-studs and double bottom rail.

For racking performance of walls with OSB sheathing on one side and PB sheathing on other side of the wall, the tests were conducted using the dimension and configurations similar to those mentioned above.

4.2.2 Test set-up

The racking tests were carried out according to BS EN 594:2011 requirements (BSI, 2011). With reference to Figure 4.3, a sole plate was positioned between the bottom rail of each wall specimen and the test rig base, and the bottom rail was fixed to the test bed by four 12 mm diameter bolts. The load was then applied by a load actuator at the top-left corner of the wall, whilst two linear transducers (LVDT-1 and LVDT-2) were used to take readings of the horizontal deformations.

The racking deformation of the wall (Δh) was calculated as the difference between the horizontal displacement of the header beam (LVDT-1) and the rigid body horizontal translation of the wall (LVDT-2). In order to avoid lateral movement of the wall specimens tested, a system of bracing and rollers was devised for the purpose.

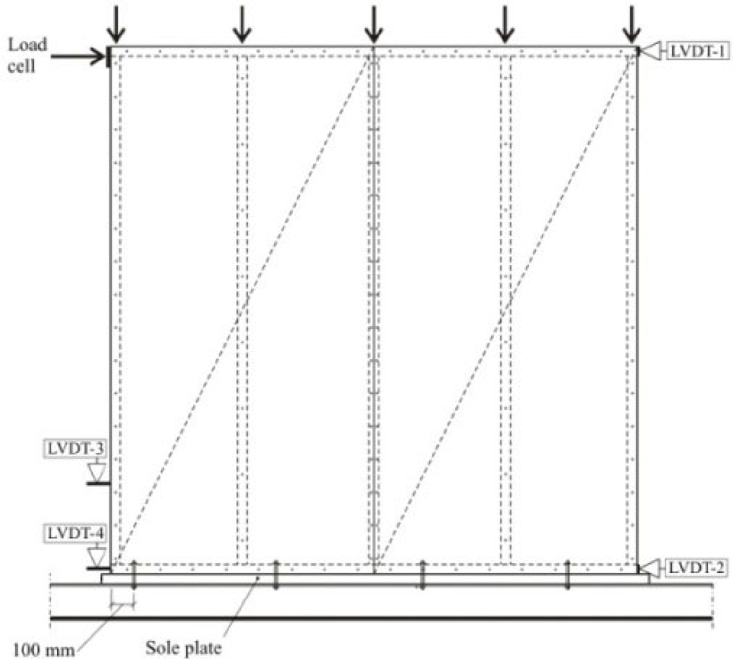


Figure 4.3 Racking test set up in accordance with BS EN 594:2011

Vertical load

The vertical load, where relevant, was applied by the use of a pressurised airbag, sandwiched between two plywood panels, and located between the header beam of the wall specimen and the overlying loading rig cross-bar (see Figure 4.4).

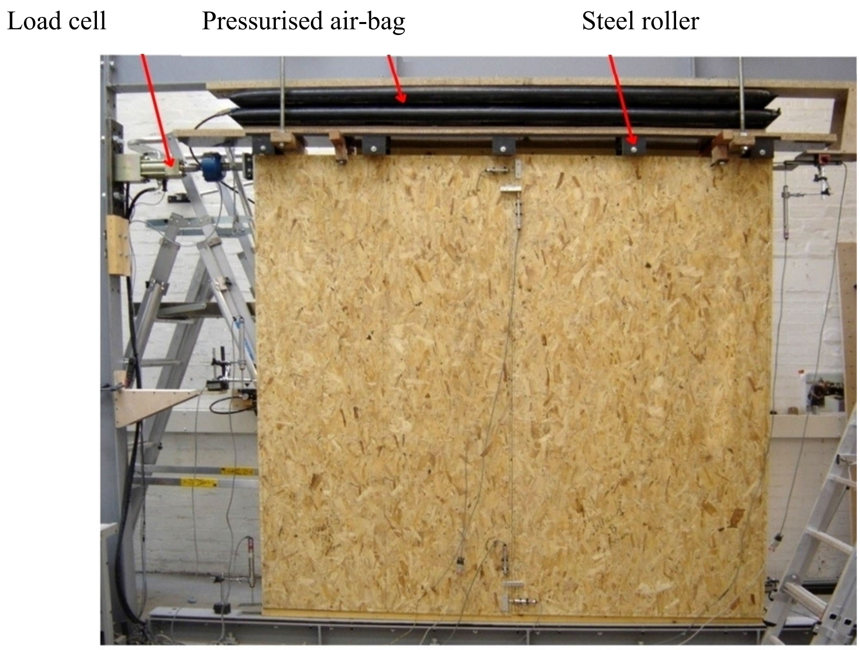


Figure 4.4 Application of vertical loading by air-bag device and steel rollers system.

Moisture content and density

Representative values of moisture content and density were determined from samples of the timber, OSB, and PB sheathing material used for the wall racking tests. The values are reported in Table 4.1.

Table 4.1 Moisture content and density values from tested walls.

Material	**Average density [kg/m^3]**	**Average moisture content [%]**
Timber - C16	375	13.0
OSB/3	591	5.5
PB	538	21

To avoid frictional forces affecting the racking test results, the air-bag device was placed on steel rollers positioned close to top of each stud, hence simulating the path of vertical loading transferred to the wall from horizontal floor joists. The required air pressure was calibrated for different increments of total vertical loading.

4.2.3 Test series

For racking tests on OSB walls, four series of tests on wall specimens, all with constant height of 2.4 m, were carried out, totalling 17 wall specimen tests. A detailed description of each wall specimen, corresponding test result and test series, are given in Table 4.2 (also see Figure 4.5). For racking tests on gypsum plasterboard walls, three series of tests on wall specimens, all with constant height of 2.4 m, were carried out, totalling 15 wall specimen tests; the detailed description of each wall specimen with their corresponding test result and test series are given in Table 4.3 (also see Figure 4.6) and for double sided walls with OSB on one side and PB sheathings on other the corresponding test results and test series are given in Table 4.4.

Table 4.2 Wall specimens with OSB sheathing - summary of test series and results

Test ID	Wall length (mm)	Frame type	No. of studs	Vertical load (kN)	Nail size (mm)	Nail[a] spacings (mm)	Experimental results: Strength (kN)	Stiffness[b] (kN/mm)
I-1	2400	standard	5	0	2.8 × 49	50	23.13	1.647
I-2						100	19.79	0.708
I-3						150	13.10	0.408
II-1	2400	standard	5	25	3.0 × 52	50	40.72	1.774
II-2						100	30.18	1.483
II-3						150	21.46	1.430
III-1	300	standard[c]	2	0	2.8 × 49	100	0.89	0.015
III-2	600		2				2.36	0.066
III-3	900		3				3.06	0.162
III-4	1200		3				7.24	0.206
III-5	1800		4				9.08	0.358
IV-1	300	Double end studs & double bottom rail	2	0	2.8 × 49	100[d]	1.04	0.017
IV-2	600		2				3.53	0.059
IV-3	900		3				6.72	0.182
IV-4	1200		3				10.74	0.278
IV-5	1800		4				16.69	0.599
IV-6	2400		5				25.82	0.938

[a]of the perimenter panel-to panel connections.

[b]as from Eq. (4.10)

[c]see Figure 4.5

[d]along two staggered rows, as shown in Figure 4.2 b.

Table 4.3 Wall specimens with Plasterboard sheathing - summary of test series and results

Test ID	Wall length (mm)	Frame type	No. of studs	Screw size (mm)	Screw spacings (mm)	Experimental results: Strength (kN)	Stiffness[b] (kN/mm)
PBI-1	2400	standard	5	3.5 × 40	50	15.25	0.772
PBI-2					100	9.15	0.856
PBI-3					150	7.16	0.521
PBI-4					300	3.81	0.375
PBII-1	300	standard[c]	2	3.5 × 40	100	0.48	0.013
PBII-2	600		2			1.5	0.039
PBII-3	900		3			3.30	0.124
PBII-4	1200		3			4.6	0.179
PBII-5	1800		4			6.44	0.497
PBIII-1	300	Double end studs & double bottom rail	2	3.5 × 40	100	0.77	0.014
PBIII-2	600		2			2.06	0.048
PBIII-3	900		3			5.99	0.173
PBIII-4	1200		3			7.16	0.283
PBIII-5	1800		4			8.55	0.544
PBIII-6	2400		5			15.94	0.873

[b]as from Eq. (4.10); [c]see Figure 4.5

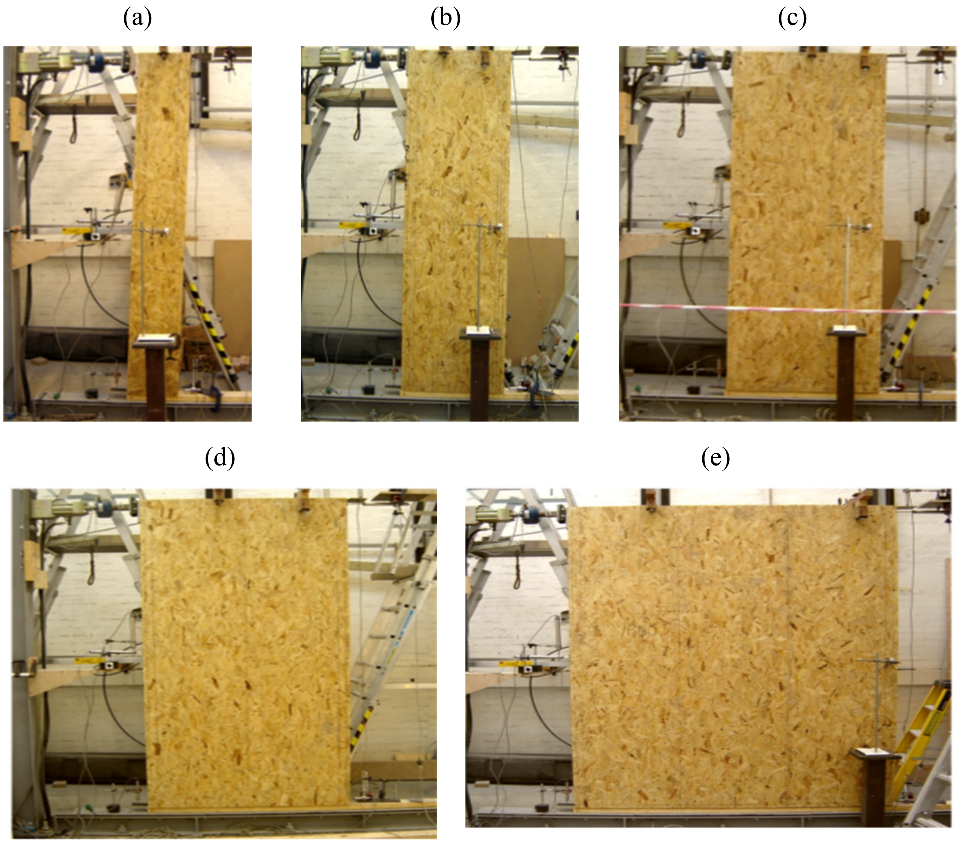


Figure 4.5 Racking test series III, Table 4.2, on timber walls with different length, L: (a) L = 300 mm, (b) L = 600 mm, (c) L = 900 mm, (d) L = 1200 mm, (e) L = 1800 mm.

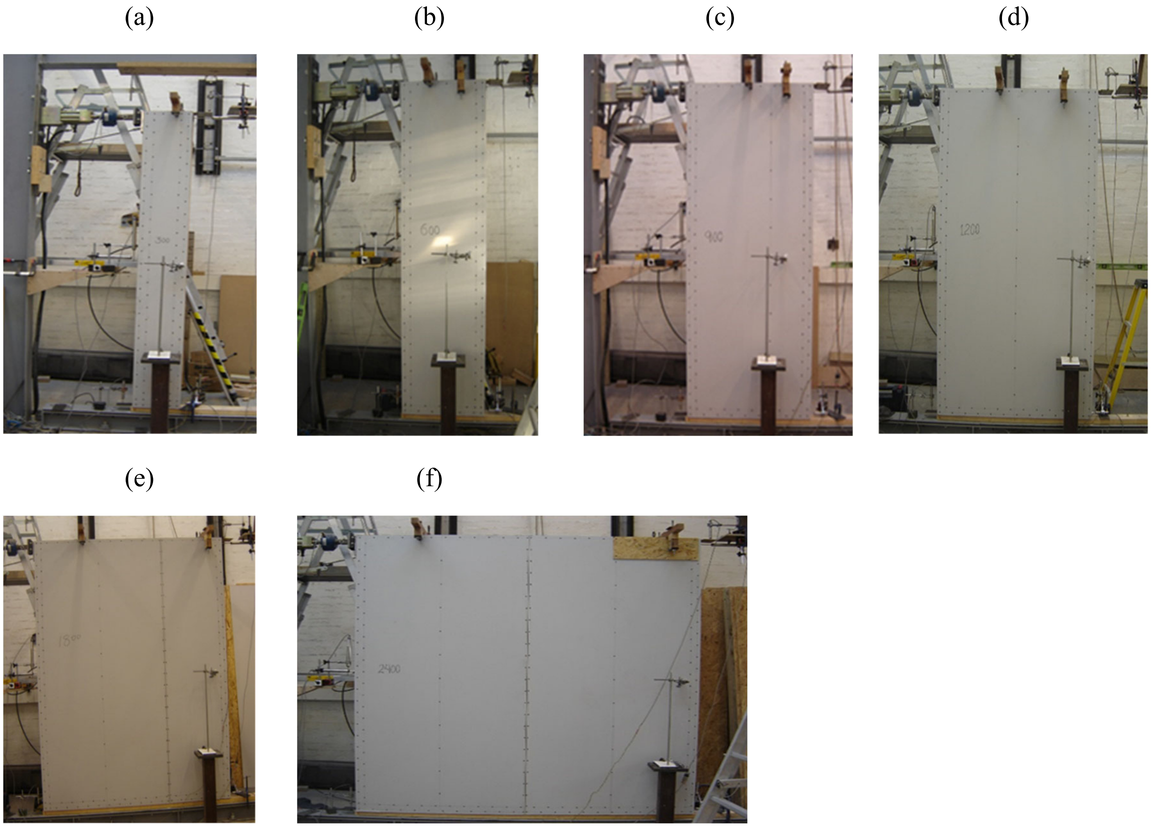


Figure 4.6 Racking test series III Table 4.3 on timber walls with different length, L: (a) L = 300 mm, (b) L = 600 mm, (c) L = 900 mm, (d) L = 1200 mm, (e) L = 1800 mm, (f) L = 2400 mm

Table 4.4 Double sided wall specimens with OSB and plasterboard sheathings - summary of test series and results

<table>
<tr><th rowspan="2">Test ID</th><th rowspan="2">Wall length
(mm)</th><th rowspan="2">Frame type</th><th rowspan="2">No. of studs</th><th rowspan="2">Vertical load
(kN)</th><th rowspan="2">Fastener size
(mm)</th><th rowspan="2" colspan="2">Fastener spacings
(mm)</th><th colspan="2">Experimental results:</th></tr>
<tr><th>Strength
(kN)</th><th>Stiffness[b]
(kN/mm)</th></tr>
<tr><td>I-1</td><td rowspan="3">2400</td><td rowspan="3">OSB standard</td><td rowspan="3">5</td><td rowspan="3">0</td><td rowspan="3">2.8 × 49</td><td colspan="2">50</td><td>23.14</td><td>1.022</td></tr>
<tr><td>I-2</td><td colspan="2">100</td><td>19.76</td><td>0.736</td></tr>
<tr><td>I-3</td><td colspan="2">150</td><td>13.1</td><td>0.417</td></tr>
<tr><td>PBII-1</td><td rowspan="4">2400</td><td rowspan="4">PB standard</td><td rowspan="4">5</td><td rowspan="4">0</td><td rowspan="4">3.0 × 52</td><td colspan="2">50</td><td>15.25</td><td>0.772</td></tr>
<tr><td>PBII-2</td><td colspan="2">100</td><td>9.46</td><td>0.734</td></tr>
<tr><td>PBII-3</td><td colspan="2">150</td><td>7.16</td><td>0.521</td></tr>
<tr><td>PBII-4</td><td colspan="2">300</td><td>3.81</td><td>0.375</td></tr>
<tr><td>DIII-1</td><td rowspan="3">2400</td><td rowspan="3">Double sided standard[c]</td><td rowspan="3">5</td><td rowspan="3">0</td><td rowspan="3">On OSB - 2.8 × 49

On PB - 3.0 × 52</td><td>50</td><td rowspan="3">100</td><td>23.66</td><td>1.202</td></tr>
<tr><td>DIII-2</td><td>100</td><td>21.58</td><td>0.991</td></tr>
<tr><td>DIII-3</td><td>150</td><td>21.1</td><td>0.947</td></tr>
</table>

[b]as from Eq. (4.10); [c]see Figure 4.5

4.2.4 PD 6693-1 method overview

The method described in PD 6693-1 is a semi-empirical approach mainly based on the development of a plastic theory model introduced by Källsner and Girhammar (2004, 2005) to predict the racking strength of partially anchored framed wall diaphragms. According to the PD method: when the panel-to-frame fasteners are fixed at uniform spacings, a lower bond value for the racking strength of the wall (indicated in this research as $P_{h,max}$) can be determined by considering the panel-to-frame fastener strength per unit length, f_{pd}, cumulated along a certain length, l_{eff} , and acting at the bottom of the wall:

$$P_{h,max}=f_{pd}\,l_{eff} \tag{4.1}$$

i. Fastener strength per unit length

The value of f_{pd} is derived by dividing the mean strength value of the panel-to-frame fasteners, $F_{v,mean}$, by the fastener spacing s:

$$f_{\mathrm{pd}}=\frac{F_{v,mean}}{s} \tag{4.2}$$

As pointed out in (Porteous and Kermani, 2013), the reason for using a mean strength value in Eq. (4.2) instead of a characteristic 5-percentile value, is because when a significant number of fasteners are loaded in a line configuration (e.g. along the bottom of the wall) it is unlikely that all these fasteners will only achieve the minimum failure strength i.e. characteristic strength value. According to the PD 6693-1 method, the mean strength value for the panel-to-frame connections is derived from the characteristic (5-percentile) value, $F_{v,Rk}$, increased by a minimum of 20 % (for s = 50 mm) up to a maximum of 30 % (i.e. for s = 150 mm):

$$F_{v,mean}=(1.15+s)F_{v,Rk} \tag{4.3}$$

In order for Eq. (4.3) to be valid, the value of s has to be expressed in m. For OSB panel-to-frame connections, the value of $F_{v,Rk}$ can be derived by following the EC5 procedure based on the Johansen plastic model (1949) to determine the strength of laterally loaded connections formed using metal dowel fasteners. As all of the fasteners will be in single shear for all of the wall test configurations, the characteristic load-carrying capacity of the connection will be obtained from EC5 Eq. (8.6), and the critical mode of failure for both nail sizes and materials considered in this study, will be failure mode (d):

$$F_{v,Rk}=1.05\,\frac{f_{h,1,k}t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}d_{t1^2}}}-\beta\right]+\frac{F_{ax,Rk}}{4} \tag{4.4}$$

in which:

t_1	thickness of the sheathing panel, in mm.
d	nominal nail diameter, in mm.
$f_{h,2,k}$	characteristic embedment strength of the sheathing panel in N/mm^2, which for OSB panels is taken as equal to $65d^{-0.7}t_1^{0.1}$ (EC5 Eq. (8.22)).
$\beta=\frac{f_{h,2,k}}{f_{h,1,k}}$	with $f_{h,1,k}$ being the characteristic embedment strength, of the timber frame members, in N/mm^2, which is equal to $0.082\rho_k d^{-0.3}$ (EC5 Eq. (8.15)), with ρ_k = 310 kg/m^3 (BSI, 2001)
$M_{y,Rk}$	characteristic yield moment of the nail in Nmm, taken as equal to: $0.3 f_u d^{2.6}$, (EC5 Eq. (8.14)), and the wire tensile strength f_u, is taken to be 600 N/mm^2.
$F_{ax,Rk}$	withdrawal capacity of the nail, taken as the minimum value between that obtained from EC5 Eq. (8.24) and 60% of the first term in Eq. (4.4), i.e. in agreement with the requirement of EC5 clause 8.2.2.(2) for round nails.

The mean load carrying capacity, $F_{v,mean}$, for OSB panel-to-frame connection made with bright smooth wire nails, has been calculated from Eqs. (4.3) and (4.4). In addition, for the same type of connection, $F_{v,mean}$ has also been derived from experimental tests on OSB panel-to-frame connection samples. The test procedure used, together with the results, are briefly described in Appendix 4.1 and a summary of the $F_{v,mean}$ values is given in Table 4.5.

Table 4.5 Load carrying capacity of the OSB panel-to-frame connection, $F_{v,mean}$

Nail size (mm)	**Nail spacing, s** (mm)	$F_{v,mean}$ **as from EC5**[a] (N)	$F_{v,mean}$ **as from tests**[b] (N)
2.8 × 49	50	667	779
	100	694	
	150	722	
3.0 × 52	50	730	1256
	100	760	
	150	791	

[a]Eqs. (4.3) and (4.4) in this chapter.

[b]See Appendix 4.2.

ii. Effective anchoring length

Having derived the relevant values of f_{pd}, the remaining parameter to insert into Eq.(4.1) in order to obtain the theoretical racking strength of the wall, is the effective anchoring length l_{eff}, which is obtained from:

$$l_{eff} = \frac{H}{\mu} + \left[\frac{H^2}{\mu^2} + L^2\left(1 + \frac{2M}{\mu f_{pd} L^2}\right)\right]^{0.5} \tag{4.5}$$

Where, H and L are the height and base length of the wall respectively; M is the stabilising moment at the leeward side of the wall, which, for the walls being tested, will equate to:

$$M = Q\frac{L}{2} \tag{4.6}$$

and Q is the total load in kN acting along the top of the wall:

The term μ in Eq. (4.5) is the ratio between the withdrawal capacity of the connections fixing the wall to the underlying structure per unit length (f_{ax}) and the panel-to-frame fastener strength per unit length (f_{pd}):

$$\mu = \frac{f_{ax1}}{f_{pd}} \tag{4.7}$$

For values of strength ratio per unit length f_{ax}/f_{pd} greater than 1, μ must be set equal to unity. This is because when $f_{ax} > f_{pd}$, the failure condition will be dictated by the strength of the panel-to-frame connections. For all of the racking tests described in this research, the base rail of the walls are anchored to the test rig basement by bolts (see section 4.3.2), and so μ = 1. Another validity requirement concerns the value of the effective anchoring length, which is subjected to the following inequality conditions:
If l_{eff} as from Eq.(4.5),

$$\begin{cases} > 1 \Rightarrow l_{\text{eff}} = \text{L} \\ < 0 \Rightarrow l_{\text{eff}} = 0 \end{cases} \tag{4.8}$$

Finally, for walls formed using wood based panel material, in order to limit the racking deflection to an acceptable serviceability load condition, the empirical relationship given in clause 21.5.2.3 of the PD-6693-1 document must be met. The relationship has been rearranged to suit the format used in this chapter, taking into account the type of walls being investigated, and is:

$$\frac{f_{d,pd}\, l_{1,eff}}{L} \leq 8\frac{L}{H} \tag{4.9}$$

Where, $f_{d,pd} = (k_{mod} f_{pd}) / \gamma M$. For the type of materials used in the wall and for the test programme undertaken under service class 1 conditions, the values for the modification factors are set according to the UK National Annex to EC5 (BSI, 2009c) i.e. $k_{mod} = 1.0$ and $\gamma M = 1.3$. The value for $l_{1,eff}$ is derived from Eq. (4.5) with f_{pd} being replaced by $f_{d;pd}$.

4.3 Results, Analysis, and Discussion

4.3.1 Racking strength

The experimental load-displacement curves, obtained for the OSB, PB and double sided wall specimens tested are shown in Figure 4.7, Figure 4.8**,** and Figure 4.9 respectively. From these figures, it has been possible to derive the variation of racking strength as a function of the nail spacing and wall length parameters (section 4.3.1 i and ii), enabling a comparison to be made between the experimental results and the values calculated by using the analytical procedure described in the PD 6693-1 method. The experimental load-displacement curves allowed also a quantitative investigation on how the variation of nail spacing and wall length affect the racking stiffness of the timber framed wall (section 4.3.2 i and ii). The experimental values for the ultimate racking load and racking stiffness values for OSB, PB, and double sided walls are given in Table 4.2,

Table 4.3 and Table 4.4 respectively. The analytical procedure described in section 4.2.4 has been used to compute the racking strength values of the tested walls, and comparison with the test results is provided in the following subsections.

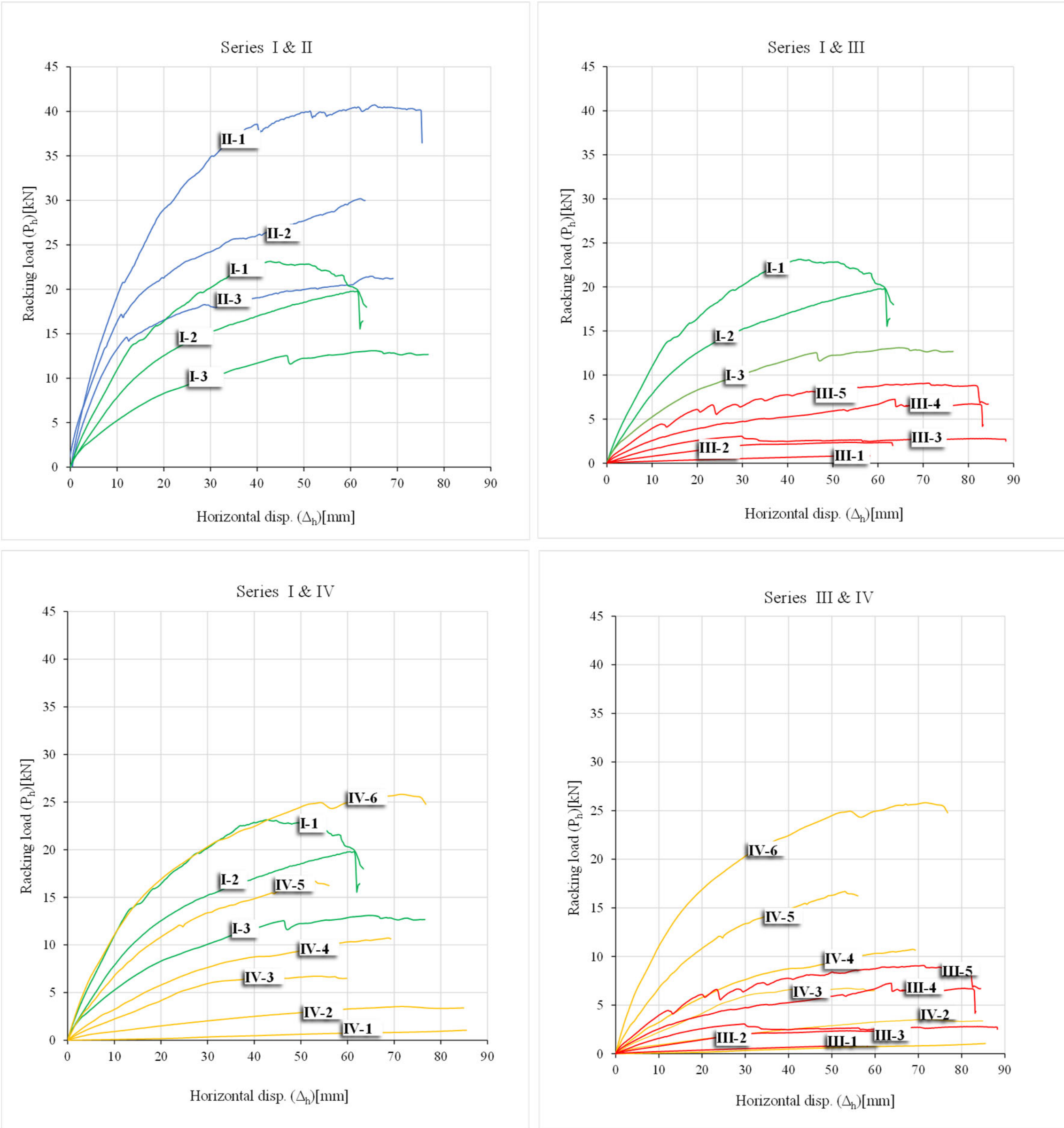


Figure 4.7 P_h-Δ_h curves and corresponding test ID, as given in Table 4.2 for OSB walls.

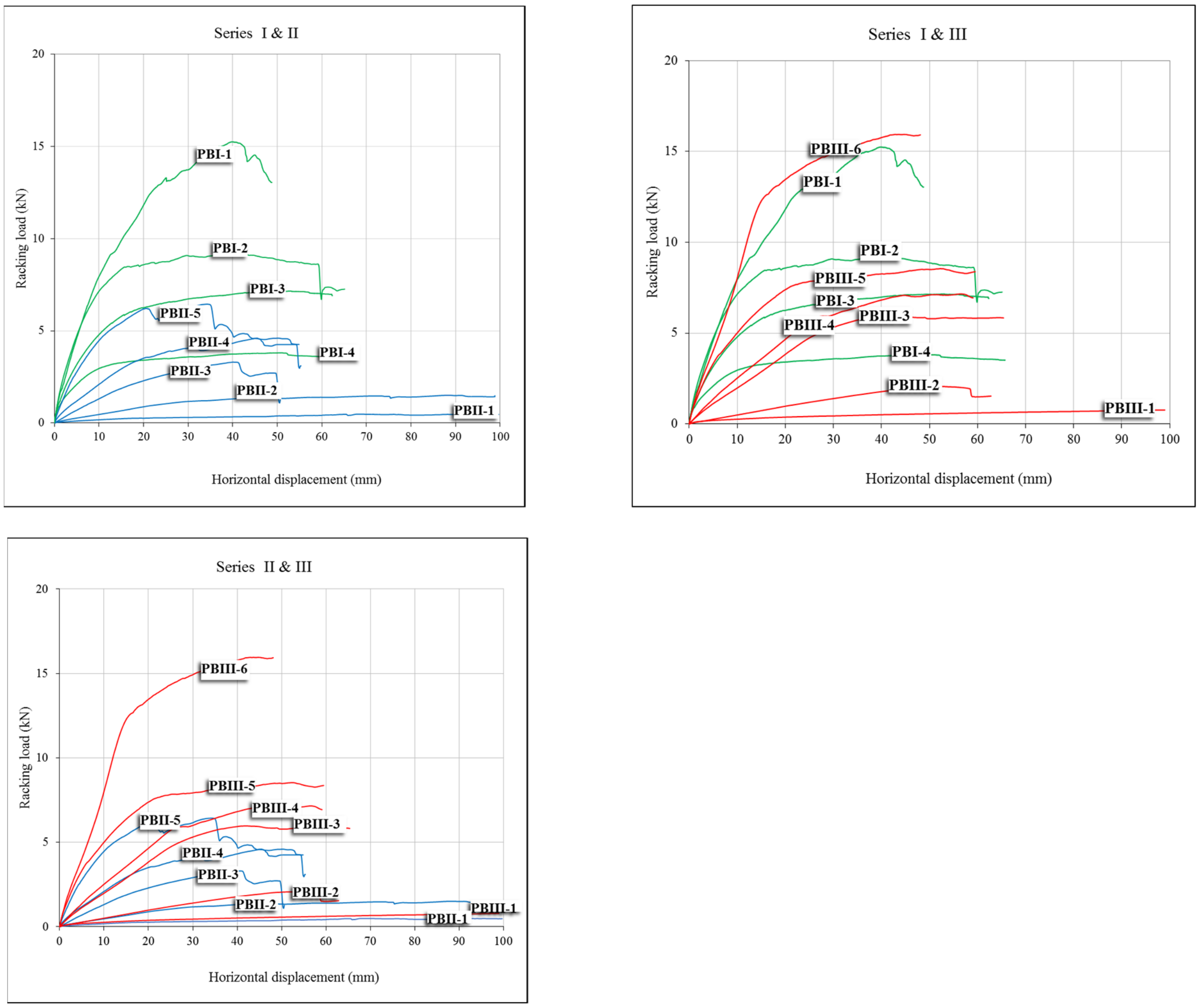


Figure 4.8 P_h-Δ_h curves and corresponding test ID, as given in Table 4.3 for PB walls

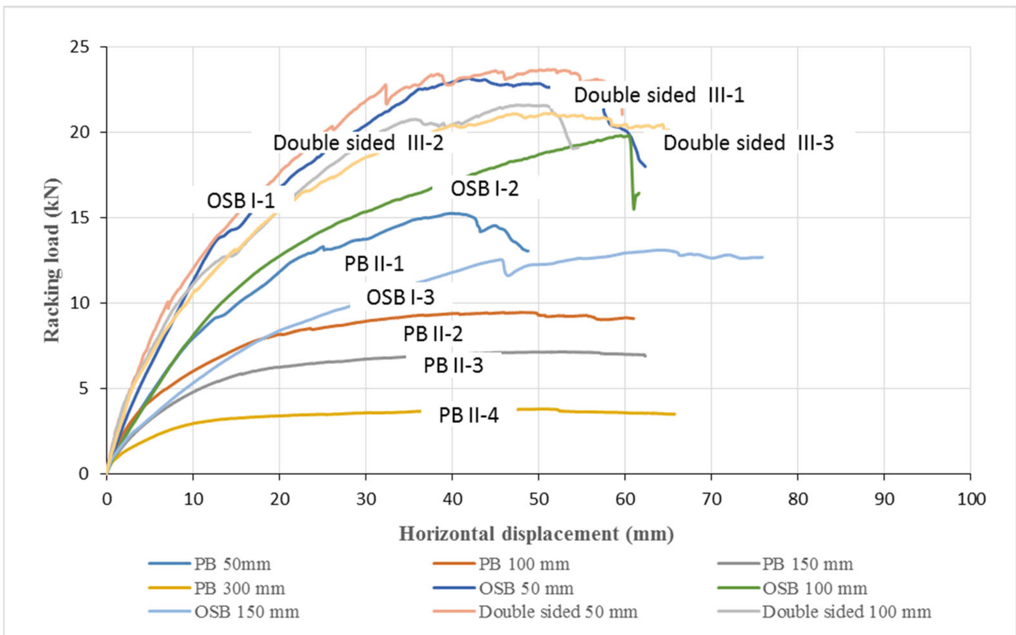


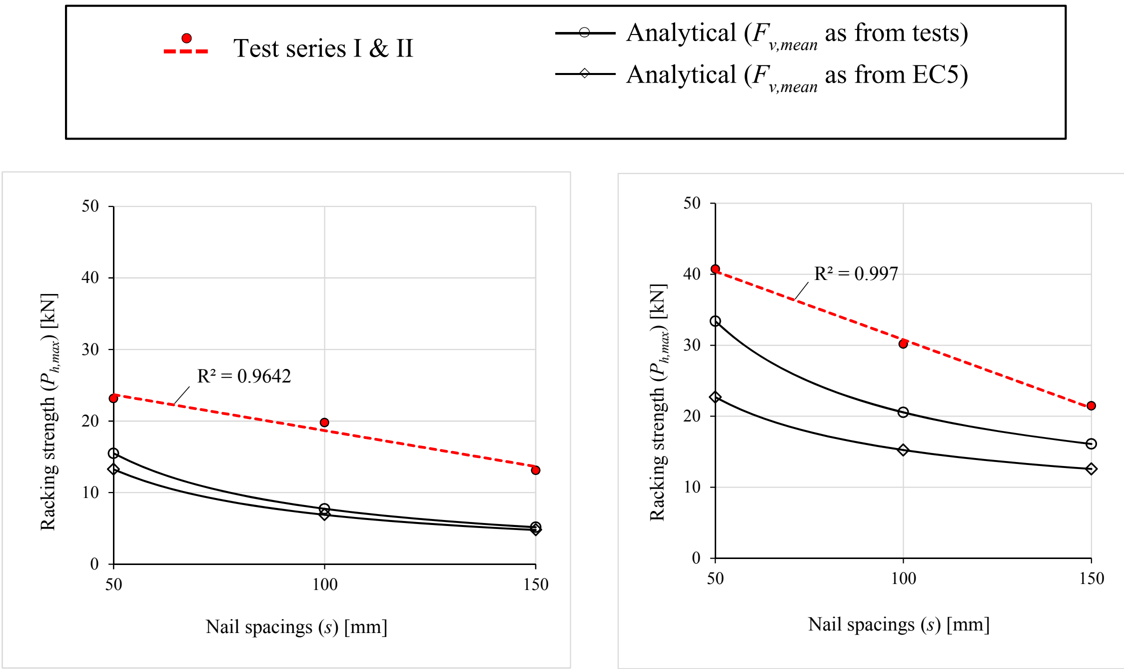
Figure 4.9 P_h-Δ_h curves and corresponding test ID, as given in Table 4.4 for OSB, PB, and double sided walls.

i. Effect of nail spacings on the racking strength of OSB walls

Figure 4.10-a and -b show the variation of racking strength as a function of the panel-to-frame nail spacings, obtained respectively from tests on wall specimens without and with vertical loading, i.e. test series II and I (see Table 4.2). To allow comparison with the corresponding analytical functions (bold lines with circles for values based on $F_{v,mean}$ derived from test results; bold lines with diamonds for values based on $F_{v,meam}$ derived from EC5 design rules), the test values have been fitted with a linear function (dashed lines) such that $P_{h,max}(s) = \alpha s + \beta$. Values for the square of the multiple correlation coefficient, R^2, are given on Figure 4.10.

On this basis, it can be seen that, regardless of the nail spacing, the racking strength values predicted analytically (by Eq. (4.1)) follow a similar trend to those derived by tests, but are consistently lower. Also, the analytical values for function $P_{h,max}(F_{v,mean})$ with $F_{v,mean}$ derived from EC5 method (Eq. (4.3)), provide lower results than those obtained by using the value for $F_{v,mean}$ derived from tests. The difference between the two analytical curves is greater for racking strength results on walls formed using the larger diameter nails (3.0 mm × 52 mm) i.e. Figure 4.10-b, and this is very much influenced by the difference between the fastener strength values of the 2.8 mm and 3.0 mm diameter nails derived from the lateral strength tests (see third and fourth columns of Table 4.5). For connections made with 2.8 mm × 49 mm nails, the mean strength value ($F_{v,mean}$) obtained from tests is 8%-16% higher than $F_{v,mean}$ as obtained from EC5 calculations, and this difference rises to 58% - 72% when looking at the mean strength of connections made with 3.0 mm × 52 mm nails.

Since in the PD 6693-1 method the wall racking strength, $P_{h,max}$, is a function of the panel-to-frame fastener strength (see Eqs. (4.1) - (4.2)), it is not surprising that the analytical function $P_{h,max}(F_{v,mean})$, with $F_{v,mean}$ derived from EC5 calculations, provides lower values compared to the same function with $F_{v,mean}$ obtained from tests. This also explains the more pronounced difference between the two analytical racking curves when 3.0 mm x 52 mm nails are used to fix the panels to the frame (see Figure 4.10-b).



(a) No vertical loading (b) With 25kN vertical loading

Figure 4.10 Wall racking strength as a function of the panel-to-frame fastener spacings (s). The experimental values are referred to: (a) test series I, i.e. walls assembled with 2.8 mm × 49 mm nails and without applied vertical load. (b) test series II, i.e. walls assembled with 3.0 mm × 52 mm nails and with 25 kN vertical load (see Table 4.2).

Making a comparison between the analytical results obtained using $F_{v,mean}$ from tests (round markers with continuous curve in Figure 4.10) and the experimental racking strength results (dashed curves), the following observations are made:

- With change in the nail spacing s, between 50 and 150 mm, the difference between the experimental and the analytical curves remains roughly constant at the 50 mm and 150 mm spacings. Although staggered downward, the analytical curves seem to effectively follow the variation of racking strength due to the different fastener spacings used. With reference to Figure 4.10-a, with s ranging from 50 mm to 150 mm, the experimental value of $P_{h,max}$ decreases from 23.13 kN to 13.10 kN (-10.03 kN) and the analytical value of $P_{h,max}$ decreases from 15.49 kN to 5.16 kN (-10.32 kN). Similarly, with reference to Figure 4.10-b, the experimental value of $P_{h,max}$ drops from 40.72 kN to 21.46 kN (-19.26 kN) and the analytical value of $P_{h,max}$ from 33.39 kN to 16.09 kN (-17.29 kN).

In relative terms however, the analytical underestimation of racking strength increases with the increase of the nail spacing s. Referring to the test case with no applied vertical load (Figure 4.10-a): for s = 50 mm, the analytical function gives a racking strength that is -33% the corresponding experimental value, whilst for s = 150 this difference increases to -61%. A similar, but less pronounced difference, is found for the test case with 25 kN vertical load (Figure 4.10-b): at s = 50 mm the analytical raking strength is predicted to be -18% the corresponding experimental value, whilst for s = 150 the underestimation increases to -25%.

- The underestimation of the analytical function is much more pronounced, in both relative and absolute terms, for the test case without vertical applied load. For this case, $P_{h,max}$ is calculated on average to be -53% (9.3 kN) less than the test result (Figure 4.10-a). This compared to an average difference of -25% (-7.4 kN) for the test case subjected to 25 kN vertical load (see Figure 4.10-b).

  A possible explanation to why the analytical function gives more accurate results when a vertical load Q is applied to the top of the wall, is provided as follows. In the analytical approach, in accordance with the requirements of Eqs. (4.6) and (4.5), the racking strength of the wall increases with the increase of the stabilising moment M it supports. This is a function of the wall head loading being supported, i.e. $M = QL/2$. Another contributor to the stabilising moment will be the resistance offered by the stud-to-beam rail connections at the windward end of the wall, which is ignored in the PD 6693-1 equations for a combination of practical and conservative reasons. However, in this analysis, whilst for Q = 25 kN, such a contribution only represents a small percentage of the stabilising moment, for the case where Q = 0 kN there will be a contribution to M entirely due to the withdrawal capacity of these connections, which is ignored in the analysis. This aspects the results and will contribute to the reason why there is a different behaviour between loaded and unloaded test and analytical results.

As previously seen, the analytical racking strength function $P_{h,max}(F_{v,mean})$, computed with $F_{v,mean}$ obtained from EC5 method, provides lower results compared to the same function computed with $F_{v,mean}$ obtained from tests. With reference to Figure 4.10-a, with s ranging from 50 mm to 150 mm, the analytical value of $P_{h,max}$ (computed with $F_{v,mean}$ as from EC5 method) decreases from 13.25 kN to 4.78 kN (-8.47 kN). Similarly, with reference to Figure 4.10-b, the same analytical value drops from 22.68 kN to 12.57 kN (-10.11 kN).

ii. Effect of wall length on the racking strength of OSB walls

Figure 4.11 shows the variation of racking strength as a function of the wall length, derived from tests on walls made with OSB sheathings fixed on a standard frame (test series III plus I-2) and OSB sheathings fixed on timber frames made with double end studs and double bottom rail (test series IV). The test values have been fitted with a power function such that $P_{h,max}(L) = \alpha s^{\beta}$ since a better fit of the experimental data is achieved, compared to a linear function.

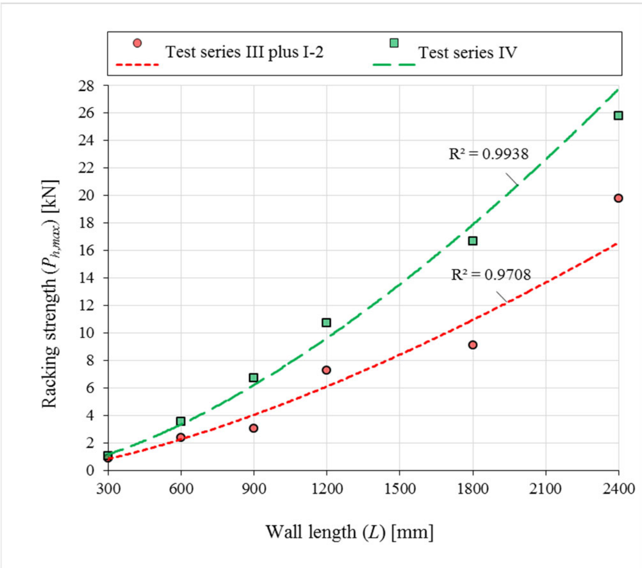


Figure 4.11 Experimental racking strength as a function of the wall length (L)

The values are referring to walls made with a standard frame (test series III plus I-2) and walls made with frames assembled with double end studs and double bottom rail (test series IV), see Table 4.2.

The wall specimens made with a standard type frame have a racking strength of 0.89 kN for L = 300 mm up to 19.79 kN for L = 2400 mm. In comparison, the walls made with double studs and a double bottom rail are much stronger, with strength values ranging from 1.04 kN for L = 300 mm, up to 25.82 kN for L = 2400 mm (i.e. about 58% higher, on average). The reason for such a strength increase is primarily due to the use of a double row of fasteners along the perimeter of the wall (see Figure 4.2), rather than any strength contribution from the double end-studs and double bottom rail. Considering the cumulated lateral strength of two rows of fasteners at 100 mm spacings to be equivalent to two rows of fasteners at 100 mm spacing, a comparison of results can be made between wall test I-1 and IV-6: wall I-1 has a racking strength of 23.13 kN, which is only 10% lower than the racking strength of wall IV-6 (25.82 kN).

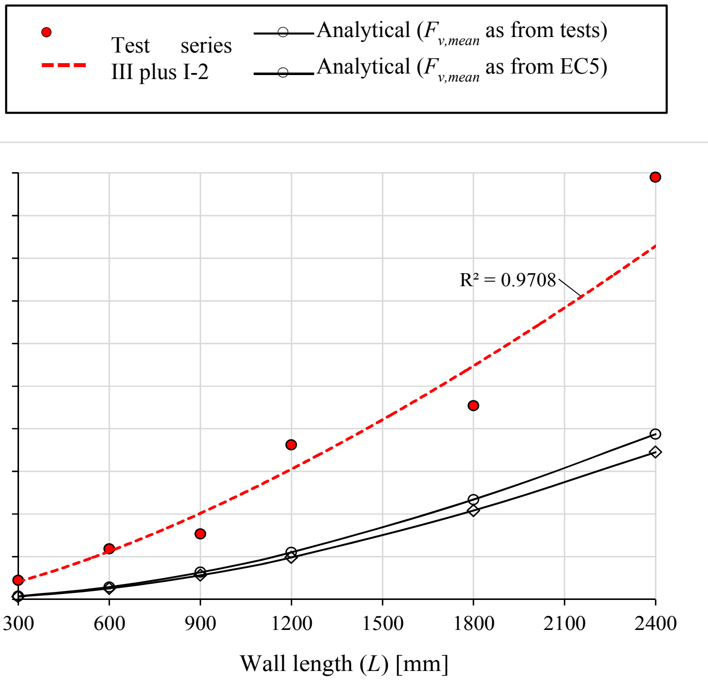


Figure 4.12 Experimental and analytical racking strength as a function of the wall length (L)
The experimental values are referring to test series III plus I-2, i.e. walls made with sheathings fixed at 100 mm spacings on a standard frame. The fastener load carrying capacity, $F_{v,mean}$, required to compute $P_{h,max}$, has been derived both from tests (see Appendix 4.2) and from EC5 procedure, i.e. Eqs. (4.3).) and (4.4).

In Figure 4.12 a comparison of racking strength results obtained from tests (test series III plus I-2), and strength values obtained analytically, based on tests and EC5 values, is shown. The experimental curve is derived from test results of walls assembled with 2.8 diameter × 49 mm long nails spaced at 100 mm, and with no vertical loading. As can be observed from the Figure, the analytical raking strength curves remain well below the experimental curve for the entire range (i.e.300 mm ≤ L ≤ 2400 mm). In particular, the relative underestimation increases as the wall length is reduced: for L = 2400 mm, the analytical racking strength is predicted between 6.90 (based solely on EC5) and 7.75 kN (based on EC5 using test values), i.e. about 65% and 61% less than the experimental value (19.79 kN). As the wall length reduces to 300 mm, the analytically predicted racking strength becomes about 80% lower than the corresponding experimental value of 0.89 kN.

4.3.2 Racking stiffness behaviour of OSB walls

For each tested wall specimen, the corresponding racking stiffness, R, has been evaluated in accordance with the requirement of BS EN 594:2011 (BSI, 2011) as follows:

$$R = \frac{0.4P_{h,max} - 0.2P_{h,max}}{\Delta_4 - \Delta_2} \tag{4.10}$$

in which Δ_4 and Δ_2 are the values of the wall deformation recorded respectively at 40% and 20% of the maximum racking load $P_{h,max}$.

The particular relationships investigated in regard to stiffness behaviour are covered in the following subsections.

i. Effect of nail spacings on the racking stiffness

Figure 4.13 shows the variation of racking stiffness, R, as a function of the nail spacing s, obtained from tests on wall specimens without vertical load (test series I) and also with 25 kN vertical load (test series II), both walls being 2400 mm long. The racking stiffness, R, was derived from tests according to Eq. 4.10. As expected, the racking stiffness is enhanced as the nail spacing is reduced. For the case with 25 kN vertical load, R rises from 1430 N/mm (for s = 150 mm) to 1774 N/mm (for s = 50 mm) i.e. an increase of 23.8%. For the same wall without vertical loading there is a much steeper increase in racking stiffness, rising from 410 N/mm (for s = 150 mm) to 1647 N/mm (for s = 50 mm), corresponding to an increase of 300%. Also, at a nail spacing of 50 mm, the racking stiffness of the unloaded wall is approximately 93% of the loaded wall condition. From this it can be seen that the stiffness of unloaded walls is more greatly influenced by nail spacing than loaded walls of the same construction, and also that as the nail spacing reduces the stiffness is primarily influenced by the nail spacing rather than the vertical loading.

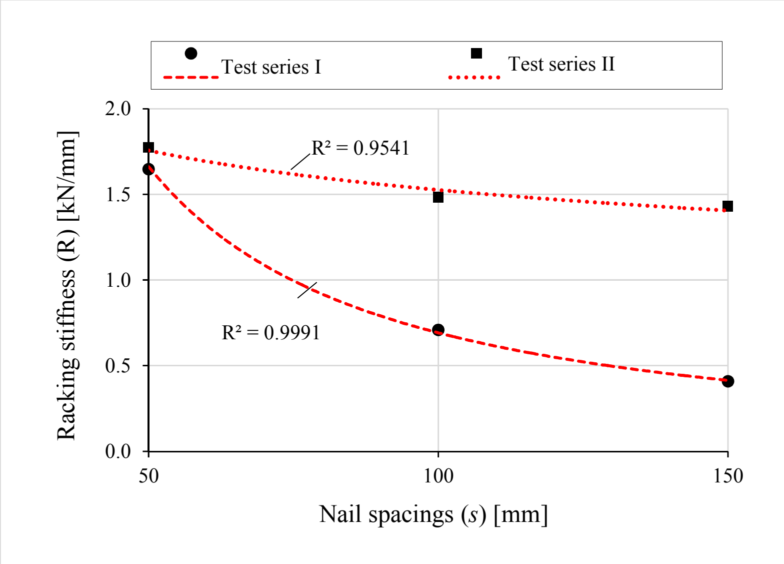


Figure 4.13 Racking stiffness as a function of the nail spacing (s).

Values referring to test series I, i.e. walls without applied vertical load, and test series II, i.e. walls with 25 kN applied vertical load.

ii. Effect of wall length and frame construction on the racking stiffness

A plot of racking stiffness values, R, against the wall length, L, is shown in Figure 4.14. The Figure gives plots of wall specimens made with OSB sheathing panels fixed to a standard frames (test series III plus I-2), and wall specimens with sheathings fixed on frames made with double end studs and double bottom rails (test series IV). In line with the stiffness to nail spacing behaviour referred to in section 4.3.2 i, the racking stiffness, as well as the rate of increase in stiffness, increases with the length of the wall. For short walls (i.e. up to 900 mm) the increase in stiffness and rate of change of stiffness are approximately linear and despite the stiffer frame construction associated with the test series IV walls, the behaviour of both types of wall is similar. Above this wall length however, the stiffness values start to increase at a more rapid rate, and for the 2400 mm walls assembled with double studs and double bottom rails the racking stiffness is about 32% stiffer than the same length of wall constructed using the standard type of frame.

For the shorter walls, the wall shear deformation per unit racking force will make a larger contribution than for longer walls as it is a function of the ratio of panel-height to panel-width. The factor will range from 8, for 300 mm long walls, to 1 for 2400 mm long walls. Therefore, for longer walls, the lateral shear deformation of the wall panels becomes less significant and the major contribution to stiffness is the behaviour of the sheathing fasteners and the racking frame. The configuration of the fasteners is similar for both types of wall, however, from the test results, doubling up on the end studs and the bottom rails has made a significant contribution to stiffness behaviour.

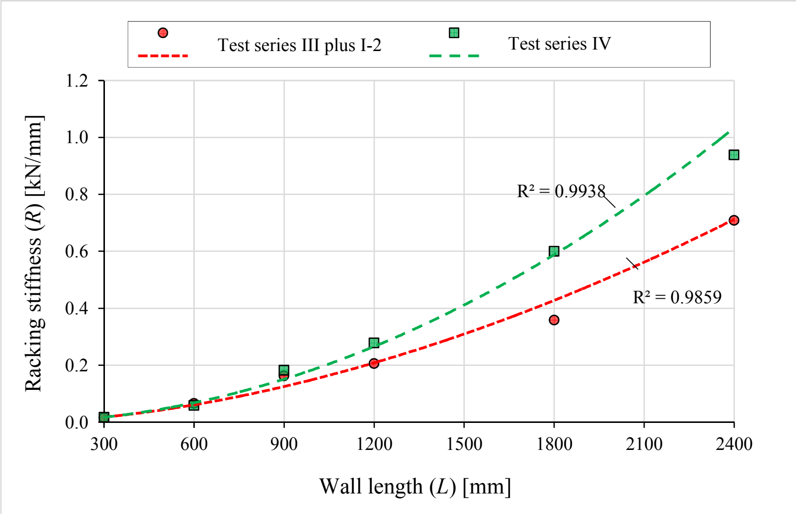


Figure 4.14 Racking stiffness as a function of the wall length (L)

The values are referred to walls made with OSB panels fixed on a standard frame (test series III plus I-2) and OSB panels fixed on a timber frame made with double studs and bottom rail (test series IV). See Table 4.2.

iii. Effect of PD 6693-1 rules on design strength and stiffness values

In the PD 6693-1 document, in order to limit the racking deflection of a wall, a stiffness criterion has been introduced and to suit the format used in this chapter it has been re arranged and is given in Eq(4.9). In accordance with the functions used in PD 6693-1, this empirical relationship can be expressed in terms of the design racking load, P_{ULS}, of the wall at the Ultimate Limit State (ULS), where:

$$P_{ULS} = f_{d,pd} l_{1,eff} \tag{4.11}$$

enabling Eq. (4.9) to be rewritten as:

$$\frac{P_{ULS}}{8L^2} \leq 1 \tag{4.12}$$

The value of the design racking load for each wall test has been calculated in accordance with the procedure defined in PD6693-1, with the $f_{d,pd}$ values derived using the values of the panel-to-frame fastener strength, $F_{v,mean}$ obtained by the application of the EC5 design procedure, given in Table 4.5. Inserting the relevant functions into Eq. (4.12) for walls I, II and III, a plot of the results is shown in Figure 4.15.

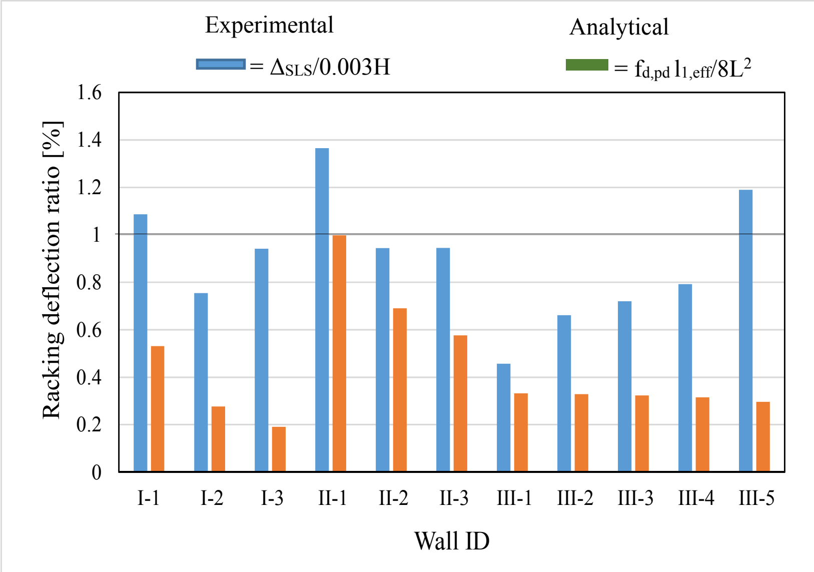


Figure 4.15 Comparison between the racking deflection limit ratios based on test results ($\Delta_{SLS}/0.003H \leq 1$) and PD 6693-1 rules

Since no limiting relationship for an acceptable value of racking stiffness is given in BS EN 594:2011, the deflection limit of 0.003 times the panel height, given in BS 5268-6.1:1996 (BSI, 1996a), has been used as the limiting deformation that would be acceptable. It is also anticipated that this deflection limit will be incorporated into the next revision of the UK National Annex for BS EN 1995-1-1 as the maximum lateral deformation that will be permitted at the Serviceability Limit State (SLS), for such walls. Based on the test results, a plot of the ratio $\Delta_{SLS}/0.003H$ for walls I, II and III is given on Figure 4.15 to allow comparison with the empirical relationship for the limitation of displacement at the serviceability state given in PD6693-1, restructured as presented in Eq. (4.12). All walls tested were 2400 mm high, resulting in a deflection limit of 0.003H = 7.2 mm. As the stiffness criteria relationship in equation Eq. (4.12) is based on characteristic design values, to obtain equivalent load values from the test curves, the test load results have been modified by a factor of 0.8, as given in Table 8 of BS 5268-6.1:1996. Also, to derive the deflection at the serviceability state, Δ_{SLS}, associated with the P_{ULS} design load, the value has been taken to be that obtained from the modified test results at a load of $P_{ULS}/1.5$.

From the Figure it can be seen that based on the above procedure, all walls will pass the stiffness criterion set by the PD6693-1. However, when comparing with the deflection limit criterion $\Delta_{SLS}/0.003H \leq 1$, walls I-1, II-1 and III-5 will fail. In all cases, the results from the PD6693-1 criterion indicate that the walls are generally well within the limiting value except for wall II-1, which is on the limit of acceptability. When analysed using the deflection limit approach, $\Delta_{SLS}/0.003H$, three walls fail (walls I-1, II-1 and III-5), and further three are close to the failure (I-3, II-2 and II-3) and in every instance this approach indicates there is a smaller margin against compliance than in the case where the PD6693-1 criterion is used. In practice, vertically loaded walls will be selected over unloaded walls to provide racking resistance to a structure and so the walls of particular interest in a stiffness comparison exercise are walls II-1, II-2 and II-3. For these three walls, the ratio of the experimental to analytical results is on average 1.45 and as the fastener spacing reduces the walls stiffness gets closer to the limiting stiffness condition, with wall II-1 exceeding the limit when based on the experimental approach.

4.3.3 Racking strength behaviour of plasterboard (PB) walls

i. Effect of screw spacings on the racking strength of PB walls

Figure 4.16 shows the variation of racking strength as a function of the panel-to-frame screw spacings, obtained from tests on wall specimens without vertical loading, i.e. test series PB I (see Table 4.3).

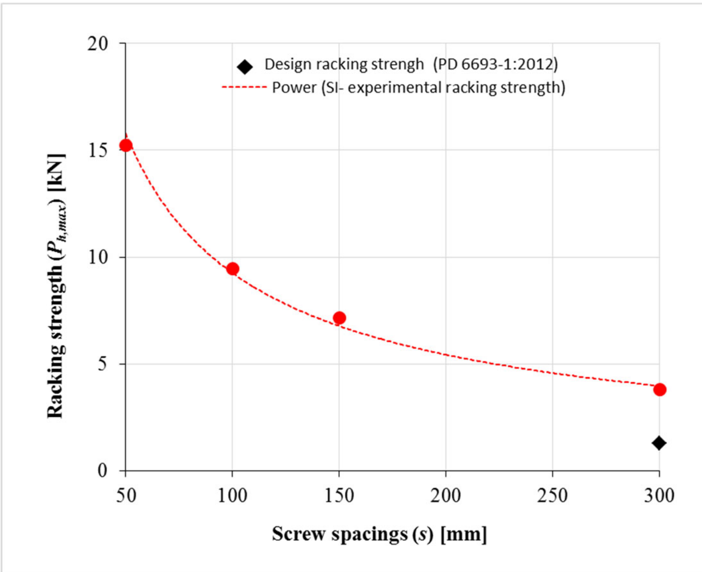


Figure 4.16 Wall racking strength as a function of the panel-to-frame fastener spacings (s) in PB walls

The experimental values are referred to test series I walls assembled with 3.5 mm × 40 mm drywall screws without applied vertical load (see Table 4.3).

With reference to Figure 4.16, with s ranging from 50 mm to 300 mm, the experimental value of $P_{h,max}$ decreases from 15.25 kN to 3.81 kN (- 11.44kN). The analytical value (diamond mark with 1.23 kN) is given only for fastener spacing s = 300 mm spacing in accordance with PD 6693-1:2012 considering total design shear capacity per unit length of the perimeter fastener, f_{pdt} =1.27 kN/m which is lower than the experimental value by 209%.

ii. Effects of wall lengths on racking strength of gypsum plasterboard walls

Figure 4.17 shows the variation of racking strength as a function of the wall length, derived from tests on walls made with PB sheathings fixed on a standard frame (test series PBIII plus PBI-2) and PB sheathings fixed on timber frames made with double end studs and double bottom rail (test series PB IV).

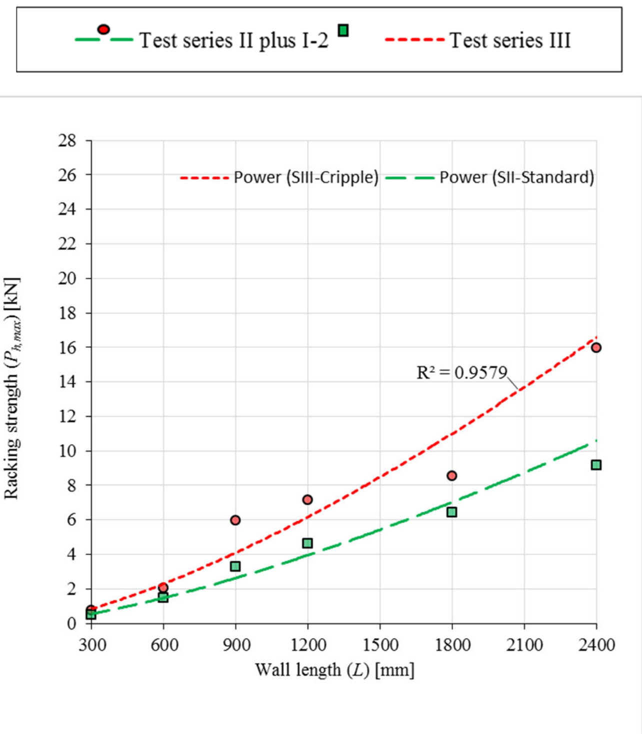


Figure 4.17 Experimental racking strength as a function of the wall length (L) for PB walls
The values are referring to walls made with a standard frame (test series PBII plus PBI-2) and walls made with frames assembled with double end studs and double bottom rail (test series PBIII), see Table 4.3.

The wall specimens made with a standard type frame have a racking strength of 0.48 kN for L = 300 mm up to 9.15 kN for L = 2400 mm. In comparison, the walls made with double studs and a double bottom rail are much stronger, with strength values ranging from 0.77 kN for L = 300 mm, up to 15.94 kN for L = 2400 mm (i.e. about 58% higher, on average). The reason for such a strength increase is primarily due to the use of a double row of fasteners along the perimeter of the wall (see Figure 4.2), rather than any strength contribution from the double end-studs and double bottom rail.

4.3.4 Racking stiffness behaviour of plasterboard (PB)

i. Effect of fastener spacings on the racking stiffness of PB

Figure 4.18 shows the variation of racking stiffness, obtained from tests on wall specimens without vertical load (test series PB I). The racking stiffness was derived from tests according to Eq. (4.10). As expected, the racking stiffness is enhanced as the screw spacing is reduced. There is an increase in racking stiffness, rising from 0.375 kN/mm (for s = 300 mm) to 0.772 kN/mm (for s = 50 mm), corresponding to an increase of 105%.

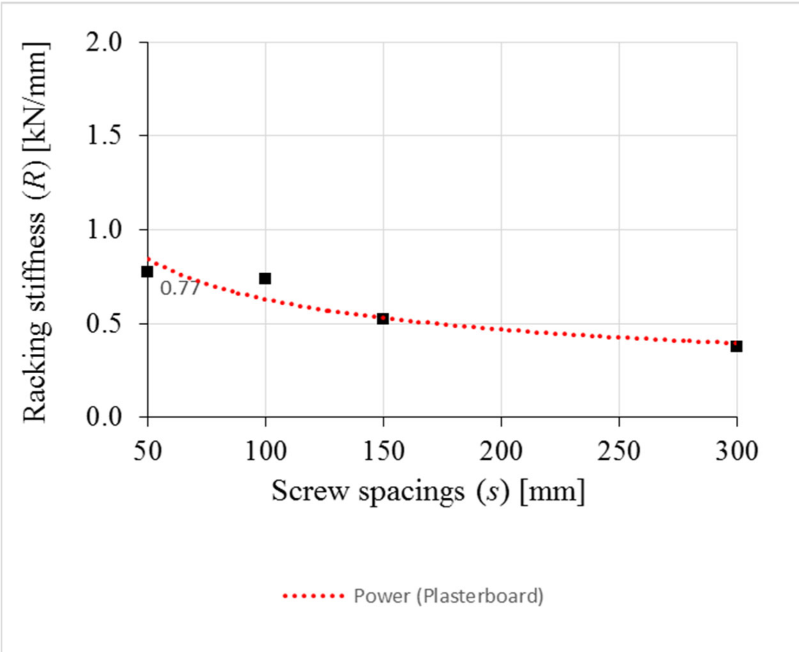


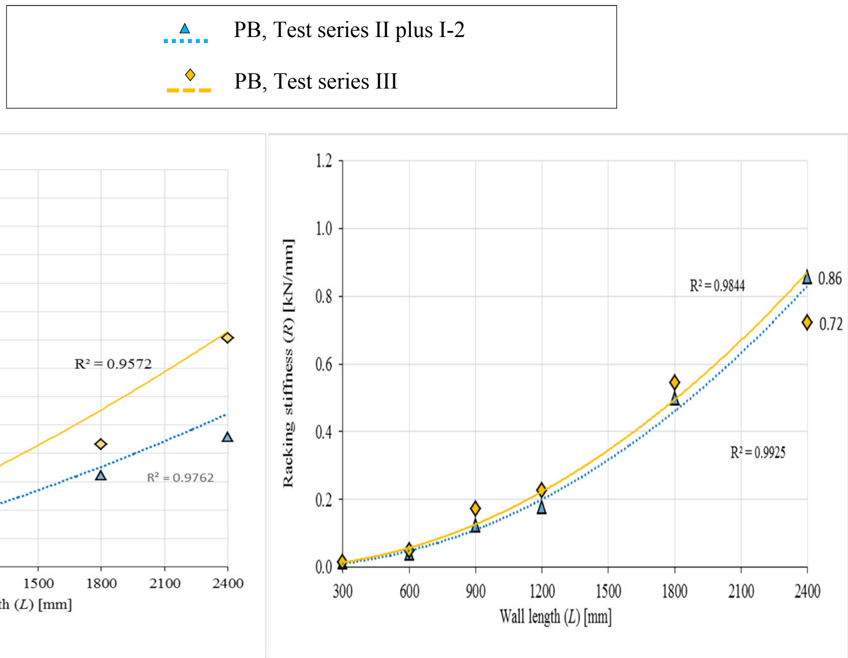
Figure 4.18 Racking stiffness as a function of the screw spacing (s)
Values referring to test series PB I, walls without vertical load.

ii. Effect of wall length and frame construction on the racking stiffness

In Figure 4.19-a, a comparison of racking strength results for standard frames and frames assembled with double end studs and double bottom rail is shown. The racking strength of series PB III was higher than series PB II for all wall lengths in both cases. Typically, the racking strength of PB sheathed wall of length 2400 mm (series PB III) is 74% higher than that of corresponding length of PB sheathed wall (series PB II) and when length reduced to 300 mm, it is 60% higher (series PB III) than that of corresponding length of PB sheathed wall (PB series II).

In Figure 4.19-b, a comparison of racking stiffness results for standard frames and frames assembled with double end studs and double bottom rail is shown. The racking stiffness of series PB III is higher than series PB II for wall lengths ranging from300 mm ≤ L ≤ 1200 mm.

In standard frames, the racking stiffness of series PB III for wall length L = 1200 mm is 58% higher than that of corresponding wall lengths in series PB II (0.179 kN/mm) and for L = 300 mm, it was 7.6% higher than that of corresponding wall length of series PB II (0.013 kN/mm).



(a) Strength

(b) Stiffness

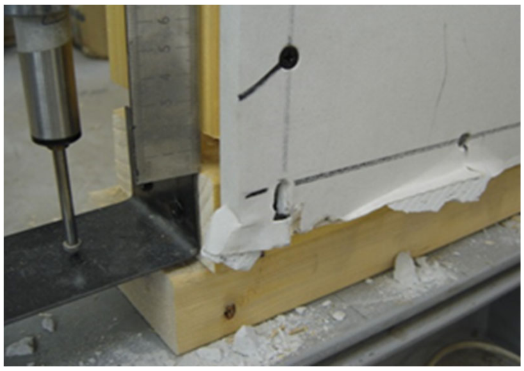
Figure 4.19 Comparison of the effects of wall length on the racking performance_of PB walls with and without double end studs and double bottom rail

The values are referring to walls made with standard frames: for PB, test series II plus I-2 as in Table 4.3; frames assembled with double end studs and double bottom rail: for PB, test series III as in Table 4.3

iii. Comparison of experimental test series III between OSB and PB to examine the effect of wall length with double end studs and double bottom rails on racking performance of wall panels

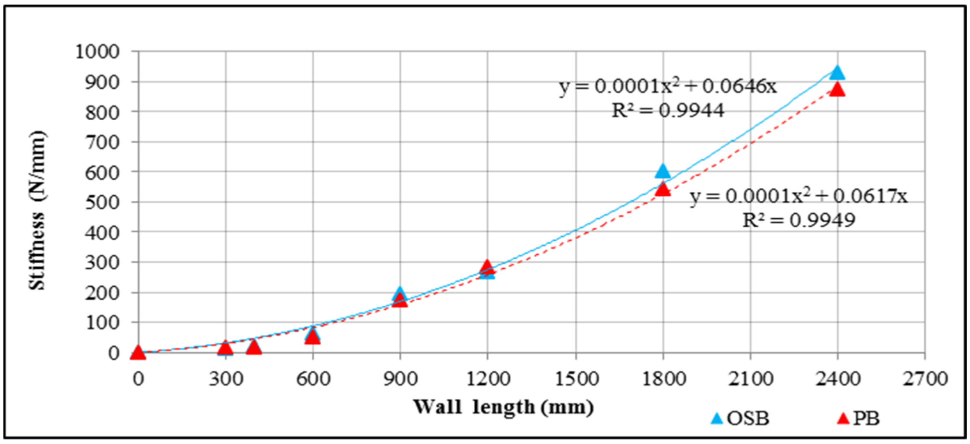
A Series of tests was conducted to check the racking performance of wall panels with different length using double end-studs and double bottom rail of walls sheathed with OSB and walls sheathed with PB panels as shown in Figure 4.22. The experimental results and calculated results in accordance with PD 6693-1:2012 for plasterboard walls are shown in Table 4.3. The results show that walls using double studs for both wall panels with OSB and PB are stiffer and stronger than walls using single studs as shown in Figure 4.21 (a) and (b). Typical failure mode of walls sheathed with OSB and walls sheathed with PB panels are shown in Figure 4.20 (a) and (b) (also see Appendix 4.3).

(a) OSB

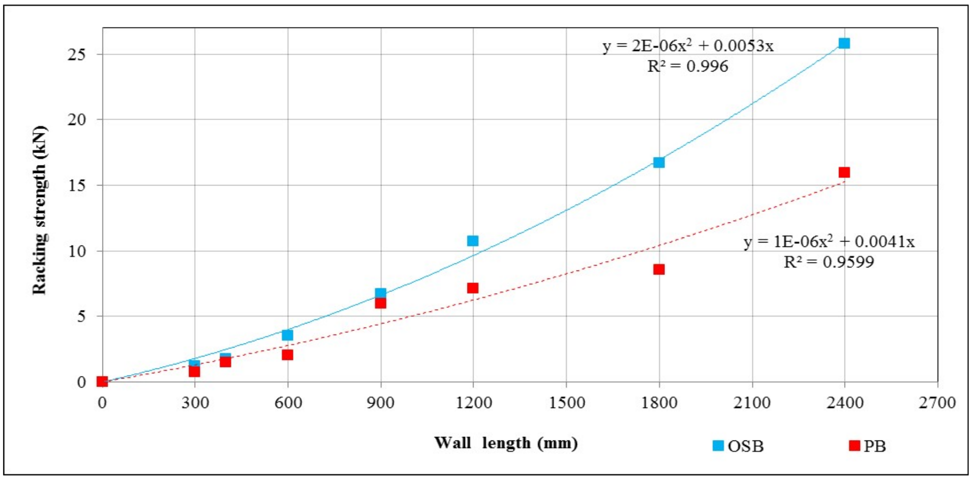


(b) PB

Figure 4.20 Failure of 1800 mm long (a) OSB and (b) 1800 mm plasterboard walls



(a) Stiffness



(b) Strength

Figure 4.21 Comparison of racking strength and stiffness as a function of wall length between OSB and PB of test series III

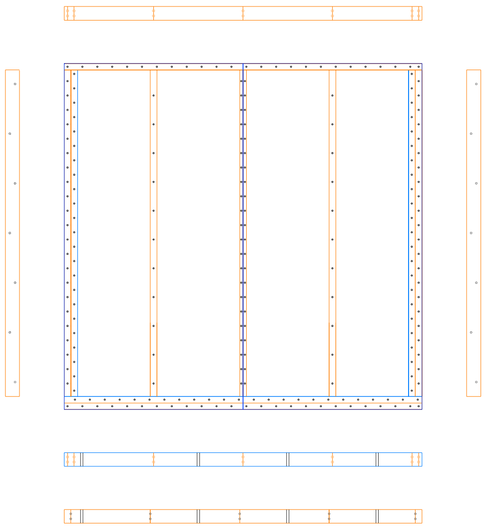
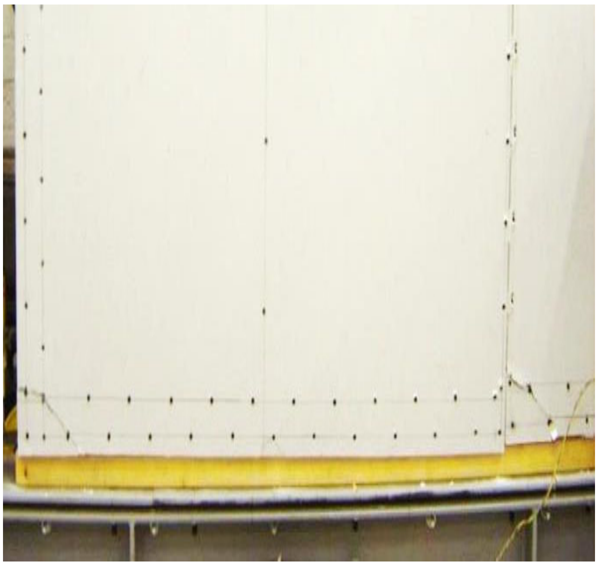
 

Figure 4.22 Wall frame with cripple end studs and double bottom rail used in Test Series PB III (2400 mm × 2400 mm)

4.3.5 Racking performance of double sided walls (with OSB and PB)

A series of double sided walls sheathed with 9mm OSB/3 on one side and 12.5mm British gypsum PB on the other were tested to examine their performance characteristics. The tests were conducted with three different nail spacing of 50, 100 and 150mm using 2.8 x 50 mm round headed smooth nail. The test results were compared with the single sheathed walls of either OSB or PB panels, and with existing design methods in accordance with EC5 (Method A and B) and PD 6693-1:2012 which is shown in Table 4.6 and the strength and stiffness are shown in Figures 4.23 and 4.24. Method A calculation for both sided wall was calculated in accordance with EC5 clause 9.2.4.2 (7) which states "if different types of sheets are used, 75% of the racking-carrying capacity of the weaker side may, unless some other value is shown to be valid, be taken into consideration if fasteners with similar slip moduli are used. In other cases not more than 50% should be taken into consideration". Since the walls tested here were used with the nails 2.8 x 50 mm at 50/100, 100/200 and 150/300 mm for the OSB and 3.5 x 40 mm drywall screws for PB at 100/200 mm spacing pattern. Therefore, 50% of the weaker side strength rule was considered. The design racking load for both sides with 50 mm fastener spacing obtained from Method A is 36.85 kN when 50% of racking load of PB (i.e. 50% of 15.25kN) and full load of OSB (i.e. 29.22kN) were considered. For other spacing, similar calculations are done as shown in Table 4.6. However, experimentally the racking load obtained for both sides did not follow the value calculated in accordance with Method A, which was supposed to be 30.77 kN. Instead, it was recorded as 23.66 kN which is 23% lower than the design load. The reason for this could be due to the premature failure of the Plasterboard or due to the decision limited on only one test of the wall with each fastener spacing.

Table 4.6 Comparison of test results of different configuration of walls with existing design method

2400 x 2400 mm sized wall		Nail spcing on one side of the walls with OSB (mm)			Nail spacing on double sided walls (with OSB & PB) (mm)			Nail spacing on one side of the walls with PB (mm)			
		50	**100**	**150**	**50**	**100**	**150**	**50**	**100**	**150**	**300**
Experimental	Ultimate racking load (kN)	23.14	19.79	13.1	23.66	21.58	21.1	15.25	9.46	7.16	3.81
	Characteristic racking load (kN)	19.76	16.90	11.19	20.20	18.44	18.01	13.03	8.08	6.122	3.254
	Design racking resistance (kN) (Kmod = 1 and ym = 1.25 assumed)	15.82	13.53	8.96	16.17	14.76	14.42	2.61	1.62	1.224	0.651
	Racking stiffness (N/mm^2) S4	1022.20	736.06	417.17	1202.08	991.14	947.09	771.90	734.32	520.8	375.4
Design to EC 5 (Method A)	Design racking load (kN)	29.22	14.61	9.74	36.85	19.34	13.32	N/A			
Design to EC 5 (Method B)	Design racking load (kN)	13.89	9.5	7.22	13.89	9.5	7.22	N/A			
Design to PD 6693-1:2012	Design racking load (kN)	13.3	7.15	5.09	11.12	6.01	4.31	N/A			1.23

Both Methods A and B do not provide values for walls sheathed with PB therefore, the no value is in the table for the walls tested with PB on single sided walls. For 300 mm screw spacing throughout the single sided sheathed with PB walls, total design shear capacity per unit length of the perimeter fastener, f_{pdt} was considered 1.27 kN/m in accordance with PD6693-1:2012 Clause 23 Table 9.

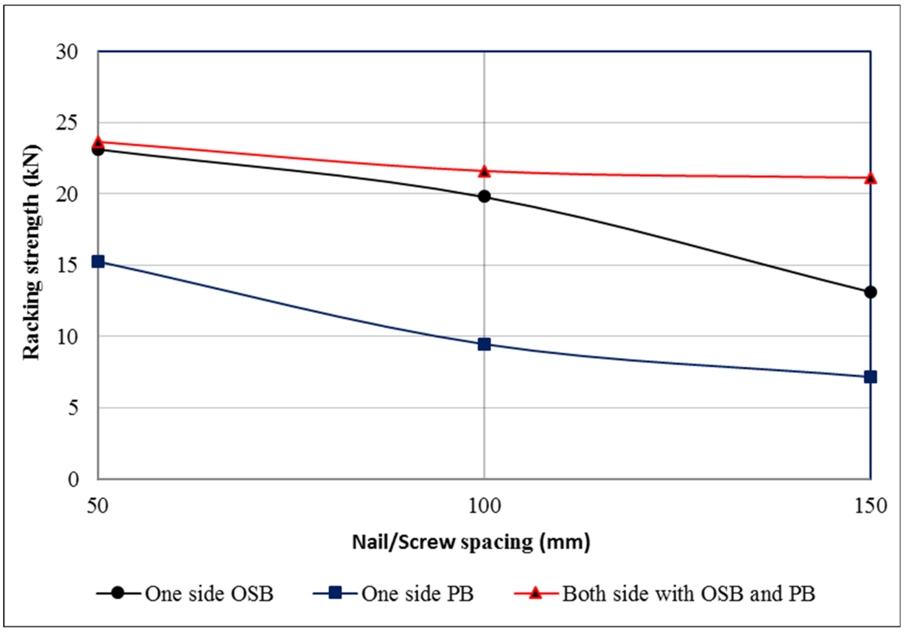


Figure 4.23 Comparison of racking strengths with three different wall configurations

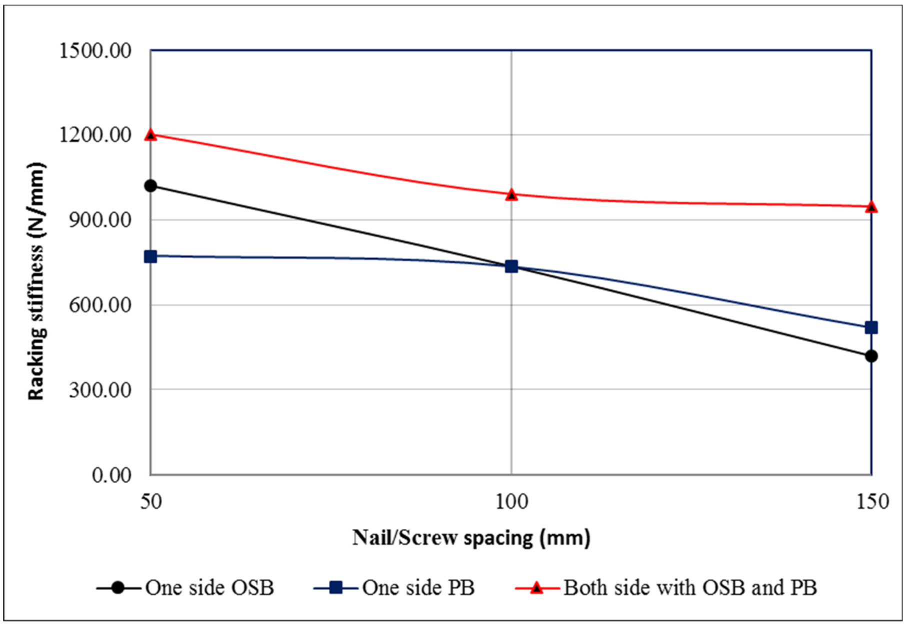


Figure 4.24 Comparison of racking stiffness with three different wall configurations

4.4 Conclusions

The present work aimed to assess, by means of experimental tests, how the variation of some common parameters, such as fastener spacing and wall length, affect the racking behaviour of timber Platform framed walls, enabling evaluation of the accuracy of the formulae proposed in the design code to determine the racking strength and stiffness of the walls. In particular, the investigation has been focused on partially anchored racking walls, the most common method of construction adopted for timber framed walls in the

UK. Consequently, the procedure described in the PD 6693-1 document, as recommended by the UK NA to EC5, has been adopted. From the analyses and test results described in section 4.3, the following conclusions are drawn:

- In general, the racking strength of the wall is more sensitive to variations in the fastener spacings when it is subjected to a vertical loading. Conversely, when the wall has no vertical loading, its racking stiffness becomes more sensitive to change in fastener spacings.
- The effect of panel-to-frame fastener spacing is more pronounced when the wall is subjected to an applied vertical loading. For example, the gain in strength for walls without vertical loading, when the fastener spacing was reduced from s = 150 mm to 50 mm, was 76% compared to the increase of 89% for a similar wall under a vertical loading of Q = 25 kN.
- In the case of racking stiffness, for walls without vertical loading, the gain in stiffness was up to 300% when the fastener spacing was reduced from s = 150 to s = 50 mm. However, such gain in stiffness did not occur in similar walls when they were subjected to a vertical loading of Q = 25 kN, with stiffness increasing by only 24%.
- The comparison of the experimental results of the full-length (2400 mm) wall specimens, irrespective of their panel-to-frame fastener spacings (50 mm to 150 mm), with the results from the design code formulae, showed that on average the design code underestimated the racking strength by 25% for walls under vertical loading of Q = 25kN and by 54% for walls without vertical loading. Noting that the analytical model only provides a lower bound value for the racking strength of the wall, the most likely explanation why such an underestimation is greater for walls without applied vertical load, is due to the contribution to the stabilising moment, M in Eq. (4.6) due to the withdrawal capacity of the stud-to-beam connections.
- Compared to walls made with a standard type of frame, the use of double studs and double bottom rails provides (on average) an increase in racking strength and stiffness of about 64% and 37% respectively. Nonetheless, the enhanced racking capacity may be (solely) attributed to the use of increased number of panel-to-frame fasteners along the perimeter of the wall.

- Considering stiffness behaviour, all walls comply with the requirements of the empirical relationship given in clause 21.5.2.3 of the PD-6693-1 document. However, when deriving stiffness behaviour from the experimental results, i.e. using the Δ_{SLS}/ 0.003H approach, walls I-1, II-1 and III-5 fail. It is difficult to draw any general conclusions on the accuracy of the PD 6693-1 criterion, however, as the more important situation in practice will relate to the behaviour of walls that carry vertical loading, i.e. walls II-1, II-2 and II-3, the behaviour of these walls show that both approaches result in an increase in value as wall stiffness is increased and for the stiffest wall, II-1, the experimental result shows the wall will fail whilst the PD-6693-1approach concludes it will pass. As acceptable stiffness behaviour has to be achieved in the design of racking walls, it is to be questioned that the empirical relationship given in equation PD6693-1 may require to be reviewed.

- Both the strength and the stiffness increased as the length of the walls increased, but decreased as the fasteners spacing increased. Both the strength and stiffness of the walls increased when the walls with double end studs and double bottom rails were tested in comparison to that of the walls with single studs. This could be because of the use of double studs and double row of fasteners. For plasterboard, though in this experiment the fasteners spacing were varied from 50 mm to 300 mm, but the design racking resistance were only compared with the spacing of 300mm in accordance with PD 6693-1:2012, which is shown in Table 4.6 and Figure 4.16 to examine the behaviour of the racking performance of the walls. The analytical value (diamond mark with 1.23 kN in Figure 4.16) is given only for fastener spacing s = 300 mm spacing in accordance with PD 6693-1:2012 considering total design shear capacity per unit length of the perimeter fastener, f_{pdt} =1.27 kN/m which is lower than the experimental value 3.81 kN by 209%.

- When the walls were tested by sheathing on both sides using OSB on one side and PB on the other side, the results for both the strength and stiffness were observed to be higher than the walls with single sheathed with either one of the materials.

Chapter 5 Effects of openings on racking performance

5.1 Introduction

A racking wall may comprise more than one wall diaphragm and if it contains discontinuities as a result of openings for windows and doors, the racking behaviour of the wall would be affected. Hence, the main focus of this chapter is to examine the effects of openings/discontinuities which incorporate windows and doors on the racking performance of OSB sheathed walls. As described in Chapter 4, double end studs and double bottom rail have a positive influence on the racking performance of walls. This Chapter also examines the possible influence of cripple studs, double trimmers, footers around opening on the performance of the racking walls. The experimental tests were conducted in accordance with BS EN 594:2011. The tested results were then compared to examine the accuracy with the existing design methods in accordance with EC5 (Method B and PD 6693-1:2012). Method A does not consider the opening for calculating racking performance; hence, it is not included in this chapter.

5.2 Theoretical procedures for calculating racking performance

In the construction of timber framed walls, the strength and stiffness of timber frame walls under lateral load is an important requirement for developing structural design rules. Many aspects have been ignored in the design methods for the structures, as for instance the effect of openings is not effectively addressed in EC5. Hence, this chapter focuses on better understanding of the racking performance of timber frame construction assessing a range of configurations particularly on the effects of different sizes and positions of openings for windows and doors in the shear wall using a predetermined constant nail spacing pattern. In the UK, the design procedure for calculating the racking strength of timber-framed walls is based on the rules in Eurocode 5 and given in PD 6693-1. EC5 provides two simplified analysis of wall diaphragm: Method A and Method B. The walls containing openings are not considered in the Method A.

Method B

The Method B of EC5 is applicable to walls made from sheet of wood-based panel products, fastened to a timber frame. According to EC5, clause 9.2.4.3, the width of the wall should be at least the height of panel divided by 4 in order to contribute to the racking strength. The fasteners should be either nails or screws and should be equally spaced around the perimeter of the sheet. The fasteners within the perimeter of a sheet should be spaced at not more than twice the perimeter fastener spacing. In case of the panels with opening, the length of panel on each side of the opening are considered as separate panels.

The racking strength of a wall assembly, $F_{v,Rd,}$ is defined as,

$$F_{v,Rd} = \sum F_{i,v,Rd} \tag{5.1}$$

where,

$F_{i,v,Rd}$	design racking strength of a wall and calculated as,

The $F_{i,v,Rd}$ is calculated as,

$$F_{i,v,Rd} = \frac{F_{fRd} b_i}{s_0} k_d k_{i,q} k_s k_n \tag{5.2}$$

where,

F_{fRd}	lateral design capacity of an individual fastener
b_i	wall length in m
s_0	basic fastener spacing in m
k_d	dimension factor for the wall
$k_{i,q}$	uniformly distributed load factor for wall *i*
k_s	fastener spacing factor
k_n	sheathing material factor

The values of s_0, k_d, $k_{i,q}$, k_s, k_n are calculated as (also see (5.4) - (5.7))

$$s_0 = \frac{9.7\, d}{\rho_k} \tag{5.3}$$

Where,

d	fastener diameter in mm
ρ_k	characteristic density of the timber frame in kg/m^3

$$K_d = \begin{cases} \dfrac{b_i}{h} & \text{for } \dfrac{b_i}{h} \leq 1.0 \\ \left[\dfrac{b_i}{h}\right]^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i \leq 4.8 \text{ m} \\ \left[\dfrac{4.8}{h}\right]^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i > 4.8 \text{ m} \end{cases} \tag{5.4}$$

where, h is the height of the wall in m

$$k_{i,q}=1+\left(0.083\,q_i-0.0008\,q_i^2\right)\left[\frac{2.4}{b_i}\right]^{0.4} \tag{5.5}$$

where, q_i is the equivalent uniformly distributed vertical load acting on the wall, in kN/m (also see eq.(5.8)

$$k_s=\frac{1}{0.86\frac{s}{s_0}+0.57} \tag{5.6}$$

where, s is the spacing of the fasteners around the perimeter of the sheets

$$k_n=\begin{cases} 1.0 & \text{for sheathing on one side} \\ \dfrac{F_{i,v,Rd,max}+0.5F_{i,v,Rd,min}}{F_{i,v,Rd,max}} & \text{for sheathing on both sides} \end{cases} \tag{5.7}$$

where,

$F_{i,v,Rd,max}$	design racking strength of the stronger sheathing
$F_{i,v,Rd,min}$	design racking strength of the weaker sheathing

Note: Refer to Appendix 5.1 for the calculation of existing design methods as in above equations.

Since the walls are to be tested with sheathing on one side only, the value of k_n is taken as 1.0 in this study. The q_i to calculate $k_{i,q}$ should be determined using only permanent actions of loads and any net effects of wind together with the equivalent actions arising from concentrated forces, including anchorage forces, acting on the panel. For the purposes of calculating concentrated vertical forces, these should be converted into an equivalent uniformly distributed load on the assumption that the wall is a rigid body e.g. for the load $F_{i,vert,Ed}$ acting on the wall as shown in the Figure 5.1.

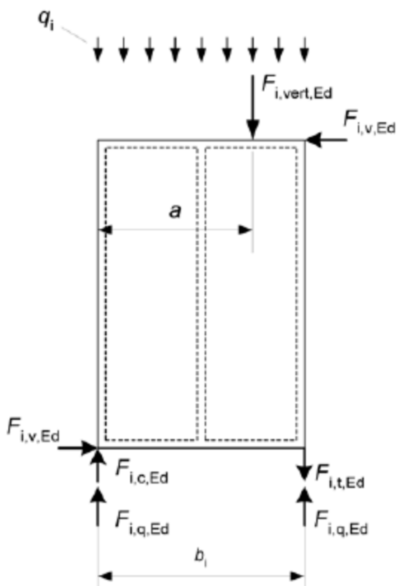


Figure 5.1 Vertical action q_i and reaction forces from vertical and horizontal actions

Source: BSI (2009c)

$$q_i = \frac{2\,a\,F_{i,vert,Ed}}{b_i^2} \tag{5.8}$$

where,

a	horizontal distance from the force F to the leeward corner of wall
b	length of the wall

All the walls were tested without any vertical load; hence, the value of k_{iq} is taken as 1 in this study.

PD 6693-1:2012

According to the Clause 21.2.2 of PD 6693-1: 2012 (Vessby et al., 2010b), the racking discontinuities are considered if the openings for doors or windows exceed any of these limits: a) The vertical dimension of the opening is greater than 0.65 times the wall diaphragm height and b) the height to the underside of the opening is less than 0.25 times the wall diaphragm height (see Figure 5.2).

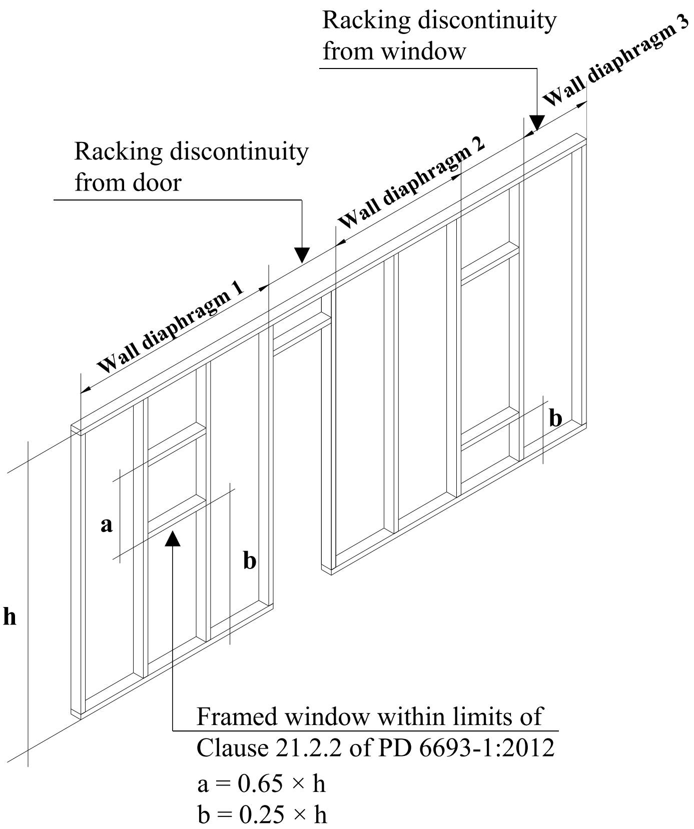


Figure 5.2 Wall diaphragms and racking discontinuities according to PD 6693-1:2012

Source: Interpretation based on BSI (2010b)

The wall diaphragm may comprise framed openings of dimensions within the limits given in Clause 21.2.2, provided that their effects on racking strength and stiffness are taken into account (see eq. (5.15)). Small openings within a length of wall diaphragm comprising only full height sheathing sheets may be allowed without reducing racking resistance if all of these conditions are met: a) The opening does not exceed 300 mm in both length and height where the opening is framed; b) The opening does not exceed 150 mm in both length and height or 200 mm in diameter where the opening is unframed; c) The edge distance from the opening to any edge of a sheathing sheet is at least the maximum dimension of the opening; d) only one such opening is allowed in a sheathing sheet and the spacing between such openings is at least 1200 mm. No more than two sheathing sheets of a length less than 600 mm should be used consecutively along the length of wall diaphragm. According to Clause 21.2.6, wall diaphragm with a framed opening of dimensions within the limits given in Clause 21.2.2 a) and b) may be designed to resist racking (see eq. (5.10)) provided that these conditions are met: a) each full height sheathing sheet on either side of the opening should have a minimum length of 0.25 times the width of the opening or one-eighth of the wall height, whichever is the larger. Alternatively, there should be a full width sheathing sheet (nominally 1200 mm) within a distance of one-eighth of the wall height from the vertical edge of the opening; b) the connection between the edge stud of the panel below the opening and the cripple stud immediately adjacent to the opening should have a design shear capacity per unit length of no less than $f_{p,d,t}$ (see eq. (5.11)).

The racking strength $F_{v,Rd}$ for racking wall made up of more than one wall diaphragm is calculated as,

$$F_{v,Rd} = \sum F_{i,v,Rd} \tag{5.9}$$

where, $F_{i,v,Rd}$ is the design racking strength of each wall diaphragm and is calculated as follows,

$$F_{i,v,Rd} = K_{opening} K_{i,w} f_{p,d,t} L \tag{5.10}$$

where,

L	length of the wall diaphragm
$f_{p,d,t}$	summation of the design shear capacities per unit length of the perimeter sheathing fastener in kN/m (also see (5.11)
$K_{i,w}$	modification factor taking into account wall length, vertical load and holding-down arrangements (also see (5.14)

$K_{opening}$ modification factor taking into account the effect of framed openings

The total design shear capacity per unit length of the perimeter sheathing fasteners $f_{p,d,t}$ is calculated as

$$f_{p,d,t} = f_{p,d,1} + K_{comb} f_{p,d.2} \tag{5.11}$$

with $f_{p,d,2} \leq f_{p,d,t}$

where,

$f_{p,d,2}$ design shear capacity per unit length of perimeter sheathing fasteners of the second sheathing layer in kN/m (also see (5.13))

K_{comb} sheathing combination factor having the values in Table 5.1

Note: Refer to Appendix 5.1 for the calculation of existing design methods as in above equations.

Table 5.1 Values of sheathing combination factor, K_{comb}

Details of second sheathing	$\boldsymbol{K_{comb}}$
None	0
On opposite side of framing to first sheathing layer but having sheathing sheets and fasteners of the same type, dimension, and spacing	0.75
On opposite side of framing to first sheathing layer but having sheathing sheets and fasteners of the different type, dimension, and spacing	0.5
On same side of framing to first sheathing layer	0.5

Source: BSI (2012b)

In order to limit racking deflection, the following condition should be applied

$$K_{i,w} f_{p,d,t} \leq 8(1+k_{comb}) \left(\frac{L}{H}\right) \tag{5.12}$$

Where, H is the height of the sheathed area of the wall diaphragm in m.

The design shear capacity per unit length of the perimeter fasteners to a sheathing sheet, $f_{p,d}$ is calculated as,

$$f_{p,d} = \frac{F_{f,Rd}[1.15+s]}{s} \tag{5.13}$$

where,

$f_{p,d}$ design lateral capacity of an individual fastener in kN

s sheathing perimeter fastener spacing in m

The modification factor $K_{i,w}$ is calculated from equation,

$$K_{i,w}=\left[1+\left(\frac{H}{\mu L}\right)^{2}+\left(\frac{2M_{d,stb,n}}{\mu f_{p,d,t}L^{2}}\right)\right]^{0.5}-\left(\frac{H}{\mu L}\right) \tag{5.14}$$

where, the equation 5.15 gives a value of $K_{i,w} > 1$, $K_{i,w}$ should be taken as 1.0 and where it gives a value of $K_{i,w} < 0$, $K_{i,w}$ should be taken as 0.

where,

$$M_{d,stb,n}=M_{d,stb}-M_{d,dst,top}$$

$$\mu=\min\left[1,\ f_{w,d}/f_{p,d,t}\right]$$

and where,

$f_{w,d}$ design withdrawal capacity of bottom rail-to-floor conncection per unit length in kN/m

$M_{d,stb}$ design stabilizing moment in kNm, about the leeward end of the wall diaphragm from design permanent load, reduced by any vertical component of design wind load

$M_{d,dst,top}$ design destabilizing moment in kN/m about the top of the wall diaphragm from design wind load

For a wall diaphragm with a framed opening of dimensions within the limits given in Clause 21.2.2 and meeting the provisions of Clause 21.2.6, $K_{opening}$ should be taken as:

$$K_{opening}=1-1.9\rho \tag{5.15}$$

where,

$$\rho=\frac{A}{HL} \tag{5.16}$$

and

A Aggregate area of openings in wall diaphragm in m^2. The area of the opening is to be taken as 0.5 $(L_{open})^2$ if the vertical dimension of an opening is less than half its horizontal dimension (L_{open}).

Note: Refer to Appendix 5.1 for the calculation of existing design methods as in above equations.

5.3 Method

5.3.1 Wall specimen

Similarly to the experimental work discussed in the previous chapters , all wall specimens tested were assembled using C16 (BSI, 2009a) timber with a cross-section of

44 mm × 95 mm for the frame members. The connection of the timber frames were formed by using wood screws of dimension 3.55 × 100 mm long, 9 mm thick Oriented Strand Boards (OSB/3) (BSI, 2006) were used for sheathing, and were connected to the frame timber using 2.8 × 50 mm long round smooth bright nails. The bottom rail was fixed down to the test bed using four M12 × 200 mm long bolts.

5.3.2 Test set up

The lateral (racking) load was applied in accordance with BS EN 594:2011 specification. The studs were connected to the top and bottom rails, sheathed on one face only and fixed with the nails. The general set-up of the walls and the application of loads and measurement of determinations were as described in Chapter 4, section 4.3.2. The density and moisture content were determined from samples of the timber and OSB sheathing materials in accordance with BS EN 322 and BS EN 323 respectively; and are shown in Table 5.2.

Table 5.2 Moisture content and density values from tested walls

Material	Average density [kg/m^3]	Average moisture content [%]
Timber - C16	365.76	15.25
OSB/3	591.61	4.82

5.3.3 Test series

A programme of work was designed to examine the effect of opening for windows and doors on the racking performance.

i. Openings for windows in wall panels

The effect of opening on the racking performance (strength and stiffness) was examined by testing different opening sizes on randomly selected sizes such as 300 × 600, 600 × 600, 900 × 600, 1200 × 900, and 1500 × 1200 mm on wall panels of 2400 × 2400 mm. The configurations of the walls with opening are illustrated in the Figure 5.3. The openings were positioned at a distance of 600 mm from the outer edge of the top rail and leading stud. Additional intermediate studs were introduced for the window sizes of 300 × 600, 900 × 600, and 1500 × 1200 mm and fixed with the horizontal rails (also refer Figure 5.3.- a). The panel-to-frame connections were done by using nails spaced at

100 mm and 200 mm in the horizontal and vertical members respectively where the opening is located. It is to be noted that in the wall with opening size of 900 × 600 mm for window, one of the threaded bars for holding down was adjusted to 1595 mm from its original position of 1500 mm from the leading edge of the bottom rail, this to avoid the contact with additional vertical stud.

Two of the walls with larger openings for the windows i.e. 900 × 600 and 1500 × 1200 mm were reassembled and tested with additional spreader on the top rail and trimmers (see Figure 5.3. -b). The additional intermediate stud/s was introduced for both window sizes. The panel-to-frame connections were built by using nails spaced at 100 mm and 200 mm in the horizontal and vertical members respectively. Trimmers were fixed on the studs by 3 × 75 mm screws at spacing of 150 mm.

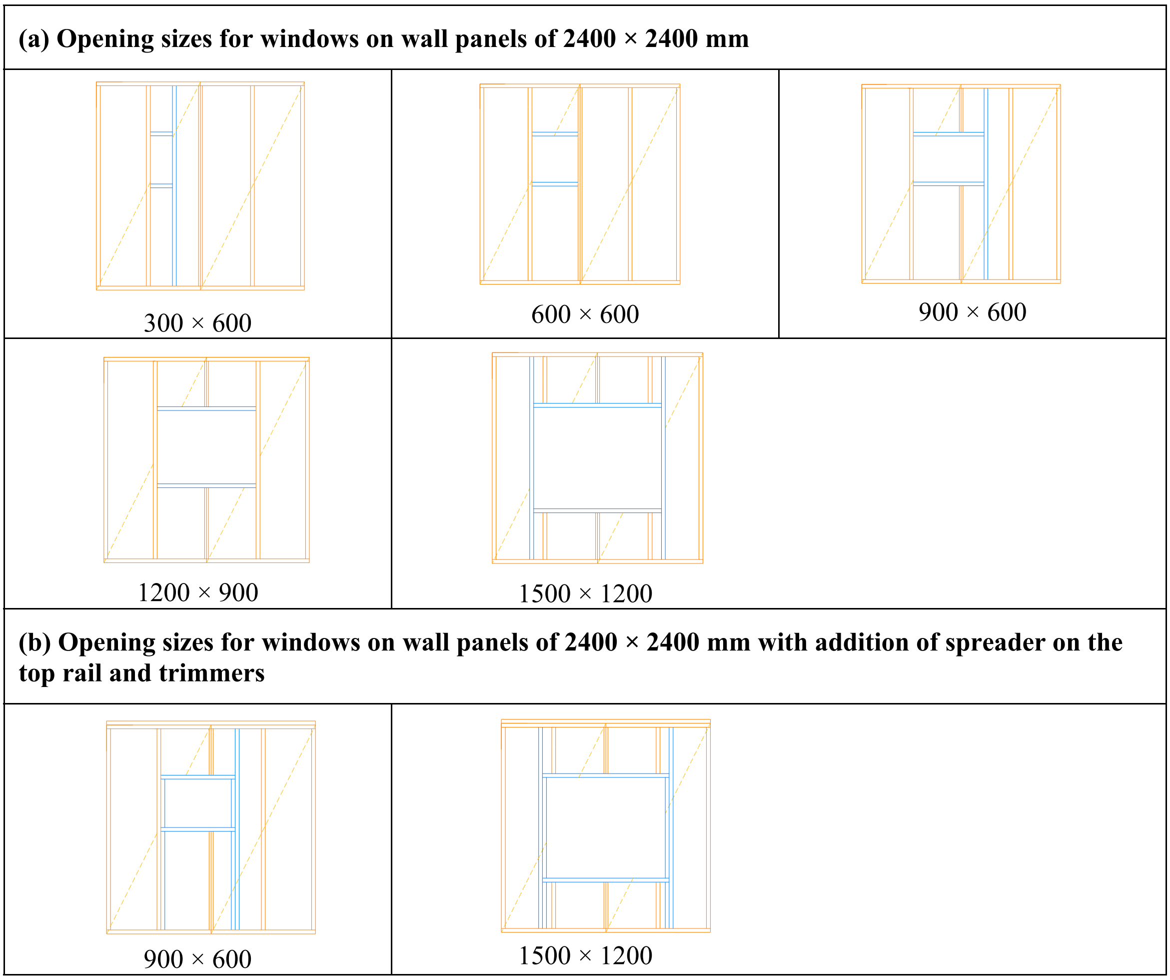


Figure 5.3 Configurations of walls with openings for windows

ii. Openings for doors in wall panels

The effect of the size of openings for doors on the racking performance of the walls were examined by testing walls with openings on randomly selected sizes such as 600 × 2050, 900 × 2050, 1200 × 2050, 1500 × 2050, and 1800 × 2050 mm on wall panels of 2400 × 2400 mm (Figure 5.4 - a). For the wall with opening size of 1200 × 2050 mm, the actual height of middle stud was kept as 284 mm. The length of bottom rail was fixed as 622 mm each on the leading and rear part of the wall instead of running through the entire wall length of 2400 mm. Again, similar to test on openings for windows, two additional walls with openings for doors of 900 × 600 and 1500 × 1200 were constructed and internal studs were doubled, see Figure 5.4 (b). The additional intermediate studs were fixed on both leading and rear part of the wall. For the wall with opening size of 1800 × 2050 mm, cripple studs were used along the edges of the openings and rails were also provided on the horizontal edges of the openings.

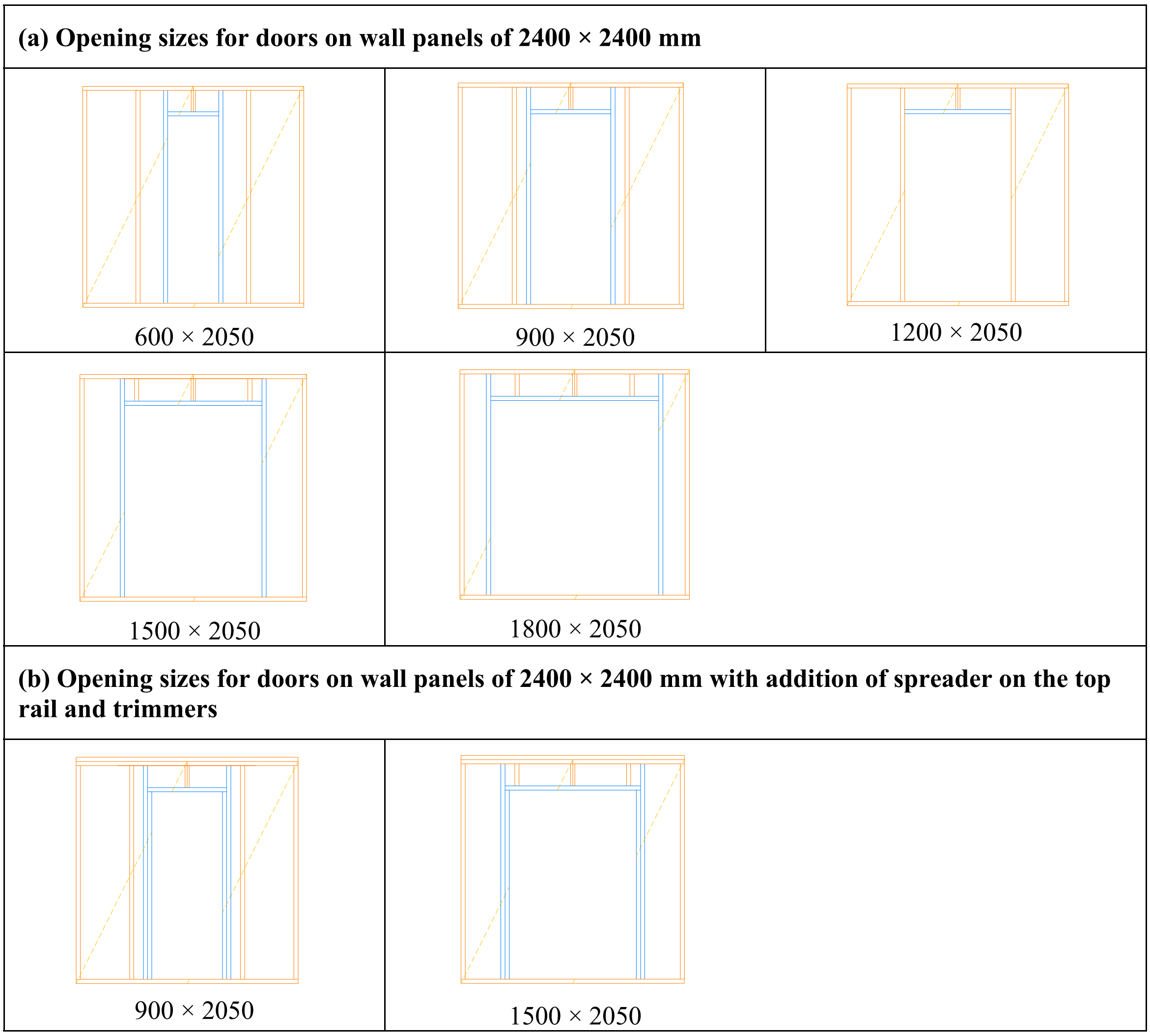


Figure 5.4 Configurations of walls with openings for doors

Sugiyama and Matsumoto (1994) and Dolan and Johnson (1997) developed the empirical formulas for the calculation of stiffness and strength of the walls which are detailed in equations 5.18 and 5.19 and the parameters are shown in Figure 5.20. The results obtained from the tests were compared with these empirical equations (refer Table 5.7). These equations considered the sum of the area of opening and sum of the length of full height of sheathing. However, this research is limited with the single opening, i.e either for window or for door, on the timber frame wall.

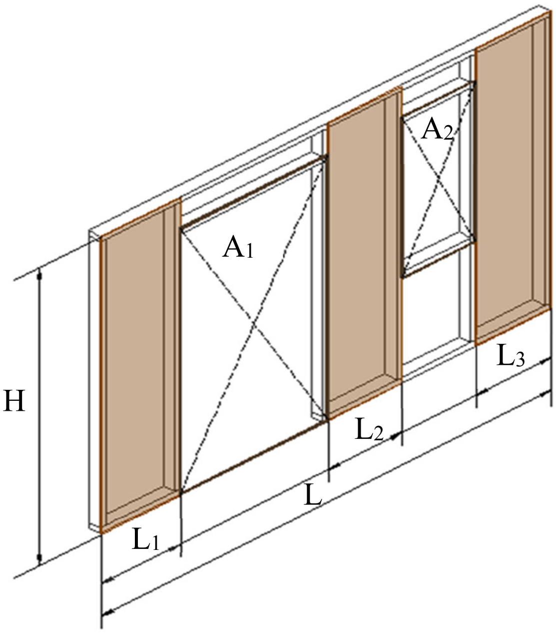


Figure 5.5 Wall showing the discontinuity of length of wall due to openings
Source: Interpretation based on Dolan and Johnson (1997)

5.4 Results, Analysis, and Discussion

The strength and stiffness of the walls for the experimental tests were determined in accordance with recommendation of BSEN 594. The analytical procedure described in section 5.2 has been used to compute the racking performance of the walls. The comparison with test results is provided in the subsections 5.4.2 and 5.4.4 for windows and doors respectively.

5.4.1 Effect of size of opening for windows with and without spreader on the top rail and trimmers

The load-deformation behaviours recorded are shown in
Table 5.3 and Figure 5.6 and 5.6. The result showed that the wall that has opening size of 300 × 600 mm (3.13 % opening areas) has higher strength and stiffness than all other

walls with openings (Figure 5.8 – a and b) as well as walls without opening. This could be due to the additional intermediate stud and the small opening size in the wall. Table 5.4 shows that the stiffness and strength decreases as the opening size increase except in the wall that has opening size of 900 × 600 mm. This wall has higher strength and stiffness than opening size of 600 × 600 mm. The provision of extra studs may have influenced the strength and stiffness of the walls. The effects of adding framing around the openings and cripple studs are shown in Table 5.3 and Figure 5.6. For opening area of 9.38% the effect was small and not conclusive, but had more positive effect when used with larger opening of 31.25%.

Table 5.3 Strength and stiffness for different sizes of window openings in OSB walls with and without spreader on the top rail and trimmers

Percentage of openings	**Window length**	**Window height**	**Stiffness**	**F_{max} (Max load)**	**Strength ratio**	**Stiffness ratio**
(%)	mm	mm	(N/mm)	(kN)		
0	-	-	736.06	19.79	1.0	1.0
3.13	300	600	912.52	21.84	1.10	1.24
6.25	600	600	704.51	17.44	0.88	1.96
9.38	900	600	755.51	18.51	0.94	1.02
9.38 with additional framing	900	600	549.24	17.65	0.89	0.75
18.75	1200	900	698.14	16.33	0.83	0.95
31.25	1500	1200	526.68	11.67	0.59	0.72
31.25 with additional framing	1500 (1368)	1200 (1156)	542.54	14.18	0.72	0.74

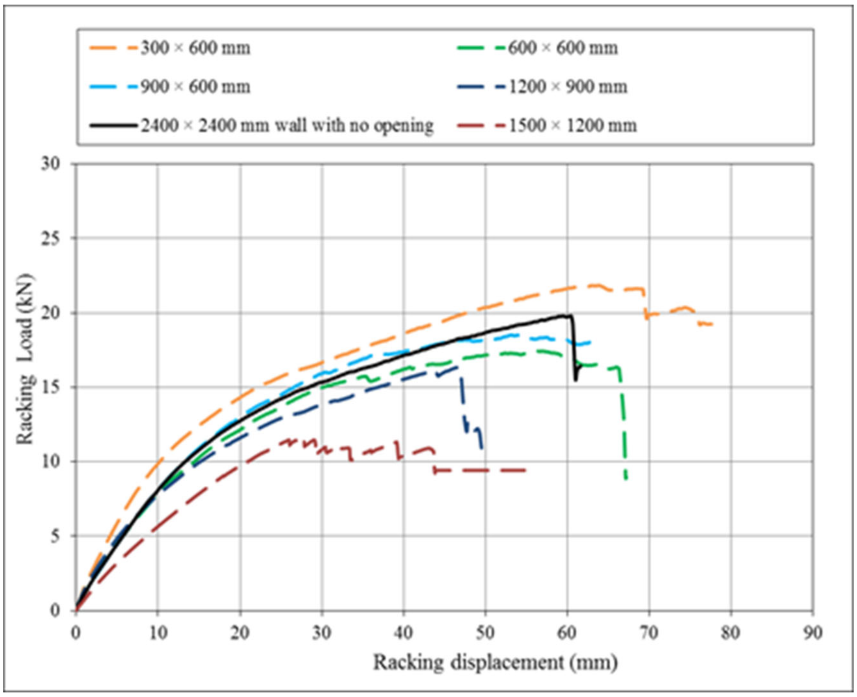


Figure 5.6 Load-deformation behaviour of OSB walls with openings for windows

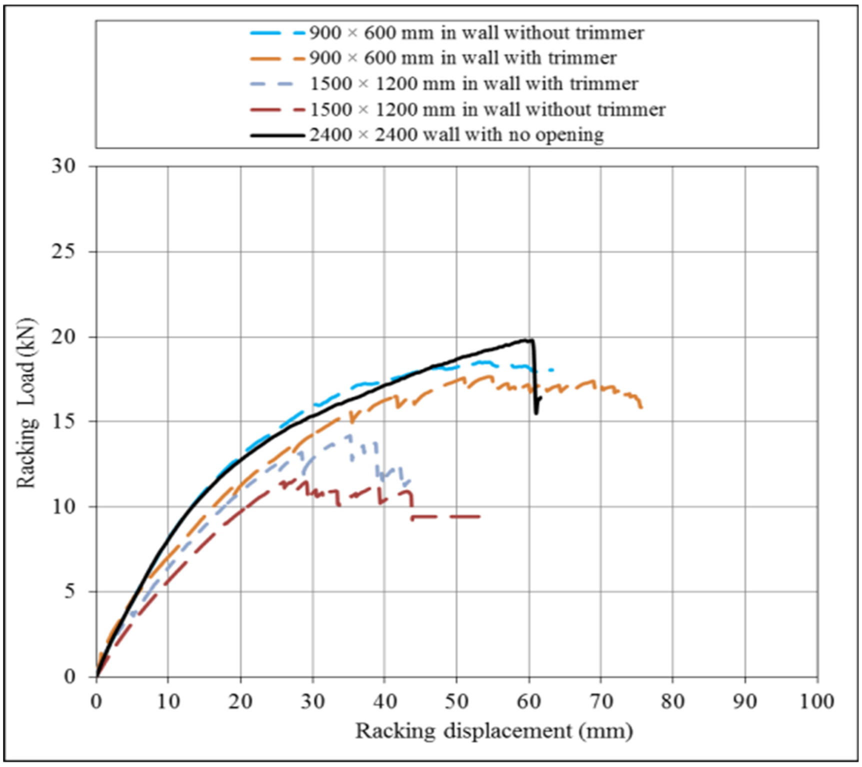
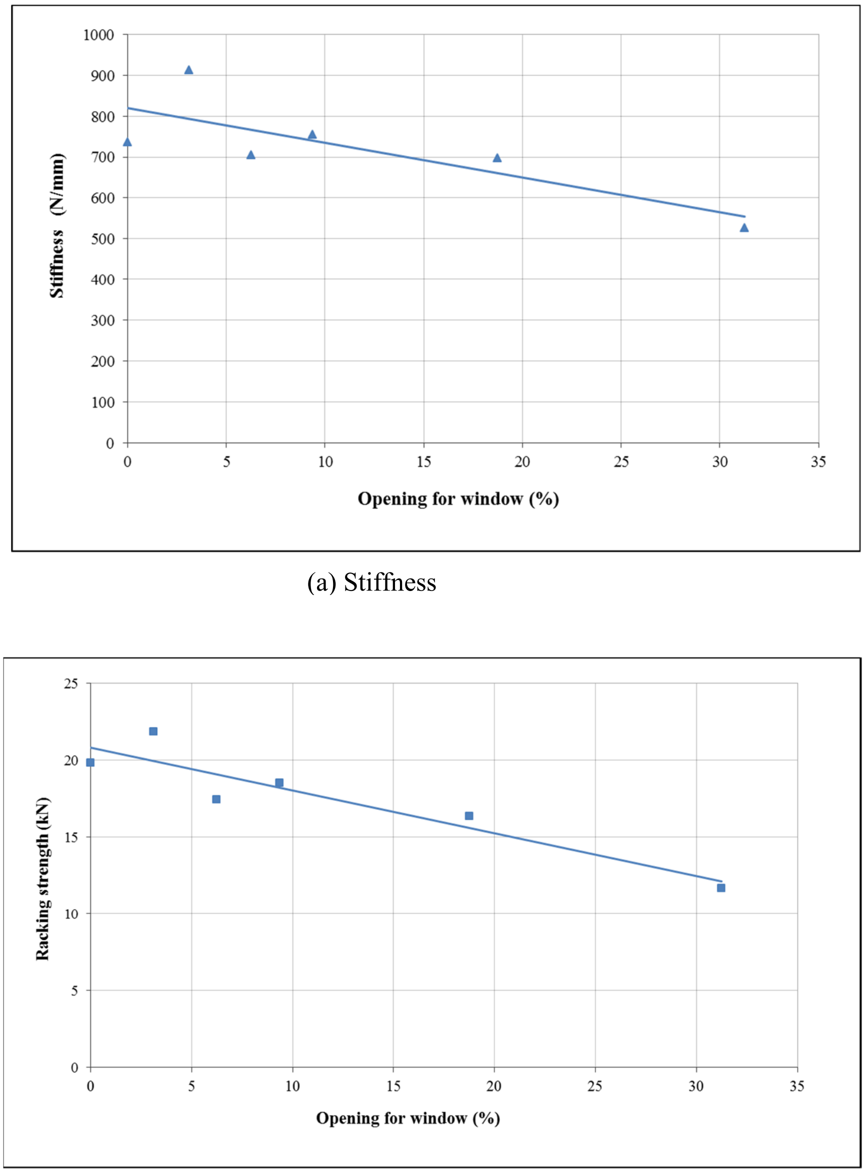


Figure 5.7 Load-deformation behaviour of the openings for windows in OSB wall panels with and without spreader on the top rail and trimmers



(a) Stiffness

(b) Strength

Figure 5.8 Racking performances: of OSB wall as a function of percentage of opening for windows.

The failure modes observed were combination of lifting-up of the leading stud from the bottom rail, pull-out of the nails from timber and pull-through of the nails in OSB, and shearing of OSB at opening corners, particularly in walls with openings of 1200 × 900 and 1500 × 1200 mm, (Figure 5.9 and 5.9). There is a relatively linear relationship between the stiffness and percentage of opening; the stiffness decreases with the increase in area of the opening. The wall with 3% and 9% area openings are stiffer than the wall without any opening, which could be due to an additional stud fixed in the walls to support horizontal studs of the opening.

Figure 5.9 Walls with different sizes of openings for windows during testing and at failure.

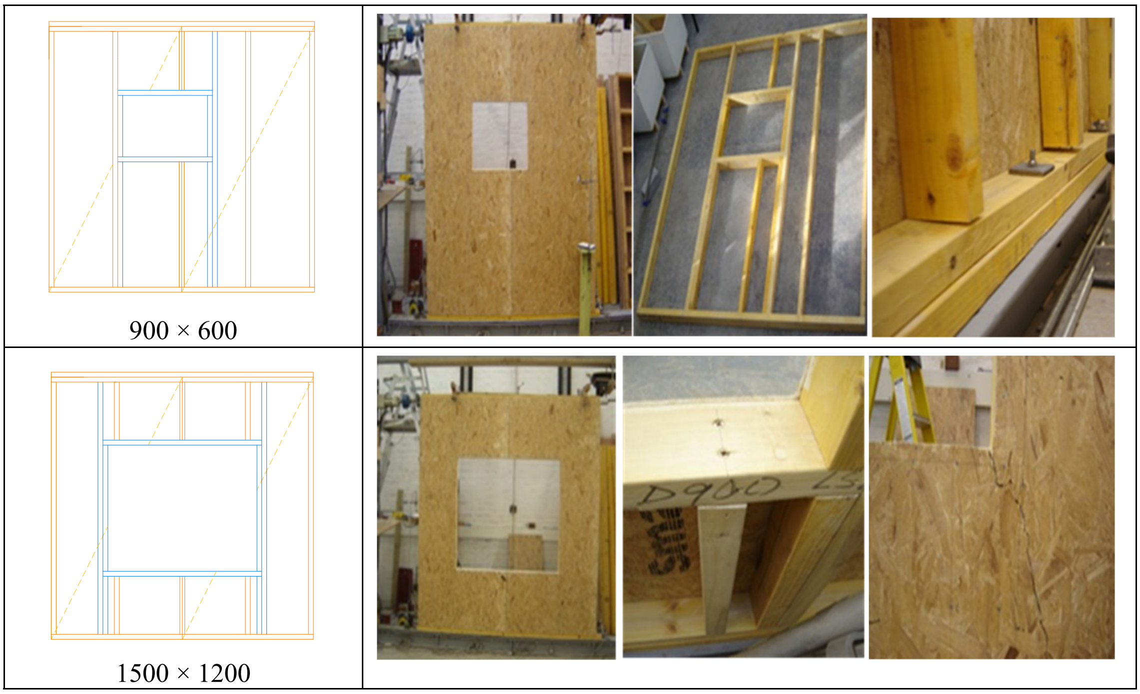


Figure 5.10 Walls with different sizes of openings for windows during testing and at failure.

5.4.2 Comparison of the experimental results and existing design methods: EC5, Method B, and PD 6693

The design racking loads were calculated in accordance with EC5 Method B; the lengths of panel on each side of the opening were considered as separate panel (EC5, Clause 9.2.4.3.1 (3) and (4)). In the wall with opening size of 1500 × 1200 mm, when separate panels on each side of the openings are 450 mm (i.e. less than the 600 mm as required by EC5 Clause 9.2.4.3.1 (2)), no racking resistance value is given in the Table 5.4.

When the racking performance was assessed based on EC5, PD6693-1:2012, the sizes of openings for windows did not influence the racking performance as defined in Clause 21.2.2 (a) and (b) and 21.2.6 (a) of PD6693-1:2012, hence for all sizes of openings in walls have the value of 6.01kN.

Table 5.4 Comparison between the experimental results and existing design methods of openings for windows with and without the spreader and trimmer

Opening for windows (l x h) mm		300 × 600	600 × 600	900 × 600	900 × 600 with spreader and trimmer	1200 × 900	1500 × 1200	1500 × 1200 with spreader and trimmer
Percentage of openings (%)		3.13	6.25	9.38		18.75	31.25	
Experimental	Ultimate racking load with openings (kN)	21.84	17.44	18.51	17.65	16.33	11.67	14.18
	Racking load test result without opening (kN)	19.79						
	Characteristic racking load (kN)	18.62	14.88	15.77	15.07	13.97	9.99	12.12
	Design racking resistance (kN) (Kmod = 1 and γ_m= 1.25 assumed)	14.91	11.92	12.63	12.07	11.19	8.00	9.70
Design to EC5 (Method B)	Calculated design racking load (kN) - (Method B)	6.89	5.91	4.92		3.94	0	
Design to PD6693-1:2012	Calculated design racking load (kN) - (PD)	6.01						

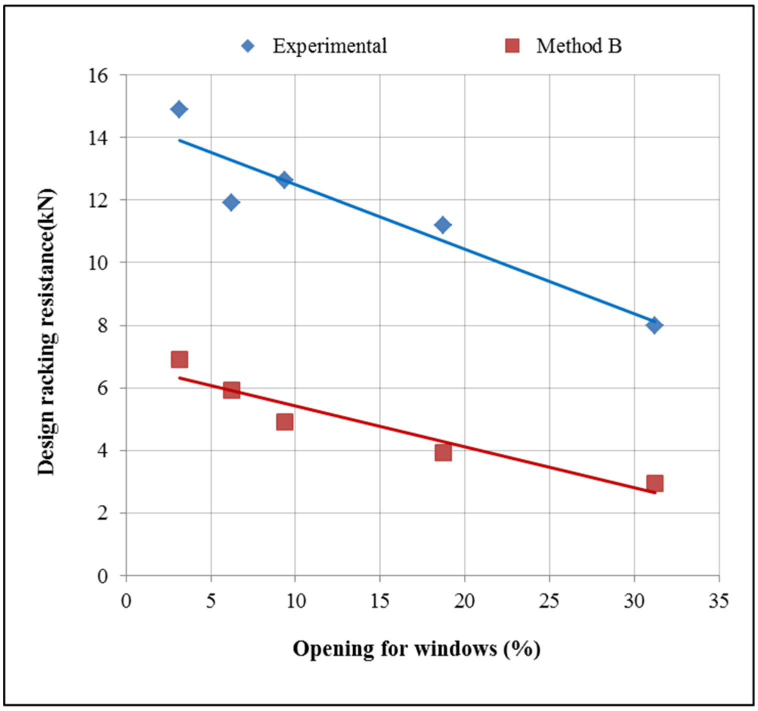


Figure 5.11 Comparison of the design racking resistance between experimental results and Method B for openings for windows

The Table 5.4 shows that the calculated design values of all openings sizes are lower than the experimental values (see Figure 5.10). The Clause 9.2.4.3.1 (2) in Method B of EC5 specified that large opening size such as 1500 x 1200 mm do not contribute to racking load. However, the experimental result of the same opening size recorded the design load of 8 kN which is higher than the calculated racking loads of all opening sizes in accordance with Method B and PD6693-1:2012.

5.4.3 Effects of size of opening for doors with and without spreader on the top rail and trimmers

The load deformation behaviour of the walls with an opening for door of pre-determined size is illustrated in Figure 5.12 and 5.12. The maximum racking load for each wall recorded is shown in Table 5.5. Wall with opening size of 600 × 2050 mm showed higher stiffness and strength than other walls with opening. Figure 5.12 and 5.12 illustrates the effect of the opening size on the racking performance of the walls.

In general, the failure modes observed were by shearing of OSB sheathing on both top corners of the opening and partial buckling at rear top corner of the opening which was considered to be high stress concentration at this area.

Table 5.5 Strength and stiffness for different size of the door openings in OSB wall with and without the spreader on the top rail and trimmers

Percentage of openings	**Door size (l)**	**Door height (h)**	**Stiffness**	**F_{max} (Max load)**	**Strength ratio**	**Stiffness ratio**
(%)	mm	mm	(N/mm)	(kN)		
0	-	-	736.06	19.79	1.0	1.0
21.35	600	2050	373.33	12.42	0.63	0.51
32.03	900	2050	335.80	10.53	0.53	0.46
32.03 with spreader and trimmers			284.98	10.11	0.51	0.38
42.71	1200	2050	210.57	5.98	0.31	0.29
53.39	1500	2050	198.80	5.46	0.28	0.27
53.39 with spreader and trimmers			204.04	7.36	0.37	0.28
64.06	1800	2050	92.01	5.09	0.26	0.13

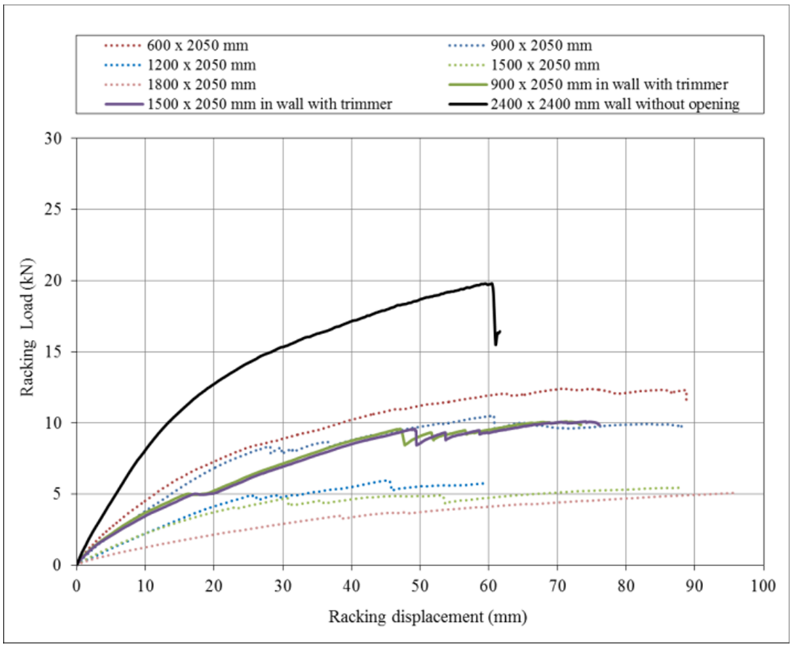


Figure 5.12 Load-deformation behaviour of the size of the opening for doors in OSB walls

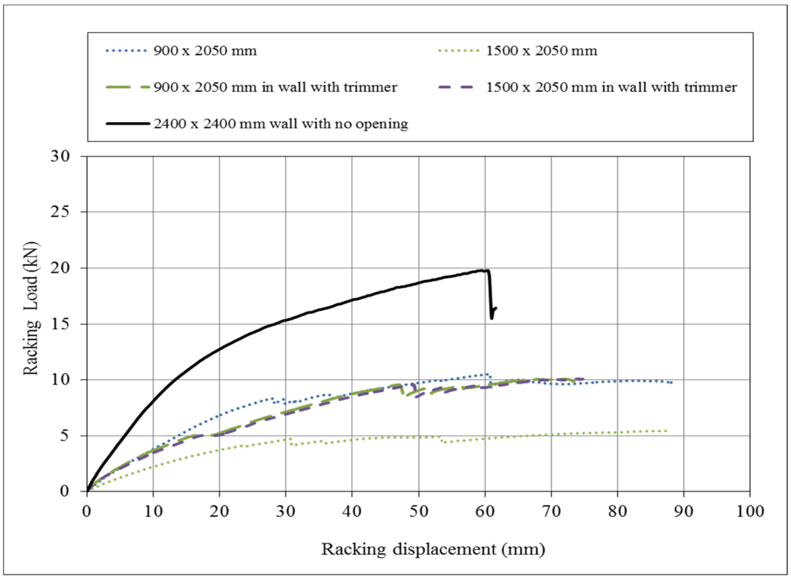
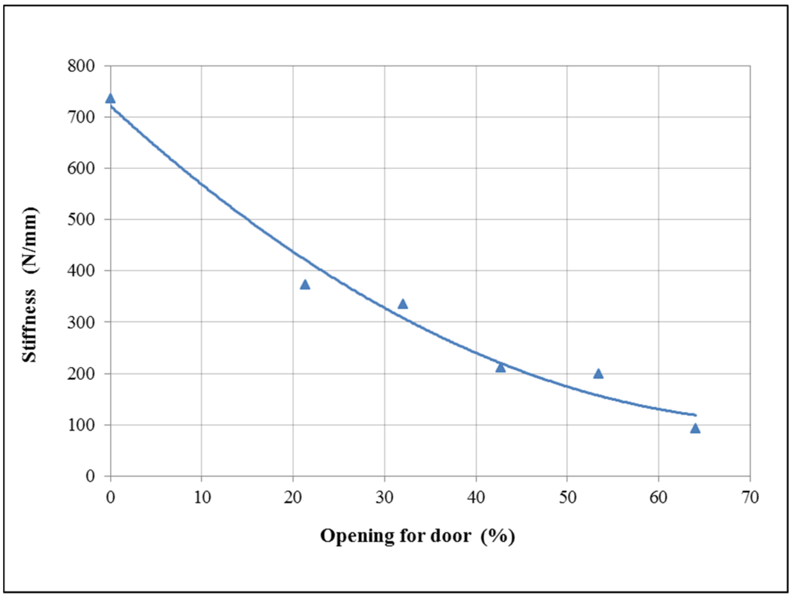
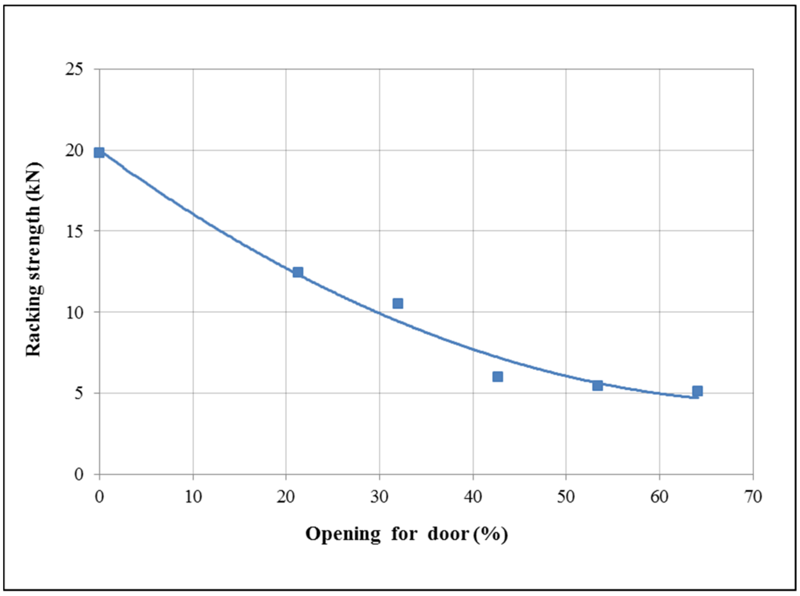


Figure 5.13 Load-deformation behaviour of the openings for doors in OSB wall panels with and without spreader on the top rail and trimmers

In the wall with 1800 × 2050 mm sized door opening, cripple studs were used along the edges of the openings and timber sections were also provided on the horizontal edges of the openings. The failure of OSB walls occurred again by shearing and buckling on top rear part of an opening. Both stiffness and racking strength decreased with increase in the size of openings for door as shown in Figure 5.14 - a and b.



(a) Stiffness



(b) Strength

Figure 5.14 Effect of different sizes of door openings on racking performances of OSB wall

The possible effects of addition of spreader beams on the top rail was examined by testing two further walls with opening of 900 x 2050 and 1500 x 2050. The results are shown in Table 5.5 and Figure 5.12 and 5.12. In general, no significant change in performance compared with walls without spreader beams were noted. Walls with different sizes of openings for doors during testing and at failure modes are shown in Figures 5.14 and 5.15.

Figure 5.15 Walls with different sizes of openings for doors during testing and at failure.

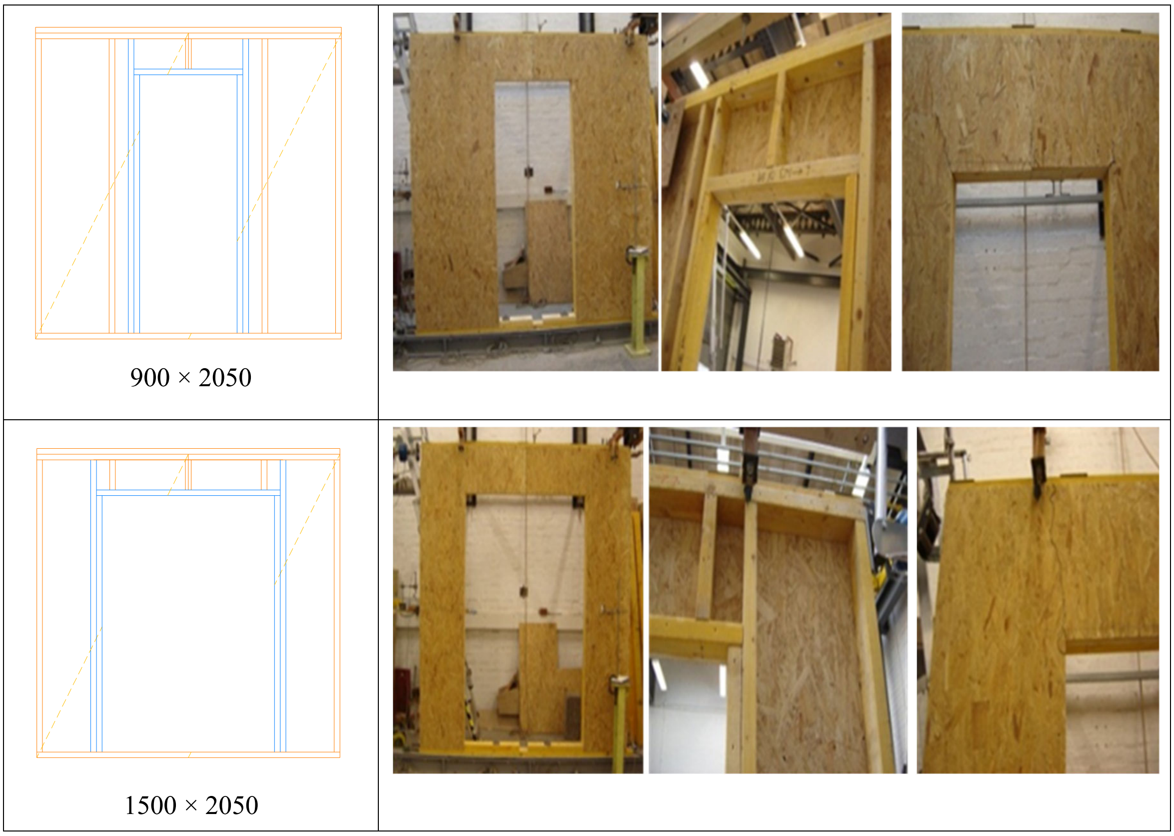


Figure 5.16 Walls with different sizes of openings for doors during testing and at failure.

5.4.4 Comparison with design methods to Eurocode 5 for wall panels with and without the spreader on the top rail and trimmers

The experimental results were compared with Method B of EC5 and PD 6693-1:2012 and are shown in Table 5.6. Similar to the openings for windows, the design racking load using Method B was calculated in accordance with Clause 9.2.4.3.1 (2) and (3), considering the separate panels on either side of the openings. The sizes of the panels on either sides of walls with opening to be 1500 × 2050 and 1800 × 2050 mm were considered too small based on (EC5, Method B requirements) to be considered for calculating racking strength. Also, taking into considerations of PD6693-1:2012, the design racking load was calculated with racking discontinuity caused by large sized openings for doors (Clause 21.2.2 (a) and (b)).

The racking resistances of the walls with openings for doors obtained from experimental result are higher than Method B and PD 6693-1:2012 (Table 5.6 and Figure 5.16).

Table 5.6 Comparison between the experimental results and existing design methods of openings for doors

Opening for doors (l × h) mm		**600 x 2050**	**900 x 2050**	**900 x 2050 with S & T**	**1200 x2050**	**1500 x 2050**	**1500 x 2050 with S & T**	**1800 x 2050**
Percentage of openings (%)		21.35	32.03		42.71	53.39		64.06
Experimental	Ultimate racking load (kN)	12.42	10.53	10.11	5.98	5.46	7.36	5.09
	Ultimate racking load without opening (kN)	19.79						
	Characteristic racking load (kN)	10.62	9.00	8.61	5.09	4.68	6.27	4.36
	Design racking resistance (kN) (Kmod = 1 and γ_m= 1.25 assumed)	8.50	7.21	6.90	4.07	3.74	5.02	3.49
Design to EC5 (Method B)	Design racking load (kN)	6.06	5.08		4.08	0		0
Design to PD 6693-1:2012	Design racking load (kN)	3.77	2.72		1.82	1.10		0.57

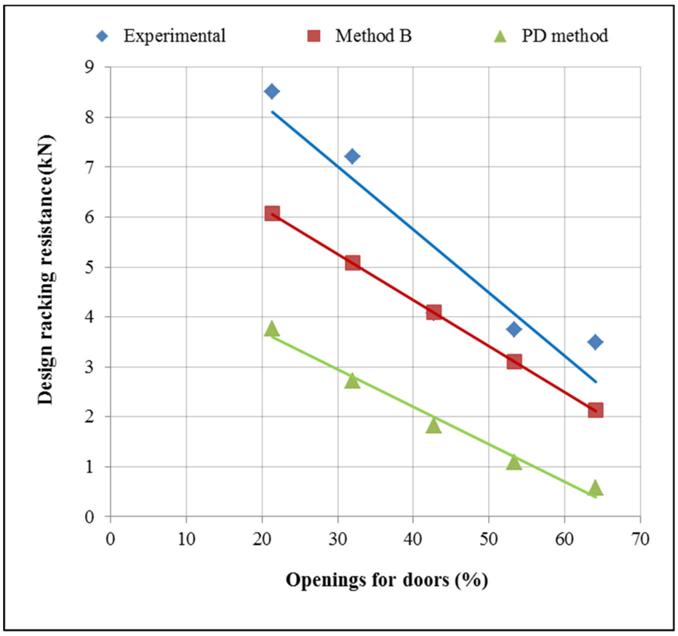


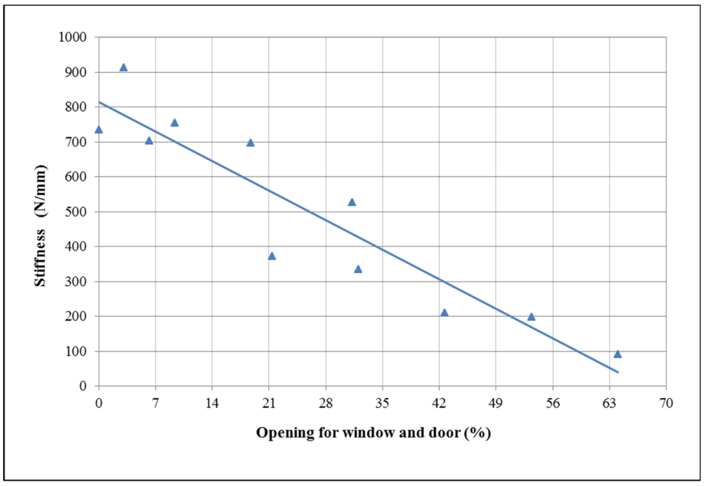
Figure 5.17 Comparison between Experiment results with EC5 (Method B) and PD for openings for doors

In general, the design racking resistance for the wall with openings for doors were greater than the design values obtained using EC5 Method B and PD 6693-1:2012. With openings up to 1200 x 2050, the Method B values are closer to the experiment results but it provides no racking resistance where panel sizes on either side of the opening becomes less than 600 mm.

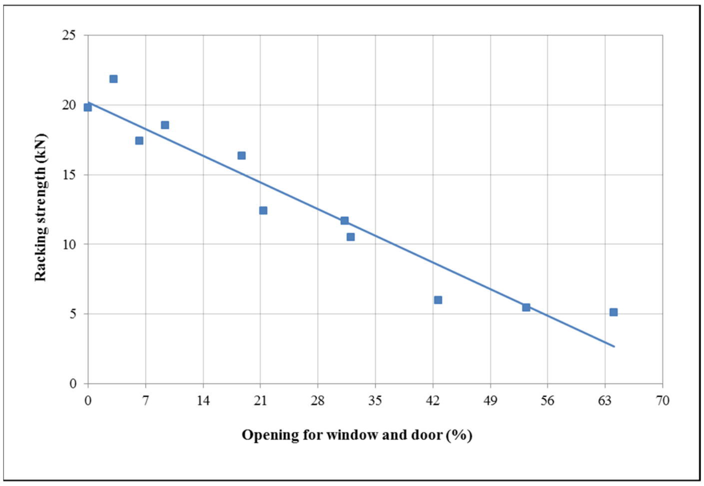
5.4.5 Overall comparison on racking performance

i. Opening percentages

From the above discussions, it is clear that the racking performance of wall with opening is influenced by opening size and configurations. The effect on racking stiffness and strength due to opening sizes (windows and doors) in the walls without the spreader and trimmer are shown in Figure 5.18-a and b respectively (see also Figures 5.9, 5.10, 5.15 and 5.16).



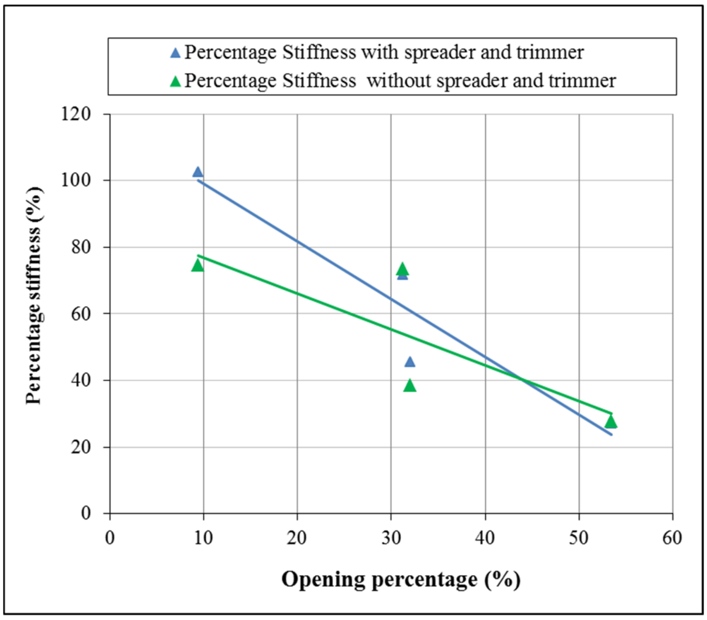
(a) Stiffness



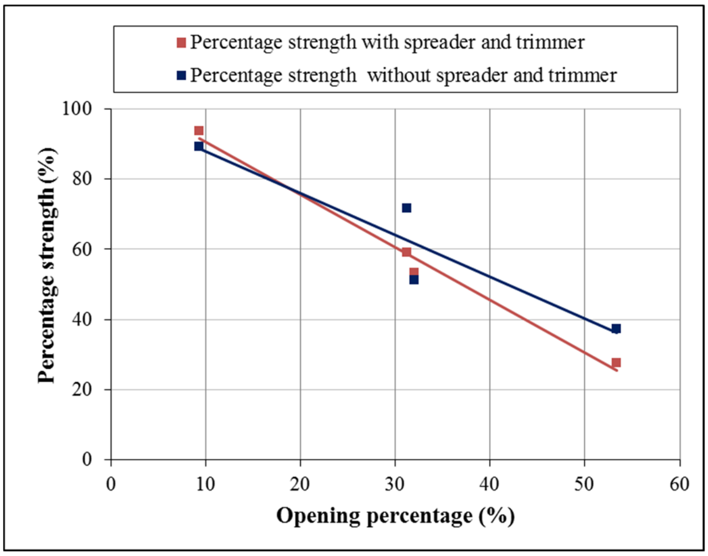
(b) Strength

Figure 5.18 Comparison of effect of the opening percentages on racking performance of OSB wall

The figures show that there is relatively linear relation between the openings and the racking performance of the walls. The stiffness and strength decrease with the increase in opening sizes (as percentage of the wall).



(a) Stiffness



(b) Strength

Figure 5.19 Comparison between the percentages of stiffness and strength and opening percentages in walls with and without spreader on top rail and trimmer

From the Figure 5.19 and Figure 5.20, it can be seen that as the opening increases the strength and stiffness decreases.

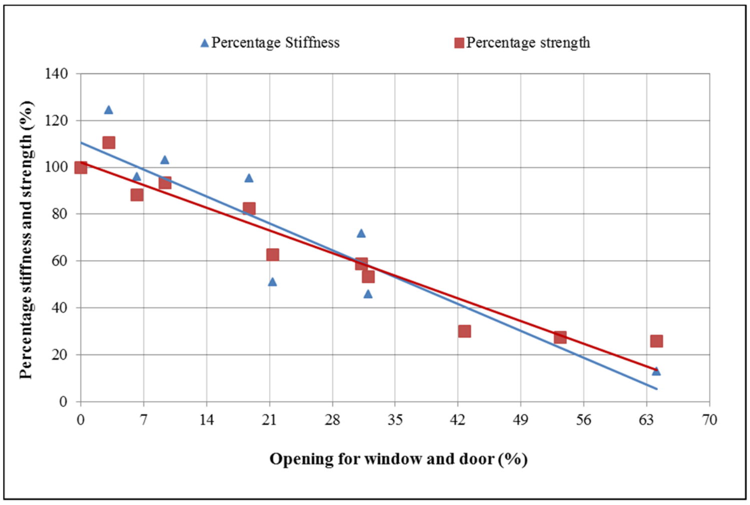


Figure 5.20 Relation between the percentages of stiffness and strength and opening percentages in walls without spreader on top rail and trimmer

ii. Existing theories

The design of traditional shear wall includes multiple shear wall segments if it contains openings for windows and doors (Dolan and Johnson, 1996). According to them, it is also essential that each wall panels has to be fully sheathed and has overturning restraint provided by the structure's weight and/or mechanical anchors. In this case, the design capacity of shear walls is assumed to equate the sum of the capacities of each shear wall segment. The sheathing which is placed above and below the openings is typically not considered to contribute in the overall performance of the wall. Another empirical-based approach to the design of shear walls with openings is the perforated shear wall method that consists of a combination of prescriptive provisions and empirical adjustments to design values in shear wall selection for the design of shear wall segments containing openings. With an application of this method, when designing for a given load, shear walls will have a reduced number of overturning restraints than a similar shear wall constructed with multiple traditional shear wall segments. The prescriptive provisions and empirical adjustments are based on the parameters of various studies conducted on shear walls with openings. The empirically derived adjustment factors, or shear capacity ratios, for the perforated shear wall method are based on an equation developed by Sugiyama and Matsumoto (1994) for predicting shear capacity ratios. According to which, the shear capacity ratio, or the ratio of the strength (or stiffness) of a shear wall segment with openings to the strength (or stiffness) of a fully sheathed shear wall segment without openings is determined by,

$$F = \frac{r}{3 - 2r} \tag{5.17}$$

where,

F	ratio of shear strength with/without openings
r	sheathing area ratio

The parameters for sheathing area ratio are illustrated in Figure 5.5.

Sheathing area is calculated as,

$$r = \frac{1}{\left(1+\frac{\alpha}{\beta}\right)} \tag{5.18}$$

$$\alpha = \frac{\sum \mathrm{Ai}}{\mathrm{H.L}}$$

$$\beta = \frac{\sum L_i}{\mathrm{L}}$$

where,

α	opening area ratio
β	wall length ratio
ΣL_i	sum of the length of full height sheathing
H	height of wall
$\sum Ai$	Sum of the area of opening

The ratio of shear stiffness according to Dolan and Johnson (1996) cited in (Dujic et al., 2007) conducted on monotonic and cyclic test results on full size wood frame walls with various openings is recognised as,

$$F = 1.27r - 0.28 \tag{5.19}$$

where,

F	Ratio of shear strength with/without openings
r	Panel/sheathing area ratio

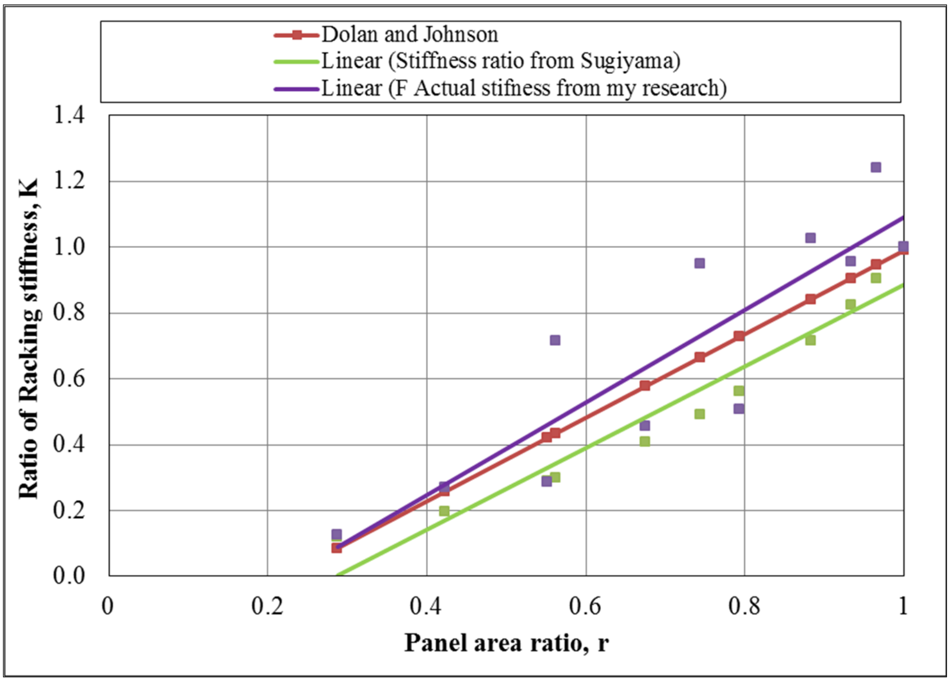


Figure 5.21 Comparison of F_{actual} stiffness and strength ratio with Sugiyama and Matsumoto (1994) and Dolan and Johnson (1997)

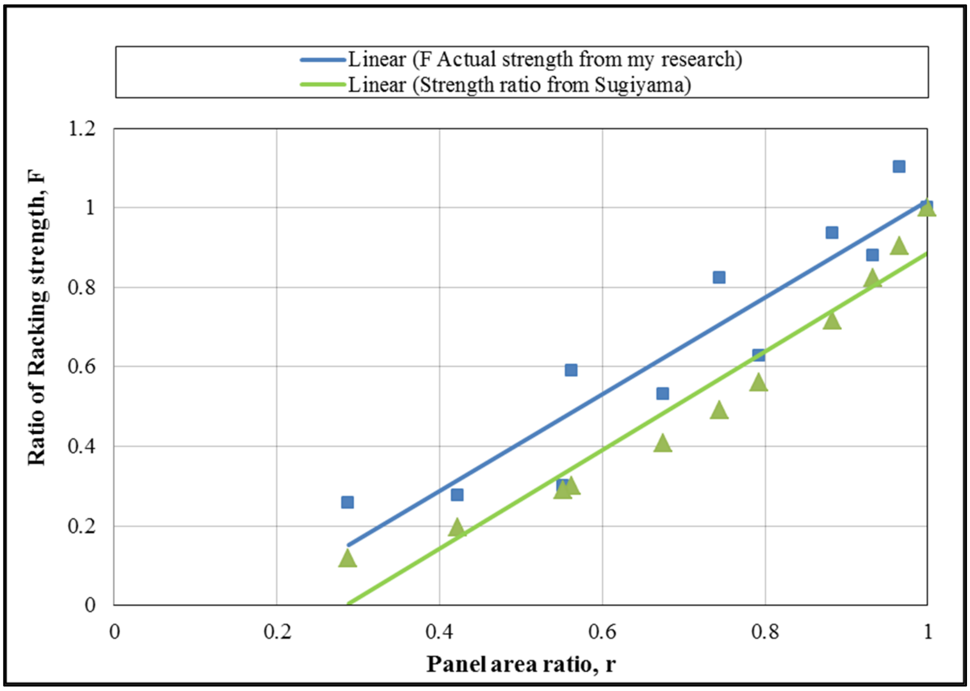


Figure 5.22 Comparison of linear relation of F_{actual} strength ratio and panel ratio of openings with Sugiyama and Matsumoto (1994)

Table 5.7 Comparing with Sugiyama and Matsumoto (1994) and Dolan and Johnson (1997) methods

	Size (mm)	L(mm)	L(mm)	H(mm)	ΣAi	α=ΣAi/HL	ΣLi	β=ΣLi/L	r=1/(1+α/β)	Stiffness from test	Strength calculated using empirical equation by Sugiyama and Matsumoto (1994) (K= r / (3-2r))	Actual stiffness from test; F, $_{Actual}$ K= stiffness of opening / stiffness without opening	F_{max} from test	Strength calculated using empirical equation by Sugiyama and Matsumoto F=r / (3-2r)	Stiffness calculated using empirical equation by Dolan and Johnson (1997) (F=1.27r-0.28)	Actual strength; F, $_{Actual}$ F=strength of opening / strength without opening
No openings	0	0	2400	2400	0	0	2400	1	1	736.06	1	1	19.79	1	1.0	1
Openings for windows	300x600	300	300	600	180000	0.03	2100	0.88	0.97	912.52	0.90	1.24	21.84	0.90	0.95	1.10
	600x600	600	556	556	309136	0.05	1800	0.75	0.93	704.51	0.82	0.96	17.44	0.82	0.91	0.88
	900x600	900	556	856	475936	0.08	1500	0.63	0.88	755.51	0.72	1.03	18.51	0.72	0.84	0.94
	600x2050	600	556	2028	1127568	0.20	1800	0.75	0.79	373.33	0.56	0.51	12.42	0.56	0.73	0.63
	1200x900	1200	856	1156	989536	0.17	1200	0.50	0.74	698.14	0.49	0.95	16.33	0.49	0.67	0.83
	900x2050	900	856	2028	1735968	0.30	1500	0.63	0.67	335.8	0.41	0.46	10.53	0.41	0.58	0.53
	1500x1200	1500	1156	1456	1683136	0.29	900	0.38	0.56	526.68	0.30	0.72	11.67	0.30	0.43	0.59
Openings for doors	1200x2050	1200	1156	2028	2344368	0.41	1200	0.50	0.55	210.57	0.29	0.29	5.98	0.29	0.42	0.30
	1500x2050	1500	1456	2028	2952768	0.51	900	0.38	0.42	198.8	0.20	0.27	5.46	0.20	0.26	0.28
	1800x2050	1800	1760	2028	3569280	0.62	600	0.25	0.29	92.01	0.12	0.13	5.09	0.12	0.09	0.26

The ratio of racking stiffness from the test results as well as those obtained by using equations (5.17) and (5.19) are shown in Table 5.7 and Figure 5.21.The trend line of the experimental stiffness (F_{Actual}) is close to the line of Dolan and Johnson (1997), showing similar linear behaviour that indicates the good harmony between the experimental result and the empirical equation of Dolan and Johnson. Therefore, only actual stiffness is close to the empirical equation of Dolan and Johnson and for actual strength, it is close to the empirical equation derived by Sugiyama and Matsumoto (1994) as shown Figure 5.22.

5.5 Conclusion

The effects of openings/discontinuities for windows and doors on racking performance of walls were examined on wall panels. The experimental works that were conducted in accordance with BS EN 594:2011 were compared with the existing design methods to EC5 (Method B and PD 6693-1:2012). From the analyses and test results described in section 5.4, the following conclusions are drawn:

- The size of opening has significant influence on racking performance. The increase in the size of opening (windows and doors) decreases racking performance. However, an anomalous case occurred when the opening size of 9% for windows showed a higher stiffness and strength values compared to wall without opening. This was considered to be as a result of addition of framing around the small opening which enhanced its performance characteristics.

- In general, the design racking resistance for the wall with openings for doors were greater than the design values obtained using EC5 Method B and PD 6693-1:2012. With openings up to 1200 x 2050, the Method B values are closer to the experiment results but it provides no racking resistance where panel sizes on either side of the opening becomes less than 600 mm.

- As the opening increases the strength and stiffness decreases.

- Comparing the experimental stiffness F, $_{Actual\ K}$ with the empirical equations derived by Yasumura and Sugiyama and Dolan and Johnson, the trend line of $F_{Actual\ K}$ indicates the similar linear behaviour as the line of Dolan and Johnson, whereas trend line F, $_{Actual\ F}$ from the experimental strength behaves similar to the strength ratio line of Sugiyama and Matsumoto (1994). Therefore, only actual stiffness is close to the empirical equation of Dolan and Johnson (1997) and for actual strength, it is close to the empirical equation derived by Sugiyama and Matsumoto (1994) respectively as shown in above Figure 5.21.

Chapter 6 Mid-ply wall (MPW)

6.1 Introduction

In timber frame construction, the standard walls are designed to support vertical loads whereas shear walls are designed to carry vertical as well as transferring in-plane lateral (racking) loads generated by wind and seismic actions. A shear wall effectively operates as a cantilever in terms of transferring the lateral load to the foundation. A key assumption of their design is that the lateral forces are evenly distributed along the length of the wall by existing member either a roof or floor diaphragm (Breyer et al., 2007). Normally, in the standard shear walls, the fastener works in single shear and after application of the racking load, the failure mode occurs with the pull through and withdrawal system, thus reducing lateral load carrying capacity of the shear wall. The new system of shear wall known as “Mid-ply wall” provides enhanced lateral load capacity by the mechanism of double shear system of the fastener to protect from nail pull through failure, in which the studs are turned by 90 degree to that of standard shear wall in order to make greater edge distance reducing tearing out of panel as well as to accommodate additional sheathing, as a result it performed better than that of standard shear wall (Ni et al., 2007).

This chapter describes a programme of development and assessment work carried out on the performance characteristics, application and use of an Enhanced wall system, originally developed in Canada as “Mid-ply wall”, for internal and external load-bearing shear walls to accommodate large openings and long spans. A total of 30 timber framed wall specimens constructed using OSB3 sheathing boards under different load configurations were tested. A comparison of the racking strength and stiffness between the enhanced Mid-ply and standard shear walls was carried out. For this purpose, the experimental study was designed to examine the effect of a range of geometrical parameters, such as fastener size and spacing, wall length, sheathing thickness, size and position of studs, as well as the effect of vertical loading on the racking strength and stiffness of the walls. The experimental results were then compared with results obtained from design rules, as given in the relevant European standards, to determine the racking performance of the walls.

6.2 Mid-ply wall

In a standard shear wall, the connections between the sheathing to the frame are the key component for shear resistance of the wall. However, the contribution of the frame members to the lateral load resistance of a wall is ignored. Basically, modern timber frame shear walls are composite systems typically using sawn timber to create a pinned frame that is combined with sheathing panels fixed by fasteners to provide bracing. The sheathing is fundamental as it prevents the frame from deforming into a parallelogram. Without an application of sheathing, the wall will heavily deform at a relatively low lateral load (Doudak, 2005). Because of the high rigidity, the sheathing deforms less than the frame; the difference between these two is resisted by dowel-type fasteners. Therefore, the rigidity of the sheathing along with the fasteners' strength and stiffness are the main contributors to wall's performance (Salenikovich, 2000). Oriented Strand board (OSB) is commonly used for sheathing, but other types of panels can also be employed such as plywood, hardboard, particleboard and fibre-based plasterboards (Premrov and Dobrilla, 2010). In standard shear wall as shown in Figure 6.1, the panels are fixed on narrow edges of the studs, so only panels can be fixed on the one or both sides and the nails work in single shear system.

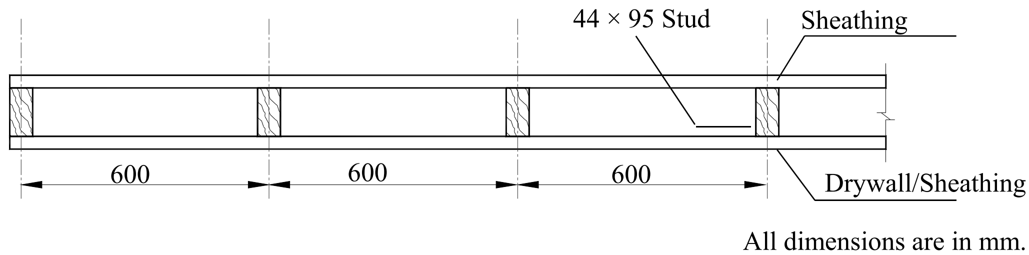


Figure 6.1 Cross-section of Standard shear wall

It is already discussed in Chapter 2 that the capacity of the shear walls reduces when the percentage of the openings for doors and windows increases. Due to the cost of land and dwelling, there is a high demand for narrow properties featuring large openings, requiring high capacities along short wall length (Griffiths et al., 2005a). In the modern times, the ranges of timber based construction products used has resulted in increased geometric irregularities of buildings, more open interiors, and numerous and larger openings which has raised concerns about the lateral resistance of timber frame buildings (Doudak et al., 2006a). These cases drive the layouts, where large lateral forces are required to be exerted on relatively small shear walls. This is achieved by either reducing the fastener spacing or the use of double sheathing and/or increasing sheathing thickness. However, these solutions are limited as these do not meet the requirements for large openings and mid-rise buildings when wind conditions are severe. This has driven the development of hybrid systems which use steel frames or reinforced concrete walls, however, these systems produce unwanted side effects such as differential shrinkage and the requirement for different materials and labour specialisms on site (Prion and Lam, 2003). Considering all these issues, a timber solution is appropriate not only to improve the performance but also to reduce the structural constraints placed upon architects.

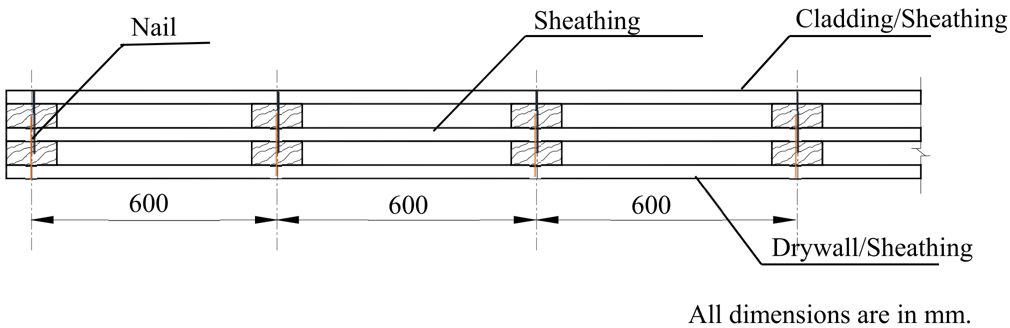


Figure 6.2 Cross section of Mid-ply wall with two exterior panels and same sized nails driven from both front and back of the walls.

In response to the above mentioned demands, the mid-ply system was introduced where its members as well as the connectors all contribute towards the development in the stiffness, lateral force resistance, and ductility of the wall. Basically, in a Mid-ply wall, as shown in Figure 6.2, the panels are fixed at the centre of the wall on the wider face of the studs which are rotated to 90 degrees (on flat) so that additional sheathing can be accommodated on both front and back of the Mid-ply and nails work on double shear system increasing the lateral load capacity. As a result, a Mid-ply shear wall has approximately twice the capacity of a regular shear wall with the same nail schedule (Pei et al., 2010). Varoglu et al.(2007) in their tests conducted for the performance of Mid-ply shear walls, concluded that the Mid-ply shear wall have dynamic load-carrying capacity of more than 2.5 times that of standard shear walls and have superior resistance against earthquake loading. Similarly, Ni et al. (2008) in their analysis for four storey building, found that Mid-ply walls have at least twice the lateral load capacity and stiffness compared to the standard shear wall with same framing members, sheathing, nail diameter and spacing.

The available literature shows that a Mid-ply shear wall provides significant improvements in stiffness, load carrying capacity and ductility in comparison with the standard shear wall. The behaviour of Mid-ply shear wall system is best illustrated in the experiment conducted for the seismic behaviour of six-storey wood frame building (The NEESWOOD building) tested on E-Defense shake table in Japan (van de Lindt et al., 2010). The Mid-ply wall contributes the alternative solutions for mid-rise timber frame construction, where standard shear walls are not adequate to provide the lateral load resistance that is required by the building (Pei et al., 2010). This new system has been integrated in the Canadian Design Code for Wood (CSA, 2014), where six-storey timber frame buildings are permitted. In Japan, a five-storey care home construction, incorporating this new system of Mid-ply, was also expected to be completed in 2015 (Hixson, 2014). While in the UK prior to this research, no investigation has been conducted to determine the performance of the mid-ply walls till date. No validated study seemed to have been conducted on the practical use of existing calculation models on the mid-ply system.

6.3 Theoretical background

6.3.1 Fasteners

Sheathing to framing fastener characteristics are the major factor for determining the racking performance of shear walls (Casagrande et al., 2016; Varoglu et al., 2006). When subjected to lateral loading, a connection formed using metal dowel fasteners may fail in a brittle or a ductile mode. To ensure that failure is in ductile rather than a brittle manner, the design rules have been developed in Table 8.2 of EC5, providing the minimum spacing, edge and end distances. (Porteous and Kermani, 2013).

i. Fasteners positions

Fasteners are positioned at the perimeter of the board and along the intermediate stud. In the sheathed timber frame shear wall with mechanical sheathing-to-framing connections, the influence of fasteners on sheathing-to-frame connections and framing joints is most important for the load carrying capacity and structural behaviour (Kallsner and Girhammar, 2009a). Kallsner and Girhammar highlighted the influence of fasteners positioned along the intermediate stud, top and bottom rails, and the leading and trailing studs on horizontal load carrying capacity and found different load carrying capacity in these positions Figure 6.3.

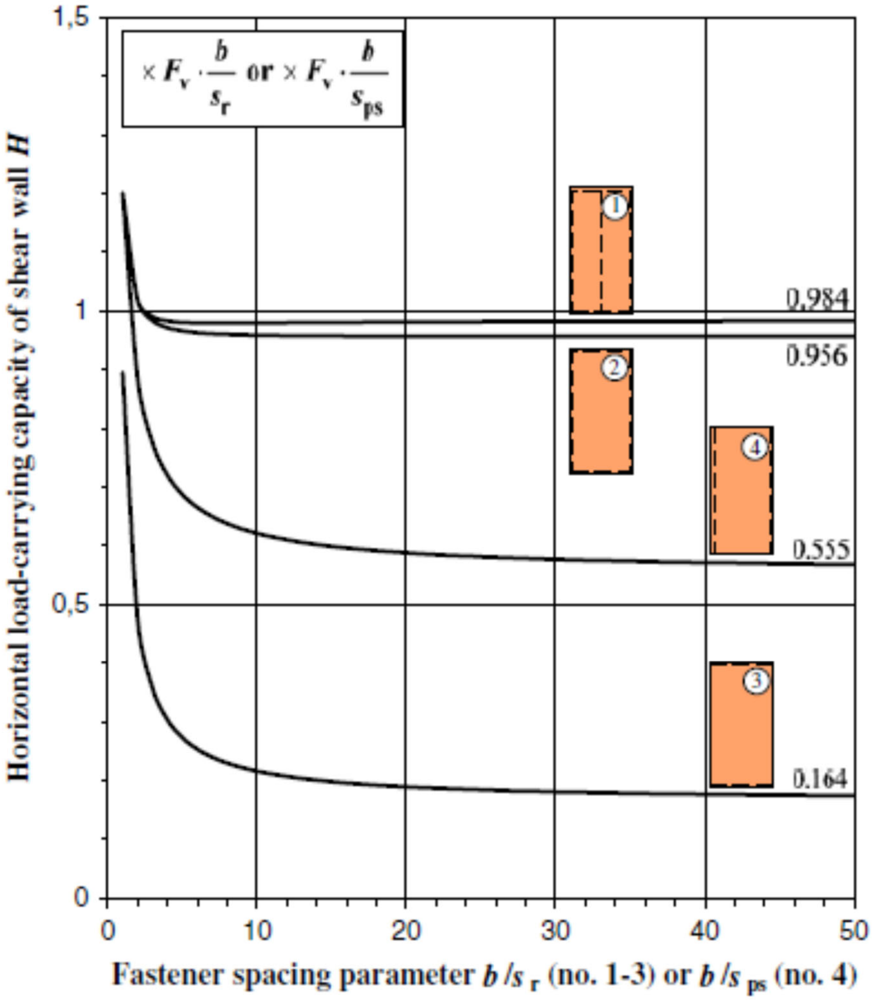


Figure 6.3 Fastener patterns in the wall

Source: Kallsner and Girhammar (2009a)

ii. Edge distance

The small edge distance lead to brittle failure of the wall with fastener pull through the sheathing. The EC5 (BSI, 2014) defined the rules for edge distance to prevent the brittle failures (Table 6.1). Basically, the fasteners are assumed to be loaded along the grain of the studs in both Method A and PD6693-1 which makes them unloaded in regards to the edge (Porteous and Kermani, 2013). However, the fasteners acting against uplift the exception to this rule as they load to the edge of the element.

Table 6.1 Minimum edge distance for fasteners according to EC5

Edge distance	Without pre-drilled holes		With pre-drilled holes
	$\rho_k \leq 420$ kg/m^3	420 kg/m$^3 < \rho_k \leq 500$ kg/m^3	
Loaded	d < 5mm: (5+2sinα) d	d < 5mm: (7+2sinα) d	d < 5mm: (3+2sinα) d
Unloaded	5d	7d	3d

Where,

α	the angle between the direction of nail force and the grain
d	nail diameter
ρ_k	the characteristics timber density in kg/m^3

The small edge distance in the sheathing panel can result the tearing out of the nails. The tests conducted by Anderson et al. (2007) on OSB did not found the tear out failure of nails at the perimeter when the minimum edge distance of at least 3⁄8 in. was used. Goodall and Gupta (2011) also confirmed that the strength and displacement at maximum load of the Gypsum Wall Board (GWB) screw connections is a function of edge distance. Increasing the strength of the GWB connection by using a larger edge distance improved GWB performance up to 1% drift, but affected the performance negatively at 2 and 3% drifts. In Mid-ply wall system, the sheathing material is fastened to the wide face of the studs provided more edge distance for fasteners on the perimeter of the sheathing panels placed in the mid plane and the exterior face of the wall, thus increasing lateral load capacity. Increased edge distance reduces the possibility of nail tear out failures (Ni et al., 2007). The tests conducted by Zheng et al (2015) on double shear nail connections in Mid-ply shear walls with OSB/3 sheathing indicated that the ultimate strength and ductility of the specimens were enhanced significantly with the increase in nail edge distance. Their study also showed that increasing the nail edge distance exhibited little influence on the initial stiffness of double shear nail connections with the same sheathing thickness and loading direction.

iii. Fastener strength

The connections can be formed with fasteners in single or double shear. The single shear has one shear plane per fastener and double shear has two shear planes per fastener (Figure 6.4)

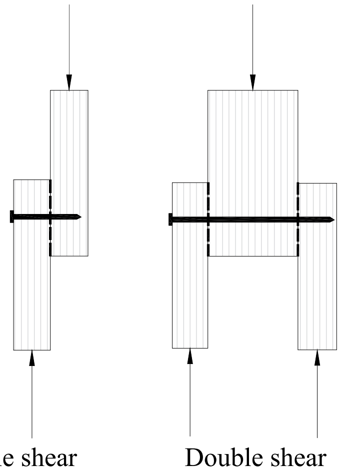


Figure 6.4 Fasteners loaded laterally in single and double shear

Source: Porteous and Kermani (2013)

The embedment strength f_{hk} provides the compressive strength of the timber under the action of a stiff straight dowel loaded as shown in Figure 6.5 (Porteous and Kermani, 2013). The strength varies depending on the diameter of nail, types of material, and whether or not predrilling is adopted.

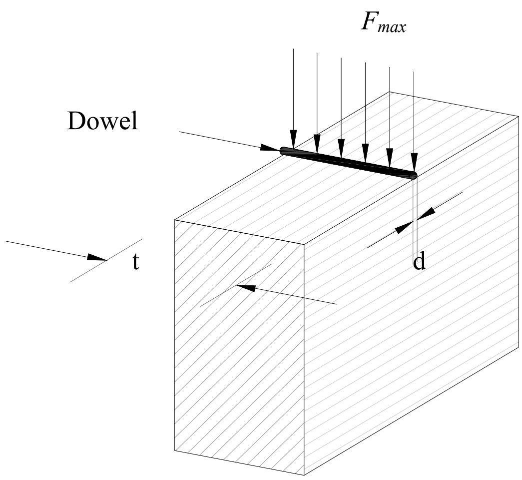


Figure 6.5 Embedment strength

Source: Porteous and Kermani (2013)

The embedment strength is calculated as,

$$f_h = \frac{F_{max}}{d.t} \tag{6.1}$$

where,

f_h	average compressive strength
F_{max}	maximum load
d	nail diameter
t	thickness of material

Characteristic embedment is determined by EC5 Clause 8.3.1.1., equation 8.15 as,

Without pre-drilled $f_{h,k} = 0.082\,\rho_k\, d^{-0.3}$ (6.2)

With pre-drilled $f_{h,k} = 0.082\,(1-0.1d)\,\rho_k$ (6.3)

where,

ρ_k	characteristic density of material
d	diameter of fastener,
$f_{h,k}$	characteristic embedment strength which is the product of fastener penetration length and diameter

The combined friction forces and withdrawal strength referred to as rope effect (Figure 6.6) are distinguished from Johansen yield load (Porteous and Kermani, 2013). However, in EC5 reference is only made to the term $F_{ax,\ RK}/4$ as the contribution from this effect. The magnitude of the rope effect is a function of the angle of the fastener's rotation and affected by the fastener's resistance to pulling out or through the material. Material density and the nail head diameter plays a role of resisting against pull through whereas material's density, diameter of the fastener, profile of the nail and distance of point side penetration plays resistance against pull out or withdrawal.

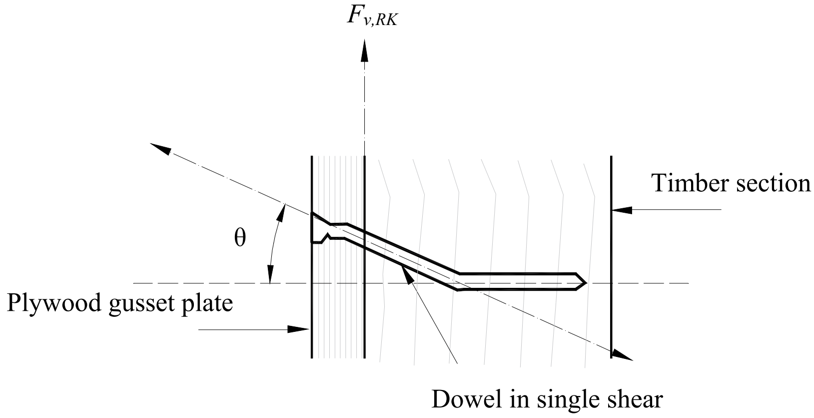


Figure 6.6 Rope effect

Source: Porteous and Kermani (2013)

According to EC5 Clause 8.3.2 (7), the point-side penetration length (length of the threaded part in the point-side member) t_{pen} for a smooth nails should be at least 8d. The nails with a point-side penetration smaller than 12d, withdrawal capacity should be multiplied by,

$$\frac{t_{pen}}{4d} - 2 \tag{6.4}$$

For threaded nails, the point-side penetration should be at least 6d. For the point of penetration smaller than 8d, the withdrawal capacity should be multiplied by,

$$\frac{t_{pen}}{2d} - 3 \tag{6.5}$$

The Enhanced mid-ply system requires longer nails to provide sufficient point-side penetration because of its double shear system (Figure 6.7). The point-side penetration defines the embedment lengths within double shear. Since the nail tip is entirely embedded into this length, it provides non-conservatism.

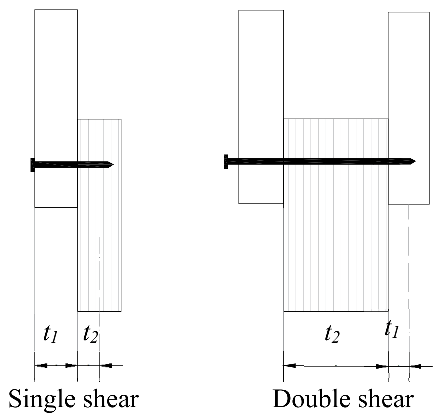
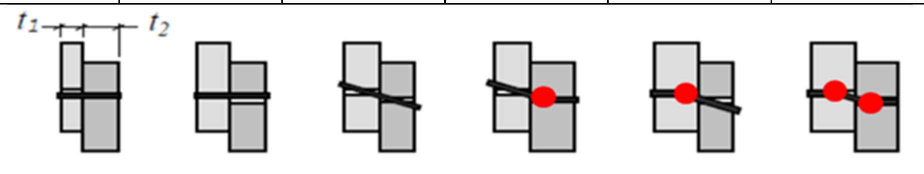


Figure 6.7 Embedment lengths
Source: BSI (2014)

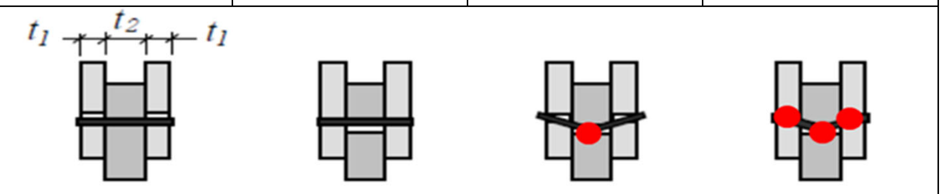
iv. Failure modes of fasteners

Johansen (1949) derived the strength equations for connections formed using metal dowel-type fasteners in timber. When using such fasteners, the possible failure modes that can arise in timber-to-timber and wood panel to timber connections. Johansen's work is also referred to as the European Yield Model (EYM) which defines different modes of failures.

Table 6.2 Characteristic load carrying capacity per fastener per shear plane based on EC5

For fastener connections in single shear						
Failure modes	a	b	c	d	e	f
						
EYM mode type	1	1	1	2	2	3

Characteristic load-carrying capacity per fastener			
	$F_{v,RK}=f_{h,1,k}\cdot t_1\cdot d$	mode (a)	(6.6)
	$F_{v,RK}=f_{h,2,k}\cdot t_2\cdot d$	mode (b)	(6.7)
	$F_{v,RK}=\frac{f_{h,1,k}\cdot t_1\cdot d}{1+\beta}\left[\sqrt{\beta+2\beta^2\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\left(\frac{t_2}{t_1}\right)^2}-\beta\left(1+\frac{t_2}{t_1}\right)\right]+\frac{F_{ax,Rk}}{4}$	mode (c)	(6.8)
	$F_{v,RK}=1.05\frac{f_{h,1,k}\cdot t_1\cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}\cdot t_1^2\cdot d}}-\beta\right]+\frac{F_{ax,Rk}}{4}$	mode (d)	(6.9)
	$F_{v,RK}=1.05\frac{f_{h,1,k}\cdot t_2\cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k}\cdot t_2^2\cdot d}}-\beta\right]+\frac{F_{ax,Rk}}{4}$	mode (e)	(6.10)
	$F_{v,RK}=1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}\cdot f_{h,1,k}\cdot d}+\frac{F_{ax,Rk}}{4}$	mode (f)	(6.11)

For fastener connections in double shear				
Failure modes	g	h	j	k
				
EYM mode type	1	1	2	3

Characteristic load-carrying capacity per fastener			
	$F_{v,RK}=f_{h,1,k}\cdot t_1\cdot d$	mode (g)	(6.12)
	$F_{v,RK}=0.5\,f_{h,2,k}\cdot t_2\cdot d$	mode (h)	(6.13)
	$F_{v,RK}=1.05\frac{f_{h,1,k}\cdot t_1\cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}\cdot t_1^2\cdot d}}-\beta\right]+\frac{F_{ax,Rk}}{4}$	mode (j)	(6.14)
	$F_{v,RK}=1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}\cdot f_{h,1,k}\cdot d}+\frac{F_{ax,Rk}}{4}$	mode (k)	(6.15)

Mode type 1 is where failure is solely by embedment of the connection material and there is no yielding of the fastener; mode type 2 is where failure is by a combination of embedment failure in the materials and a single yield failure in the fastener and mode type 3 is where there is a combination of embedment failure and double yield failure in the fastener. The connection strength equations are dependent on the geometry of the connection, the embedment strength of the timber or wood based material, the bending strength of the fastener and on the basis that the fastener will not withdraw from the connection (Porteous and Kermani, 2013). For connections in single and double shear, the characteristics load carrying capacity per shear plane per fastener $F_{v,Rk}$ is the minimum value equation as shown in Table 6.2. It is to be noted that the equation given for double shear connections only apply to symmetrical assemblies.

The entire wall with the framing, sheathing, and nail connectors all interacting closely is a highly redundant structure. As a result, the wall is not fully governed by the failure of a single connection, i.e. the failure of a single fastener will result in the load being redistributed around the remaining fasteners (Prion and Lam, 2003). The failure strength of the perimeter fastener used in wall panel, $F_{v,RK}$ is derived in accordance with the relevant Johansen strength equation in EC5 (eqs (6.6 -(6.15). $F_{v,RK}$ is multiplied by a factor of 1.2, a statistical factor used to convert a characteristic strength value to a mean strength value. The design value, $F_{f,Rd}$ is taken as,

$$F_{f,Rd} = \frac{K_{mod} \cdot \left(1.2\, F_{v,Rk}\right)}{\gamma_M} \tag{6.16}$$

where,

K_{mod}	modification factor for load duration. Service classes is given in Table 3.1 in EC5. EN 300: OSB/3 - service class: 1; load duration class - instantaneous action: 1.10
$F_{v,Rk}$	characteristic lateral load carrying capacity (also refer Table 6.2)
γ_M	partial coefficient for material properties, given in Table NA 3 in the UKNA to EC5. For OSB, $\gamma_M = 1.2$

The design lateral load carrying capacity of wall panel, $F_{i,vRd}$ is obtained from the following relationship (EC5 equation 9.21),

$$F_{i,v,Rd} = \frac{F_{f,Rd} b_i c_i}{s} \tag{6.17}$$

where,

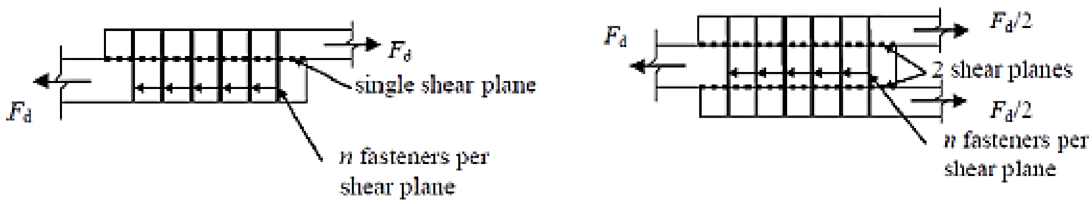
b_i	length of wall panel
s	fastener spacing around the perimeter
$F_{f,Rd}$	lateral design capacity of individual fastener
c_i	modification factor that reduce the strength of panel when its length is less than h/2, where h is the height of the wall panel

The value of c_i is obtained from (EC5 equation 9.22),

$$c_i = \begin{Bmatrix} 1 \; for \; b_i \geq \frac{h}{2} \\ \frac{2b_i}{h} \; for \; b_i < \frac{h}{2} \end{Bmatrix}$$

v. Fastener stiffness

Timber has a relatively low stiffness to strength ratio (Porteous and Kermani, 2013). The stiffness of the fastener is defined as the ratio of its lateral load per shear plane divided by its slip. The stiffness in EC5 is referred to as slip modulus, which has been derived from Type 3 failure mode that has both fastener yielding and embedment failure. By adopting this type of most common failure mode, joint strength can be evaluated (Porteous and Kermani, 2013). No clear guidance is given in EC5 on the value to be used to determine the stiffness of a connection and, irrespective of the angle of load relative to the grain, for single and double shear connection the actual number of fasteners should be used. The connection stiffness for single and double shear configurations are given in Figure 6.8.



(a) Single shear

(b) Double shear

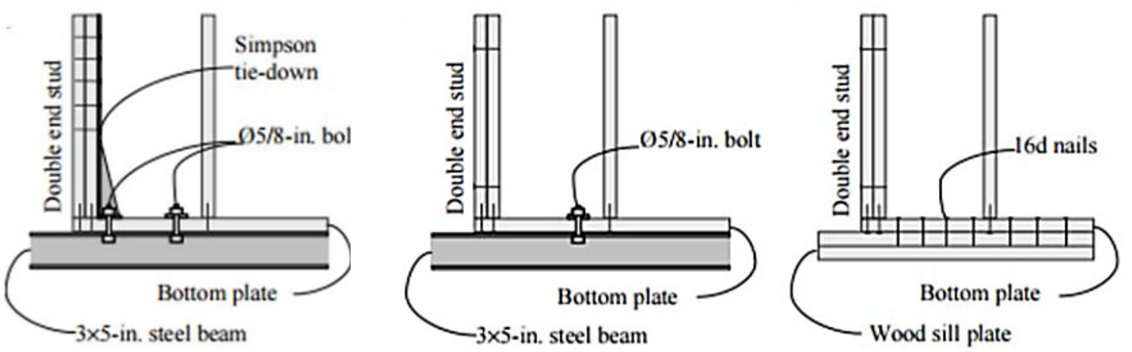
Figure 6.8 Stiffness of single and double shear connections

Source: Porteous and Kermani (2013)

In a series of test conducted by Germano et al. (2015) using smooth nails and ring-shank nails, they found that ring-shank nails generally perform 1.75 times higher strength than equivalent diameter of the smooth nails due to an increase withdrawal capacity. They also found a clear reduction in stiffness and highlighted the need for further research in to this area.

6.3.2 Anchorage

In the UK, partial anchorage is used for determining the overturning forces, whereas full anchorage is used in the Canadian systems. Different options of anchorage including the nailing method used in the UK are shown below in Figure 6.9.



(a) Full anchorage (b) Partial anchorage

Figure 6.9 Anchorage options

Source: Salenikovich (2000)

The partial anchorage has an impact of an extra stress that is placed on the fasteners, connecting the bottom rail to the substrate while transferring overturning forces (Kallsner and Girhammar, 2009b). The brittle failure occurs due to the bending stresses generated at the bottom rails (Figure 6.10). With partial anchorage, the mid-ply system's higher capacity will be intensified.

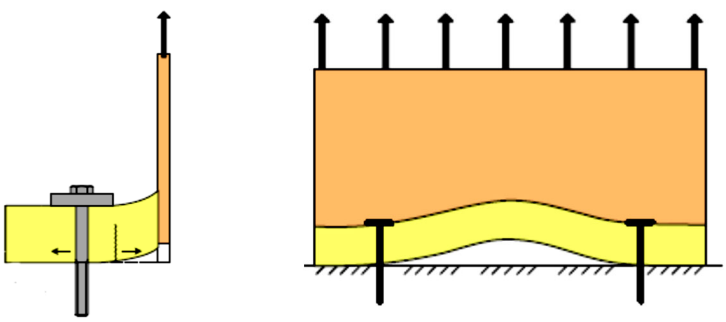


Figure 6.10 Bending failure of bottom rail

Source: Kallsner and Girhammar (2009b)

The partial anchorage reduces the number of fastener acting against the lateral forces resulting the reduction of the capacity. The common failure mode caused by partial anchorage is uplifting of the leading stud as shown in Figure 6.11 which then ultimately separates the sheathing and studs from the bottom rail as well as causes the rotation of the sheathing, resulting weak stiffness levels (Salenikovich, 2000). Because of the increased lateral load capacity, one of the key design parameters for the Mid-ply wall is to prevent hold-down failure under dynamic loading (Pei et al., 2010).

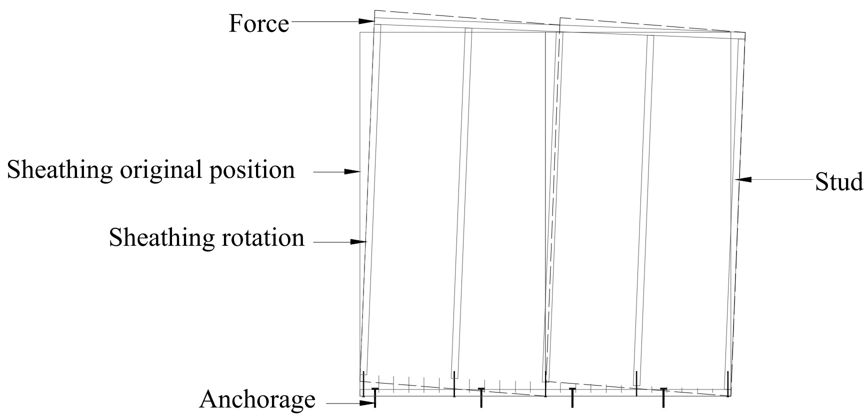


Figure 6.11 Partial anchorage failure

6.3.3 Sheathing thickness, buckling and gap between the boards

The application of sheathing in the wall need to comply with BS EN 13986 (BSI, 2015). There is similar behaviour between the application of Oriented Strand Boards (OSB) and plywood when using them as sheathing in shear walls. According to Jang (2002), there is only little difference in their performance. OSB originally developed to replace the lower grades of plywood due to its economical solution (Premrov and Dobrilla, 2010). There is not specific strength distinction between these materials in the UK design method PD 6693-1 (Vessby et al., 2010b). The buckling in the sheathing is affected by thickness of sheathing. The thin sheathing causes buckling due to the compression stresses exerted in the sheathing.

The calculation to determine the critical buckling stress is presented by Kallsner and Girhammar (2009c). According to EC5 (BSI, 2009c) , the shear buckling of the panel may be disregarded provided that,

$$\frac{\text{distance between studs}}{\text{sheathing thickness}} \leq 100$$

This shows the significance of the intermediate stud fasteners. The sheathing buckling was a common failure in the testing schedule of Leitch and Hairstans (2010), using 9mm OSB. The 3mm expansion gap between the sheathing panels, which is common practice in the UK, is provided to allow the shear movement of the panels as well as permit the expansion of sheathing in case of increased moisture levels without buckling. In standard shear wall, there is possibility of separation of sheathing due to buckling caused by outside position of the frame, but in mid-ply shear wall, this is protected as the sheathing lies in between the frame.

The investigation conducted by Vessby et al. (2010a) to find whether the gap reduces the strength capacity or not, concluded that the insignificant influence of contact caused by the gap signifies that the forces were transmitted via the sheathing-to-framing fasteners along the top rail and to some extent also via the sheathing-to-framing fasteners along the upper parts of the vertical studs joining the different sheets.

6.4 Method

This research focuses on optimisations of the performance of Mid-ply shear walls by testing full-scale walls with different geometrical configurations considering nail spacing, different stud sizes, and different panel thickness. It then compares the results of Mid-ply shear walls with those of the standard shear walls having similar configurations. The aim was to assess a range of configurations for developing an optimal solution for design and construction of high-performing timber wall systems. A series of pre-determined geometrically configured walls of 2.4 m × 2.4 m in size, comprising 9 mm and or 11 mm OSB/3 sheathing, studs/timber sections of 38 × 89, 44 × 95, and 45 × 45 mm, approx. 3.0 mm diameter round wire/ring-shanked nails (driven by hand or fired by nail-gun), nail spacing/patterns of 150/300 and 100/200 mm were tested under partial anchorage system. The smooth round wire nails of sizes 3.0 × 60 mm and 3.35 × 65 mm were hand driven, whereas 3.1 × 75 mm ring-shanked nails and 3.1 × 90 mm smooth nails were fired by a nail-gun as shown in Figure 6.13 (a). Typical types of smooth nails and ring shank nails are shown in Figure 6.12.

(a) Smooth nails (b) Ring shank nails

Figure 6.12 Types of nails

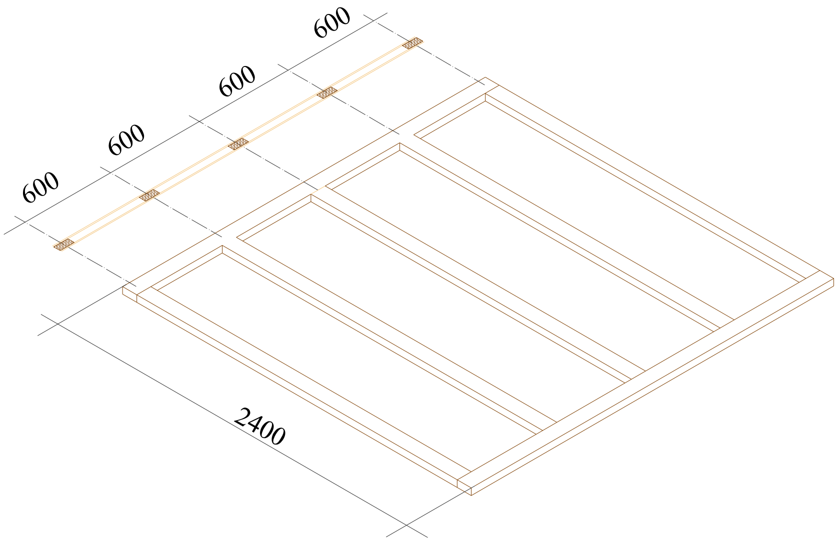
The difference between nails driven by hand and nail gun (Fig 6.13 (a)) is the easy removal of nails if there is miss shot in case of hand driven and difficult removal of nails in case of nail gun. The possibility of miss firing of nail gun occurs if there is any presence of knots in the timber. The presence of knots in the timber prevented the nails from full penetration causing the nails to bend out of the alignment. This type of the problem occurred around 5% in each wall; these could be avoided and corrected where possible. However, it was difficult to know the straightness of the nail once they are driven which might reduce the edge distance and spacing of the nails as shown in Figure 6.13 (b). This type of the problem also occurred in the commercial fabrication.

(a) Nail gun (b) Bend out of ring shank nail using nail gun

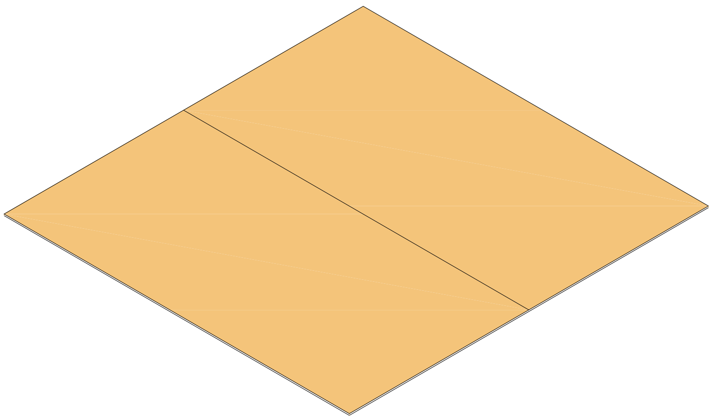
Figure 6.13 Nail gun and its impact on timber when knots are present in the timber

Smart-ply 2400 x 1200 mm OSB/3 panels to BS EN 13986 (BSI, 2015) were used for sheathing as per industry practice. The choice of Grade 3 panels is due to its suitability for external walls in humid conditions. The Mid-ply shear walls are constructed in different stages as shown in **Figure 6.14** (see also Appendix 6.3 and 6.4). The Mid-ply shear walls were benchmarked against the Standard wall. These included: standard walls (SW) with nailing density/patterns of 150/300, 100/200 and/or 50/100 as datum for benchmarking and enhanced (Mid-ply) walls (Design 1, D1) (Figure 6.15) with nailing density/patterns of 150/300 and 100/200.

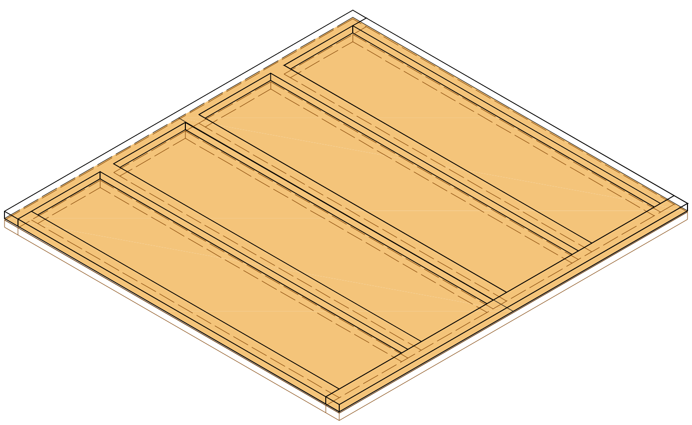
Step 1: Assembling frame - side 1



Step 2: Placing sheathing at top



Step 3: Assembling Frame - side 2



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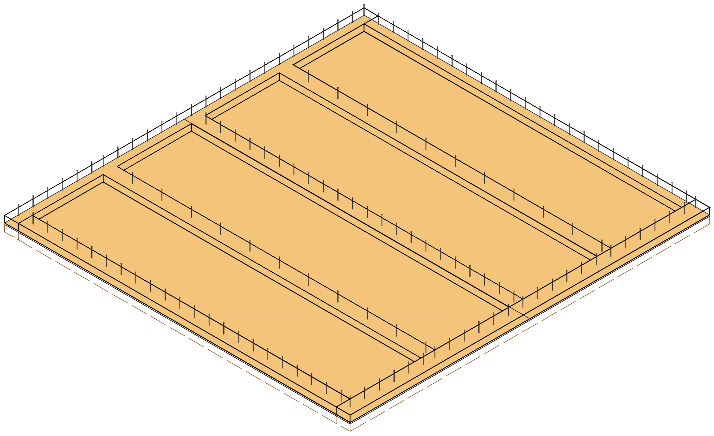
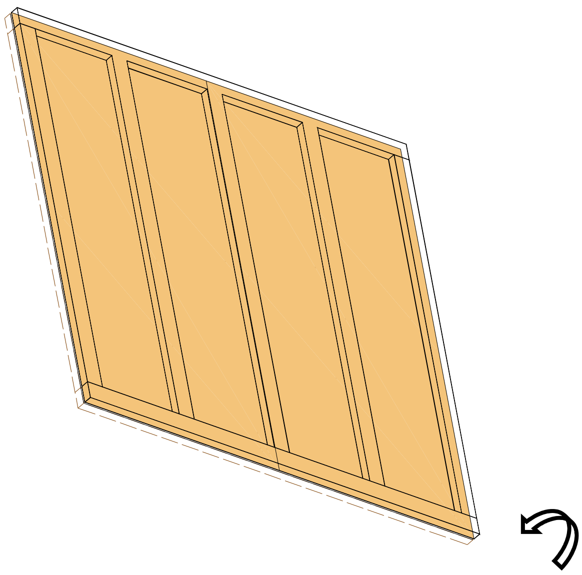
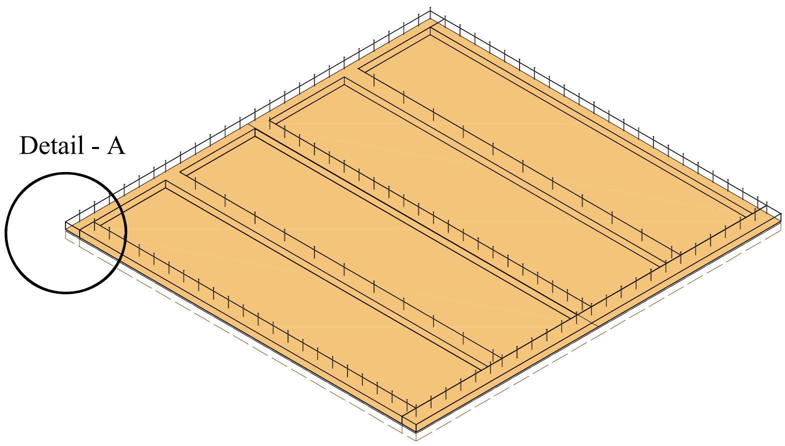
Step 4: Nailing side 2



Step 5: Rotating frame – side 1



Step 6: Nailing side 1



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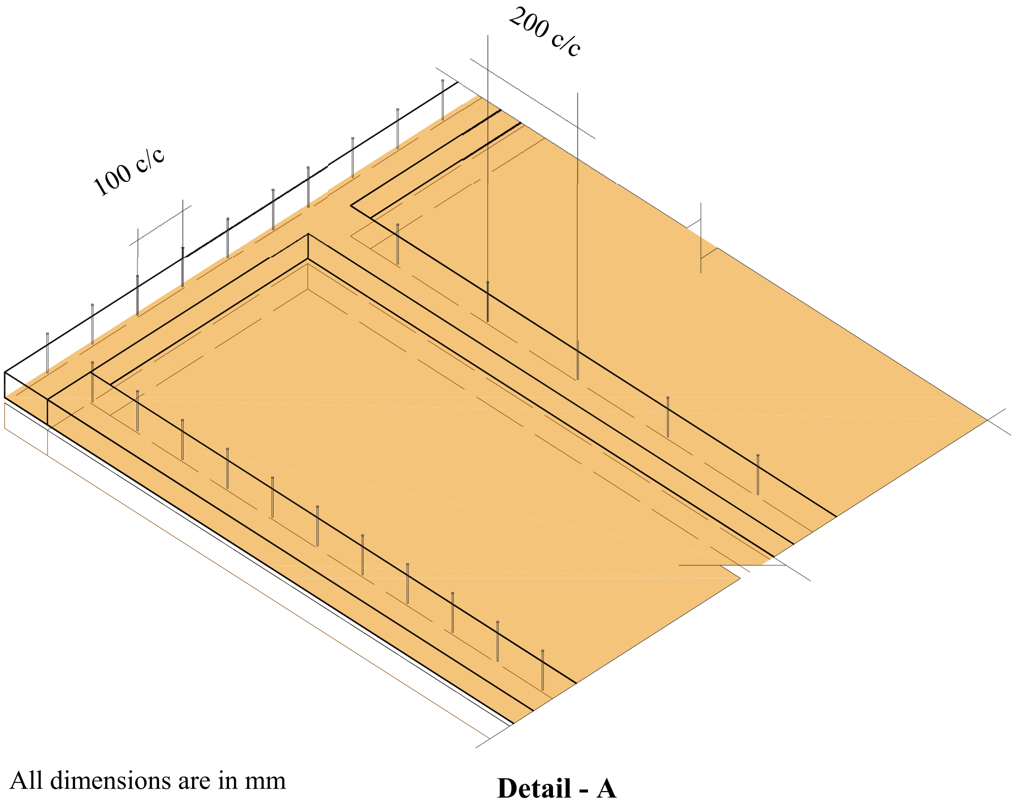
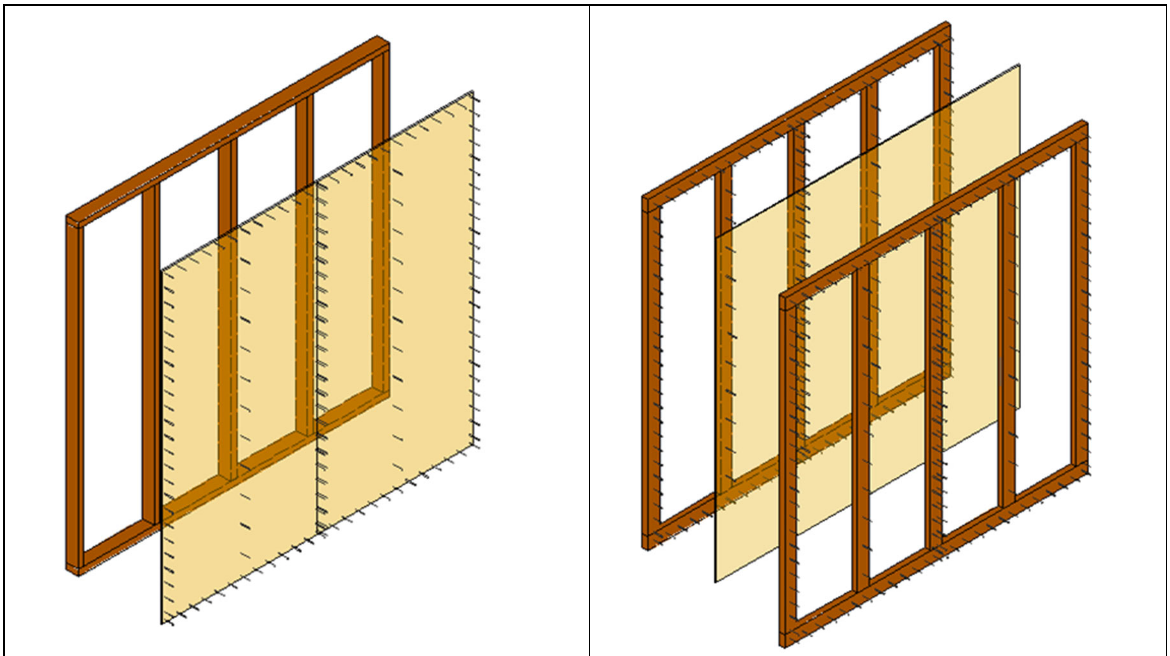


Figure 6.14 Mid-ply walls construction



Wall types	(a) Standard walls (SW)	(b) Mid-ply walls (Design 1, D1)
Nailing patterns:	150/300, 100/200, 50/100	150/300 and 100/200.

Figure 6.15 Nailing patterns of shear walls

6.4.1 Test programme

The experimental work broadly followed the test procedure and recommendations of BS EN 594:2011 (see Figure 6.16) and were concerned with the variables such as: fastener type; fastener spacing; stud and rail sizes; OSB thickness; and openings for doors. The wall panels were tested in upright position. The bottom rail was fixed down to the test bed using four pairs of M12 x 150 mm long bolts. To prevent lateral distortion of the wall during testing, lateral restraints were provided by means of two pairs of rollers at the top plate (header level) which permitted free in-plane movement of the wall both in vertical and horizontal directions. The loads were applied using two separate loading systems: (i) the vertical loading (where appropriate) was applied via pressurised air-bag. A stabilising UDL vertical load of 25 kN in total was applied via the air bag to the head binder at the stud positions and maintained for 120 sec. The load was then removed and the panel was allowed to recover for a period of 600 sec before the strength test was carried out. For walls under vertical loading, a constant UDL vertical load of 25kN was applied via the air bag to the head binder at the stud positions and maintained throughout the racking test and (ii) the racking load was applied via a horizontal jack connected to an automatic /computerised loading and data-acquisition system which followed a pre-programmed loading procedure based on BS EN 594:2011. This load was applied at a steady rate in which 90% of the maximum load was reached within 300 $\pm$ 120 sec. Displacement transducers were used to record the horizontal displacements of the walls at the leeward base and the header levels and the vertical uplift of the lead stud as well as the vertical movement of the sole plate at the loaded side of the wall. The calculation of lateral deformation, v was carried out as the difference between the horizontal displacement of the header beam (LVDT-1) and the rigid body horizontal translation of the wall (LVDT-2). According to (Doudak et al., 2006a), this calculation process for the deformation helps in correction and remove the rigid body translation caused by slip in the fasteners, connecting the rails to the substrate.

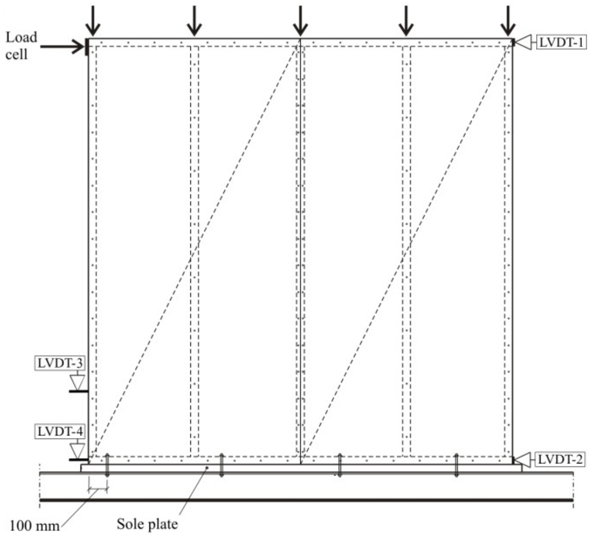
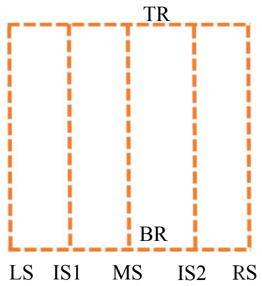


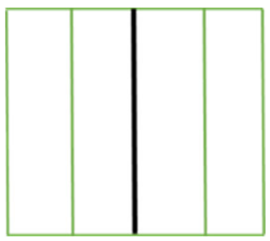
Figure 6.16 Typical set up of racking wall in accordance with BS EN 594:2011

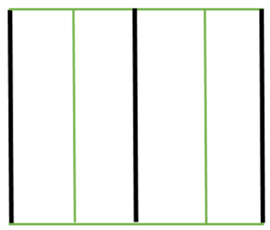
6.4.2 Test series

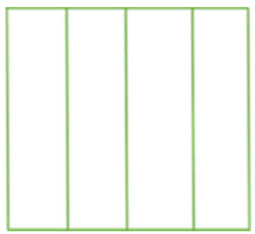
A total of 30 wall tests were conducted. The numbers of Mid-ply walls were grouped in 12 different categories based on the use of studs of size and position of the wall panel as shown in Table 6.3.

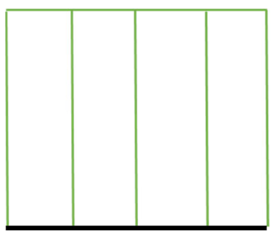
Table 6.3 Test programme for Enhanced Mid-ply shear walls

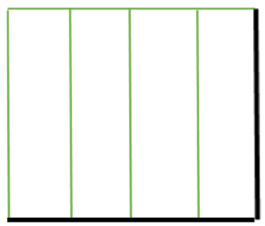
Wall type: Type 1: Wall numbers - 1, 2, 7, 8		
	Stud section:	LS/IS1/MS/IS2/RS = 38 × 89
	Rail	TR/BR = 38 × 89
	Nail spacing:	150/100
	Nail size:	3.35 × 65
	OSB thickness:	11

Wall type: Type 2, Wall numbers - 3, 4, 9		
	Stud section:	LS/IS1/ IS2/RS = 45 × 45 MS/ = 44 × 95
	Rail	TR/BR = 45 × 45
	Nail spacing:	150/100
	Nail size:	3.35 × 65 3.1 × 75
	OSB thickness:	11

Wall type: Type 3, Wall numbers - 5, 6, 12		
	Stud section:	LS/ MS/ RS = 44 × 95 IS1/ IS2 = 45 × 45
	Rail	TR/BR = 45 × 45
	Nail spacing:	150/100
	Nail size:	3.35 × 65
	OSB thickness:	11

Wall type: Type 4, Wall numbers - 10, 11		
	Stud section:	LS/IS1/MS/IS2/RS = 45 × 45
	Rail	TR/BR = 45 × 45
	Nail spacing:	150/100
	Nail size:	3.1 × 75
	OSB thickness:	9

Wall type: Type 5, Wall numbers - 13, 14		
	Stud section:	LS/IS1/MS/IS2/RS = 45 × 45
	Rail	TR = 45 × 45 BR = 44 × 95
	Nail spacing:	150/100
	Nail size:	3.1 × 75
	OSB thickness:	11

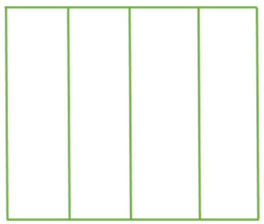
Wall type: Type 6, Wall numbers - 15, 16		
	Stud section:	LS/IS1/ MS/IS2 = 45 × 45 RS = 44× 95
	Rail	TR = 45 × 45 BR = 44 × 95
	Nail spacing:	150/100
	Nail size:	3.1 × 75
	OSB thickness:	11

All dimensions are in mm

LS= Lead stud, IS = Intermediate stud, MS = Middle stud, RS = Rear stud, TR = Top rail, BR = Bottom rail

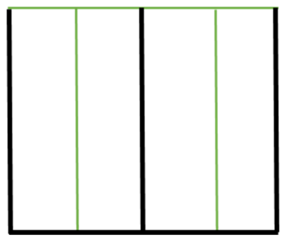
Colour code for stud/rail size: orange = 38 × 89 mm; green = 45 × 45 mm; black = 44 × 95 mm

Wall type: Type 7, Wall numbers - 17, 18



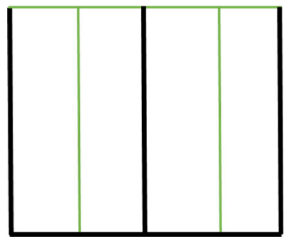
Stud section:	LS/IS1/MS/IS2/RS = 45 × 45
Rail	TR/BR = 45 × 45
Nail spacing:	150/100
Nail size:	3.1 × 75
OSB thickness:	11

Wall type: Type 8: Wall numbers - 19, 20, 27, 28



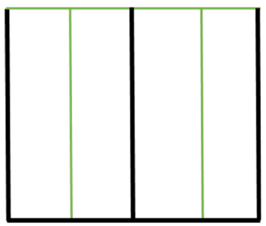
Stud section:	LS/ MS/ RS = 44 × 95 IS1/ IS2 = 45 × 45
Rail	TR = 45 × 45 BR = 44 × 95
Nail spacing:	150/100
Nail size:	3.1 × 75
OSB thickness:	11

Wall type: Type 9: Wall numbers - 21, 22



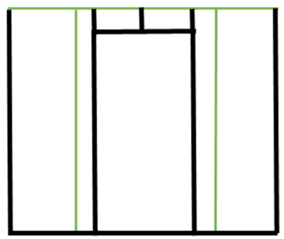
Stud section:	LS/ MS/ RS = 44 × 95 IS1/ IS2 = 45 × 45
Rail	TR = 45 × 45 BR = 44 × 95
Nail spacing:	150/100
Nail size:	3.1 × 90
OSB thickness:	11

Wall type: Type 10, Wall numbers - 23, 24, 29,30



Stud section:	LS/ MS/ RS = 44 × 95 IS1/ IS2 = 45 × 45
Rail	TR = 45 × 45 BR = 44 × 95
Nail spacing:	150/100
Nail size:	3.1 × 75
OSB thickness:	11

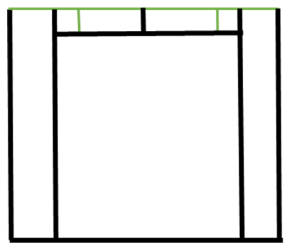
Wall type: Type 11, Wall numbers - 25



Wall frame with opening for door size 900 × 2050

Stud section:	LS/ MS/ RS = 44×95 IS1/ IS2 = 45 × 45
Rail	TR = 45 × 45 BR = 44 × 95
Nail spacing:	100
Nail size:	3.1 × 75
OSB thickness:	11

Wall type: Type 12, Wall numbers - 26



Wall frame with opening for door size: 1500 × 2050

Stud section:	LS/IS1/ MS/IS2 = 44 × 95 RS = 45× 45
Rail	TR = 45 × 45 BR = 44 × 95
Nail spacing:	100
Nail size:	3.1 × 75
OSB thickness:	11

All dimensions are in mm

Colour code for stud/rail size: orange = 38 × 89 mm; green = 45 × 45 mm; black = 44 × 95 mm

6.4.3 Moisture contents and density

The density and moisture contents were measured from samples of the timber and OSB sheathing material used for the wall racking tests in accordance with BS EN 322 and BS EN 323 respectively. These were recorded as shown in Table 6.4 below,

Table 6.4 Moisture content and density values from tested walls

Material	**Average density [kg/m³]**	**Average moisture content [%]**
9/11mm OSB/3	573 (548, 613)	7.3%
C16, 45 × 45 mm timber	406 (3690, 418)	13.6%
C16, 45 × 95 mm timber	397 (350, 412)	14.4%
C16, 38 × 89 mm timber	393 (376, 421	12.3%

6.4.4 BS EN 594:2011 method overview

As defined in BS EN 594 Clause 6.4.3, the maximum load, F_{max}, reached when either the panel collapses or the panel attains lateral deformation of 100 mm. The Stiffness, R (N/mm) is determined by taking a secant modulus within the elastic range which is in the range of 20% and 40% of the maximum load.

$$R=\frac{F_4 - F_2}{v_4 - v_2} \tag{6.18}$$

where,

F_2 racking load of 0.2 F_{max} in Newtons

F_4 racking load of 0.4 F_{max} in Newtons

v_2 and v_4 deformation at 0.2 F_{max} and 0.4 F_{max} respectively in millimetres

The rate of loading applied, as stated on BS EN 594 Clause 6.4.1, should ensure that 90% of F_{max} (F_{90}) should be reached within (300 ± 120) seconds, with a recommended mean time to F_{90} of 300 seconds.

6.4.5 Fastener strength

Double shear connection tests, considering the variables such as loading angle, framing thickness, nail profile and length, were conducted to determine the sheathing-to-framing fastener strength and stiffness. A set of joint tests for stud sizes of 38 × 89 mm and 44 × 95 mm using loads parallel and perpendicular to the grains were conducted using two types of nails, 3.1 × 75 mm ring-shank nails and 3.35 × 65 mm smooth nails (Table

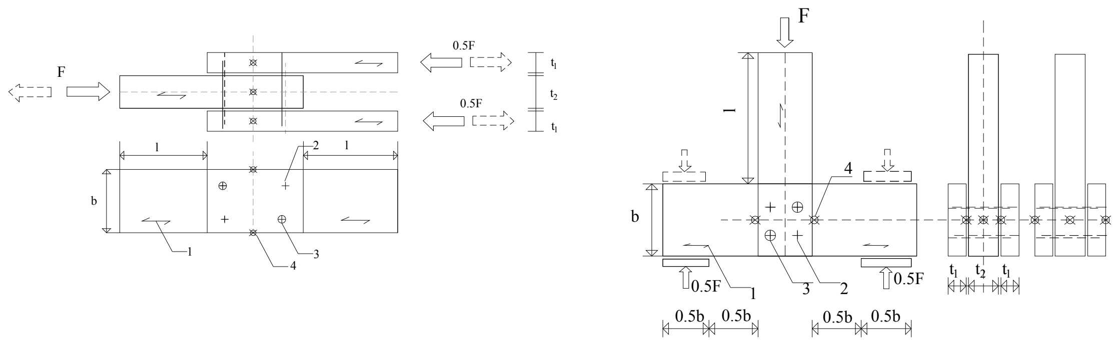
6.5). The tests were configured in accordance with BS EN 1380:2009 and shown in Figure 6.17.

Table 6.5 Joint test schedule

Loading angle of the grain	**Stud size**			
	38 × 89 mm studs		**44 × 95 mm studs**	
Parallel	3.1 × 75 mm ring-shank	3.35 × 65 mm smooth	3.1 × 75 mm ring-shank	3.35 × 65 mm smooth
Perpendicular	3.1 × 75 mm ring-shank	3.35 × 65 mm smooth	3.1 × 75 mm ring-shank	3.35 × 65 mm smooth

The conducted tests show the embedment strength and withdrawal capacity of both sets of nails. These tests also check the calculations of strength and stiffness of the fasteners in accordance with EC5 clarifying whether the strength is conservative and stiffness is non conservative.

Parallel Perpendicular



Note:

0 grain direction

1 not protruding end

2 protruding end

3 displacement measurement point

4 l free length

5 t_1 side member width

6 t_2 middle member

Figure 6.17 Joint test arrangements

Source: BSI (2009b)

6.4.6 BS EN 26891:1991 overview for deformation characteristics of fastener joints

The maximum load, F_{max} is defined in BS EN 26891 (1991) Clause 8.2 at either joint failure or joint slip of 15 mm. To determine the stiffness, initially two specimens were tested to obtain an estimated maximum load, F_{est}. The loading pattern was employed in accordance with the standard. The load should be applied up to 0.4 F_{est} and maintained for 30 s. The load should then be lowered to 0.1 F_{est} and maintained for 30 s. Thereafter the load should be increased until the ultimate load or slip of 15 mm is reached. The purpose of maintaining the load constant for 30 s is to allow adequate time for the loading to be reversed. The acquired loading profile provides two phases; the first phase is for the stiffness and the second is for the strength. The loading rate below 0.7 F_{est} should be within 0.2 F_{est} ±25 percent per minute and for above 0.7 F_{est}, a constant rate should be used until the slip of 15 mm is reached in 3 to 5 minutes. The total duration of test is about 10 to 15 minutes.

The slip modulus, k_s, defined in Clause 8.5 is given as,

$$K_s = 0.4\frac{F_{est}}{v_{i,mod}} \tag{6.19}$$

$$v_{i,mod} = \frac{4}{3}\,(v_{04} - v_{01}) \tag{6.20}$$

where,

F_{est}	estimated maximum loads
v_{04}	slip at 40 % of estimated maximum load
v_{01}	slip at 10 % of estimated maximum load
$v_{i,mod}$	modified initial slip

Slip, v, is determined by taking an average from two displacement transducers fixed on sheathing minus the average reading taken from two transducers fixed on the timber to eliminate deformation of the timber due to loading. K_s is equivalent to EC5's calculated values of K_{ser}.

6.4.7 BS EN 14358:2006 for characteristic strengths

Characteristic 5-percentile values were determined in accordance with BS EN 14358:2006 (Wood Panel Industries Federation, 2015) which requires a minimum of three samples.

According to which,

$$m_k = \exp\left(\bar{y} - K_s S_y\right) \tag{6.21}$$

where,

y	mean value
m_k	the characteristic value
m_i	the test value
n	the number of test values
S_y	is the standard deviation

$$\bar{y} = \frac{1}{n}\sum_{i=1}^{n} ln\, m_i \tag{6.22}$$

$$S_y = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(ln\, m_i - \bar{y})^2} \tag{6.23}$$

K_s is taken as 3.15 as per BS EN 14358:2006.

The joint test were conducted using a Schenk Trebel Instron 5500 loading machine fixing the load cell and the transducers as shown in the Figure 6.18 below.

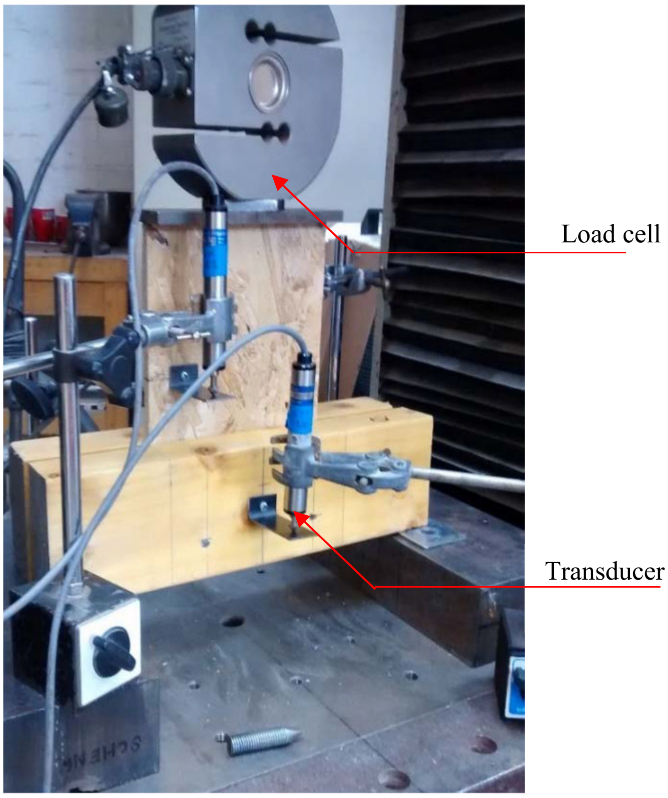


Figure 6.18 Typical Joint test

6.5 Results, analysis, and discussion

The results of the tests conducted on the wall systems indicating ultimate racking loads and stiffness values calculated according to eq.(6.18). The information on wall components and configurations used for the Mid-ply (D1) and standard (SW) shear walls are reported in Appendix 6.1 and Table 6.6 respectively. The Test wall no. 0-1 to 0-10 refers to standard racking walls (OSB/3 sheathing to one side) with no opening and Test wall no. 0-11 and 0-12 refer to standard racking walls (OSB/3 sheathing to one side) with an opening for door. 0-1 Datum test is calculated by averaging the 5 wall tests (with OSB/3 boards), 0-1 to 0-4 test results from Phases 1 & 2 and 0-5 to 0-12 are test results from recent in-house research. The Test wall no. 1-n, 2-n, 3-n or 4-n refer to the Mid-ply walls. Variations in nail size and length, stud, header and footer sizes and vertical load of 25 kN are also indicated accordingly in the Appendix 6.1 (see also Appendix 6.2).

Table 6.6 Test results of standard shear walls

Wall no.	Nail spacing	Vertical load	F_{max}	Stiffness	Comments
	mm	kN	kN	N/mm	
0-1	SW-150 Datum	0	13.41	597	Average of 5 wall test (from Phase 1)
0-2	SW-150	25	20.7	1060	Test results from Phase 1 & 2
0-3	SW-150	0	12.76	650	"
0-4	SW-150	25	20.8	1207	"
0-5	SW-150	0	13.10	417.17	From other tests
0-6	SW-100	0	19.79	736.06	"
0-7	SW-50	0	23.14	1022.20	"
0-8	SW-150	25	20.70	1053.00	"
0-9	SW-100	25	29.19	1744.00	"
0-10	SW-50	25	43.15	2470.00	"
0-11	SW-100	0	10.53	435.00	Openings for door 900 x 2050
0-12	SW-100	0	5.46	255.00	Openings for door 1500 x 2050

The following observations were made for the Mid-ply shear walls during the tests.

- The bending of bottom rails basically occurred due to small sized timber i.e. 45 × 45 mm.

- The buckling of the rear stud was observed at high loads of around 35 kN and over except in wall no.13 where the buckling occurred at a load of around 25 kN (see also Appendix 6.5).

6.5.1 Effects of nail spacing and nail length

For Standard (conventional) walls, the effect of nail spacing (density/pattern) are well established. The 150/300 mm nail pattern is the most common pattern used for standard walls (with standard racking capacity). For walls requiring higher racking resistance closer nailing i.e. 100/200 or even 50/100 nail patterns are often used. The results of further tests on Standard (conventional) walls examining the effect of nail spacing (density/pattern) are detailed in Table 6.7. The results of tests on Enhanced (Mid-ply) walls examining the effect of nail spacing (density/pattern) using two different nail sizes are shown in Table 6.8.

Table 6.7 Effects of nail spacing (density/pattern) on performance of Standard (conventional) walls.

Ref no.	Nail spacing	OSB3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
	mm	mm	mm	kN	Φ × length	kN	N/mm
0-5	SW-150	9	44 × 95	0	3.0 × 60	13.10	492
0-6	SW-100	9	44 × 95	0	2.8 × 50	19.8	720
0-7	SW-50	9	44 × 95	0	2.8 × 50	23.14	998
0-8	SW-150	9	44 × 95	25	3.0 × 60	20.70	1053
0-9	SW-100	9	44 × 95	25	3.0 × 60	29.19	1744
0-10	SW-50	9	44 × 95	25	3.0 × 60	43.15	2470

The results clearly indicate that the reduction in nail spacing is very effective in enhancing both strength and stiffness of Mid-ply shear walls, noting that 3.35 × 65 mm nails were smooth round wire nails and 3.1 × 75 mm were ring-shanked nails, see Table 6.8. However, when smooth but longer nails (3.1 × 90 mm) were used instead of ring-shanked ones, 3.1 × 75, the length of the nails did not have the expected positive effect, see Table 6.9 (see also in Appendix 6.6). Here, the increase was only noted in strength magnitudes whereas stiffness values were noticeably lower. The cause of this could be associated with a number of issues including: nail pull-through/out due to smoothness of nails, extra firing force of the nail gun for longer nails and possibility of crack formation (although not noticeable) and the dimensions of the timber sections used; noting that the spacing requirements of the code were satisfied.

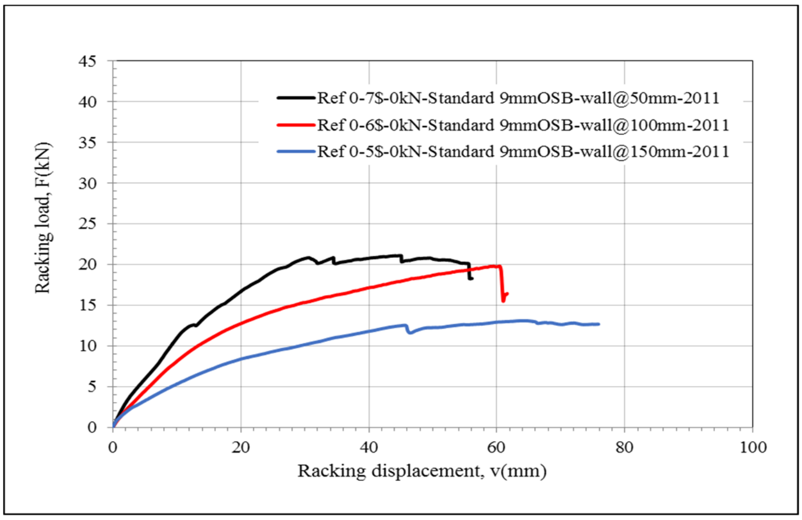
Table 6.8 Effects of nail spacing (density/pattern) on performance of Mid-ply walls.

Ref no.	Nail spacing	OSB3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
	mm	mm	mm	kN	Φ × length	kN	N/mm
1-1	D1-150	11	All 38 × 89	0	3.35 × 65	32.68	1180
1-2	D1-100	11	//	0	3.35 × 65	41.70	1533
1-7	D1-150	11	All 38 × 89	0	3.1 × 75	36.33	1207
1-8	D1-100	11	//	0	3.1 × 75	48.00	1762

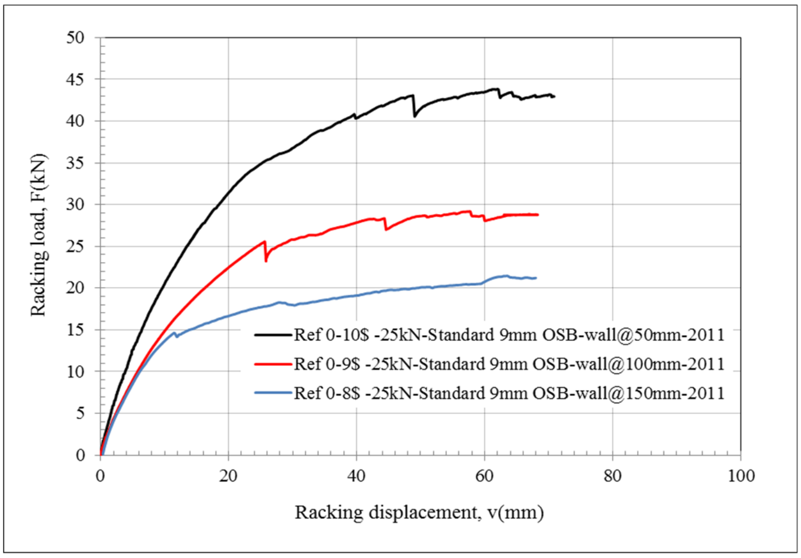
Table 6.9 Effects of nail spacing and length on performance of Mid-ply walls

Ref no.	Nail spacing	OSB3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
	mm	mm	mm	kN	Φ × length	kN	N/mm
1-19	D1-150	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75 ring-shank	35.74	1492
1-20	D1-100	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75 ring-shank	44.94	1305
1-21	D1-150	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 90 smooth	40.46	1299
1-22	D1-100	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 90 smooth	46.15	1159

The performance characteristics of the standard (conventional) walls with nailing patterns of 150/300, 100/200 and 50/100 under vertical loads of 0 kN and 25 kN is shown in Figure 6.19.



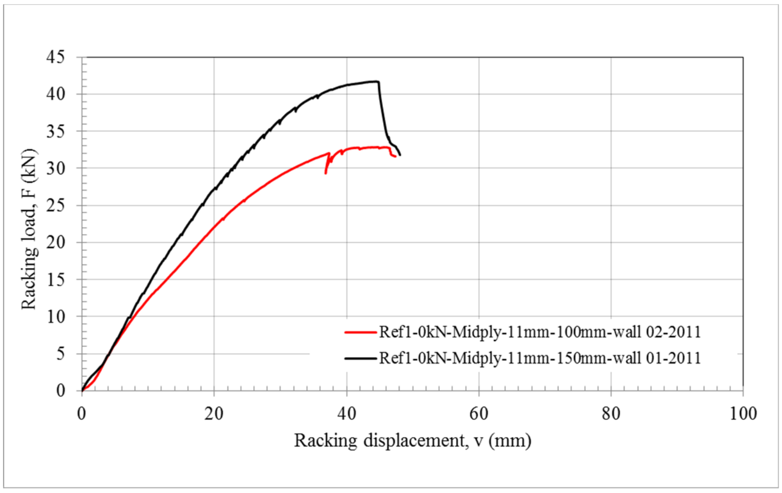
(a) vertical load – 0 kN



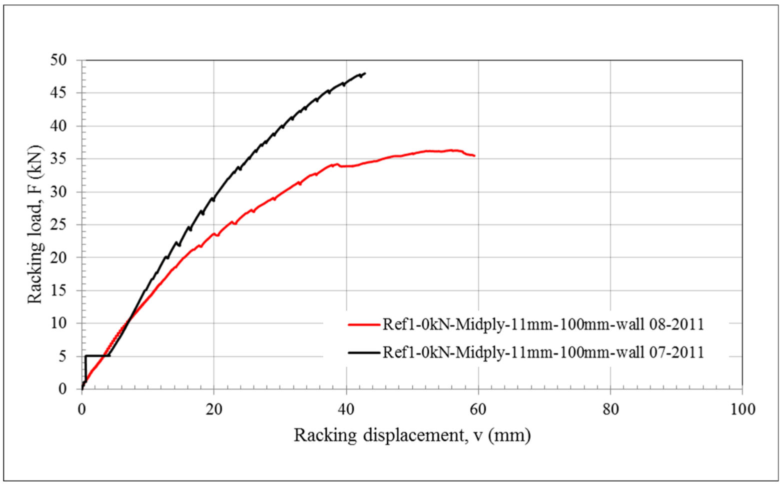
(b) vertical load - 25 kN

Figure 6.19 Effects of nail spacing (density/pattern) on performance of Standard (conventional) walls

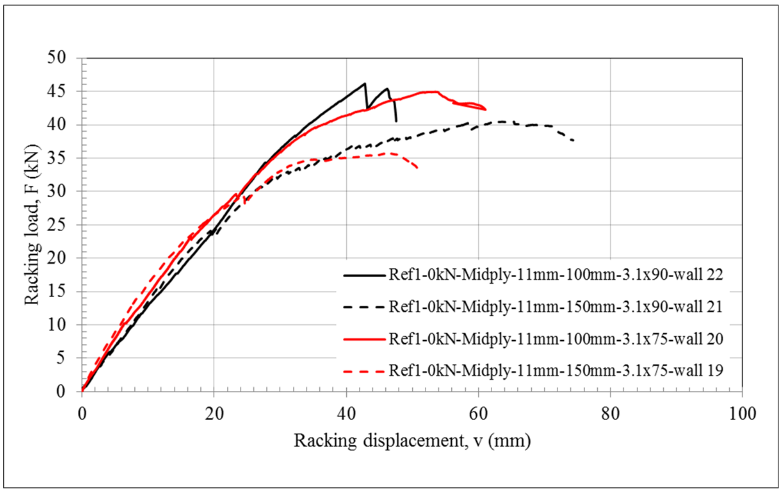
The racking performance of the Mid-ply walls and effects of nail spacing and length are shown and compared Figure 6.20 (a), (b), and (c).



(a) vertical load – 0 kN, Nails 3.35 × 65mm hand driven



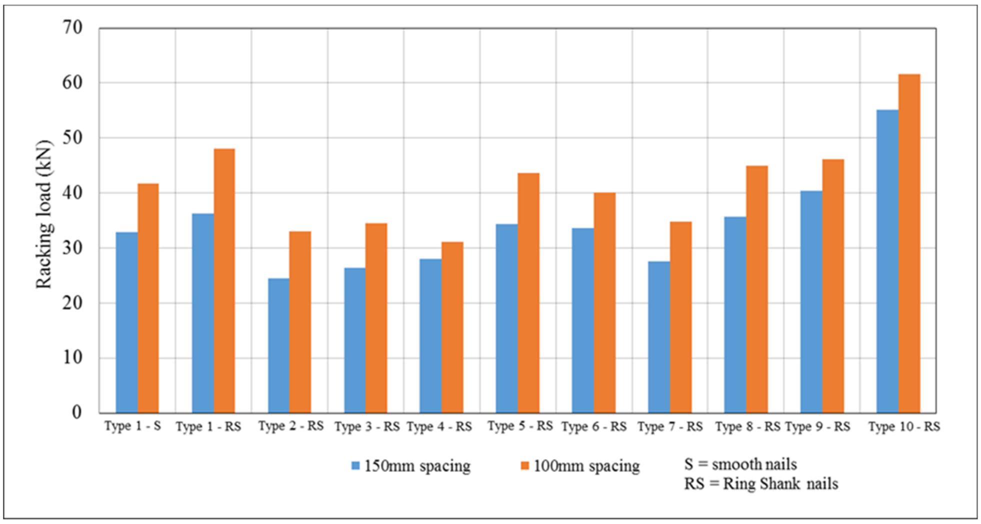
(b) vertical load - 0 kN, Nails 3.1 × 75mm nail-gunned



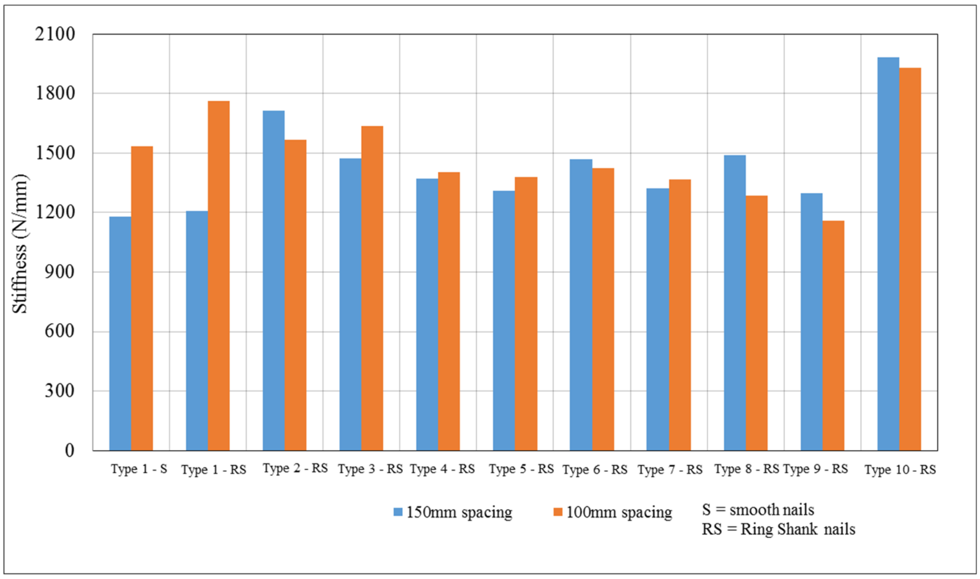
(c) vertical load – 0 kN, Nails 3.1 × 75mm and 3.1 × 90 mm nail-gunned

Figure 6.20 Effects of nail spacing (density/pattern) on performance of Mid-ply walls

The effect of fasteners spacing and types in terms of strength and stiffness are further illustrated and compared in Figure 6.21 (a) and (b) respectively. The results show that smaller spacing of the fasteners gives better wall performance, expect for the stiffness of Type 2 with ring shank nails. This might be due to splitting of bottom rail in wall number 9 of Type 2.



(a) strength



(b) stiffness

Figure 6.21 Racking performance of nail spacing at 150 mm and 100 mm

6.5.2 Calculation of fasteners strength and stiffness to EC5:

The yield moment of 3.1 x 75 mm ring-shank nails calculated in accordance with PD 6693-1 Clause 13 (Vessby et al., 2010b) is higher than that of the manufacturer value of 3286 Nmm (see Doudak et al., 2006b).

According to PD 6693 method;

$$M_{y,Rk}=0.3\ f_u d^{2.6} \tag{6.24}$$

$$M_{y,Rk} = 0.3 \times 700 \times 3.1^{2.6} = \mathbf{3978.87}\ \text{Nmm}$$

where;

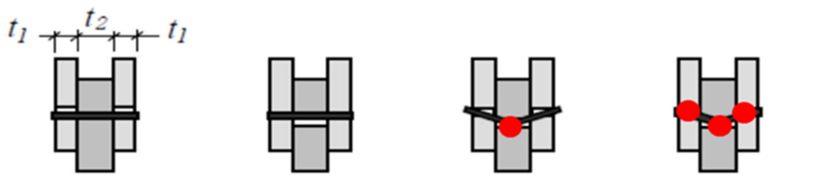
f_u tensile strength of wire (minimum of 700 N/mm^2 from manufacturer)

d nail diameter

The lower yield moment given by the manufacturer was used for calculation. Because the smooth nail with 65 mm long length do not meet the minimum EC5 point-side penetration length (8d) to be considered acting in double shear, the calculations were not entirely accurate. Non-conservatism of EC5 including the nail tip in the embedment length becomes more significant below the minimum penetration length as the tip account for a larger percentage of the point-side length. When the characteristics load carrying capacity of nails are calculated assuming single shear behaviour, the EC5 values drop to 443.81N which is for both studs widths due to failure Mode (yielding in the stud, see Table 6.2 and eq.(6.10)).

The characteristics load carrying capacity per fastener per shear plane with their associated failure modes were calculated for the Mid-ply shear walls of two different stud sizes, 38 × 89 mm and 44 × 95 mm, both comprised 11 mm thick OSB and ring-shank nails, according to equations ((6.12 - (6.15) and shown in Table 6.10.

Table 6.10 Characteristics load carrying capacity, $f_{v,Rk}$ (N) of different sized studs

Stud size	Nails	Failure modes			
mm	Φ × length	mode (g)	mode (h)	mode (j)	mode (k)
		eq.(6.12)	eq.(6.13)	eq.(6.14)	eq.(6.15)
					
		Characteristics load carrying capacity, $f_{v,Rk}$ (N)			
38 × 89	3.1 × 75	1459	638 critical	828	930
44 × 95	3.1 × 75	1120	638	637 critical	831.46

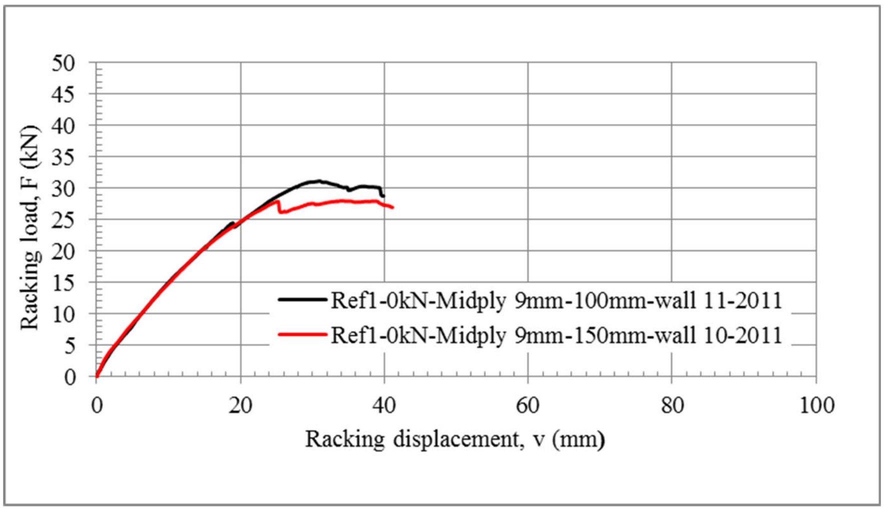
For the stud size of 38 × 89 mm, the failure mode (h) is not preferable as it does not brings double shear advantage because EC5 equation 8.7 (eq.(6.13 in this study) effectively splits the OSB in half for each shear plane. In stud size of 44 × 95 mm, the failure mode (j) was observed due to lower rope effect caused by poor point-side penetration. When the EC5 mid-ply double shear values were compared with the same fasteners in single shear using traditional wall stud orientation, higher of 60% was found in mid-ply system.

6.5.3 Effects of OSB/3 sheathing thickness

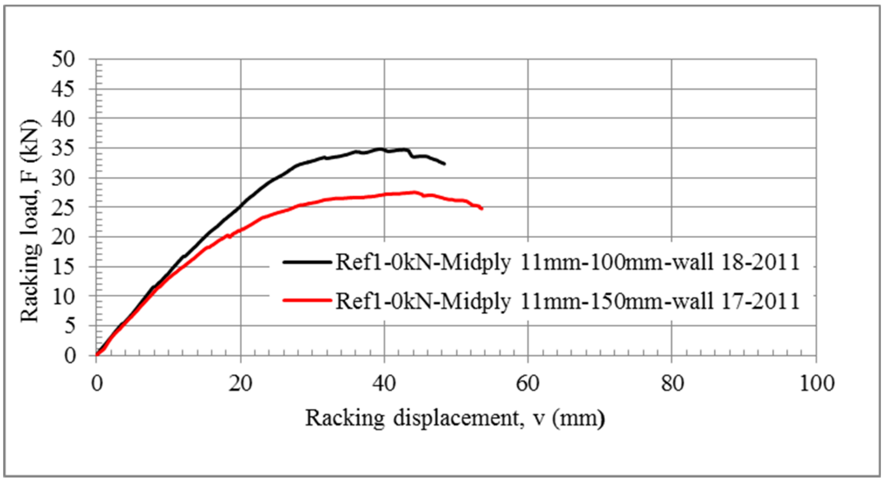
In Table 6.11, the results of tests on Mid-ply walls examining the effect of use of 9 mm and 11 mm OSB3 sheathing are detailed (also see Figure 6.22). The results are not conclusive but overall indicate a possible enhancement in performance when 11mm OSB is used instead of the typical 9 mm boards.

Table 6.11 Effects of use of 9 mm and 11 mm OSB sheathing on performance of Mid-ply walls

Ref No.	**Ref code nail spacing**	**OSB/3 thickness**	**Stud/timber size/type**	**Vertical load**	**Nails**	**Strength**	**Stiffness**
		(mm)		(kN)	Φ × length	(kN)	(N/mm)
1-10	D1-150	9	All 45 × 45	0	3.1 × 75	28.01	1343
1-11	D1-100	9	//	0	3.1 × 75	31.15	1148
1-17	D1-150	11	All 45 × 45	0	3.1 × 75	27.51	1318
1-18	D1-100	11	//	0	3.1 × 75	34.83	1370



(a) 9 mm OSB sheathing nails 3.1 × 75 mm nail-gunned



(b) 11 mm OSB sheathing nails 3.1 × 75 mm

Figure 6.22 Effects of use of 9mm and 11mm OSB sheathing on performance of Mid-ply walls

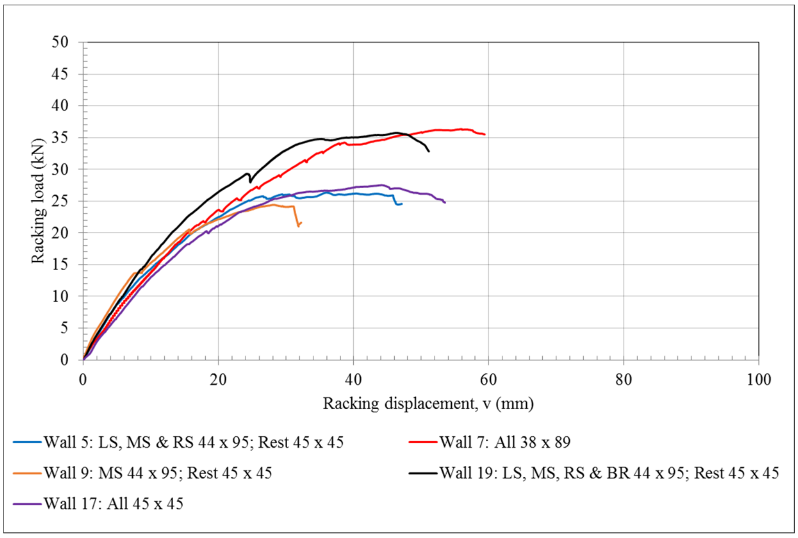
6.5.4 Effects of timber section sizes (studs, header and footer)

In Table 6.12 the results of tests on Mid-ply walls examining the effect of use of different sizes of timber sections for studs, header and footer plates are detailed. The aim was to examine the possibility of minimising the use of timber and hence to optimise performance/material use in design and construction of the walls, within the scope of the project.

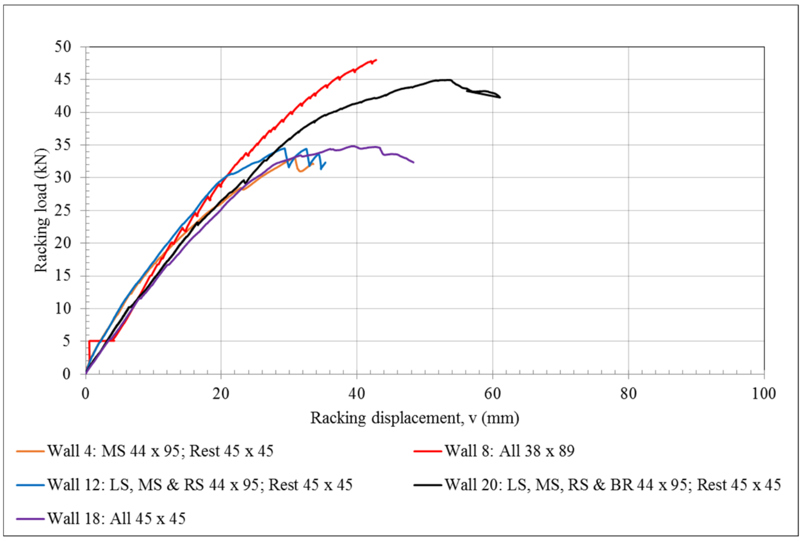
Table 6.12 Effects of use of smaller timber sections on performance of Mid-ply walls

Ref No.	Ref code -nail spacing	OSB3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
		(mm)	(mm)	(kN)	Φ × length	(kN)	(N/mm)
1-7	D1-150	11	All 38 × 89	0	3.1 × 75	36.33	1207
1-8	D1-100	11	//	0	3.1 × 75	48.00	1762
1-9	D1-150	11	MS 44 × 95 Rest 45 × 45	0	3.1 × 75	24.41	1716
1-4	D1-100	11	//	0	3.1 × 75	33.07	1567
1-5	D1-150	11	LS, MS and RS 44 × 95 Rest 45 × 45	0	3.1 × 75	26.34	1475
1-12	D1-100	11	//	0	3.1 × 75	34.34	1636
1-19	D1-150	11	LS, MS and RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	35.74	1492
1-20	D1-100	11	//	0	3.1 × 75	44.94	1305
1-17	D1-150	11	All 45 × 45	0	3.1 × 75	27.51	1318
1-18	D1-100	11	All 45 × 45	0	3.1 × 75	34.83	1370

The close inspection of the performance characteristic of the walls made with a set of predetermined timber section sizes showed that along the junctions (where panels are joined to one another or at base etc), a larger section timber which provides greater edge and end distances for the boards/timber, performs better, for example walls 19 and 7 or 20 and 8, as shown in Figure 6.23 (a) and (b).



(a) nail spacing 150 mm



(b) nail spacing 100 mm

Figure 6.23 Effects of use of different timber section sizes on performance of Mid-ply walls

6.5.5 Effect of opening for door (reduced shear area)

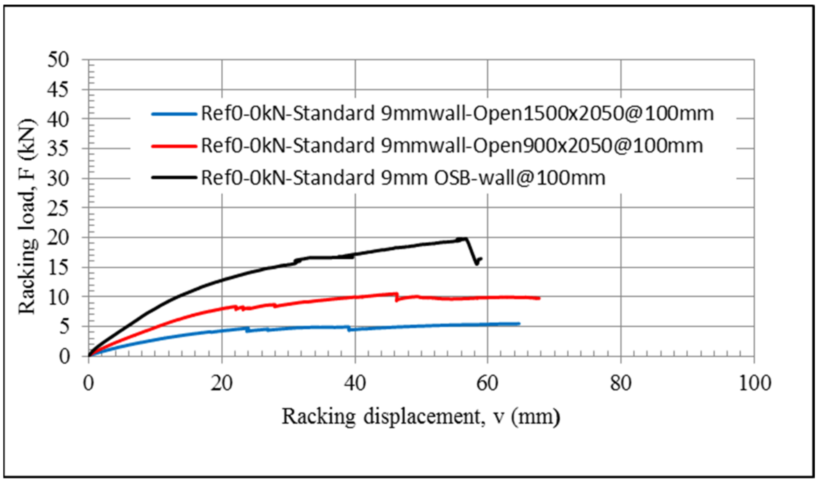
In order to compare the performance of Mid-ply walls, with openings for doors, with similar Standard (conventional) walls, results of a set of similar tests carried out earlier are included in Table 6.13 and their load-deformation characteristic are compared in Figure 6.24 (a). In Table 6.14 the results of tests on Mid-ply walls examining the effect of openings for two different sizes of doors 900 × 2050 mm and 1500 × 2050 mm are detailed and in Figure 6.24 (b) their performance is compared. The effects of opening for doors on Standard and Mid-ply walls in terms of stiffness are shown in Figure 6.25.

Table 6.13 Effect of opening for doors on performance of Standard (conventional) walls.

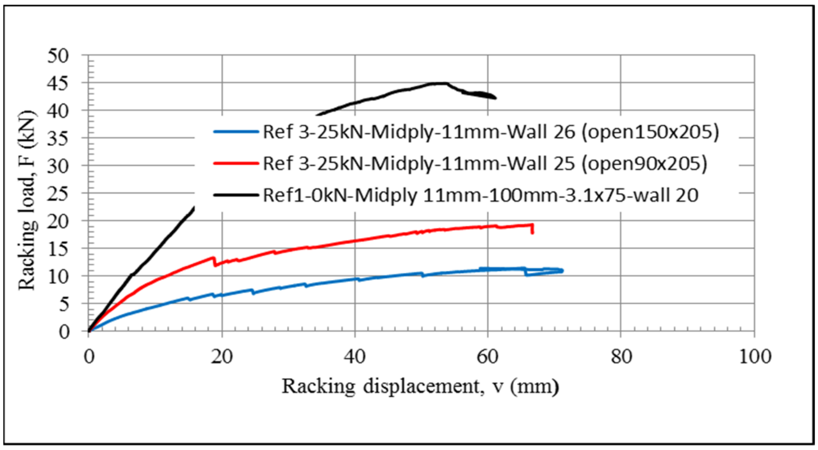
Ref No.	Ref code -nail spacing	Opening	OSB/3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
		size & %	(mm)	(mm)	(kN)	Φ × length	(kN)	(N/mm)
0-6	SW-100	Solid wall 0%	9	44 × 95	0	2.8 × 50	19.8	720
0-11	SW-100	Door 900 × 2050 32%	9	44 × 95	0	2.8 × 50	10.53	435
0-12	SW-100	Door 1500 × 2050 53%	9	44 × 95	0	2.8 × 50	5.46	255

Table 6.14 Effect of opening for doors on performance of Mid-ply walls

Ref No.	Ref code -nail spacing	Opening	OSB/3 thickness	Stud/timber size/type	Vertical load	Nails	Strength	Stiffness
		size & %	(mm)	(mm)	(kN)	Φ × length	(kN)	(N/mm)
1-20	D1-100	Solid wall 0%	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	44.94	1305
3-25	D1-100	Door 900x2050 32%	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	19.37	858
3-26	D1-100	Door 1500x2050 53%	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	11.59	356



(a) Standard (conventional) walls



(b) Mid-ply walls

Figure 6.24 Effect of opening for doors on Standard and Mid-ply walls

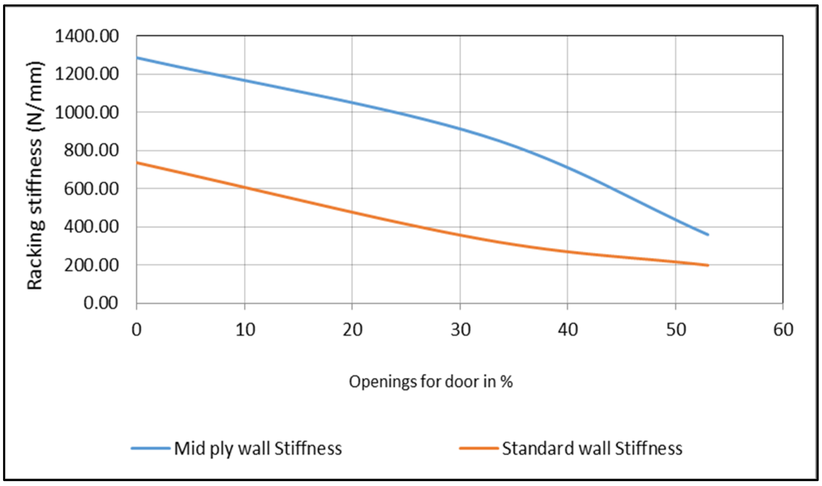


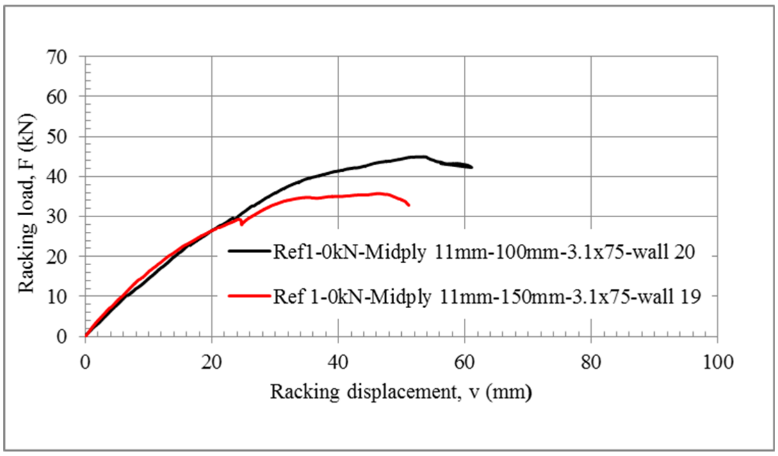
Figure 6.25 Effect of opening for doors on Standard and Mid-ply walls in terms of stiffness

6.5.6 Effect of application of vertical load

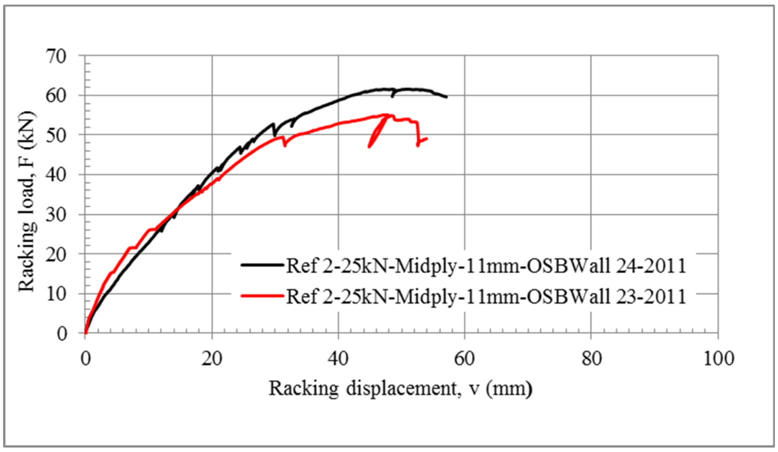
The results of tests on Mid-ply walls examining the effect of applied vertical load are detailed in Table 6.15 and in Figure 6.26. The observation of the performance of the walls during tests indicated that the walls, as expected, demonstrated a comparatively high strength and stiffness when subjected to combined vertical and racking loads. However, the failures were mainly governed by the buckling of the leeward stud at high loads.

Table 6.15 Effects of applied vertical load on performance of Mid-ply walls

Ref No.	**Ref code - nail spacing**	**OSB/3 thickness**	**Stud/timber size/type**	**Vertical load**	**Nails**	**Strength**	**Stiffness**
		(mm)	(mm)	(kN)		(kN)	(N/mm)
1-19	D1-150	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	35.74	1492
1-20	D1-100	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	0	3.1 × 75	44.94	1305
2-23	D1-150	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	25	3.1 × 75	55.12	1985
2-24	D1-100	11	LS, MS, RS and BR 44 × 95 Rest 45 × 45	25	3.1 × 75	61.60	1930



(a) vertical load 0 kN



(b) vertical load 25 kN

Figure 6.26 Effects of applied vertical load on performance of Mid-ply walls

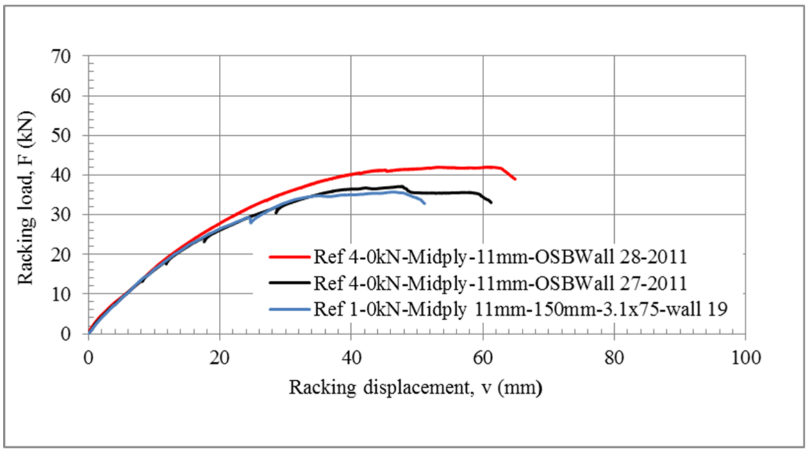
6.5.7 Characteristic strength and stiffness

The characteristic strength and stiffness values for the Mid-ply walls were determined by testing 3 wall systems under zero vertical loading and 3 walls under 25kN vertical load. The results are summarised in Table 6.16 a and b. The load-deformation behaviours are shown in Figure 6.27 a and b.

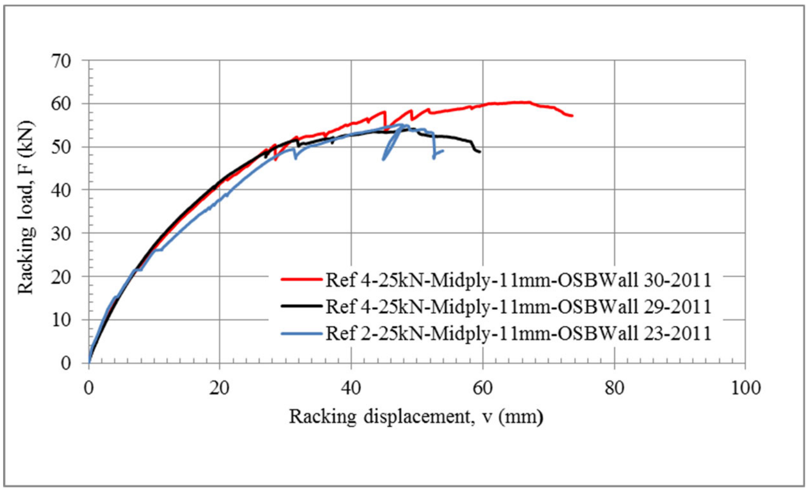
Table 6.16 Mid-ply wall, 11 mm OSB, 3.1 × 75 mm nails, 150 mm spacing

(a) 0 kN		
Test Ref	**Ultimate**	**Stiffness**
Test 1 (1-19)	35.74	1492
2 (4-27)	37.13	1469
3 (4-28)	42.04	1443
Mean	**38.30**	**1467.99**
SD =	**3.31**	**24.70**
C.V.=	0.09	0.02
Charc.V.=	32.64	1253.95

(b) 25 kN		
Test values	**Ultimate**	**Stiffness**
Test 1 (2-23)	55.12	1986
2 (4-29)	54.11	2571
3 (4-30)	60.32	2276
Mean	**56.52**	**2277.58**
SD =	**3.33**	**292.40**
C.V.=	0.06	0.13
Charc.V.=	48.23	1934.94



(a) vertical load 0 kN



(b) vertical load 25 kN

Figure 6.27 Mid-ply wall, 11 mm OSB, 3.1 × 75 mm nails, 150 mm spacings under 0 kN and 25 kN vertical loads tested to EN 594:2011

6.5.8 Conclusion

The results of this study show that the performance characteristics of the Mid-ply walls are significantly higher than the Standard (traditional) walls in both strength and stiffness properties. As a comparison, the mean strength and stiffness values obtained for Mid-ply walls were 38.3 kN and 1467 N/mm, respectively, compared to those of the Standard walls of 13.4 kN and 597 N/mm, when no vertical loading was applied. The results indicate an enhancement in strength of around 280% and in stiffness of around 210%. A similar pattern of enhancements was observed when the walls were subjected to a vertical load of 25 kN.

Also, the comparison of the results indicates that Mid-ply wall with a large opening for a door can exhibit a racking resistance that is comparable to that of a solid Standard wall.

Based on the tests carried out and comparison of the results, it was shown that the Mid-ply walls provide a considerable potential for use in UK and European timber frame construction as they provide a marked improvement in both strength and stiffness properties over the Standard walls. The use of Mid-ply walls can result in the elimination of the costly steel portals commonly used around the areas in walls with large openings and where the standard designs fail to meet the racking resistance requirements. In addition, the Mid-ply walls can eliminate the costly problems associated with the known OSB warping and stud distortions during construction.

Chapter 7 Conclusion and Recommendation

7.1 Introduction

Timber Platform frame construction is extensively acknowledged as an efficient building method for multi-storey dwellings. Previous research on the racking performance of timber frame walls had limited objectives in relation to examining the length of walls, size and position of openings for doors and windows or possible design configurations, fixing types, size and positions. This research has aimed to consider these issues in determining and better understanding of the racking performance of the standard walls as well as Mid-ply walls and are summarised below.

7.2 Key findings and research output

Objective (i) was to examine the compatibility and suitability of two different test procedures detailed in BS EN 594:1996 and BS EN 594:2011 versions on the racking performance of timber framed walls. This objective was addressed and discussed in Chapter 3.

In the UK, various test methods have been introduced to calculate racking resistance, superseding the older versions superseded by newer ones to achieve better racking performance. The BS 5268-6.1:1996, though was superseded by Eurocode 5, is still used and it includes a calculation method using BS EN 594:1996. The BS EN 594:2011 that superseded BS EN 594:1996 introduced significant changes in the test procedure, such as the loading cycle requirement up to 40% of the failure load is removed and the overall test duration is greatly reduced.

The racking performance conducted on a range of OSB/3 panels, Air/Vapour barrier OSB, Medite Vent panel, Medite Tricoya panel and Fire resistant OSB of varying thicknesses, fixed to one side only of the timber framing in accordance with 1996 and 2011 versions of BS EN 594 showed that the racking strength load was similar for both test procedures and for the walls tested there was an average variation between results of just -2.7%. For stiffness behaviour, there was a clear difference between the results of the wall panels tested under the two procedures. Tests to BS EN 594:1996 consistently resulted in stiffness values greater than those derived from BS EN 594:2011; on average, the stiffness was over 46% greater than the stiffness of identical wall panels to BS EN 594:2011. Because of this difference, when applying the procedure given in BS

5268-6.1 to calculate the value of the basic racking resistance, it was significantly lower under the procedure in BS EN 594:2011 than the procedure in BS EN 594: 1996. Furthermore, the basic racking resistance values derived from the tests to BS EN 594:2011 were always less than the basic resistance value tested to BS EN 5268-6.1. When the racking walls using different sheathing materials were tested in accordance with BS EN 594:2011 and 1996 under vertical loading of 0 kN and 25 kN, the strength and stiffness of the walls increased significantly tested in BS EN 594:1996 under the vertical load of 25 kN. The basic racking resistance of all tested walls for all sheathing materials were higher than the requirement indicated in BS 5268-6.1 (category 1) for both with and without vertical loads. The Medite Tricoya due to their enhanced material properties performed exceptionally well. However, the existing literature showed that the most effective sheathing materials for shear resistance are plywood Iizuka (1975) and OSB Toothman (2003).

Objective (ii) was to determine the effects of parameters such as panel-to-frame fastener spacing, wall length, arrangement and composition of studs and bottom rail members (e.g. use of double studs and double bottom rail), magnitude of vertical loading on the racking performance of OSB and PB sheathed walls. This objective was addressed and discussed in Chapter 4.

The experimental tests were focused on partially anchored racking walls and the procedure described in the PD 6693-1 document, as recommended by the UK NA to EC5 was adopted. In general, the racking strength of the wall was found more sensitive to variations in the fastener spacings when it was subjected to a vertical loading. Conversely, when there was no vertical loading, the racking stiffness was more sensitive to change in fastener spacings.

The effect of panel-to-frame fastener spacing was more pronounced when the wall was subjected to an applied vertical loading. When the walls were tested under a vertical loading of 25 kN with reduced fastener spacing from 150 to 50 mm, the strength was 89% higher compared to the strength of similar wall without vertical loading. The racking stiffness of walls without vertical loading increased by 300% when the fastener spacing was reduced from 150 to 50 mm. However, when the walls were subjected to a vertical loading of 25kN, the increase in stiffness was only by 24%.

The comparison of the experimental results of the full-length (2400 mm) wall specimens, irrespective of their panel-to-frame fastener spacings (50 to 150 mm), with the results from the EC5 design code formulae, showed that on average the design code underestimated the racking strength by 25% for walls under vertical loading of 25 kN and by 54% for walls without vertical loading. The most likely explanation why such an underestimation of the racking strength is greater for walls without applied vertical load is due to the contribution to the stabilising moment due to the withdrawal capacity of the stud-to-beam connections.

Compared to walls with a standard type of frame, the use of double studs and double bottom rails provided (on average) an increase in racking strength and stiffness of about 64% and 37% respectively. Nonetheless, the enhanced racking capacity may be (solely) attributed to the use of increased number of panel-to-frame fasteners along the perimeter of the wall.

Considering stiffness behaviour, all walls comply with the requirements of the empirical relationship given in clause 21.5.2.3 of the PD-6693-1 document. However, when deriving stiffness behaviour from the experimental results, i.e. using the ΔSLS/0.003H approach, walls I-1, II-1 and III-5 (refer Table 4.2) failed. Hence, it is difficult to draw any general conclusions on the accuracy of the PD 6693-1 criterion. However, both approaches resulted in an increase in value as wall stiffness was increased. For the stiffest wall, II-1, the experimental result showed that the wall would fail while the PD-6693-1 approach concludes it would pass. As acceptable stiffness behaviour has to be achieved in the design of racking walls, it is to be questioned whether the empirical relationship given in equation PD6693-1 would require to be reviewed.

The test for double sided walls sheathed with OSB/3 on one side and British gypsum PB on the other were conducted with existing design methods in accordance with EC5 (Method A and B) and PD 6693-1:2012. The racking load obtained from Method A for double sided wall recorded the value of 23% lower than the design load. This could be due to the premature failure of the Plasterboard or due to the decision limited on only one test of the wall with each fastener spacing. The racking load of double sided walls with OSB and PB is lower than the sum of two individual wall panels sheathed with OSB and PB. However, this contradicts the study conducted by Patton-Mallory et al. (1984) who claimed that the behaviour of the double-sided wall can be predicted by summing single-sided wall values. But the test conducted by Sartori (2012) for evaluating the behaviour of panels with three configurations; OSB on both sides, gypsum fiberboard (GFB) on both sides and OSB on one side and GFB on the other side, indicated that all three configurations have similar stiffness and strength for the wall sheathed with GFB or with the mixed configuration was lower than the wall sheathed with OSB. On the other hand, the study conducted by Branco et al.(2017) concluded that the stiffness almost doubles in relation to one side with OSB board to both sides with OSB boards. While in this research, the stiffness of double sided sheathed walls with OSB on one side and other with PB was better than wall sheathed one side with OSB only.

Objective (iii) was to determine the effects of openings/discontinuities of windows and doors on racking performance of OSB walls with and without using the trimmers as well as spreader. This objective was analysed in Chapter 5.

The strength and stiffness of the walls with opening for windows and doors were examined in accordance with BS EN 594:2011. The tested results were then compared to examine the accuracy with the existing design methods in accordance with EC5 (Method B and PD 6693-1:2012). Since, the Method A does not consider the opening for calculating racking performance; it was not examined. The tested results showed that the racking performance was significantly influenced by the size of the openings. The results confirmed that the increase in opening percentages (windows and doors) decreases the racking performance. This also confirms with the studies conducted by He et al. (1999), Hayashi (1988), Yasumura (2010), and Steensels et al. (2017) regardless of any loading conditions and sheathing materials.

In the experiment conducted on walls with openings for windows, the strength and stiffness was also affected by the addition of framing around the openings and cripple studs. Comparing the walls with small opening percentage with and without the additional framing, the effect was small; whereas, the wall with large opening percentage with and without the additional framing had the positive effect. The failure modes observed were the combination of lifting up of the leading stud from the bottom rail, pull-out of the nails from timber and pull-through of the nails in OSB, and shearing of OSB at opening corners.

Comparing the experimental results and existing design methods (Method B and PD 6693-1:2012) of openings for windows with and without the spreader and trimmer, the racking resistance of all sized window openings was recorded higher in experimental tests. The Clause 9.2.4.3.1 (2) in Method B of EC5 stated that large opening sizes do not contribute to racking load. Conversely, the experimental result of the large opening size for windows in the walls with and without spreader and trimmer recorded the design load which is higher than the existing design methods.

Comparing the walls with small opening percentage for doors with and without the additional framing, the effect in racking performance was small. The failure modes observed shearing of OSB sheathing on both top corners of the opening and partial buckling at rear top corner of the opening. The racking resistances of the walls with openings for doors obtained from experimental result are higher than Method B and PD 6693-1:2012 and the design loads of PD 6693-1 seemed to be conventional than those of experimental tests and Method B. Moreover, Comparing the experimental stiffness with the empirical equations derived by Sugiyama and Matsumoto (1994) and Dolan and Johnson (1997), the trend line of experimental stiffness indicates the similar linear behaviour as that of Dolan and Johnson, whereas for actual strength, it is close to the empirical equation derived by Sugiyama and Matsumoto.

Objective (iv) was to examine the effect of a range of geometrical parameters such as fastener size and spacing, wall length, sheathing thickness, size and position of studs, as well as the effect of vertical loading on the racking strength and stiffness of the Mid-ply shear walls. This objective was analysed in Chapter 6.

This chapter focused on optimisations of the performance of Mid-ply shear walls by testing a series of geometrically configured walls with OSB/3 sheathing. The experimental works followed the test procedure and recommendations of BS EN 594:2011. The experimental results showed that the reduced nail spacing was very effective in increasing strength and stiffness of Mid-ply shear walls. The length of the nails did not have the expected positive effect, with increase in strength only and stiffness values were noticeably lower. When the EC5 mid-ply double shear values were compared with the same fasteners in single shear, higher of 60% was found in Mid-ply wall. Ni et al. (2008) in their analysis also found that Mid-ply walls have greater lateral load capacity and stiffness compared to the standard shear wall with same framing members, sheathing, nail diameter and spacing.

The experimental results for examining the effect of use of different size OSB/3 sheathing, though were not conclusive, the overall indicated a possible enhancement in performance in high thickness than the lower one. The results of tests on Mid-ply walls examining the effect of use of different sizes of timber sections for studs, header and footer plates showed that along the junctions (where panels are joined to one another or at base etc), a larger section timber which provides greater edge and end distances for the boards/timber, performed better.

The results of tests on Mid-ply walls examining the effect of applied vertical load demonstrated a comparatively high strength and stiffness when subjected to combined vertical and racking loads.

7.3 Future research

- Development of a new/revised method for determination of basic racking resistance/load for the test results to replace those in the old BS Code: The racking resistance provided in BS 5268 is used to conduct the test in accordance with old BS Code. Therefore, it is necessary to develop either the new BS Code or the new racking resistance which could be used for new BS Code.

- Development of a design procedure for Mid-ply walls which more accurately represent their performance: There is not BS Code for Mid-ply walls therefore there is necessary to develop a design procedure for this type of the walls according to its performance.

- It is recommended to test three or more replicates of the double sided walls of which one side sheathed with OSB with fastener spacing at 50, 100, and 150 mm and other side sheathed with Plasterboard with fixings at 100mm spacings to obtain more accurate results and confirm the findings of this research.

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Appendix 1.1 Summary of materials used in this research

Elements	Sizes	Materials
Studs and rails	44 × 95 mm	C16, C24 (BS EN 338:2003)
	38 × 89 mm	
	45 × 45 mm	
Sheathing boards	9 mm, 11 mm OSB (1200 × 2400 mm)	OSB/3 (BS EN 300)
	12.5 mm plaster board (PB) (1200 × 2400 mm)	
	12.5 mm A/V Barrier OSB	
	12 mm Medite Vent	
	9 mm Medite Extreme Tricoya	
	11 mm FR OSB	
Fasteners (Nails and Screws)	2.8 x 50 mm bright smooth round headed nail	EN 14592
	3 × 60 mm bright smooth round headed Rynails	EN 14592
	3 × 50 mm bright smooth round headed Rynails	EN 14592
	3.1 × 75 mm ring shank D-headed nails (Paslode)	EN 14592
	3.1 × 90 mm smooth D-headed nails (Paslode)	EN 14592
	3.35 x 70 mm bright smooth nails	EN 14592
	3.35 × 65 mm bright smooth wire nails	EN 14592
	3.5 x 40 mm drywall screws	EN 14592
Screws for fixing the frame	3.5 × 100 mm wood screws	EN 14592
	3 × 75 mm wood screws	EN 14592
Holding down position anchors with plate	M12 threaded bars with 75 × 75 × 10 mm sized square plate	
Loading plate	90 × 170 mm with 12 mm thickness and 100 x 170 mm with 25 mm thickness	
Plates for vertical loading roller	95 × 120 mm with 3 mm thickness	

Appendix 3.1 Racking test results and calculation process (work out example for Test 1 of Table 3.3)

F_{max} (kN): Racking strength of the walls in kN

Stiffness (N/mm): Racking stiffness in accordance with EC5 BS EN 594:2011 on 6.5 (a) (also see equation 3.5)

Racking stiffness in accordance with EC5 BS EN 594:1996 (see equation 3.4)

$F_{max, min}$: Minimum F_{max} from three similar type of tested panels

$F_{max, min}$ is minimum of (19.85kN, 20.70kN, and 19.32kN)

R, Average stiffness (N/mm): (1298.01+1225.00+1105.86)/3 which is 1210 N/mm

$F_{, fail}$ (kN): ($F_{max,min}$ x K109)/ Factor of safety

= 19.32 x 0.93 / 1.6

=11.23 kN

$F_{,stiff}$ racking stiffness load (kN): (R x 0.002 x Hwp x 1.25 x K109) /1000

= (1210 x 0.002 x 2400 x 1.25 x 0.93)/1000

= 6.75 kN

R_b Basic racking resistance (kN/mm): min (F_{stiff}, F_{fail}) / (2.4 K111)

= 6.75 / (2.4 x 1.77)

= 1.59 kN/mm

F_{1stiff} (kN): (R x 0.003 x 2400 x K109)/1000

= (1210 x 0.003 x 2400 x 0.93)/1000

= 8.10 kN

R_{1b} = R_b for BS EN 594:1996

For BS EN 594:2011,

= min (F_{1stiff}, F_{fail}) / (2.4 K111)

= 8.10 / (2.4 x 1.77)

= 1.91 kN/mm

Appendix 4.1 Panel-to-frame connection tests

In order to derive the relevant value of $F_{v,mean}$, a total of twelve panel-to-frame connection samples, each comprising four bright wire smooth nails, were tested. Six samples were assembled using 2.8 mm diameter × 49 mm long nails, and a further six were assembled using 3.0 mm diameter × 52 mm long nails. As shown in Figure A4.1, two different types of test set-up were considered. For each nail size, three connection samples were tested by loading the OSB panel towards its edge (to cover for possible edge splitting failure) and three more samples with the OSB panel loaded away from its edge. The strength value, $F_{v,max}$, obtained from each sample test divided by 4 (the No. of nails per sample) is reported in Appendix 4.2, whilst the values of $F_{v,mean}$ reported in the fourth column of Table 4.5 in Chapter 4 were taken as the average of the $F_{v,max}$ values reported in Appendix 4.2.

(a)

(b)

Figure A4.1: Test set up to assess the strength of the panel-to-frame connections.

(a) set up with the panel loaded towards its edge and (b) set up with the panel loaded away from its edge.

Appendix 4.2 Summary of test results for the panel-to-frame connections.

Test No.	Nail size [mm]	$F_{v,max}$[a][N]	Test set up[b]
1	2.8 x 49	732.0	(a)
2	2.8 x 49	789.7	(a)
3	2.8 x 49	827.2	(a)
4	2.8 x 49	720.7	(b)
5	2.8 x 49	835.5	(b)
6	2.8 x 49	770.2	(b)
Average =		779.2	
Standard deviation =		43.4	
Standard deviation / Average =		5.6 %	
7	3.0 x 52	1437.5	(a)
8	3.0 x 52	1287.7	(a)
9	3.0 x 52	1529.7	(a)
10	3.0 x 52	1090.2	(b)
11	3.0 x 52	1018.7	(b)
12	3.0 x 52	1174.2	(b)
Average =		1256.3	
Standard deviation =		182.3	
Standard deviation / Average =		14.5%	

[a]Referring to the strength test result divided by the number of nails per sample (i.e. 4)

[b]As from Figure A4.1.

Appendix 4.3 Failure modes of smooth nails at connection between leading stud and bottom rail on standard wall and double end studs and double bottom rails walls.

Appendix 5.1 Calculation of existing methods in accordance with EC5.

General properties

Width of each stud, b (mm)	44
Depth of each stud, h (mm)	95
Wall height, hp (m)	2.4
Wall panel width, Pw (m)	1.2
Lateral spacing of each stud, Ss Stud (mm)	600

Wall panel ratio must be less than 4: r = hp/hw = 2 OK

Wall length, bp (m) =2*Pw	2.4
Thickness of OSB/3; tosb (mm)	9
Fastener diameter, dn (mm)	2.8
Fastener spacing, s (m)	0.1

Lateral capacity of an individual fastener

Diameter, d (mm)	2.8
Length, L (mm)	50
Diameter of nail head, dh (mm)	5.5
Tensile strength, Fu (N/m^2)	600
Thickness of first member, t1 (mm)	9
Pointside penetration, t2 (mm), tpen	41
Characteristic density of timber, ρk (kg/m^3)	310
Characteristic density of OSB3, ρk OSB (kg/m^3)	550
Characteristic yield moment of fastener for round nails (kN m)	
$M_{y,Rk}=0.3\ f_u d^{2.6}$	2617.48

Characteristic embedment strength (without predrilled holes) (N/mm^2)

For timber: $f_{h,1,k} = 0.082\,\rho_k\, d^{-0.3}$	18.67
For OSB/3: $f_{h,2,k} = 65 d^{-0.7} t^{0.1}$	39.38
Ratio between the embedment strength of the members: $\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$	2.11
Withdrawal strength , Timber (N/mm^2) $f_{ax,Rk}$=$20* 10^{-6}\rho_k{}^2$	1.92
Pull through strength in OSB, (N/mm^2) $f_{head,k}$=$70* 10^{-6}\rho_k{}^2$	21.08
Withdrawal capacity for smooth nails (N) $f_{ax,Rk1} = f\rho_{ax,Rk1} d t_{pen}$	220.65
$f_{ax,Rk2} = f h_{ax,k} d t_1 + f_{head,k} d^2 h$	688.98
Therefore, withdrawal capacity of smooth nail, (N) $f_{ax,Rk}$ = minimum of $f_{ax,Rk1}$ and $f_{ax,Rk2}$	220.65

Lateral load carrying capacity of individual fastener for panel to timber

- joint in a single shear connection
- for fasteners in single shear, $f_{ax,Rk}$= min of 6 + 2 equations (N) — 634.17
- $f_{ax,Rk}$= minimum of A to H (from equation 8.9 EC5)

$F_{v,Rd} = \frac{K_{mod.med} * F_{v,Rk}}{\gamma_{M,connection}}$ — 536.61 N

Method A

k_{mod}	(short + Int)/2 = (0.5+1.1)/2 = 1
γ_M	1.25
$F_{f,Rd} = \frac{K_{mod} \cdot (1.2\, F_{v,Rk})}{\gamma_M}$	608.81 N

$F_{i,v,Rd}= \frac{F_{f,Rd} b_i c_i}{s}$

For; S = 100 mm, $F_{v,Rd}$ 14.61 kN

Method B

Modification factors,

$s_0= \frac{9.7\ d}{\rho_k}$ 0.088

$k_{i,q}=1+\left(0.083\ q_i\text{-}0.0008\ q_i^2\right) \left[\frac{2.4}{b_i}\right]^{0.4}$ 1.00 (vertical load = 0 kN)

Panel dimension factor, 1.00 (b = 2.4m, h = 2.4 m)

$$K_d = \begin{cases} \frac{b_i}{h} & \text{for } \frac{b_i}{h} \leq 1.\ 0 \\ \left[\frac{b_i}{h}\right]^{0.4} & \text{for } \frac{b_i}{h} > 1.\ 0 \text{ and } b_i \leq 4.8 \text{ m} \\ \left[\frac{4.8}{h}\right]^{0.4} & \text{for } \frac{b_i}{h} > \end{cases}$$

Sheathing material factor 1

$$k_n = \begin{cases} 1.0 \\ \frac{F_{i,v,Rd,max} + 0.5F_{i,v,Rd,min}}{F_{i,v,Rd,max}} \end{cases}$$

Fastener spacing, s (m) 0.1

Fastener spacing factor, ks 0.64

$$k_s = \frac{1}{0.86 \frac{s}{s_0} + 0.57}$$

$F_{i,v,Rd} = \frac{F_{fRd} b_i}{s_0 * 1000} k_d k_{i,q} k_s k_n$ 9.50 kN

PD method

Design shear capacity per unit length of perimeter fasteners

For 100 mm nail spacing:	0.1
$f_{p,d}= \frac{F_{f,Rd}[1.15+s]}{s*1000}$(kN/m)	6.71
Bottom rail fixity factor:	
μ	1
$K_{i,w}=\left[1+\left(\frac{H}{\mu L}\right)^2+\left(\frac{2M_{d,stb,n}}{\mu f_{p,d,t}L^2}\right)\right]^{0.5}-\left(\frac{H}{\mu L}\right)$	0.443 1 (assumed)
Design racking strength of panel:	
K_{comb}	0
$K_{opening}$	1
For 100 mm nail spacing: 0 kN	
$F_{i,v,Rd}= K_{opening}K_{i,w}f_{p,d,t}L$ (kN)	7.15

In order to limit racking deflection

$K_{i,w}f_{p,d,t}\leq 8(1+k_{comb})\left(\frac{L}{H}\right)$	2.97 ≤ 8.01 OK

Appendix 6.1 Test results of Mid-ply walls

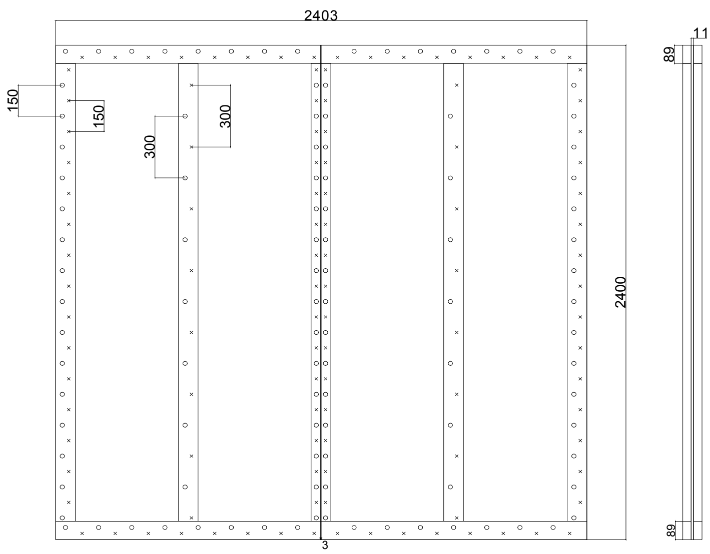
Wall Type	Wall no.	Nail spacing	Vertical load	F_{max}	Stiffness	Comments
			kN	kN	N/mm	
1	1-1	D1-150	0	32.86	1180.42	Partial separation of BR from lead panels
	1-2	D1-100	0	41.70	1535.22	
	1-7	D1-150	0	36.33	1208.62	Compression at a base point of RS and BR
	1-8	D1-100	0	47.99	1761.01	Buckling of RS
2	1-3	D1-150	0	21.16	1237.26	Partial separation of BR from lead panels
	1-4	D1-100	0	33.07	1567.44	Uplift of sole plate from test frame
	1-9	D1-150	0	24.41	1711.93	Splitting of BR on leading part
3	1-5	D1-150	0	26.34	1473.78	Control on uplift of sole plate from test frame due to increase in thickness of sole plate
	1-6	D1-100	0	29.13	1207.45	Partial separation of LS/IS1/MS from boards on both sides
	1-12	D1-100	0	34.48	1636.82	Repeat test for Wall 6. OSB ripped off at corner of LS and BR
4	1-10	D1-150	0	28.01	1372.96	Uplift of BR
	1-11	D1-100	0	31.15	1403.88	Zig-zag pattern of nails in MS
5	1-13	D1-100	0	43.60	1378.16	Buckling of RS at about 25kN
	1-14	D1-150	0	34.33	1310.79	Partial separation of BR from panels
6	1-15	D1-150	0	33.66	1470.45	Additional configurations. BR lifted up at leading part
	1-16	D1-100	0	40.11	1422.31	BR lifted up at leading part
7	1-17	D1-150	0	27.51	1320.67	RS buckled
	1-18	D1-100	0	34.83	1367.31	BR lifted up
8	1-19	D1-150	0	35.74	1487.24	Use of ring shanked nails. Separation of BR from leading board
	1-20	D1-100	0	44.94	1285.76	Use of ring shanked nails at centre.
	4-27	D1-150	0	37.13	1468.72	Buckling of RS
	4-28	D1-150	0	42.04	1443.08	Buckling of RS
9	1-21	D1-150	0	40.46	1299.38	Use of smooth longer nails. Buckling of RS
	1-22	D1-100	0	46.15	1159.50	Use of smooth longer nails. No separation of BR from leading board
10	2-23	D1-150	25	55.12	1984.61	Metal plate at test frame lifted up..IS2/RS buckled
	2-24	D1-100	25	61.60	1929.61	IS2/RS buckled
	4-29	D1-150	25	54.11	2579.54	Buckling of RS
	4-30	D1-150	25	60.32	2275.63	Buckling of IS2 and RS
11	3-25	D1-100	0	19.37	879.93	Tear failure in OSB
12	3-26	D1-100	0	11.47	359.13	Tear failure in OSB

Appendix 6.2 Summary of test and test result of Mid-ply wall

							OSB/3	Studs (mm)					Rails (mm)		% difference	
Type	Wall no.	Ref code-nail spacing	Vertical load (kN)	Fmax (kN)	Stifness (N/mm)	Nails (mm)	Thickness (mm)	LS	IS1	MS	IS2	RS	TR	BR	Strength %	Stiffness %
1	1-1	D1-150	0	32.86	1180.42	3.35x65	11	38x89	38x89	38x89	38x89	38x89	38x89	38x89	65.99	60.37
	1-2	D1-100	0	41.70	1535.22	3.35x65	11	38x89	38x89	38x89	38x89	38x89	38x89	38x89	110.69	108.57
2	1-3	D1-150	0	21.16	1237.26	3.35x65	11	45x45	45x45	44x95	45x45	45x45	45x45	45x45	6.89	68.09
	1-4	D1-100	0	33.07	1567.44	3.1x75	11	45x45	45x45	44x95	45x45	45x45	45x45	45x45	67.08	112.95
3	1-5	D1-150	0	26.34	1473.78	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	45x45	33.09	100.23
	1-6	D1-100	0	29.13	1207.45	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	45x45	47.15	64.04
1	1-7	D1-150	0	36.33	1208.62	3.1x75	11	38x89	38x89	38x89	38x89	38x89	38x89	38x89	83.53	64.20
	1-8	D1-100	0	47.99	1761.01	3.1x75	11	38x89	38x89	38x89	38x89	38x89	38x89	38x89	142.45	139.25
2	1-9	D1-150	0	24.41	1711.93	3.1x75	11	45x45	45x45	44x95	45x45	45x45	45x45	45x45	23.33	132.58
4	1-10	D1-150	0	28.01	1372.96	3.1x75	9	45x45	45x45	45x45	45x45	45x45	45x45	45x45	41.50	86.53
	1-11	D1-100	0	31.15	1403.88	3.1x75	9	45x45	45x45	45x45	45x45	45x45	45x45	45x45	57.38	90.73
3	1-12	D1-100	0	34.48	1636.82	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	45x45	74.21	122.38
5	1-13	D1-100	0	43.60	1378.16	3.1x75	11	45x45	45x45	45x45	45x45	45x45	45x45	44x95	120.25	87.23
	1-14	D1-150	0	34.33	1310.79	3.1x75	11	45x45	45x45	45x45	45x45	45x45	45x45	44x95	73.45	78.08
6	1-15	D1-150	0	33.66	1470.45	3.1x75	11	45x45	45x45	45x45	45x45	44x95	45x45	44x95	70.03	99.77
	1-16	D1-100	0	40.11	1422.31	3.1x75	11	45x45	45x45	45x45	45x45	44x95	45x45	44x95	102.64	93.23
7	1-17	D1-150	0	27.51	1320.67	3.1x75	11	45x45	45x45	45x45	45x45	45x45	45x45	45x45	39.00	79.42
	1-18	D1-100	0	34.83	1367.31	3.1x75	11	45x45	45x45	45x45	45x45	45x45	45x45	45x45	75.94	85.76
8	1-19	D1-150	0	35.74	1487.24	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	80.56	102.05
	1-20	D1-100	0	44.94	1285.76	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	127.04	74.68
9	1-21	D1-150	0	40.46	1299.38	3.1x90	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	104.40	76.53
	1-22	D1-100	0	46.15	1159.50	3.1x90	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	133.14	57.53
10	2-23 (N) V	D1-150	25	55.12	1984.61	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	178.46	169.63
	2-24 V	D1-100	25	61.60	1929.61	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	211.18	162.15
11	3-25 D1	D1-100	0	19.37	879.93	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	-2.14	19.55
12	3-26 D2	D1-100	0	11.47	359.13	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	-42.06	-51.21
8	4-27	D1-150	0	37.13	1468.72	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	87.56	99.54
	4-28	D1-150	0	42.04	1443.08	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	112.38	96.06
10	4-29 V	D1-150	25	54.11	2579.54	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	173.38	250.45
	4-30 V	D1-150	25	60.32	2275.63	3.1x75	11	44x95	45x45	44x95	45x45	44x95	45x45	44x95	204.73	209.16
	0-1	SW-150 Datum	0	13.41	597	3.0x60	9	44x95							-32.25	-18.89
	0-2	SW-150	25	20.7	1060	3.0x60	9	44x95							4.58	44.01
	0-3	SW-150	0	12.76	650	3.0x60	12.5vb	38x89							-35.54	-11.69
	0-4	SW-150	25	20.8	1207	3.0x60	11	38x89							5.08	63.98
	0-5$	SW-150	0	13.10	417.17	3.0x60	9	44x95							-33.81	-43.32
	0-6$	SW-100	0	19.79	736.06	2.8x50	9	44x95							0.00	0.00
	0-7$	SW-50	0	23.14	1022.20	2.8x50	9	44x95							16.90	38.87
	0-8$	SW-150	25	20.70	1053.00	3.0x60	9	44x95							4.58	43.06
	0-9$	SW-100	25	29.19	1744.00	3.0x60	9	44x95							47.47	136.94
	0-10$	SW-50	25	43.15	2470.00	3.0x60	9	44x95							117.99	235.57
	0-11	SW-100	0	10.53	435.00	2.8x50	9	44x95							-46.80	-40.90
	0-12	SW-100	0	5.46	255.00	2.8x50	9	44x95							-72.42	-65.36

Appendix 6.3 Construction of Mid-ply walls in Heavy Structure Lab at Edinburgh Napier University

Appendix 6.4 Typical example of wall Type 1 with 150 mm nail spacing



Appendix 6.5 Typical set up of Mid-ply wall and their failure modes with lifting up of leading studs and intermediate studs

Appendix 6.6 Comparison between types of walls, nail types and nail spacing

Type	Wall no.	Fmax (kN)	Stifness (N/mm)	Nails (mm)	Nail spacing (mm)
Type 1 Smooth nail	1	32.86	1180.42	**3.35x65**	150
Type 1 Ring-shank	7	36.33	1208.62	3.1x75	150
Type 2 Ring-shank	9	24.41	1711.93	3.1x75	150
Type 3 Ring-shank	5	26.34	1473.78	3.1x75	150
Type 4 Ring-shank	10	28.01	1372.96	3.1x75	150
Type 5 Ring-shank	14	34.33	1310.79	3.1x75	150
Type 1 Smooth nail	2	41.70	1535.22	**3.35x65**	100
Type 1 Ring-shank	8	47.99	1761.01	3.1x75	100
Type 2 Ring-shank	4	33.07	1567.44	3.1x75	100
Type 3 Ring-shank	12	34.48	1636.82	3.1x75	100
Type 4 Ring-shank	11	31.15	1403.88	3.1x75	100
Type 5 Ring-shank	13	43.60	1378.16	3.1x75	100