OPTIMISATION OF TIMBER PLATFORM FRAME CONSTRUCTION

by

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A thesis submitted in partial fulfilment of the requirements of Napier University for the degree of Doctor of Philosophy

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DECLARATION

No portion of the work referred to in this thesis has been submitted in support of an application for another degree or qualification in this or any other University or institute of learning.

Robert Hairstans

SYNOPSIS

Title: **Optimisation of Timber Platform Frame Construction**

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Timber platform frame has evolved as an efficient method of construction for domestic dwellings and is experiencing continual growth in the UK due to it lending itself to off-site modern methods of construction (MMC), being environmentally efficient and exhibiting structural robustness. The challenge faced by the industry in the UK is to continue the evolutionary process such that the future demands of off-site MMC and regulatory changes are met.

By conducting a study of the development of timber platform frame construction and reviewing the current and future requirements of the domestic dwelling construction market the challenges for the industry were highlighted. The business drivers of a timber platform frame manufacturer were considered and in conjunction with the information from the review an agenda of research programmes was derived. The objective of the research, although primarily from a structural timber engineering perspective, was to address the challenges faced by the industry employing a holistic approach with a view to implementing applied research.

The UK procurement process for domestic dwelling construction is such that building layout is determined by architectural requirements. Building layout can have an adverse effect on structural stability and result in an inefficient system. A design review was conducted to determine the influencing factors which impinge upon system stability as a result of which recommendations for improvements were made. From the investigation the transfer of shear from a wall diaphragm to the foundation was deemed critical. Therefore, an experimental study was carried out which has resulted in an optimised specification. Further to this mathematical modelling techniques were used to demonstrate the impact that architectural layout has on stability, quantifying the financial penalty of inefficient layout and making recommendations to improve current designs.

One of major priorities of the UK Government is to reduce climate change by implementing a low carbon economy with sustainable production and consumption; all with duty of care towards natural resources. Improvements to the Building Regulations (2006), in conjunction with other requirements, will result in wall U-values in domestic dwellings to be between 0.27 to 0.30W/m^{2}K. To determine an efficient method of meeting the new regulations an all encompassing research programme was

conducted with the primary function being to develop a sustainable method of achieving thermal efficiency. Another method of wall construction is Structural Insulated Panels (SIPs) and this option was reviewed. Initial work by Kermani (2005) on the structural performance of SIPs was extended to examine their racking characterisitcs with comparative studies to European and British structural codes of practice carried out.

One of the key industry drivers which the review highlighted was the need for the implementation of lean technologies. The fabrication of flitch beams (timber-steel-timber sandwich configuration), used in cases of onerous load span conditions and limited depth of section, was improved through the implementation of a shot fired dowel connection method. To optimise the method of fabrication and achieve implementation an extensive laboratory study was carried out the results of which are compared to European structural codes of practice with recommendations made for design.

The implementation of off-site MMC methods results in a change in associated risk during construction from minor consequence and high risk to major consequence and low risk. The crane erect method of timber platform frame construction optimises on-site performance in terms of both time and cost and reduces the requirement of working at height, which on average causes almost one fatality every week. The biggest health and safety risk associated with the crane erect method is failure of the roof system when being lifted into position. Using an analytical model, verified by full scale laboratory testing, a range of lifting conditions were researched and a best practice lifting procedure was developed which allows the safe lifting of standard roof systems used in domestic dwelling construction.

ACKNOWLEDGEMENTS

First most and foremost thanks to Professor Abdy Kermani for help, support, guidance and knowledge. A gentleman and perfectionist I want to thank Abdy for his friendship.

Thank you to the workforce of Oregon Timber Frame, I hope that every piece of knowledge you have imparted has been considered in this work. In particular I would like to give thanks to the two men who were central to the initial Knowledge Transfer Partnership team, Mr Roderick Lawson and Mr Robin Dodyk two men I hold in the highest regard.

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Thank you to my family who have provided me with stability, advice and support over the years, trying my best to make you all proud.

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 General

Timber has been used since ancient times by mankind to provide protection and shelter. Its use in construction has evolved over the centuries and in particular it has met the demands of the volume housing market as a result of being readily available, easily worked and environmentally sustainable.

Originating in North America, timber platform frame has become a prominent method of domestic dwelling construction around the world primarily due to its efficiency. As a method of construction it has shown steady year on year growth in the UK and now accounts for 20% of the market. The growth of timber platform frame in the UK is due in part to its procurement and construction procedures being in line with the principles of the Construction Task Force Report (1998), its ability to conform to tighter building regulations and its environmental credentials.

The timber platform frame industry has encouraged partnering arrangements with both the private and public sector and as a result the construction process has improved making it faster and more efficient than other forms of domestic dwelling construction. Timber platform frame lends itself to off-site construction and there is now an accredited quality assurance scheme, Q-Mark (The UKTFA Quality Scheme) which covers design, manufacturing and erection. In addition to this a *timber frame erector* is now a recognised trade and the recently launched City & Guilds accredited training programme in the UK will further enhance its profile in the industry.

The benefits of off-site construction are mainly reduced time, cost and improved quality (Gibb and Isack, 2003) and this is reflected in timber platform frame construction. Generally the level of off-site construction of timber platform frame is currently the pre-assembly of wall diaphragms and floor cassettes. However, future advancements in this area would be the application of insulation, inclusion of services and installation of windows and doors resulting in a finished factory made pre-assembled component.

The purpose of this PhD research is to improve the competitive position, products and services provided by the timber platform frame manufacturing industry and in particular Oregon Timber Frame

Ltd through the implementation of applied research. Oregon Timber Frame Ltd design, manufacture and erect structural timber platform frames for the U.K housing market. The company commenced trading in February 1998 with 12 employees from a Jedburgh base and have since grown to over 90 employees and moved to a larger factory in Selkirk (March, 2006). Currently the company manufactures over 1600 units per annum for house building companies and social housing contractors in the UK with an expected turnover of £18million for 2007. The ethos within the organisation is to encourage house builders to specify sustainable and environmentally friendly construction methods, reduce accident frequency rates by utilising off-site construction methods, and increase efficiency and quality.

The research work conducted was initiated under a Knowledge Transfer Partnership (KTP). KTP is a UK government funded research programme to improve the competitiveness and productivity of an industrial partner through the better use of knowledge, technology and skills that reside within the UK knowledge base (DTI, 2005). This programme received a national award for excellence in 2005 from the Department of Trade and Industry (DTI). The knowledge base partner for the project was Napier University due to the level of expertise in the field of structural timber engineering held there and also the Universities track record of working successfully with industry.

1.2 Research Objectives

The project was core to the strategic development of the business, and although the main focus was from a structural timber engineering perspective, the objective of the project was to refine the whole process from design and manufacture to erection, leading to overall improvements in efficiency and cost.

The following were identified as key needs central to the on-going success of the business and as a result formed the nucleus of the research programme:

- Improved system efficiency and robustness by means of whole house engineering.
- Endorsement of regulation change and revised codes of practice.
- Reduce the environmental impact of the building envelope.
- Application of lean techniques to manufacturing procedures.
- Safe and robust implementation of Modern Methods of Construction (MMC).
- Improve and simplify the design, off-site and on-site processes through applied research.

From the outset it was understood that the research work conducted needed to take a holistic approach whereby an understanding of, and balance between, each business sector was required. From an initial study conducted the business was split into three sectors and from these the key drivers for departmental success were determined, as detailed in Table 1.1. In terms of an overall company perspective the function of the business is to be profitable. Therefore, although not shown in Table 1.1 specifically, financial implications were required to be considered. It is understood that added value can come at a cost, however, if the client is willing to pay a premium for the improved product or service then additional profit can be made.

Table 1.1 Business Sectors

Business sector	Key drivers	Explanation of drivers
Design	Added value	▪ Proving the added value of what is being done. ▪ Improving the value of a system or component without impinging on another business sector.
	Robustness	▪ Improving the robustness of the system without impinging on another business sector. ▪ Proving the robustness of improved products or services.
	Health & Safety	▪ Ensuring Health & Safety guidelines for all products and services are adhered to.
	Sustainability	▪ Reducing the environmental impact of all products and services.
Off-site	Health & Safety	▪ Ensuring Health & Safety guidelines for all products and services are adhered to.
	Manufacturing efficiency	• Implementing a lean strategy.
	Quality assurance	• Ensuring that quality is not reduced as a result of the endorsement of any new product or service.
On-site	Health & Safety	▪ Ensuring Health & Safety guidelines for all products and services are adhered to.
	Construction efficiency	▪ Improving the efficiency of erection procedures through the endorsement of Best Practice Procedures.
	Quality assurance	▪ Ensuring that quality is not reduced as a result of the endorsement of any new product or service.

1.3 Contents

The thesis is divided into eight chapters. Chapter 2 reviews the development of timber platform frame construction and following on from this the sequence of the thesis is such that the main impact of the research work conducted in each chapter corresponds to the three business sectors, Design, Off-site and On-site. Finally the conclusions of the thesis are drawn in Chapter 8.

Chapter 2 – Development of Timber Platform Frame Construction

The literature review charts the development of timber platform frame construction, it covers the influence that the material properties of timber have on design and how the use of timber as a structural material has evolved specifically in terms of domestic dwelling construction. Further to this Modern Method of Construction (MMC), regulation change and revised codes of practice are reviewed and the resulting future challenges to the timber platform frame industry are identified.

Chapter 3 – System Stability of Timber Platform Frames

A comparative study of typical UK timber platform frame houses is carried out in terms of system stability. The concepts of stiffness proportionality, redundancy, continuity and robustness are explored in relation to current UK timber platform frame design detailing. In particular the application of Eurocode 5 for the design of the sole plate to foundation connection is considered with guidance given to allow safe but economical design to be carried out. The findings of an extensive laboratory investigation into alternative methods of providing sole plate to foundation fixity are presented with recommendations made to improve the value and robustness of the connection system.

Chapter 4 – Design for Stability: Development of Semi-Empirical Models

The development of semi-empirical models which quantify the influence of building parameters, site location and wall detailing on stability are detailed. The developed models are then combined and used to measure the financial implications of building layout requirements and recommendations are made to improve system efficiency.

To improve the design procurement process a simplified method of design was deemed necessary to improve the capacity within the industry to provide initial design calculations. It is demonstrated that the developed models provide an efficient method of carrying out initial design whilst maintaining a degree of transparency. Further evidence of this is the use of the derived model in a simplified design technique for determining racking resistance requirements which has been published in "*The Scottish Buildings Standards Agency: Domestic Technical Handbook 2007*" (SBSA, 2007).

Chapter 5 – Wall Diaphragms

The first part of this chapter considers the impact of the EU Directive 2002/91/EC on the energy performance of buildings, which has the aim of promoting energy performance within the EU, on current timber frame construction in the UK. Detailed in this section is the derivation of optimum wall options giving due consideration to practicality, cost, sustainability and structural performance.

The second part of this chapter considers Structural Insulated Panels (SIPs) as an alternative to traditional timber frame wall panel construction. An overview of the benefits of SIPs is given and information from a research programme conducted at Napier University where the structural performance of wall systems constructed of SIPs was evaluated is presented.

Chapter 6 – Shot Fired Dowel Flitch Beams

The traditional method of flitch beam fabrication was deemed to be inefficient due to the time required for fabrication. As a result an improved lean technique of fabrication using a shot fired dowel connection was investigated. Laboratory testing of the connection method and of beams formed using the connection method were conducted considering different steel thickness and timber elements (solid section timber and timber composites). As a result of the testing the strength of this type of connection is quantified as well as the influence of the number of nails employed on beam strength. Further to this the study also investigated the level of strength capacity and stiffness achieved by flitch beams formed employing an optimum number of nails which subsequently resulted in a standardised nailing specification being implemented assisting both design and production.

Chapter 7 – Crane Erect of Timber Platform Frame Construction

A study of the crane erect method of construction which utilises on-site preparatory work and off-site fabrication is detailed in this chapter. The project planning alterations and implications which are required for crane erect construction to be successful, and the feasibility of crane erect in relation to improved time, cost and safety, are examined.

Of the crane erect method of construction the operation of lifting truss rafter roof systems was identified as the operation which had the highest associated risk. Therefore, it was deemed necessary to determine a best practice procedure for roof lifting. Presented in this chapter is the derivation of a computer model capable of analysing truss rafter roof systems under lifting conditions which, due to the nature of the support conditions, required to be verified by means of laboratory testing. The verified computer model was then used to derive two best practice procedures, the use of which depends on the level of system complication. Further to this the chapter also contains the information required to ensure that lifting operations are carried out safely.

Chapter 8 – Conclusions and Recommendations for Future Work

This chapter summaries the most important findings of the research programme and presents proposals for future work to be carried out.

CHAPTER 2

DEVELOPMENT OF TIMBER PLATFORM FRAME CONSTRUCTION

2.1 Timber as a Building Material

Wood is a natural, heterogeneous, anisotropic, hygroscopic composite material, (Smith et al, 2003). Its structural properties are highly variable as a result of a whole range of influencing factors. From growth to use, the structural properties of wood are affected. What has to be considered is the level of effect the influencing factors have in relation to the structural properties of the timber section being considered. If it can be considered negligible in the overall scale of investigation then it can be ignored.

The structure of timber can be considered in four levels:

1. Micro: Cell level (Figure 2.1a)
2. Messo: Growth ring level (Figure 2.1b)
3. Macro: Clear wood level
4. Massive: Sawn timber

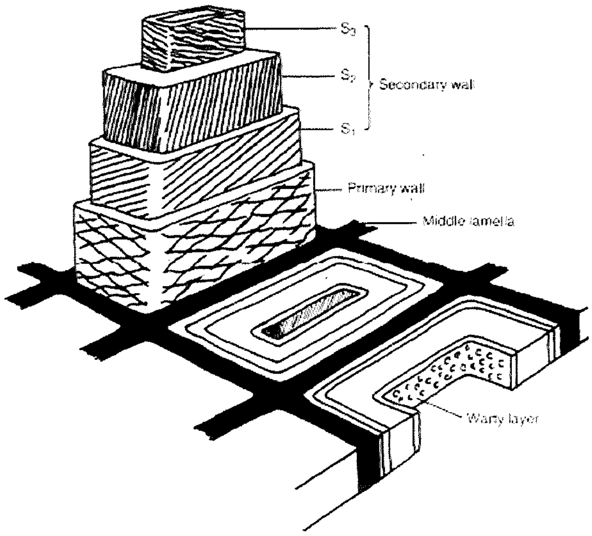
When considering timber as a building material although an appreciation of the four levels is beneficial it is the "Massive" structure which is most relevant. Massive wood means large dimension timber produced by sawing, generally the smallest cross-sectional dimension of which is more than 100mm, or a combination of solid wood members created by gluing together relatively small pieces of lumber (i.e. glulam). It is massive wood which is used in structural design and because of the inherent presence of imperfections; structural wood has lower strength than clear wood of the same species. Therefore, when designing with timber it is important to have an appreciation of what affects its strength.

Density

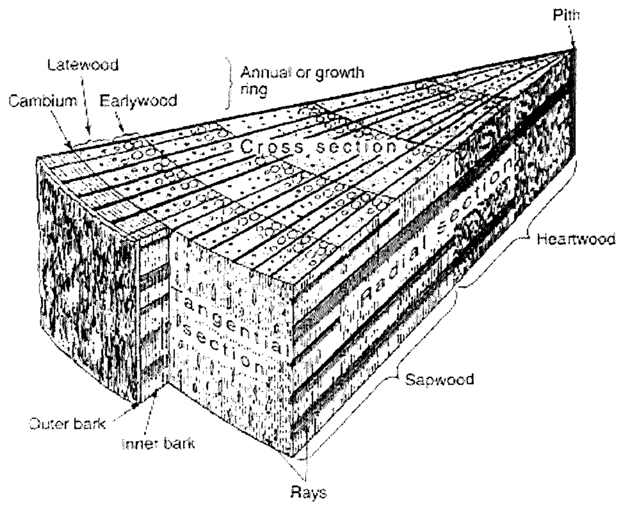
Density is considered the most important physical characteristic of timber. Most physical properties of timber are positively correlated to density as is the load carrying capacity of timber joints, (Hoffmeyer, 1995).

At a macro level, slow growth rate and consequently closeness of grain results in thick-walled tracheids which are tightly packed resulting in higher strength. The cell structure of softwoods consists of 90 to 95% tracheids, (Hoffmeyer, 1995), which are normally 25 to 45μm in diameter and 3 – 5mm in length. The tracheids are aligned radially, which results in softwood being strongly anisotropic in the cross-sectional plane, (Smith et al, 2003). The anisotropic nature is enhanced by the fact that cell wall thicknesses vary as do the size of cavities. The larger the cavity the better the cell is at conduction and conversely, the thicker the walls and the smaller the cavity, the less suitable the cell is for conduction, but the better it is at providing strength, (Desch, 1993).

A solid piece of wood, i.e. a piece only consisting of the cell wall material, where there are no cell wall cavities and intercellular spaces would have density of 1500kg/m^3. As a result of the different cell wall to air space ratio of timber, the density of wood ranges from 160 kg/m^3 to 1250 kg/m^3, (Desch, 1993). Porosity of the timber therefore determines the density and also gives an estimate of the amount of water which can be held.



a) Cell wall organisation of a mature tracheid (Eaton and Hale, 1993)



b) Diagrammatic representation of a wedge shaped segment cut from a five year old hardwood tree showing the principal structural features (Dinwoodie, 2000)

Figure 2.1 Cellular and structural features of timber

Moisture Content

Wood is hygroscopic and thus continually exchanges moisture with the surrounding atmosphere and therefore attains a moisture content which is in equilibrium with the water vapour conditions of the surrounding atmosphere: this moisture content is referred to as the equilibrium moisture content. Wood is also mechanosorbic and therefore its structural properties are influenced by the amount of moisture it holds.

Fibre saturation point is the point at which only bound water remains as the free water has been removed. Bound water is held within cell walls by hydrogen bonds and van der Waals forces. Free water exists in the cell lumen and cavities in liquid and/or vapour form. The removal of bound water requires much more energy than the removal of free water, (Smith et al, 2003).

When moisture is removed from the cell wall, timber shrinks. Shrinkage and swelling within the normal moisture range for timber structures are termed movements. The problems of dimensional movement of timber are resolved by using timber of a moisture content corresponding to the relative humidity of its environment, (Hoffmeyer, 1995).

Most mechanical properties of defect free wood improve with decreasing moisture content below the fibre saturation point. It is generally accepted that the overall increase in strength with reduction in moisture is because of the shortening and consequent strengthening of the hydrogen bonds linking together the microfibrils which make up the cell wall in three layers (s1, s2 and s3) as shown in Figure 2.1.

The overall relationship between mechanical properties and moisture content is not linear, but it is approximately so from 8 – 22% moisture content, (Smith et al, 2003). Table 2.1 shows the average changes in mechanical properties of clear wood due to one percent change in moisture content. Moisture variations above the fibre saturation point have no effect on mechanical properties, since such variations are related to free water.

Table 2.1 Approximate change (%) of clear wood properties for a one percentage change of moisture content (Hoffmeyer, 1995)

Property	Change (%)
Compressive strength parallel	5
Compressive strength perpendicular	5.5
Shear strength parallel	3
Modulus of rupture parallel	4
Modulus of elasticity parallel	2
Tension strength parallel	2.5
Tension strength perpendicular	1.5
Note: Properties at 12% moisture content form the datum.	

Temperature

At temperatures within the range +200°C to -200°C and at constant moisture content strength properties are linearly (or almost linearly) related to temperature, decreasing with increasing temperature, (Dinwoodie, 2000).

Time

Timber is a viscoelastic and rheological material. Viscoelasticity menas that the behaviour of the material is time-dependent; at any instant in time under load its performance will be a function of its past history, (Martensson, 2003). Rheological means that its behaviour is also a function of the thermal and moisture histories, and their interaction with the loading history, (Smith et al, 2003).

The modulus of rupture (maximum bending strength) will decrease in proportion, or nearly in proportion, to the logarithm of the time over which the load is applied; failure in this particular time-dependent mode is termed creep rupture or static fatigue (Dinwoodie, 2000).

Like some other materials, wood exhibits three creep phases under constant stress:

1. Primary creep is an initial phase during which the rate of deformation accumulation decreases with any increase in elapsed time.
2. Secondary creep is a phase during which the rate of deformation accumulation is constant.
3. Tertiary creep is a phase during which the rate of deformation accumulation increases with any increase in elapsed time.

Whether secondary or tertiary creep phases are entered depends upon the stress level and the elapsed time. At high enough stress and temperature levels, all three stages will occur, resulting in fracture of the material at the end of the tertiary stage, (Young et al, 1998).

Grain Deviation

Grain deviation is an important influencing factor on timber strength properties. In straight-grained timber, the fibres or tracheids are more-or-less parallel to the vertical axis of the tree. As a result of the tracheids being more-or-less parallel to the vertical axis the S2 layers of the tracheids, which is the thickest and most influential layer, are parallel to the vertical axis. The microfibrils of the S2 layer are therefore at a small angle to the vertical axis of the timber, as a result of the high level of anisotropy of timber this results in a high strength of timber in tension along the grain while a low strength perpendicular to the grain, approximately 48:1, (Dinwoodie, 2000).

Knots

There are two different types of knot to consider tight knots and loose knots. Tight knots are intergrown with the surrounding tree as a result of the girth of the trunk increasing and the successive growth rings forming over the stem and branches. If the limb dies or is broken off then subsequent growth rings added to the main stem simply surround the dead limb stub and the dead part of the stub becomes an encased knot. It is not intergrown and often has bark entrapped and is called a loose knot, (Hoffmeyer, 1995).

Knots have a negative effect on most mechanical properties of wood because they distort the flow of the grain. Consequently eccentricities inevitably develop in the flow of forces within components containing knots. Whatever the nominal stress condition for a timber component is there will be stress components perpendicular to grain and shear stress parallel to grain. How critical this is depends on the positioning of the knot(s) within a component, its size, soundness, and geometry, (Smith et al, 2003).

To assist in the process of design strength classes are used. A strength class system groups together grades and species with similar strength properties thus making them interchangeable. This then permits an engineer to specify a chosen strength class and the characteristic strength values of that class in design calculations.

Grading

Traditionally grading was by visual inspection and the most important strength determining factors were rate of growth, indicated by annual ring width, and the strength reducing factors such as knots, slope of grain, fissures, reaction wood, fungal and insect damage and mechanical damage. With the use of grading machines, used in North America, UK, Australia, New Zealand and Scandinavia since the 1960s (Johansson, 2003), it is possible to determine other characteristics such as bending modulus of elasticity, which are better correlated with strength properties (Glos, 1995). Shown in Table 2.2 are the characteristic values for some common strength classes of solid softwood:

- C16 – framing material for timber frame stud walls.
- C24 – commonly used as lintel material over openings.
- C27 – used to form timber trusses for housing.

Table 2.2 Characteristic values for some common strength classes of solid softwood (from BS EN 338)

Property	Symbol	Units	Strength class		
			C16	C24	C27
Characteristic bending strength,	$f_{m,k}$	N/mm²	16	24	27
Characteristic tensile strength parallel to the grain,	$f_{t,0,k}$		10	14	16
Characteristic tensile strength perpendicular to the grain,	$f_{t,90,k}$		0.5	0.5	0.6
Characteristic compressive strength along the grain,	$f_{c,0,k}$		17	21	22
Characteristic compressive strength perpendicular to grain,	$f_{c,90,k}$		2.2	2.5	2.6
Characteristic shear strength,	$f_{v,k}$		1.8	2.5	2.8
Mean value of modulus of elasticity parallel to the grain,	$E_{0,mean}$		8000	11000	11500
Fifth percentile value of modulus of elasticity,	$E_{0,05}$		5400	7400	7700
Mean value of modulus of elasticity perpendicular to the surface grain	$E_{90,mean}$		270	370	380
Mean value of shear modulus,	G_{mean}		500	690	720
Characteristic density,	ρ_k [a]	kg/m³	310	350	370
Mean density,	ρ_{mean} [b]		370	420	450

[a] Used for calculating the strength of mechanically fastened connections
[b] Used for calculating weight

2.2 Evolution of Timber Construction Forms, Products and Methods

Based on the definition that a tree is a *"woody plant growing on a single stem usually to a height of over two metres"* (Hunt, 1996) there are approximately 21,000 different species. A broad range of species diversity exists, but commercially they are divided into two categories softwoods and hardwoods. Softwoods are generally evergreen with needle-like leaves (which in biological terms are known as gymnosperms, plants bearing naked seeds) and include the spruces and pines etc. Hardwoods are generally broad-leaved (deciduous) trees that lose their leaves at the end of each growing season, birch, oak etc.

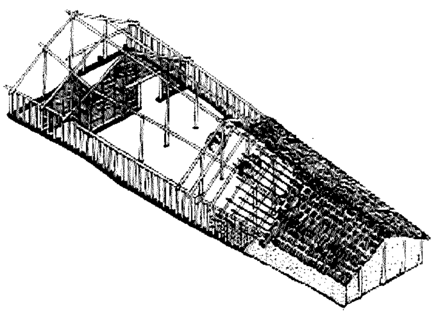
As a result of the diversity of trees most climatic zones have at least one species that has adapted to the prevailing conditions within that area. Timber is therefore available in most habitable regions of the world (Chilton, 1995) ranging from the Scots Pine in Scotland to the Kauri tree at the other side of the world in New Zealand (Figure 2.2).

a) Scots Pine (Scotland) b) Kauri tree (New Zealand)

Figure 2.2 Gymnosperms on either side of the world

Prior to understanding the micro or messo levels of timber the human race was using it in "construction" to provide a means of shelter and protection. Timber in many respects is the ideal construction material. It is strong in both tension and compression. It has a high strength to weight ratio and can be relatively easily worked. It is the only truly sustainable material with every cubic metre ot timber used in place of other building materials saving 0.8 tonnes of CO_2 from being released from the atmosphere (Harris, 2004). Trees can with good management improve land quality and soil fertility and are also a prime sink for carbon (carbon fixed from the atmosphere CO_2 by photosynthesis) (Stehn, 2002).

The climatic conditions and natural environment largely dictated the form and function of early timber structures. Early examples of timber structures from around the world are shown in Figure 2.3.



a) Long house, central Europe, 3000 BC.

b) Borgund church, Norway, twelfth century.

c) Three storey pagoda, Yakusiji Toto, built 730.

Figure 2.3 Early examples of timber structures (Thelanderson, 2003)

In the Far East the natural environment was one where seismic activity was prevalent and as a result heavy roof structures, supported on sophisticated frame systems, which originated in China, were favoured. Of the surviving wooden structures Japan contains the most and what has survived is timber frame systems joined by intricate brackets sets which provide great strength and stability.

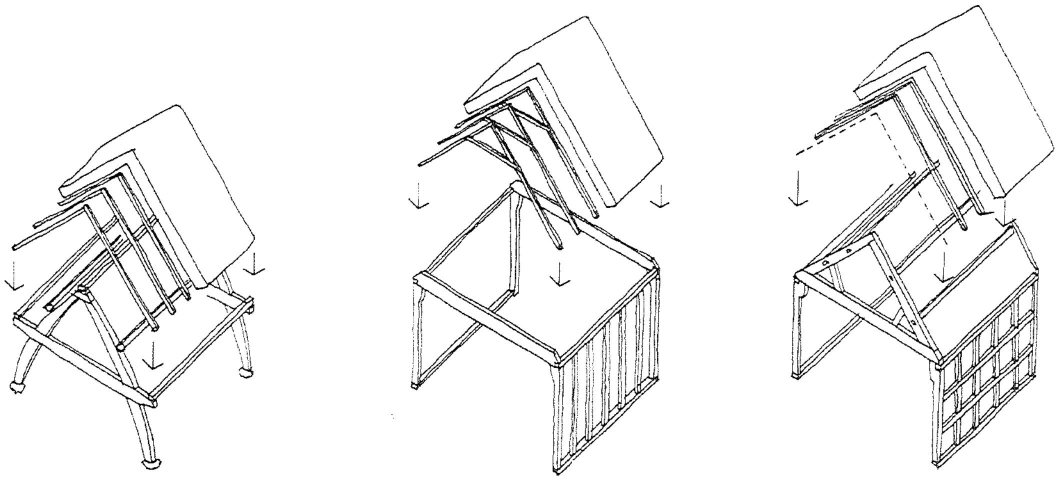
There is a ring of softwood forest which encircles the planet around the North Pole. Further south, and at lower altitudes, grow a variety of deciduous trees. Due to the abundance of resource in these regions there was no scarcity of material to build and the major traditional method was "log construction" (block work) examples of which are shown in Figure 2.4 (Pryce, 2005).

Figure 2.4 Pihlajavesi church, Central Lakeland, Finland, 1780 (Pryce, 2005)

There is evidence that block work was used in Britain, specifically Northern England, up until the sixteenth century. However, unlike the softwood timber of the major countries associated with block work construction, the hardwood timber of Britain would be unlikely to produce the long, uniform and only slightly tapered tree trunks even in primeval forests (Brunskill, 1994) which are conducive to the use of this method. As a result traditional timber construction in Britain, and in other parts of continental Europe, generally took on a different form of which there are three main categories (Figure 2.5):

- Cruck Construction
- Box Frame Construction
- Post & Truss Construction

Timber frame construction as we know it now really started to take shape in the New World during the nineteenth century. In North America the population was rapidly expanding and there was a need to rationalize the way houses were built to satisfy demand. The demand for housing coincided with the development of mechanised saw mills, which could produce accurate and small sizes of timber, and wire nails, which made redundant the requirement for intricate, hand crafted joints. As a result of this the skeletal frame work was born, not just in North America but also in the other major softwood producing countries of the world (Grimsdale, 1985).



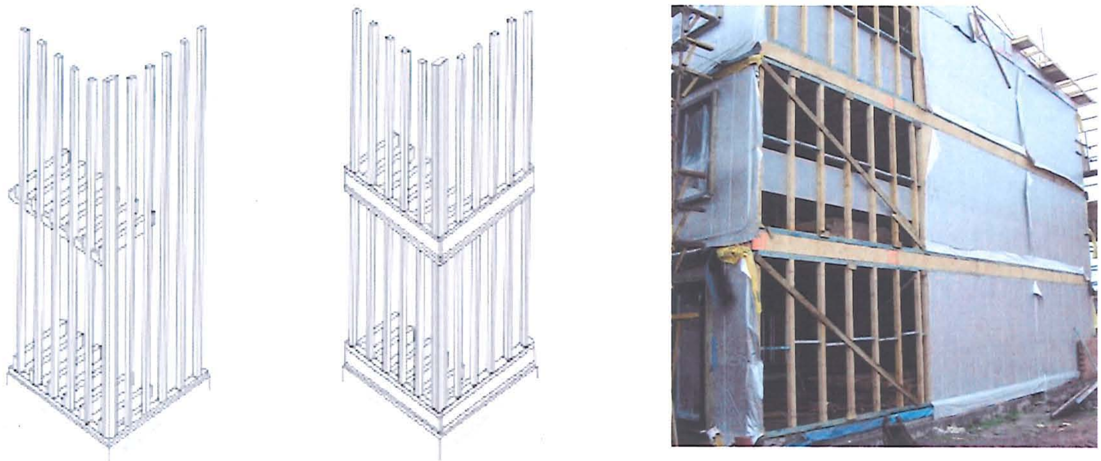
a) Cruck Construction b) Box Frame Construction c) Post & Truss Construction

Figure 2.5 Traditional methods of UK timber construction (Brunskill, 1994)

The pressures to build rapidly in North America resulted in the invention of 'Balloon Frame Construction', evidence of which can be traced back to methods employed by seventeenth century carpenters in Virginia. Balloon framing is 2- or 3-storey height timber framed and sheathed wall panels which act as vertical diaphragms and support roofs and floors acting as horizontal diaphragms (Figure 2.6).

Chicago gained a reputation for inventing the Balloon Frame. Chicagoan George W. Snow has been declared as the man who "*revolutionised construction practice*" with its invention in 1832. Chicago adopted this reputation due to the fact that factories there produced ready-made houses with balloon frames that were sold to various western cities attempting to meet the needs of rapidly expanding populations. Platform timber frame is a derivative of balloon framing and is most commonly used in Canada, the USA and the UK.

Similarities between North America and Australia can be drawn in terms of methods of construction and the reasons for it. In 1851 gold was discovered in Victoria and as a result the population of the colony swelled from 76,000 to 540,000 in three years. A large majority of the timber for the required houses was imported from the West Coast of America (often Oregon pine) and presumably with it the similar methods of timber frame construction used there (Pryce, 2005).

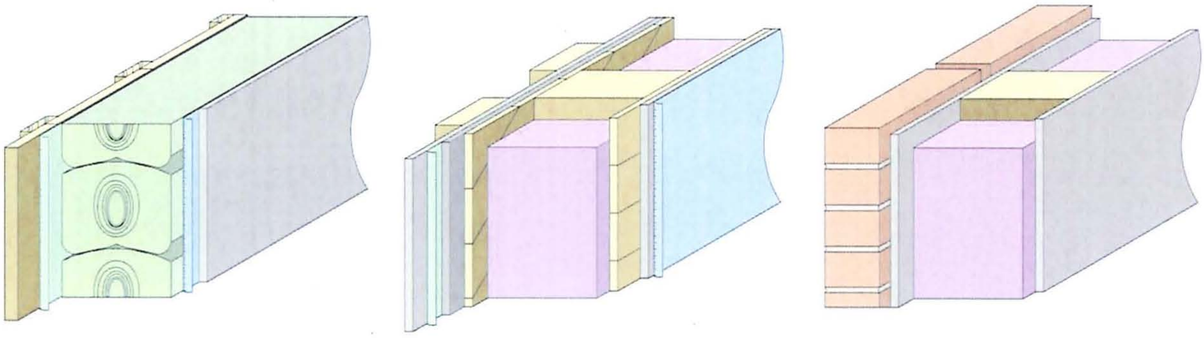


a) Balloon framing b) Platform framing c) Example of platform framing

Figure 2.6 Framing methods

Wood construction in Japan accounts for 40-50% of more than 1.2million housing starts per year (JAWIC, 2001). Annually Japan builds the second highest number of wood houses in the world the predominant form of which is Post and Beam accounts for 38% of the market (CINTRAFOR, 2001). However, timber frame methods of construction, first introduced in 1972 from North America (Cohen, 1994), now account for almost 15% of the Japanese market.

It was in the 1900's when lightweight timber frame construction was imported from North America to Finland. In the beginning the timber frame method became common very slowly with block work still the predominant method of construction as a result of readily available and relatively cheap logs. However, during the 1930's timber frame construction became the more dominant construction method in Finland as a result of it being a more efficient use of resources and less laborious. The adoption of timber frame construction in Finland resulted in an evolutionary process which by the 1980s had resulted in a method of construction which consisted of a timber wall with a brick façade and mineral wool insulation (Figure 2.7). This method of timber frame construction resembles in many respects UK timber frame wall construction today.



a) Log wall in the 1930s

b) Timber wall with saw dust insulation 1950s

c) Timber wall with brick façade and mineral wool insulation in 1970s and 1980s

Figure 2.7 Evolution of the timber wall in Finland (Heikkilä & Suikkari, 2001)

In the UK there are examples of modern type timber frame houses which date back 100 years. However, it was amendments to Building Regulations in April 1965 which limited the amount of energy that could be lost through certain elements of the fabric of the building (expressed as U-values – the amount of energy heat lost per square meter, for each degree of Celsius of temperature difference inside and outside) and a call for more houses to be built using less labour which stimulated the use of timber platform frame. Since 1965 growth has remained steady now accounting for 20% of all new housing (Scotland 65%, England 10.8%, Wales 10.9% & N. Ireland 7%) according to the UK Timber Frame Association (2005). Shown in Figure 2.8 is a standard UK timber platform frame house.

a) Under construction

b) Completed house

Figure 2.8 Standard two storey UK timber frame house

It is envisaged that the UK timber platform frame market will continue to grow as a result of its ability to deliver durable and sustainable housing at a fast rate. In the UK Housing has remained static for 5 years at 154,000 units and is at its lowest since 1924. The demand widens by 60,000 annually and the Government target is 219,000 new homes every year (UK Land directory, 2007).

The Government is also committed to developing a Code for Sustainable Homes the purpose of which is to reduce the environmental impact of the future housing stock. Domestic households currently account for 28 per cent of the UK's greenhouse gas emissions, more than half of the water consumed and ten per cent of all waste created (SERA, 2007).

2.3 Engineering Timber Products & Composites

Timber as a product and its uses have also changed over the centuries. Timber as a structural material has its limitations in its natural form due to availability; specifics of serviceability and dimensional restraint. The properties of timber vary from species to species, are dependent on the growth environment of the species and also vary across the structure of the species itself. Timbers with a cross-section of over 75x225mm and more than 5m long which can be used for structural purposes are at a cost premium. It therefore may be possible to specify a tropical hard wood for a specific design circumstance but the cost and availability make it unfeasible (Steer, 1995). The re-engineering of timber in the form of timber engineered products and timber composites has challenged these restrictions. Timber trusses (Figure 2.9) were the first engineered product which allowed timber structures to span beyond the limitations of 5 – 7m. It was the Romans who developed triangulated trusses, with spans up to 30m, for the roofs of their basilicas and these greatly influenced the form of medieval Italian and later European roof structures (Chilton, 1995). Trusses produced from structural timber are still the most commonly used method of roof construction in domestic dwellings. Modern truss systems are manufactured from sawn timber sections which are connected together at the node points by pressed metal plate connectors (Figure 2.10). Software packages are used to design the full roof system and also provide the information for cutting the timber and specifying the metal plates based on the load/span conditions of the particular design case.

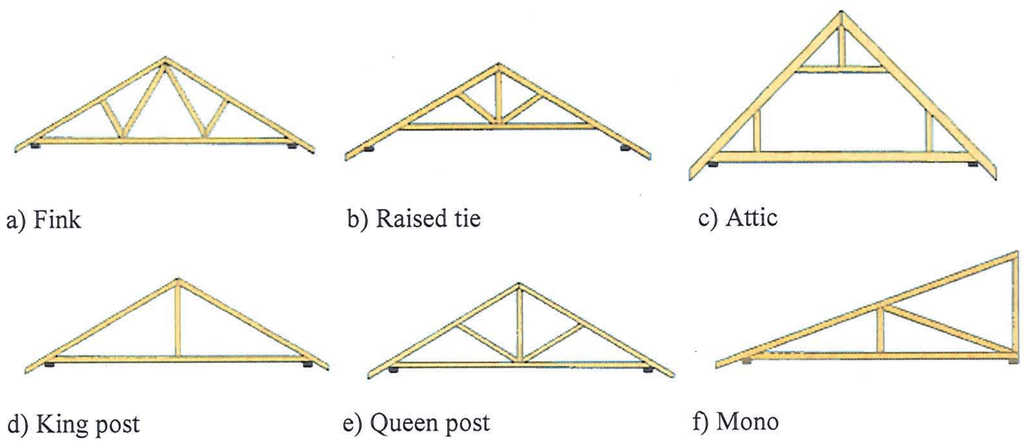


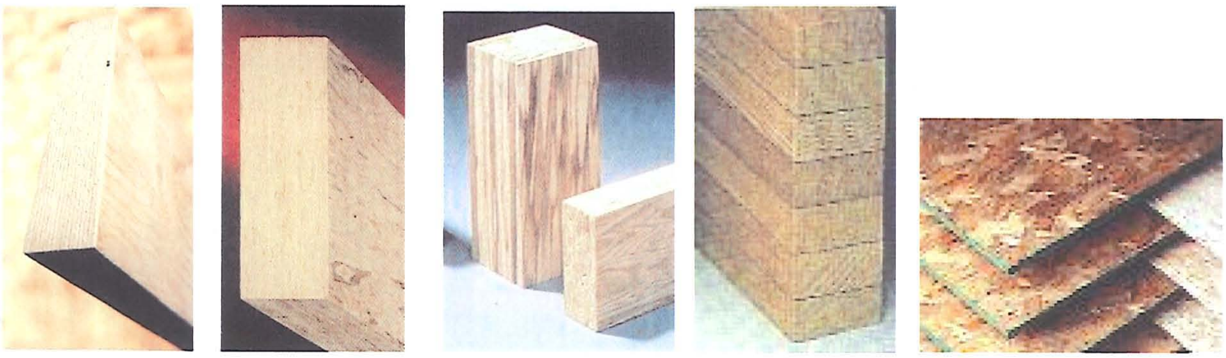
Figure 2.9 Trussed rafter configurations

There is evidence of timber composites dating back to the Egyptian Pharaohs with archaeologists having found traces of laminated wood in their tombs. However, plywood is widely regarded as the original engineered timber. The English and French are reported to have worked wood on the general principle of plywood in the 17th and 18th centuries but the industry was, according to the American Plywood Association (2007), "born" in 1905 when Gustav Carlson decided to laminate wood panels from a variety of Pacific Northwest softwoods called "3-ply veneer work". A further significant advancement in plywood was in 1934 when Dr. James Nevin, a chemist at Harbor Plywood Corporation in Aberdeen, Washington finally developed a fully waterproof adhesive and this subsequently opened up new markets for the product.

a) Placing of connection plate   b) Pressing of truss plate   c) Finished trusses

Figure 2.10 Fabrication of modern truss

Indeed it is the concept of plywood which forms the basis of most engineered timber composites which are manufactured from timber sections and reconstituted timber through adhesion. Timber composites overcome the dimensional limitations of sawn timber, improve performance, structural properties and stability, transform the natural orthotropic product into one with more homogenous properties and also optimise the use of a valuable resource whilst minimising waste (Mettem et al, 1996). Examples of timber composites which are used extensively in structural applications are glue laminated timber (glulam), laminated veneer timber (LVL), laminated strand lumber (LSL), parallel strand timber (PSL) and oriented strand board (OSB) (Figure 2.11). Depending on the country or region of use these products will have to comply with standards such that they can be specified in design by the engineer.



a) LVL b) LSL c) PSL d) Glulam e) OSB

Figure 2.11 Timber composites

Evidence of the use of timber in combination with another material was being explored in 1859 where at the Royal Arsenal; Woolwich (Desai, 2003) engineers were researching the possibilities of improving the structural properties of timber through re-engineering it in the form of a bolted timber-steel-timber sandwich construction, commonly known as flitch beams. Flitch beams (Figure 2.12) are still commonly used today, especially in housing construction, where load span conditions and limited depth of section dictate.

Figure 2.12 Fabrication of a flitch beam using laminated strand lumber

More recently improving the structural performance of timber by means of combining it with another material has been carried out using carbon fibre reinforced polymers. Gilfillan et al (2001) write that such a process can significantly enhance the performance of low grade timber in the form of Carbon Fibre Reinforced Wood (CFRWood). However, the financial viability of such a process is still in question and the use of such technologies is restricted to bespoke projects and the retro fitting of degraded timber elements in dilapidated structures.

An example of where reconstituted timber in combination with another material has been used for both improved building and structural performance is Structural Insulated Panels (SIPs). SIPs (Figure 2.13) are a sandwich construction of OSB and expanded polystyrene (EPS) to provide both structural support and insulation in a single system normally for wall and roof construction. The concept of SIPs began in 1935 at the Forest Products Laboratory (FPL) in Madison, Wisconsin. FPL engineers speculated that plywood and hardboard sheathing could take a portion of the structural load in wall applications.

Famed architect Frank Lloyd Wright used structural insulated panels in some of his affordable Usonian houses built throughout the 1930's and 1940's. SIPs took a major leap in technology when one of Wright's students, Alden B. Dow, son of the founder of Dow Chemical Company, created the first foam core SIP in 1952. By the 1960's rigid foam insulating products became readily available resulting in the production of structural insulated panels as they are known today (Structural Insulated Panel Association, 2007).

a) SIPs panels in a press

b) SIPs panels being erected

Figure 2.13 Structural Insulated Panels

Increasing the structural performance of timber by means of optimising the cross section, a concept explored by Victorian engineers to increase the structural performance of steel and iron by producing I- and box beam sections, was first researched between 1915 and 1942 in both North America and Europe. Subsequently timber I- and box-beam sections with plywood webs and solid timber flanges were widely used for aircraft and glider wing spars and struts.

Again the catalyst for mainstream use of timber, this time in the form of an engineered product, was rapid building needs. The requirement post war for schools, community centres and telephone exchanges was satisfied, with substantial volumes of industrialised building elements being produced each week by several British factory lines and many such elements remain satisfactorily in use to this day (Turnbull et al, 1998).

A modern I-beam (Figure 2.14) consists of LVL or high strength graded timber flanges and oriented strand board (OSB) or timber or metal truss webs (posi-joists). Timber I-beams are now extensively used for floor construction in the UK due to the advantages they offer over solid timber sections due to their span capabilities, light weight and ease of design and construction.

Table 2.3 is an example of the information, determined by testing, which is required to be supplied to allow the specification of LVL for a structural application.

a) OSB web b) Timber truss web c) Steel truss web

Figure 2.14 Examples of timber I-joist flooring systems

Major I-beam producers include Weyerhaeuser (USA), Finn Forest (Finland) and James Jones (Scotland) all of which supply the components for full floor systems using their specified products and software. As an example Weyerhaeuser supply a "silent" flooring system which consists of TJI joists and Timberstrand LSL rim board designed using TJ-Xpert® software. If the products and software of the company supplying the flooring system are not used in combination then the floor is not guaranteed by the joist supplier.

Table 2.3 Characteristic values for some common makes of LVL (from VTT Certificate no. 184/03 March 2004)

Symbol	Property	Units	Product		
			Kerto-S Thickness 21–90mm	Kerto-Q Thickness 21–24mm	Kerto-Q Thickness 27–29mm
	Bending strength	N/mm²			
$f_{m,0,edge,k}$	Edgewise		44.0	28.0	32.0
s	Size effect parameter		0.12	0.12	0.12
$f_{m,0,flat,k}$	Flatwise		50.0	32.0	36.0
	Tensile strength				
$f_{t,0,k}$	Parallel to grain		35.0	19.0	26.0
$f_{t,90,edge,k}$	Perpendicular to grain, edgewise		0.8	6.0	6.0
$f_{t,90,flat,k}$	Perpendicular to grain, flatwise		-	-	-
	Compressive strength				
$f_{c,0,k}$	Parallel to grain		35.0	19.0	26.0
$f_{c,90,edge,k}$	Perpendicular to grain, edgewise		3.4	9.0	9.0
$f_{c,90,flat,k}$	Perpendicular to grain, flatwise		1.7	1.7	1.7
	Shear strength				
$f_{v,0,edge,k}$	Edgewise		5.7	5.7	5.7
$f_{v,0,flat,k}$	Flatwise		4.4	1.3	1.3
	Modulus of elasticity				
$E_{0,k}$	Minimum, parallel to grain		11600	8300	8800
$E_{90,k}$	Minimum, perpendicular to grain		-	-	-
$E_{0,mean}$	Mean, parallel to grain		13500	10000	10500
$E_{90,mean}$	Mean, perpendicular to grain		-	-	-
	Shear modulus				
$G_{0,edge,k}$	Minimum, edgewise		400	400	400
$G_{0,flat,k}$	Minimum, flatwise		400	-	-
$G_{0,edge,mean}$	Mean, edgewise		600	600	600
$G_{0,flat,mean}$	Mean, flatwise		600	-	-
	Density	kg/m³			
ρ_k	Minimum		480	480	480
ρ_{mean}	Mean		510	510	510

2.4 Timber Platform Frame Construction

Timber platform frame construction for volume house builders is regarded as an "off-site" method of construction. Off-site construction has been defined by Goodier and Gibb (2002) as *"the manufacture and pre-assembly of components, elements or modules before installation into their final location"*. There is evidence of "off-site" construction throughout history and it has always been based on two key principles: efficiency and quality.

In the UK there is evidence of the concept from the medieval period where quality assurance procedures were in place. Medieval carpenters often fabricated the frames of buildings in their yards. The joints were created and the frames assembled to ensure that all the elements fitted accurately together. They were then taken apart (after all the joints were carefully marked with 'carpenter's marks'), transported to site and reassembled following the carpenter's marks.

The balloon framing techniques pioneered in North America were formed on the need for efficiency resulting in standardised materials and factory produced windows, doors and trim.

In the same way that timber frame construction advanced with the introduction of mechanised saw mills and wire nails modern methods of timber platform frame construction have evolved with the introduction of engineered timber products and composites and modern technology.

The majority of timber platform frame construction in the UK is now an integrated process from design to erection. At the design stage the roof and floor systems are designed using the software associated with the specified product. The structural frame work is designed by a structural engineer and subsequently drawings are produced of the frame using an AutoCAD based design package (Figure 2.15).

The timber frame system has a relatively high degree of compatibility and can be conceived as composite wall and floor units built up from timber framing, panel products, insulation and cladding. The composite units in a timber frame system can be utilised for the:

- Transfer of vertical loads
- Stabilisation of dynamic and seismic loads
- Physical separation
- Sound insulation
- Thermal insulation

The design is required to be made up such that all the relevant requirements are optimised. In terms of the design of walls and floors, different aspects can be identified as critical and are in order of priority (Thelandersson, 2003):

- Fire resistance
- Horizontal stabilisation
- Sound insulation
- Vertical loading

The roof trusses will be fabricated by the roof truss manufacturer and delivered to site for erection. Floor cassettes and open wall panels are fabricated off-site in a controlled factory environment (Figure 2.15). All the materials are cut from automated cutting lists, produced during the drawing process (CAD/CAM software) by using optimising saws to reduce waste. The fabrication of floor cassettes and wall panels is normally a manual process with production line procedures adopted for efficiency. As a result of the use of timber composites and engineered products in the manufacturing of wall panels and floor cassettes product dimensions are not restricted by available stock sizes but are dictated by transportation issues and the erection method.

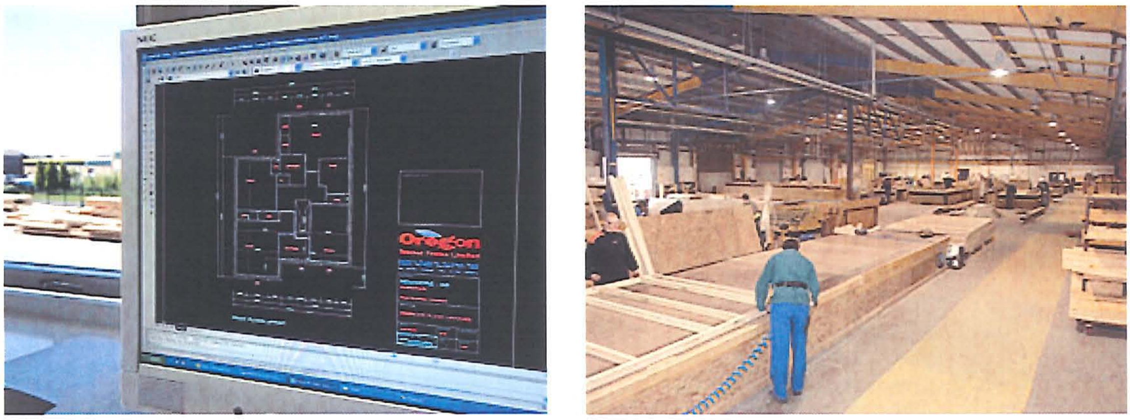
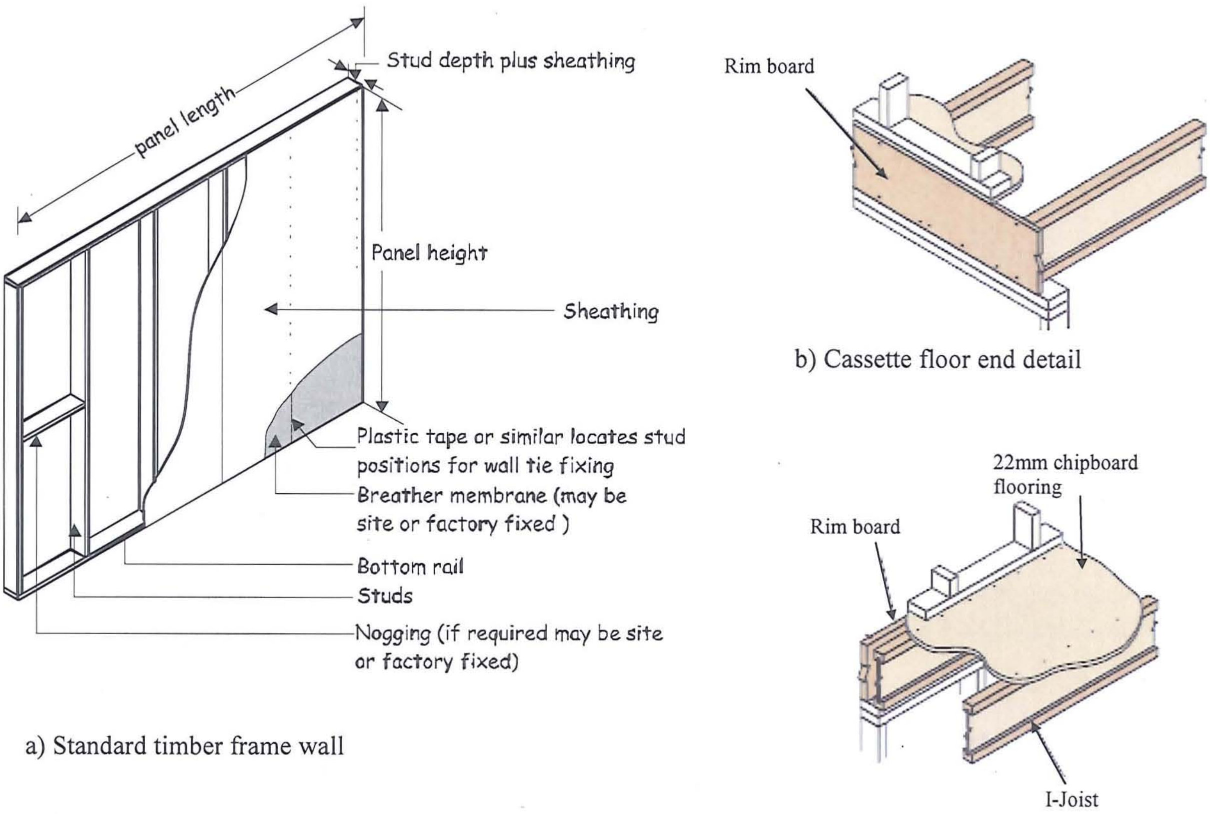


Figure 2.15 Timber frame detailing & off-site production

Standard factory produced structural wall panels (Figure 2.16) consist of sawn timber framing material (normally 38 or 45mm wide by 89, 95, 115 or 140mm deep C16 grade studs at 600mm centres) sheathed with single or double 9 or 11mm OSB (depending on the required structural performance).

The cassette flooring system (Figure 2.16) will normally consist of 240mm deep I-joists spaced at 600mm centres. The flange specification of the joists will be determined by the load span conditions and in certain design circumstances deeper I-joists at closer centres may be required.



a) Standard timber frame wall

b) Cassette floor end detail

c) Cassette floor edge detail

Figure 2.16 Standard wall and floor details

Wall panels and floor systems need to satisfy building performance issues (fire resistance, sound insulation and thermal performance). Table 2.4 and Table 2.5 show standard wall and floor specifications for a range of building performance ratings.

In terms of sustainability the thermal performance of the building envelope is important. The UK Government has prioritised reducing climate change and providing a low carbon economy with sustainable production and consumption; all with duty of care towards natural resources. In endorsing the EU directive on Energy Performance of Buildings (2002) the recent introduction of the revised Part L of the Building Regulations (2006) will lead to an improvement in the energy efficiency of buildings by around 20%. As a result the thermal transmittance values of both walls and floors will have to be improved.

Table 2.4 Illustrative specifications for load-bearing timber frame stud walls for particular performance requirements (IstructE, 2007)

Wall type	Minimum stud sizes (mm)	Plasterboard	Insulation thickness within frame	Fire resistance (minutes)	Thermal trans-mittance U (W/m²K)	Sound insulation R_w (dB)
External	38 x 89	1 layer 12.5mm Type A	90mm	30	0.35	50
External	38 x 140	1 layer 12.5mm Type A	140mm	30	0.31	55
Internal	38 x 63	1 layer 12.5mm Type A	65mm	30	N/A	40
Internal	44 x 75	1 layer 15mm Type D with staggered joints	None	30	N/A	43
Internal, party	38 x 89	1 layer 19mm + 1 layer 12.5mm Type A with staggered joints on each leaf	65mm in each frame	60	N/A	55

NOTES

- The values shown relate to walls insulated with Isowool, a glass mineral wool, and are reproduced by kind permission of the manufacturer. Insulating materials and plasterboards are produced in different densities, and the manufacturers should be consulted on the product types to which such test data apply. The insulant is laid directly on top of the plasterboard ceiling unless otherwise stated.
- External walls have 1 or 2 layers of plasterboard on the inner face internal walls have 1 or 2 layers on both faces. For thermal and sound values "External walls" assumes 100 mm brickwork or blockwork, a 50mm cavity, and 9mm thick plywood or OSB sheathing on the outer face.
- Minimum stud centres 600mm.
- Plasterboard types from BS EN 520
- Airborne sound. Improved sound performance can be obtained by the use of proprietary resilient bars.
- Type D is a high density, sound-insulating board.
- Two separate timber frames spaced 50 mm apart, consisting of timber studs with mid-height noggings. Two layers of plasterboard on the internal face of each frame.

Primarily the major advantage of timber frame construction when considering the building process relative to brick and block construction, also prevalent in the UK, is speed of construction. The removal of brick construction from the critical path can result in a wind and water tight building envelope in a matter of hours. To realise the full time saving potential of timber platform frame a method of construction has evolved which incorporates the off-site construction of the wall and floor components and the on-site preparatory construction of the roofing system at ground level (Figure 2.17), which in many respects is considered a Modern Method of Construction (MMC).

Table 2.5 Sample specifications for floor constructions for different performance requirements (IstructE, 2007)

Floor type	Minimum joist depth (mm)	Plasterboard	Insulation thickness	Fire resistance (minutes)	Sound insulation R_w (dB)
Intermediate floor, timber I-joists	240	1 layer 12.5mm Type A	65mm	30	40
Intermediate floor, solid timber joists	195	1 layer 12.5mm Type A	100mm	30	40
Separating floor, timber I-joists	240	1 layer 19mm Type A beneath walking surface. 1 layer of 19mm + 1 layer of 12.5mm Type A beneath the structural floor, staggered joints.	25mm laid on sub-deck between acoustic battens + 100mm laid on plaster-board ceiling.	60	50 airborne 53 impact

NOTES

- The illustrative performance values shown are specific to floors insulated with Isowool, a glass mineral wool. They are reproduced by kind permission of the manufacturer. The specifications of plasterboard and insulating materials can change, so always consult the product manufacturers for details of tested configurations relating to the materials currently manufactured.
- Gypsum plasterboard types from BS EN 520
- The insulant is laid directly on top of the plasterboard ceiling unless otherwise stated.
- Joists at 600mm centres screw fixed to 22mm t & g particleboard.
- Joists at 450mm centres screw fixed to 18mm t & g particleboard.
- Joists at 600mm centres. The separating floor consists of a floating floor on a structural floor. The floating floor consists of an 18 mm wood particleboard walking surface spot-bonded to 19mm Type A plasterboard supported on 70mm deep acoustic battens at 450mm centres fixed to a 15mm OSB sub-deck. The structural floor consists of the OSB sub-deck nailed or screwed to timber I-joists at 600 mm centres. The ceiling plasterboard is fixed to a resilient bar acoustic channel screw-fixed to the underside of the joists.

a) Wall panels b) Floor cassette b) Roof system

Figure 2.17 Crane erection of Off-site produced components and roof system

2.5 Modern Methods of Construction (MMC)

The report of the Construction Task Force to the Deputy Prime Minister on the scope for improving the quality and efficiency of UK construction was commissioned in 1998 as a result of concerns about the state of the industry in the UK. The overall concern was that a degenerative cycle was taking place: poor quality product was being delivered with a lack of efficiency, resulting in low profitability and subsequent reduced investment in research and development (Figure 2.19).

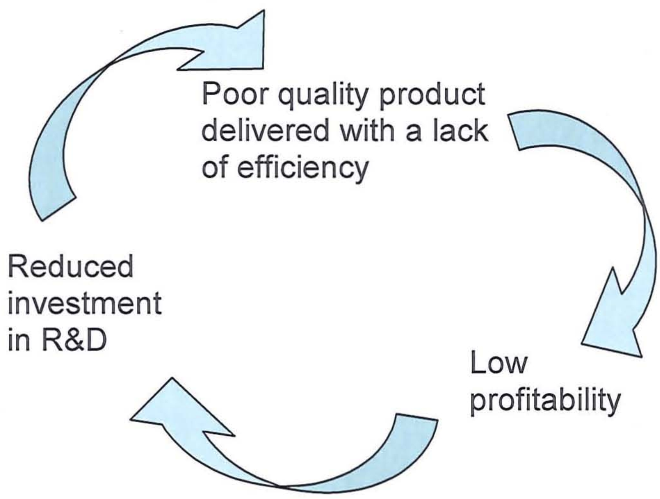


Figure 2.18 Degenerative cycle

The challenge set by the Task Force was not for the construction industry to improve its current methods but for the industry and the Government to join with major clients and do it entirely differently. In essence the proposal was to "rethink" construction.

The UK government has acknowledged that construction must be "re-thought" and that there should be a greater emphasis on off-site construction, particularly in the housing sector. Demand for housing in the UK is increasing over time driven primarily by demographic trends and rising incomes. Yet in 2001 the construction of new houses in the UK fell to its lowest level since the Second World War. Over the ten years to 2002, output of new homes was 12½ percent lower than for the previous ten years (Barker, 2004).

A UK Government briefing paper, Modern Methods of House Building (POST, 2003), identifies the need for a step change in the ability of the construction industry to meet the demand for 3 million new homes by 2016.

If the housing requirements are to be met at the expected standard the Task Force recommended to the house building industry, prior to the Barker report, the following key objectives:

- Targets for improvement, performance indicators, and arrangements for data collection, analysis and dissemination should be agreed upon.
- Principles should be established for commissioning and evaluating innovative demonstration projects and disseminating good practice.
- The procurement processes should be simplified, supply chains streamlined and components standardised.
- Long term partnering arrangements should be encouraged.

The Barker report made several recommendations to the UK Government to incentivise and improve the planning process so that it would be more streamlined and robust. The report also made several recommendations to the UK House Builders to improve their reputation and also to work in collaboration with other institutes to address the industry skills shortage, improve the vernacular appearance of new builds and endorse modern methods of construction (MMC).

Specific to MMC was recommendation 33 of the Barker report:

"The House Builders Federation, in conjunction with NHBC, Construction Skills and other interested parties, should develop a strategy to address barriers to Modern Methods of Construction (MMC). This strategy should be developed to fit alongside existing initiatives, working closely with Government to identify further measures that can be taken. A range of approaches should be explored, in particular actions by industry plus changes to policy / practice, as well as representations to Government on areas such as changes to Building Regulations."

Consequently the Barker 33 Cross-Industry Group was established in 2004 involving stakeholders from 50 separate organisations across the construction sector to examine the barriers to the greater use of MMC in the provision of new housing and the mechanisms to overcome them.

The Barker 33 Cross Industry Group defines MMC in the following context:

"Modern Methods of Construction are about better products and processes. They aim to improve business efficiency, quality, customer satisfaction, environmental performance, sustainability and the predictability of delivery timescales. Modern Methods of Construction are, therefore, more broadly

based than a particular focus on product. They engage people to seek improvement, through better processes, in the delivery and performance of construction."

According to the finding of the group implementation of MMC will be achieved through three goals:

1. Improve regulatory discipline.
2. Inspire product and process confidence through relevant and appropriate certification.
3. Exemplify benefits through practical (best practice) examples.

The UK government is committed to promoting the use of MMC in house building. In particular, the Office of the Deputy Prime Minister (ODPM) and the Housing Corporation spend £1.1 billion a year on building affordable housing using MMC, including £0.5 billion using off-site manufactured products (National Audit Office, 2005).

MMC is defined by the ODPM as being *"a process to produce more, better quality homes in less time"* of which four sectors have been derived:

1. Panelised units: produced in a factory and assembled on-site to produce a three dimensional structure. A spectrum exists starting at open panels which consist of a skeletal frame work only to advance panel systems which can incorporate lining material, insulation services, windows, doors, internal wall finishes and external cladding.
2. Volumetric construction: three dimensional modular units produced in factory conditions prior to transport to site.
3. Hybrid techniques: panellised and volumetric approaches are combined. An example of this is the use of volumetric units (also referred to as pods) being used for highly serviced and more repeatable areas such as kitchens and bathrooms, with the remainder of the dwelling or building constructed using panels.
4. Other: construction which may use floor or roof cassettes, pre-cast concrete foundation assemblies, preformed wiring looms, and mechanical engineering composites. Also included in this definition are innovative techniques such as thin-joint block work.

a) Steel skeletal system (Lawson et al, 2005)

b) 3 dimensional timber modular unit (Stehn, 2005)

c) Demonstration building using mixed steel panel and modular construction (Lawson et al, 2005)

d) Thin joint block work (Thermalite, 2005)

Figure 2.19 Different forms of Off-site MMC

The estimated UK capacity of the supply side in MMC is 30,000 – 40,000 housing units (in all materials), which represents 15 – 20% of current house building (Lawson et al, 2005) and accounts for approximately 2.1% of the total value of the construction sector, including new build, refurbishment and repair, and civil engineering (Goodier & Gibb, 2005). However, 64% of major industry housebuilders have indicated that the industry needs to increase the take-up of off-site MMC applications and 58% were planning to increase their use of off-site MMC (by volume) in the next three years (Pan et al, 2005).

The underlying economics of off-site MMC and modular construction in particular, is quite complex and requires significant production rate of repeatable components in order to be fully economic (Lawson et al, 2005). Modern methods of construction other than open panel techniques continue to be slightly more expensive than more established techniques and highly documented off-site MMC techniques (complete modular building, bathroom & toilet pods and flat pack, kitchen flat pack, off-site plant room and closed wall panels) are actually only applied to a very limited extent in housing

(Pan et al, 2005). However, higher buildings do favour volumetric MMC because construction costs rise faster for brick and block.

Other areas where a high degree of potential is perceived in off-site MMC are external walls, timber frame and roofs. The major industry drivers of which are, in order of importance: addressing skills shortages, ensuring time and cost certainty, achieving high quality and minimising on-site duration.

The removal of on-site activities to a factory environment will help to alleviate the current industry skills shortage by increasing productivity through the implementation of efficient manufacturing procedures. An example of which is the use of Enterprise Resource Planning (ERP) (a comprehensive planning mechanism which is supported by information technology based systems) which can be used to manage parts of or the whole supply chain (Crowley, 1998; Tarn et al., 2002; Al-Mashari., 2002).

MMC will also result in project time savings; if the process plans are tailored to match the method of construction. Open panel construction can for example reduce the duration of a project by up to 3 weeks (NAO, 2005).

The most significant barriers against the use of off-site MMC in the industry are higher capital cost, difficulty in achieving economies of scale, complex interfacing between systems, unable to freeze the design early on, the nature of the UK planning system and a significant level of poor perception.

Mitigating the major MMC project risks requires process discipline, good coordination and a culture that will not accept late changes (NAO, 2005). The main risks relative to MMC are identified as follows:

- *Late design changes*: difficult to absorb because factory work based on the design starts early. Well established partnering arrangements and close collaboration between parties is therefore good practice.
- *Loss of a factory production slot*: normally as a result of poor communication and can result in lengthy delays. An effective and robust communication mechanism is necessary. Standardisation of products can also result in increased flexibility.
- *Building tolerances and accuracy*: In particular foundations tolerances are required to be accurate. Products delivered to site have a high degree of dimensional accuracy and as a result reciprocal foundations tolerances are required.
- *Supply failure*: For the time saving of MMC to be realised the elements need to be supplied and erected on time. To ensure this happens it is important to implement procedures for the

management of materials, products and information flow, also known as supply chain management (SCM), (Tan et al., 1999).

In terms of product quality MMC can deliver at least as good a quality as more established building techniques, provided they are adequately specified (NAO, 2005). The report concluded that off-site manufacture may not guarantee enhanced durability greater than traditional construction methods but *"factory production should reduce the risk of non-conformities, related premature failures and consequent repairs which may be associated with on-site assembly"*.

Timber frame has evolved as a product as a result of new engineered timber products, timber composites and technologies. With both a government and industry led drive towards increased levels of off-site MMC timber frame construction will have to show an ability to increase levels of off-site fabrication mainly in terms of automation, application of insulation, inclusion of services and installation of windows and doors. This level of off-site construction is not unknown and there are examples of high levels of factory automation in the UK (Stewart Milne, Whitney) as well as insulated panel systems with factory fitted finishes (Space 4, Birmingham) (Figure 2.20).

The trend of increasing off-site MMC of timber frame houses is not restricted to the UK market. In the USA on-site construction is down to 69% from 90% 20 years ago, in Sweden 74% of one family detached houses were manufactured in a factory environment between 1990 and 2002 (Bergstrom & Stehn, 2005) and in Germany there is evidence that off-site manufacturing is on the increase (approximately 13% of the market share) although the overall market is considered to be in recession (DTI, 2004).

a) Stewart Milne: frame maker

b) Space 4: Phenolic foam insulated panel including doors & windows

Figure 2.20 Examples of off-site MMC in the UK

Although the level of timber frame off-site manufacturing is generally limited to the installation of insulation and fitted finishes (windows and doors) there is evidence from continental Europe of further advancement. In particular a recent government sponsored Global Watch Mission to Germany reported on high levels of automation being employed to produce timber frame housing with perceived future expansion.

The German system is typically a post-and-beam/closed panel hybrid system. The hybrid nature of the system gives designers increased flexibility, and the capability to produce closed panels with the inclusion of insulation, services, linings, windows, doors and cladding in the factory (although the majority of product leaving the factory is a basic frame with only linings, insulation and service conduits installed) (Figure 2.21).

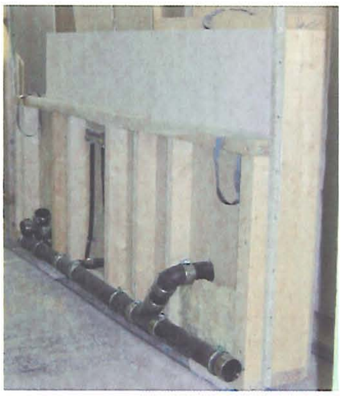
There are major differences between the UK and German Housing markets, with the German market being typical of a European model. The norm in the UK is to sell the package; house, land and location. In Germany firms sell only the house (via a show home park) with the land upon which to build the house sourced and purchased separately by the prospective homeowner. Statutory permissions and approvals together with any infrastructure costs are also the responsibility of the prospective homeowner. Therefore, the competitive position of the German firms is based on the house and associated customer related services (delivery process, maintenance etc). As a result there is a higher level of business incentive to invest in technologies and processes that will give their house an edge over competitors and the unique selling points to customers are normally in the form of incorporating new technologies.

Timber frame construction in the UK will continue to evolve as it has done throughout history. However, it does appear that the current procurement process is restrictive to the advancement of off-site MMC and not as customer centric as it could be.

The lean production concept is described by Bergstrom & Stehn (2005) as *"a holistic management philosophy, with product quality as the primary goal, which underlines the critical importance of employees, customers, improvements of the two main conversion processes, design and production, and elimination of all other activities, to achieve customisation of high volume products (Crowley, 1998; London and Kenley, 2001)"*.

The European model for procurement appears to result in a 'lean' production model (developed during the 1950s at the Japanese car manufacturer Toyota) which is conducive to value added end products and consumer confidence. A major change in UK procurement methods is not perceived therefore strong partnering arrangements between timber platform frame manufactures and house builders is required if future advancements in off-site MMC are to be realised.

a) Frame assembly (ExNorm factory)



b) Insulated sanitary ware being fitted

c) Toilet panel

d) Timber wall panel with external finishes (Elk Factory)

e) Bay window assembly

Figure 2.21 Examples of off-site MMC in Germany (DTI, 2004)

2.6 Structural Design of Timber Platform Frame

The principles of designing a timber structure are in essence similar to designing with any other material although due consideration should be given to the nature of the material itself and the properties it exhibits.

The main difference between current structural design of timber compared with steel and concrete in the UK is the basis of the code itself. Current UK structural timber design is in accordance with BS 5268-2(2002) code based on permissible stresses. Permissible (working) stress has been the basis for formal national timber codes for approximately 50 years in the UK (Bainbridge et al, 2002) and it is one in which safety factors are already incorporated into the tabulated material properties.

Steel and concrete, BS 5950-1(2000) and BS 8110-1(1997) respectively, are limit state codes of practice. Limit state codes are codes which link the structural reliability to clearly defined states beyond which the structure no longer satisfies specified performance criteria (Larsen, 1995).

By 2010 design procedures will be standardised across Europe through the adoption of the Eurocode Suite. The transition from current national codes of practice to one which covers all member states is to be a gradual implementation of change known as the harmonisation period.

The main objective of Eurocode is to facilitate further the free trade of construction products and services within Europe (Sousa, 1995). Design of Timber Structures will be in accordance with Eurocode 5 (EN 1995: 1-1) which is to be used in conjunction with Eurocode 0 (EN 1990:2002) Basis of Structural Design and Eurocode 1(EN 1991-1-1:2003) Actions on Structures. Eurocode is based on two types of limit state, ultimate limit state and serviceability limit state (Figure 2.22).

Ultimate limit states are those associated with the collapse or with other forms of structural failure. Ultimate limit states include: loss of equilibrium; failure through excessive deformations; transformation of the structure into a mechanism; rupture; loss of stability.

Serviceability limit states include: deformations which affect the appearance or the effective use of the structure; vibrations which cause discomfort to people or damage to the structure; damage (including cracking) which is likely to have an adverse effect on the durability of the structure.

a) Instance where serviceability limit state has been breached

b) Instance where ultimate limit state has been reached

Figure 2.22 Examples of instance where limit states have been breached

Limit state design provides the designer with more scope for design input. Current practice of permissible stress design is based on factored characteristic strength values. The incorporation of safety factors prior to actual design work limits the engineers input based on the particular design circumstance. An example of which is how the hygroscopic nature of timber is accounted for in Eurocode by assigning a service class depending on the moisture content of the surrounding atmosphere and its expected fluctuations. Allowing the designer to consider the service class for the design as appropriate will correlate the level of safety factor required for the given circumstance.

In theory the Eurocode should result in timber design which is economic, serviceable and ultimately safer. Although this is the aim of the Eurocode it has been queried in relation to its reliability. The Eurocode is a more rigorous design code and contains hundreds of design expressions for predicting the resistance of structural components. The expressions used are based primarily on test data and the level of accuracy of these expressions is based on the ability to correlate accurately against test data (Figure 2.23). As an example, partial safety factors on resistance, γ_M, factors, have the potential to affect significantly the economics of one construction material over another depending on the numerical value selected. Due to the inherent flaws in timber γ_M, is set at 1.3 which will prove to be onerous when considering high quality timber with low levels of imperfection compared to species of timber of low quality and high intensity of imperfections.

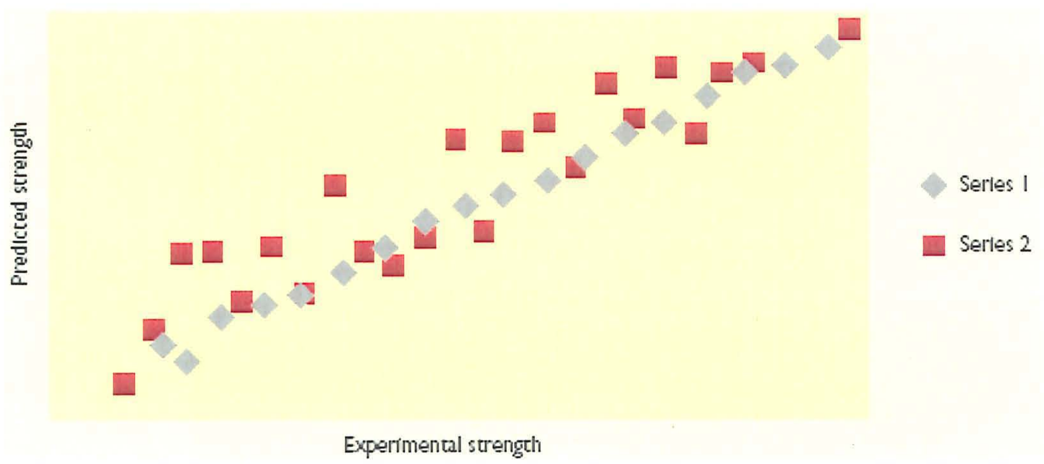


Figure 2.23 Comparison between poor and high quality design expressions (Byfield and Nethercot, 2001)

However, Eurocode 5 will facilitate a wider selection of materials and components and also provides more guidance on the design of built up components than BS 5268: Part 2. This will facilitate the incorporation in design of new engineered products and allow future products to be integrated for use (TRADA, 1994).

The procedure for designing a timber platform frame building system is shown in Figure 2.24. As explained previously it is normal practice for the floor and roof systems to be designed by the system suppliers using their accredited software packages and the structural frame to be designed by a structural engineer.

Further to the implementation of Eurocodes the structural design of timber platform frame in Scotland also has to conform to a new certification procedure. The Scheme for Certification of Design (Building Structure) was established as a result of a joint initiative by the Institution of Structural Engineers (IStructE) and the Institution of Civil Engineers (ICE) driven by the Building (Scotland) Act 2003 (SER, 2004). Principally the new legislation is aimed at improving assurances of structural safety by making a Certified Engineer responsible for ensuring that all aspect of design of the structure of a project satisfy the requirements of the Building (Scotland) Regulations 2004.

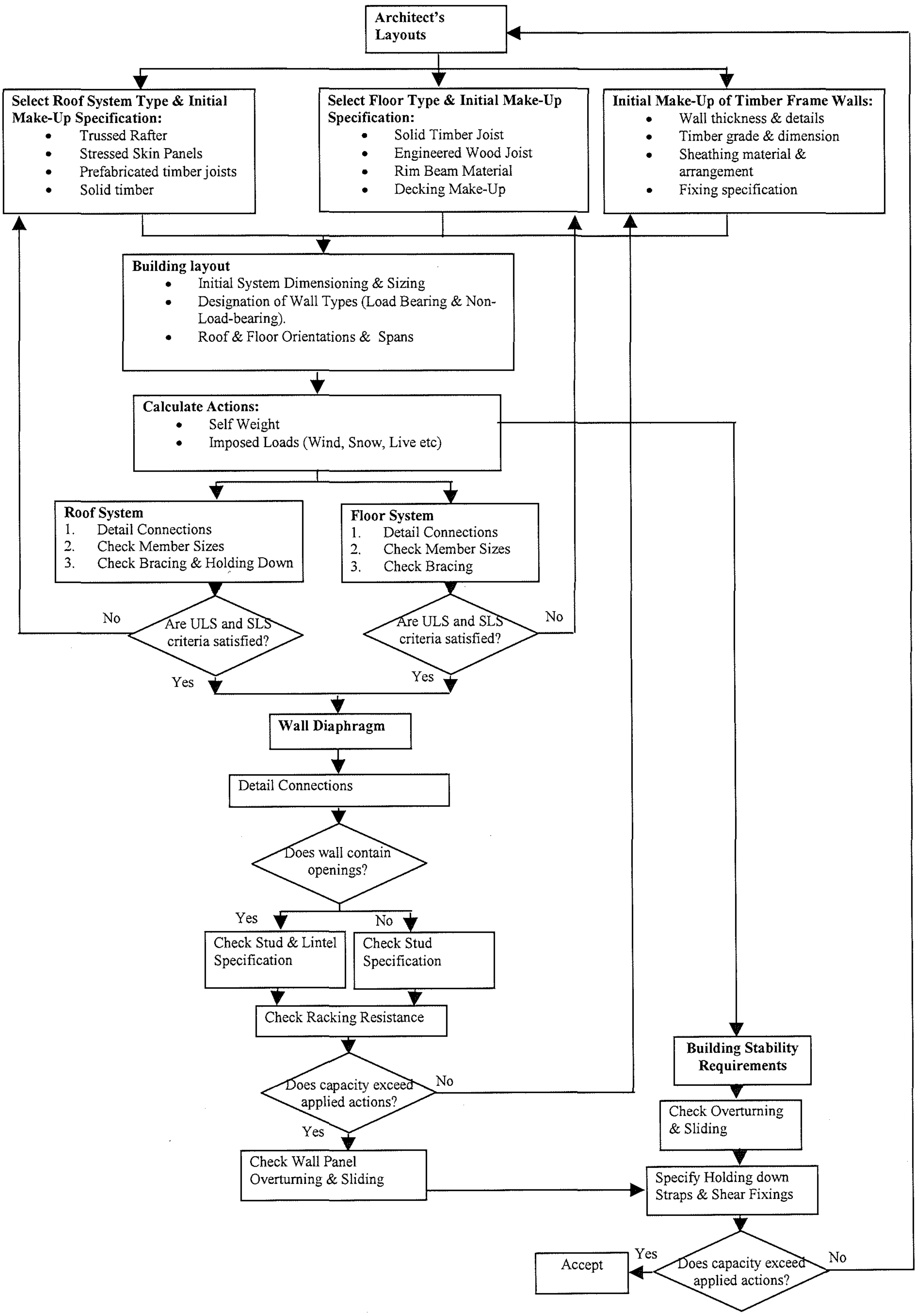


Figure 2.24 Flow chart illustrating platform timber frame design procedure (IstructE, 2007)

The fragmented structural design procurement process of platform timber frame (Figure 2.25) does not facilitate the certification process. However, as intended the new legislation will make engineers more aware of their responsibilities and duties. To ensure that the design process is safe robust communication streams are required and overall transparency needs to be exercised. Again partnering arrangements facilitate this process. Further to this there are also advances being made in CAD based whole house engineering packages which will result in fully collated engineered solutions. It is hoped that in the future this software will provide optimised design solutions ready for automated factory production.

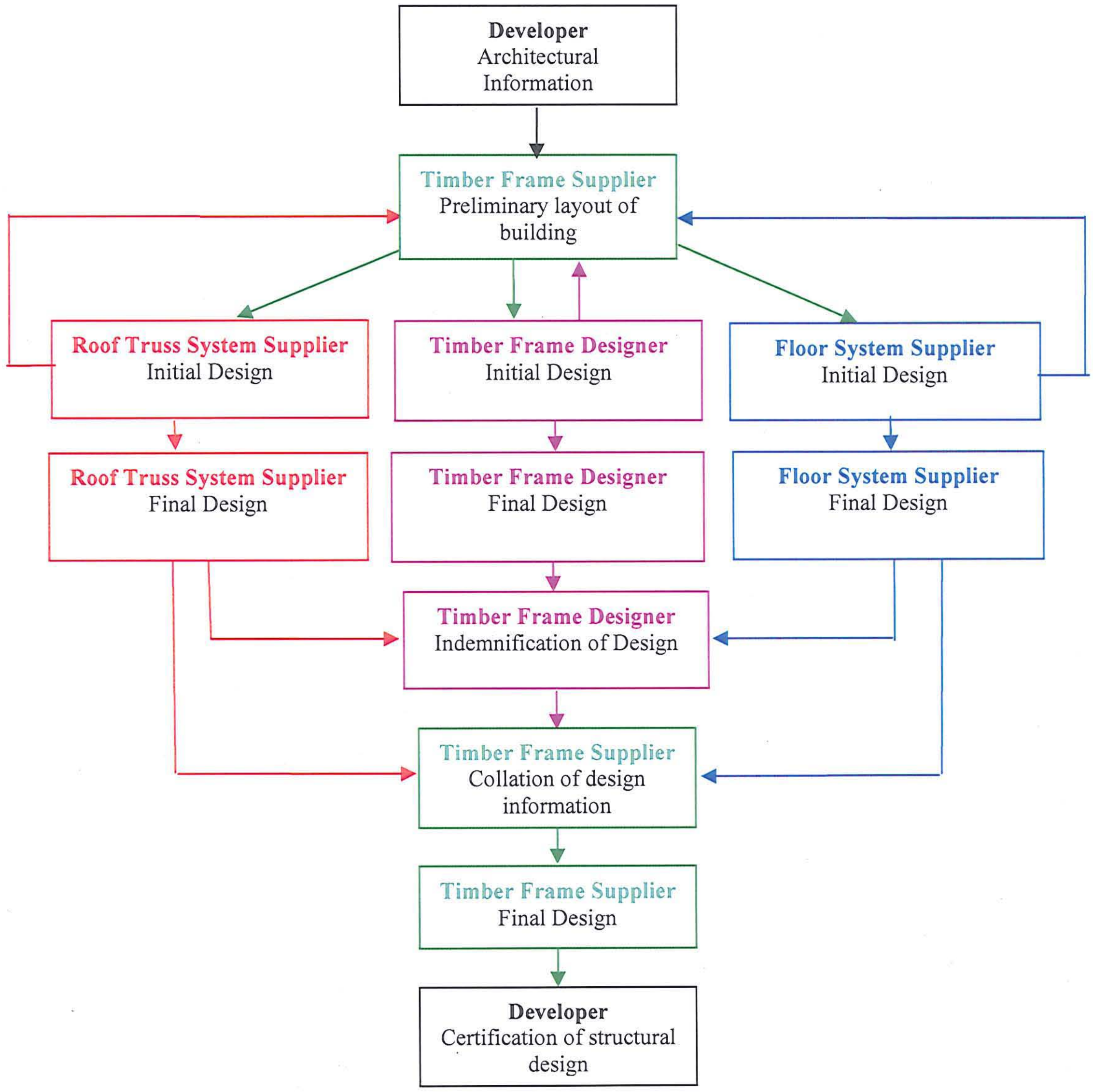


Figure 2.25 Structural design procurement process of a timber platform frame system

2.7 Summary & Research Purpose

The use of timber as a structural material, if appropriately sourced, is sustainable. Timber platform frame has evolved to be an off-site MMC due to the invention of engineered timber products and timber composites and the application of modern technologies.

If designed, detailed and erected properly it is an efficient way of providing a structurally robust system capable of longevity and meeting current building performance requirements (fire, sound and thermal).

The challenge faced by the timber platform frame industry in the UK is to continue the evolutionary process such that the future demands of off-site MMC and regulatory changes are met, these include:

- Meeting the UK Government sustainability agenda.
- Improved building performance and environmental efficiency through the optimum use of products.
- Increasing levels of off-site activity and employing automation where appropriate to alleviate an industry skills shortage.
- Eliminate client scepticism in terms of product quality and time and cost certainty.
- Providing a service and a commodity.
- Global education such that appreciation of the product, how it can be used and what it can achieve are understood at all levels.
- Improving the interfacing between systems to allow ease of construction.
- Endorsing more 'lean' techniques such that greater possibilities for variation exist without impinging upon product quality and cost.
- Building upon and improving partnership arrangements with builders to improve the procurement process.
- Harmonising with new European Structural Codes of practice which are a step change from current Permissible Stress Design to Limit State Design.
- Improving the design procurement process in order that the collation and dissemination of information is efficient so that final designs are robust, safe and serviceable.

The purpose of this research work is to improve the strategic position of the timber platform frame industry and in particular Oregon Timber Frame Ltd by way of implementing applied research. To achieve this objective it is clear the research work conducted requires not only to embrace the company ethos but also tackle the challenges faced by the industry as a whole.

The procurement process of residential construction is such that the timber platform frame normally has to fit to the architectural layout requirements. This often results in inefficient design which relates mostly to building stability issues. To improve the structural robustness of the system by means of improved detailing an audit of current designs is required. The audit should optimise and standardise specifications, provide a body of information for dissemination to exterior consultant engineers and be used for training purposes. Further to this the imbalance between the architecturally required building layout (often high levels of opening in narrow building frontages) and available engineering solutions needs to be addressed. A method of demonstrating what can be structurally achieved in terms of building layout is required to be communicated to improve the efficiency of designs or a mechanism needs to be formed which makes the tendering process more selective.

The extent to which new European environmental legislative requirements will impact upon the industry, and what effect they will have on Oregon Timber Frame Ltd, need to be quantified. The research work conducted should improve the existing product so that new legislative requirements are met. Further to this other available products in the industry are to be researched so that the threat they pose is quantified.

The manufacturing process should be reviewed and where possible new engineered products or technologies should be considered for implementation. The research work conducted will have to quantify the feasibility of the new product or technology and also provide all the relevant information required for implementation.

Oregon Timber Frame Ltd provides a service as well as a commodity. One of the most important services provided by Oregon Timber Frame Ltd is the on-site erection of the timber platform frame system. To improve the efficiency of the on-site process research is required to ensure that the methods employed are of best practice. There is an element of client scepticism towards the crane erect method of construction and as a result the research work conducted should quantify the benefits of committing to this modern method of construction and provide relevant information to ensure the process is safe.

The objectives of the research work are as follows:

- Improve the structural robustness of timber platform frame systems.
- Harmonise current designs with new European Codes of Practice and Legaslative Requirements.
- Reduce the environmental impact of the building envelope.
- Implement lean techniques into the manufacturing process.

- Implement Modern Methods of Construction safely and robustly.
- Use applied research to improve and simplify the design, off-site and on-site processes.

CHAPTER 3

STRUCTURAL STABILITY OF TIMBER PLATFORM FRAME

3.1 Introduction

The stability of timber platform frame systems is considered in this chapter with particular attention to the transmission of applied shear forces to the foundation. The concepts of stiffness proportionality, redundancy, continuity and robustness are explored by means of a comparative study of typical UK timber platform frame domestic dwellings. The objective of carrying out the work documented was to review the influencing factors in design which impinge upon system stability and make recommendations for improvements.

In particular industry standard shear fixings are considered for the transfer of shear forces from the sole plate to the foundation of the building. An experimental programme is reported on which quantifies the properties of the fixings for use in Eurocode 5 design procedures and the lateral load carrying capacity of the fixings when employed as a timber to concrete connection method. Further to this the results from the laboratory tests carried out have been used in conjunction with material cost information to optimise the structural specification with the economic cost.

3.2 General

A timber platform frame building is subjected not only to vertical loadings, such as self weight and imposed load, but also horizontal loadings caused by winds or earthquakes. In the UK earthquakes are not normally experienced at a level high enough such that they impinge on structural design.

Wind has a number of effects on a building. Its direct action is to cause pressure on one or more of the faces and suction on the others. In addition to the principal wind loads, the wind may also cause suction or pressure on the inner faces of the building (Alsmarker, 1995).

The method of determining wind actions on buildings in accordance with the Eurocode suite will be to use BS EN 1991-1-4:2005 Eurocode 1: Actions on Structures - Part 1-4: General actions – Wind. However, at the time of writing the UK National Annex for the Eurocode had not been finalised. The current method for determining the wind loads on building in the UK is in accordance with BS 6399-2:1997 Loading for buildings - Part 2: Code of practice for wind loads. The basic wind speed map of the UK is shown in Figure 3.1.

The principles of timber platform frame design are such that it is normal to consider system stability in two parts:

1. Overall system resistance to sliding and overturning as a result of the applied wind action: Since timber platform frame buildings are relatively lightweight, it is necessary to verify their overall stability under wind loading with respect to overturning, sliding and roof uplift, both during the execution phase and after completion. During the execution phase the weight of the roof tiles should be excluded. For the majority of circumstances the self weight of the system results in a holding down moment and, as a result of friction, a resistance to sliding, both of which are greater than the applied overturning and sliding forces. A point for further consideration is the common practice of levelling the sole plate due to poor foundation tolerances by inserting proprietary plastic shims, this reduces frictional resistance to sliding to an unknown level and as a result additional resistance to sliding may require to be specified.

2. The transmission of applied shear to the foundation: Applied wind loading on a building is transferred to the foundations by diaphragm action (Figure 3.2). The side walls, considered to be simply supported at roof and foundation, transfer one half the total wind load 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 foundation via shear connections and holding down straps.

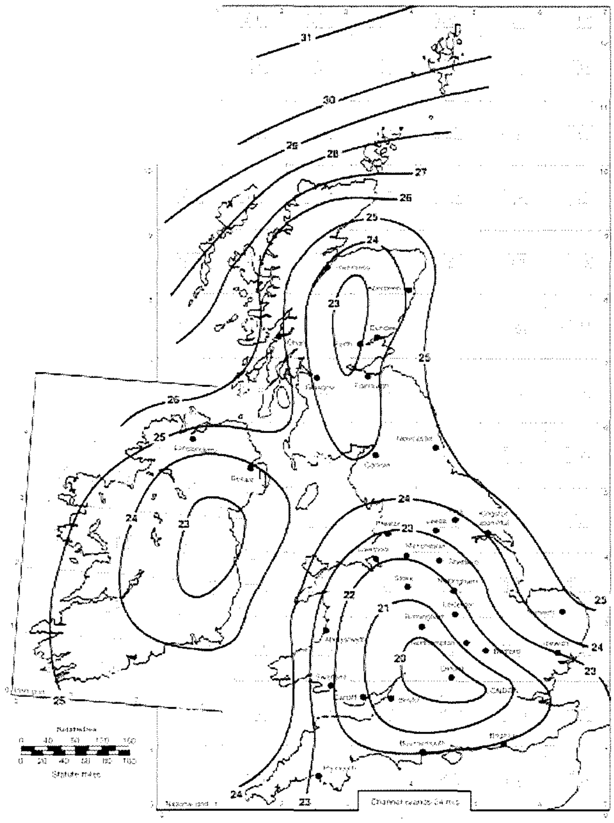


Figure 3.1 Basic wind speed *Vb* (m/s)

A shear wall in UK timber platform frame normally consist of vertical studs (normally 38 or 45mm deep by 89, 115 or 140mm wide) at 600mm centres single or double sheathed with 9 or 11mm OSB with an internal facing of 12.5mm plasterboard with an overall height of 2.4m. From a structural perspective the wall can be regarded as a cantilevered diaphragm loaded at the top plate (Figure 3.3). Using the sheathing as a bracing the applied force is transferred to the foundation in a very effective manner (Alsmarker, 1995). The sheathing will be either nailed or screwed to the frame and the type and level of fixing is of primary importance as it transfers the racking load to the sheathing. Since the connections between framing members are nominal at best, the sheathing connectors also play a crucial role in transmitting loads between framing members.

The purpose of this section of the thesis is to evaluate how different design assumptions and engineering judgements can affect the overall design of timber platform frame systems in relation to stability and as a result make recommendations to improve system design and performance.

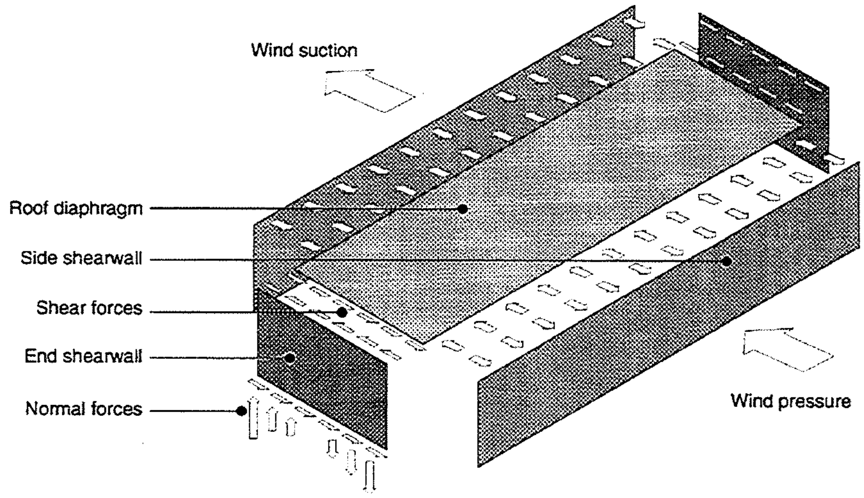
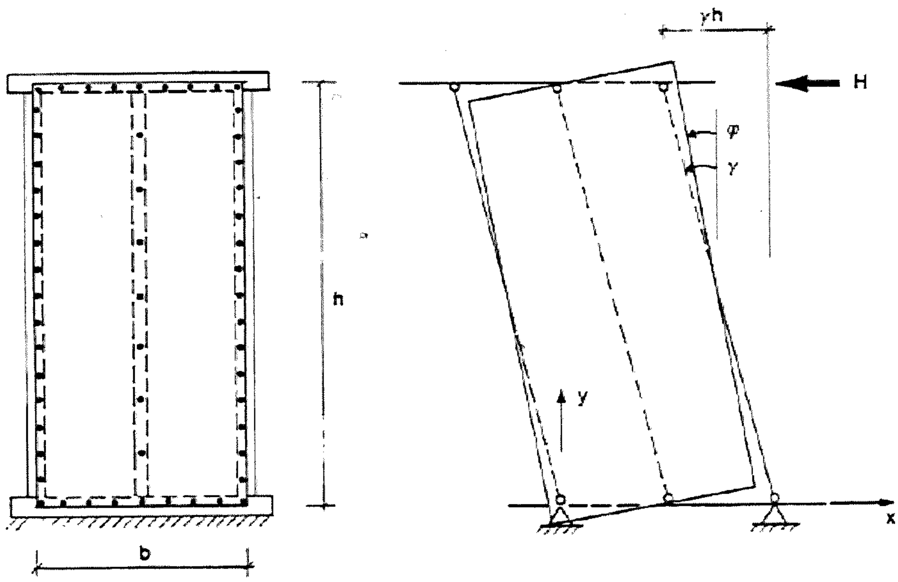


Figure 3.2 Shear wall and diaphragm action (Prion and Lam, 2003)



a) Typical wall unit b) Principal structural behaviour

Figure 3.3 Timber frame shear wall (Alsmarker, 1995)

3.3 Comparative Study

A comparative study of 3 different 2 storey timber platform frame design cases (Figure 3.4), in relation to shear transmission, has been carried out. Each design case is a representative example taken from the UK timber platform frame industry. As a result of having to use BS 6399-2:1997 to determine the wind loading for each case, good practice dictates that design of the timber frame racking walls is carried out in accordance with BS 5268: Section 6.1:1996. Externally the systems are masonry clad (this is standard practice in the UK) with the masonry tied to the timber frame with standard wall ties. Both testing and experience in the UK have demonstrated that within certain limits masonry walls will reduce the wind load onto the timber frame of buildings (IstructE, 2007 and Robertson and Griffiths, 1981). The resulting reduced wind load is considered to act uniformly over the entire area of the adjacent timber frame wall.

The roof systems consist of fink roof trusses braced in accordance with BS 5268-3:1998. The floor diaphragms for the design cases considered are constructed from I joists decked on top with 22mm chip board flooring (glued and screwed) and on the underside with a 13mm plasterboard ceiling (screwed).

The site location and building orientation is the same for all three design cases and as a result the applied wind action is consistent. However, it is to be noted that the height to ridge of Design Case 1 is 8.9m and that the pitch of the roof is 40 degrees spanning front to back. Design Cases 2 and 3 have an overall height to ridge of 7.4m and the pitch of the roof is 35 degrees spanning each individual unit, wall 1 to wall a, wall a to wall b and so on.

The timber frame wall diaphragms have an overall height of 2400mm; consisting of 45x95mm grade C16 timbers with studs at 600mm centres. The walls are sheathed internally and externally as designated in Table 3.1 and the level of opening of the external walls is given in Table 3.2. Sheathing is fixed using 3mm diameter by 50mm long galvanised wire nails at 100mm centres to external framing members and 200mm centres to internal framing members.

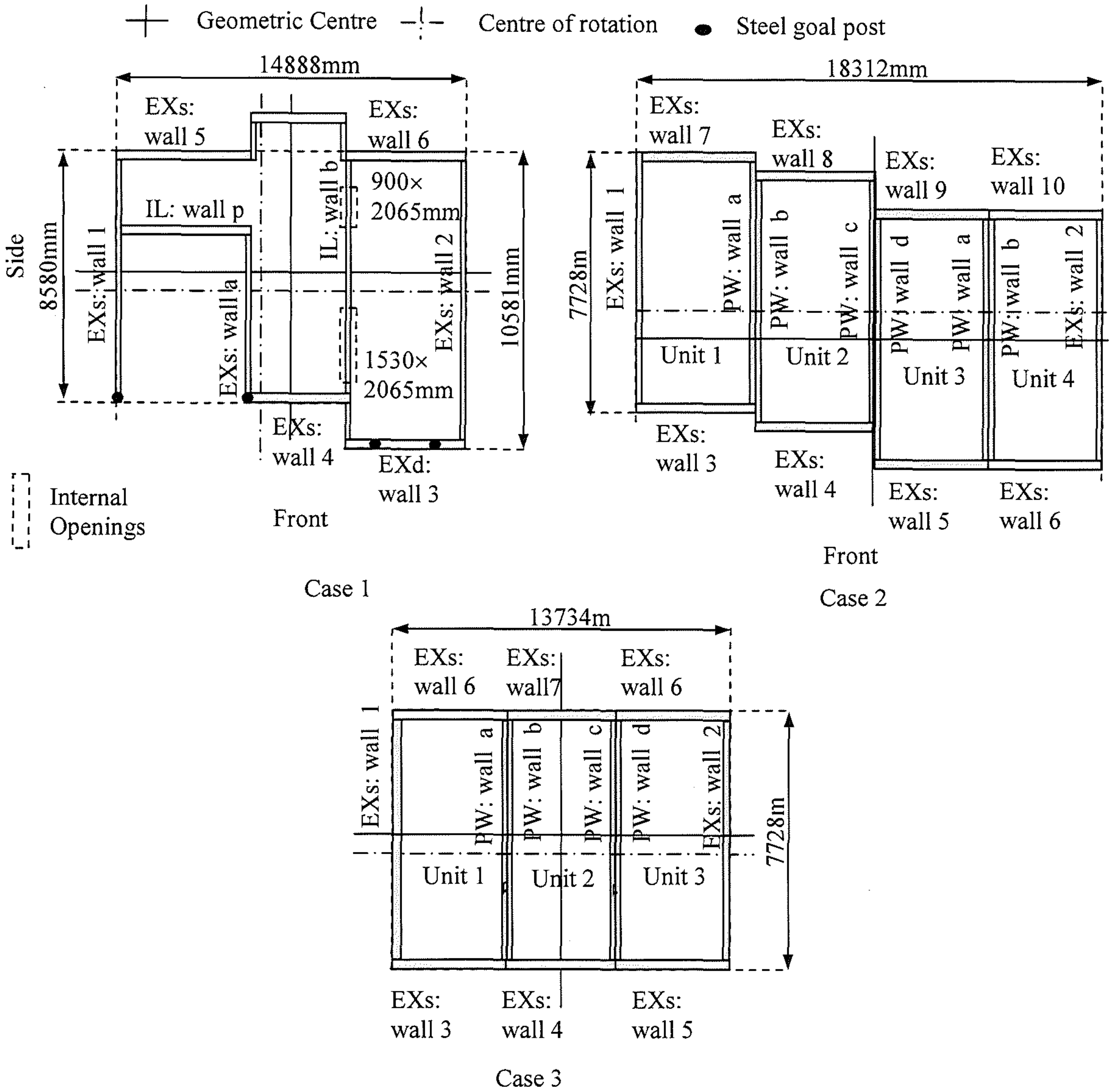


Figure 3.4 **Design cases**

Table 3.1 Wall sheathing arrangement

Type	Description	Sheathing Arrangement	
		External	Internal
Exd	External Double Sheathed	9mm OSB Grade 3	9mm OSB Grade 3
Exs	External Single Sheathed	9mm OSB Grade 3	12.5mm Plasterboard
IL	Internal Load Bearer	12.5mm Plasterboard	12.5mm Plasterboard
PW	Party Wall		12.5mm + 19mm Plasterboard

Table 3.2 Level of percentage opening in external walls

Case	External Wall	Wall Area (m^2)	Opening Area (m^2)	% of Opening
1	1	20.59	0.00	0
	2	25.39	0.00	0
	3	11.19	3.44	31
	4	10.26	4.48	44
	5	14.28	4.21	30
	6	11.88	5.60	47
	a	17.40	0	0
2	1 & 2	18.55	0	0
	3 to 10	10.99	4.10	37
3	1 & 2	18.55	0.00	0
	3 to 6	10.99	4.10	37

Rigid diaphragm action has been assumed and as a result applied shear to the system is distributed to the shear walls relative to their stiffness (Prion and Lam, 2003). It can be assumed that stiffness and shear resistance of the walls are directly related; therefore applied wind action in this study is distributed to the walls relative to their shear resistance.

By adopting a rigid analysis system torsion has to be considered. Applied torsion is dealt with by determining the centre of rotation of the system and distributing the resulting torsion forces to the walls relative to the moment resistance they provide to the system (see Appendix A for design methodology). Shown in Table 3.3 and Table 3.4 are examples of the results for Case 1 and contained in Table 3.5 is a summary of the results for all three cases. It is noted that if the torsion component is negative, which would serve to reduce the applied level of shear, it is conservatively taken as zero (Prion and Lam, 2003).

Table 3.3 Case 1 wind acting on front

Wall No.	Wall Resistance kN	Applied Shear kN	Applied Torsion kN	Applied Total kN
1	30.05	6.70	0.95	7.65
2	42.10	9.24	0.00	9.24
a	30.01	6.59	0.28	6.87
b	19.84	4.36	0.00	4.36
Σ	122.00	26.88	1.23	28.11

Table 3.4 Case 1 wind acting on side

Wall No.	Wall Resistance kN	Applied Shear kN	Applied Torsion kN	Applied Total kN
3	10.40	7.47	0.00	7.47
4	4.82	3.36	0.00	3.36
5	11.48	8.24	0.05	8.29
6	3.74	2.53	0.02	2.54
p	12.55	5.13	0.00	5.13
Σ	42.99	26.72	0.07	26.79

Table 3.5 Results of Case 1, 2 & 3 summarised (inclusive of allowable shear transfer)

Case	Wind acting on	Shear wall resistance	Allowable shear transfer*	Applied			Design Outcome
				Shear	Torsion	Total	
		kN	kN	kN	kN	kN	
1	Front	122	37.62	26.88	1.23	28.11	OK
	Side	42.99	27.20	26.72	0.07	26.79	OK
2	Front	188.44	66.65	41.81	6.59	48.4	OK
	Side	31.77	39.48	23.77	3.48	27.25	OK
3	Front	147.48	49.98	21.78	0	21.78	OK
	Side	26.22	14.81	17.8	1.56	19.36	Fail

*Note: Allowable shear transfer is as a result of the nailing specification between the wall panel and the sole plate, see Section 3.4.

It is shown in Table 3.5 that for all three cases the actual shear wall resistance is greater than the applied wind action and it is noted that for all cases the gable walls provide a high level of resistance as a result of having no openings. No openings assist racking resistance on two major counts:

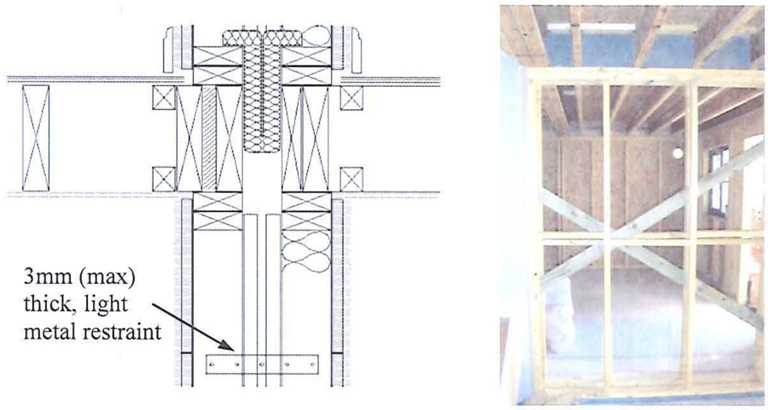
1. Increased panel area providing racking resistance.
2. Reduction in applied wind force as a result of increased masonry shielding.

For all three cases the centre of rotation is in close proximity to the geometric centre. When the centre of rotation is close to the geometric centre torsion in the system is reduced and as a result the system is capable of carrying increased wind action. This can be critical in cases of large openings; in particular if the systems in Cases 2 & 3 had not been well proportioned in relation to stiffness extra racking resistance would have been required resulting in a financial cost. Stiffness proportionality of the system therefore increases the level of direct shear the system can carry and results in more economical design.

3.4 System Continuity

System continuity is an important factor when considering the resistance of a system to applied wind action. In particular continuity across party walls is considered. Consider when the wind action is on the side of the building in Cases 2 & 3. The wall diaphragms in the first unit are incapable of carrying the total applied shear; it is the combined shear resistance of the walls of the units which resist the applied action. Therefore, residual shear has to be transferred across the party wall to the subsequent units.

As a result of thermal and acoustic performance requirements the two leaves of a party wall are unconnected for the full height except for 3mm (max) thick, light metal restraint straps tying the two leaves together (Figure 3.5). These straps are spaced at minimum horizontal centres of 1.2m, one row per storey height at or near ceiling level (TRADA, 2001).



a) Restraint strap detail b) Temporarily braced party wall

Figure 3.5 Continuity across a party wall

The connection between the metal strap and the wall stud is the critical design criteria and is normally made by 3no 3.35mm diameter 63mm long galvanised wire nails. The permissible strength of this connection is 1.65kN (calculated in accordance with EC5 and factored in accordance with BS 5268-2:2002). Therefore, the permissible residual shear which can be transferred is 1.4kN/m per storey height.

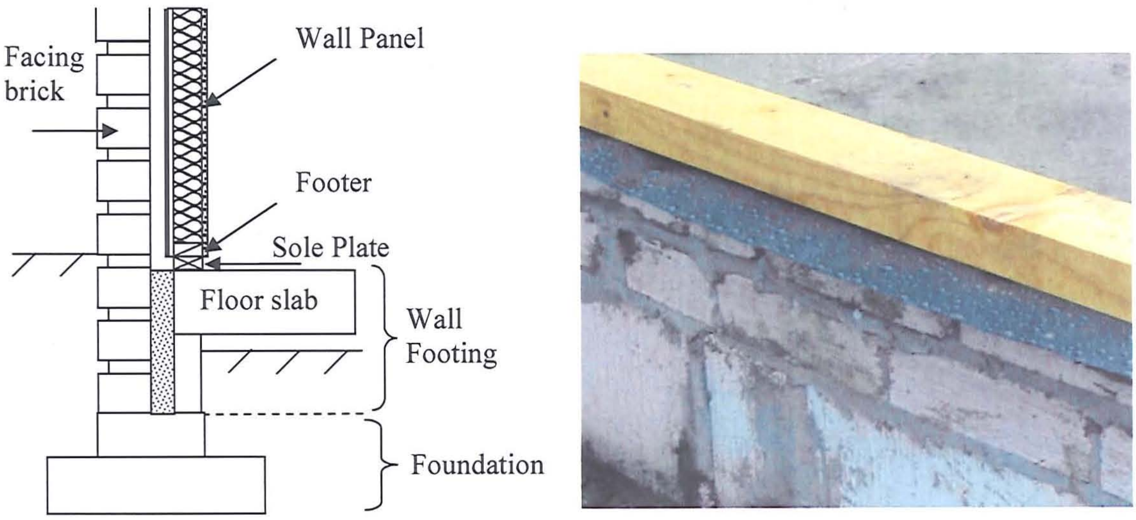
For cases 1 & 2 the transfer of shear force from unit 1 to 2 is equal (the total applied shear force on case 2 is in excess of this as a result of units 2, 3 & 4 protruding past unit 1). The party wall length in both cases is 7.728m therefore approximately 6 straps per storey can be applied, 12 straps in total. As a result the total shear which can be transferred is 19.83kN which is in excess of the residual force, 15.8kN.

It is demonstrated that continuity across the party walls for these cases is achieved through the application of restraint straps. However, it is to be noted that in certain design scenarios transfer of residual shear would be critical.

3.5 Shear Transfer to the Sole Plate

The level of shear transferred to the sole plate is dependent on the connection between the wall panel footer and sole plate (Figure 3.6). A typical nailed connection between wall panel footer and sole plate is 3.1×90mm skewed galvanised wire nails at 300mm centres (between wall studs).

The resistance to shear which can be allowed for in accordance with BS 5268-2:2002 Annex G is 323.52N per nail which equates to 1.08kN/m run. Therefore, although in Case 1 the resistance of wall 2 is stated as 42.10kN this is equal to 3.98kN/m run which requires an increase in nailing specification. However, in this case there is a degree of redundancy and the nailing specification is sufficient as the wall only requires to transmit 0.9kN/m run. The allowable shear transfer column of Table 3.5 shows the revised design racking resistance of the systems as a result of the nailing specification. It is shown that in Case 3 when the wind is acting on the side design failure occurs, therefore increased nailing of the wall panels to the sole plate is required.



a) Typical foundation detail

b) Typical sole plate to 7N/mm^2 concrete brick wall footing connection

Figure 3.6 Shear connection of timber frame to sub-structure

3.6 Timber to Concrete Connections

The shear transfer between the sole plate and the substrate is by means of a timber to concrete connection. Research on timber to concrete connections has been conducted mostly considering the application of use to be in timber-concrete composite flooring systems (Toratti and Kevarinmaki, 2006; Mungwa et al, 1999; Persaud and Symons, 2006; Aicher et al, 2003 and Dias and Cruz, 2004).

Timber-concrete composites are popular in some countries as a method of floor construction as they provide a structurally efficient system which is both rigid and light (Ceccotti, 1995). The most commonly used connection systems for timber-concrete composites are shown in Figure 3.7.

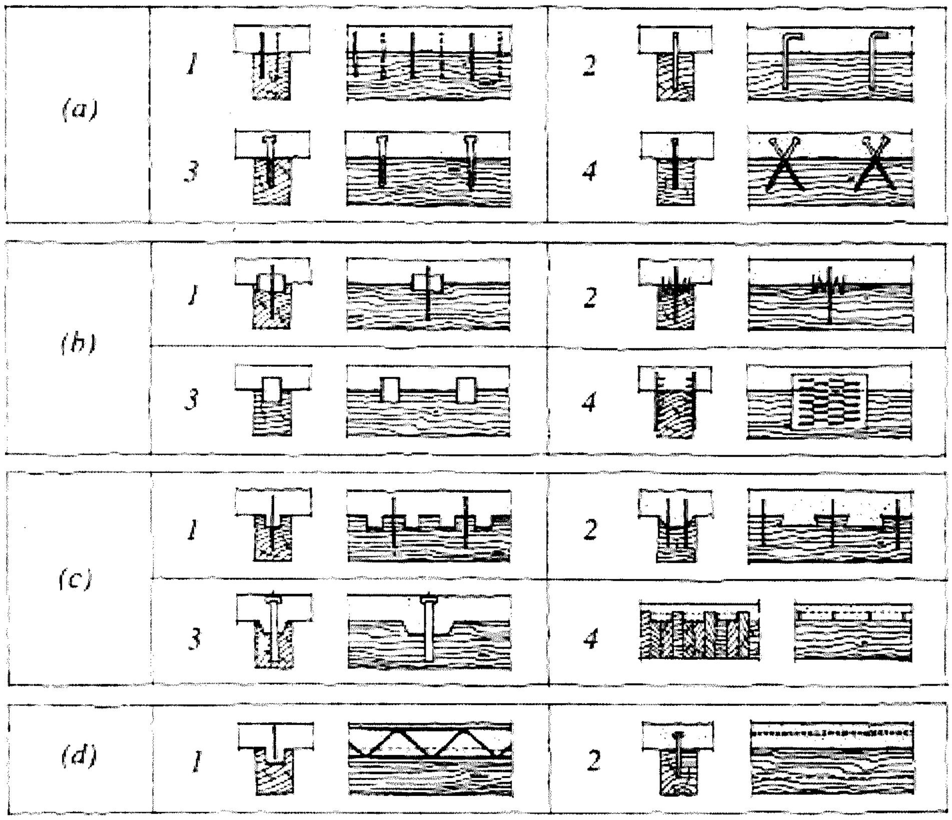
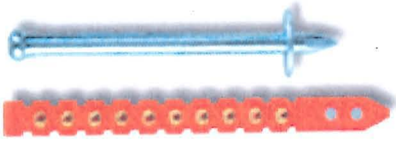


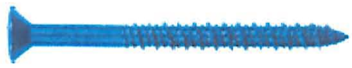
Figure 3.7 Examples of different types of timber-concrete connections (Ceccotti, 1995):
(a) nails, screws or dowel type fasteners
(b) surface connectors
(c) notched connectors
(d) bonded connectors

Shear connection of the sole plate to the substructure come in a manner of forms but the ones most commonly used for domestic dwelling construction in the UK are dowel type fasteners (Figure 3.8):

1. Hardened Zinc Plated Nails: shot fired using power actuated systems.
2. Screw Anchors: formed from carbon steel and self tapping.
3. Express Nails: formed from spring steel and hammer fixed into pre-drilled holes.



a) KMN Low velocity shot fired nail



b) MSC & BTB masonry screw

c) KF masonry anchor

d) EXPN express nail

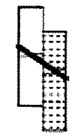
Figure 3.8 Industry standard sole plate to foundation fixings

Dowel type fasteners used in timber concrete connections are according to Ceccotti (1995), less rigid than elements connected by surface connectors and even less rigid than elements when notches have been cut into the wood itself. The stiffest connections are those where a bond between concrete and wood is obtained.

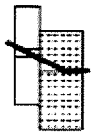
Aicher et al (2003) researched numerous timber-concrete connection methods which included medium-sized smooth nails and small-sized threaded nails used for the upgrading of timber beam ceilings and timber beam concrete slab construction respectively. The research work conducted by Aicher et al (2003) demonstrated that conventional smooth and threaded nails of medium and small sizes, when used for timber-concrete connections show the same shear capacity as calculated employing the methods of EC5 for a timber to thick steel plate in single shear (EC5 clause 8.2.3):

Failure Modes:

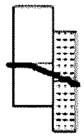
$$F_{v,Rk} = \min \begin{cases} f_{h,k} \cdot t_1 \cdot d \cdot \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,k} \cdot d \cdot t_1^{\,2}}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} \\ \\ 2.3\sqrt{M_{y,Rk} \cdot f_{h,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} \\ \\ f_{h,k} \cdot t_1 \cdot d \end{cases}$$



c



d



e

Equation 3.1

Where

$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;

$f_{h,k}$ is the characteristic embedment strength in the timber member;

t_1 is the thickness of the timber side member;

d is the fastener diameter;

$M_{y,Rk}$ is the characteristic fastener yield moment;

$F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener.

Each of the failure modes, c, d & e shown relate to the adjacent equations and were first developed by Johansen (1949). The equations predict the ultimate strength of a dowel-type joint due to either a bearing failure of the joint members or the simultaneous development of a bearing failure of the joint members and plastic hinge formation in the fastener. The precise mode of failure is determined by the joint geometry and the material properties, namely the embedding strengths of the timber or wood-based materials and the fastener yield moment (Hilson, 1995).

In accordance with EC5 characteristic embedment strength for nails up to a diameter of 8mm without predrilled holes, which would correspond to the case of the shot fired dowel (KMN fixing), is calculated as follows:

$$f_{h,k} = 0.082 \cdot \rho_k \cdot d^{-0.3}$$

Equation 3.2

For nails up to a diameter of 8mm with predrilled holes, which correspond to the remaining shear fixings (MSC, BTB, KF & EXPN as shown in Figure 3.8), the characteristic embedment strength is calculated as follows:

$$f_{h,k} = 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k$$

Where

$f_{h,k}$ is the characteristic embedment strength in the timber member;

ρ_k is the characteristic timber density,

d is the nail diameter.

3.6.1 Tensile & Yield Moment Capacity

To determine the tensile strength and yield moment capacity of the fixings so that the calculation methods of EC5 for determining the lateral load carrying capacity of the joint could be used tests were carried out. The test for yield moment, in accordance with BS EN 409:1993, requires the fixing to be subjected to 4 point bending as shown in Figure 3.9 where the dimensions l_1 and l_3 are at least twice the diameter, d, of the fixing and the free length of the nail, l_2, is between d and $3d$. The yield moment, M_y, of the fixings is taken as the bending moment at the maximum load sustained by the fixing during the test, or the bending moment at which the nail has deformed through an angle of 45° and is calculated as the greater of the two expressions ($F_1 \times l_1$) and ($F_3 \times l_3$) where F is the force.

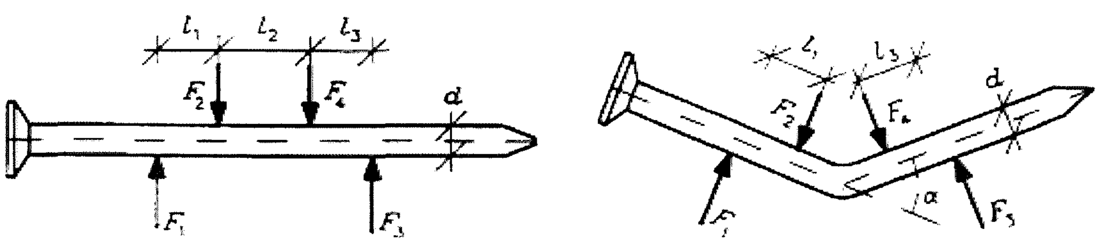


Figure 3.9 Loading and deformation of the fixing (BS EN 409:1993)

For the express nails the yield moment tests were performed on three series (A, B & C) for each diameter size with the gap in the fixing placed in three different orientations in order to determine the minimum yield moment (Figure 3.10). The minimum yield moment which would conservatively be taken in design calculations corresponded to orientation A.

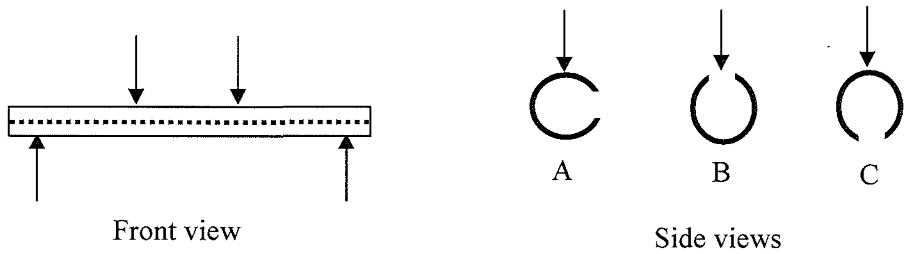


Figure 3.10 Yield moment tests, configuration for minimum yield moment of Express nails

Contained in Table 3.6 is information on the range of fixings tested and the results of the tensile and yield moment tests. To determine the characteristic yield moment, $M_{y,Rk}$, of the range of fixings which formed this study by calculation the following equation can be used in accordance with EC5 clauses 8.3.1.1 and 8.7.1:

$$M_{y,Rk} = 0.3 \cdot f_u \cdot d^{2.6} \qquad \text{Equation 3.3}$$

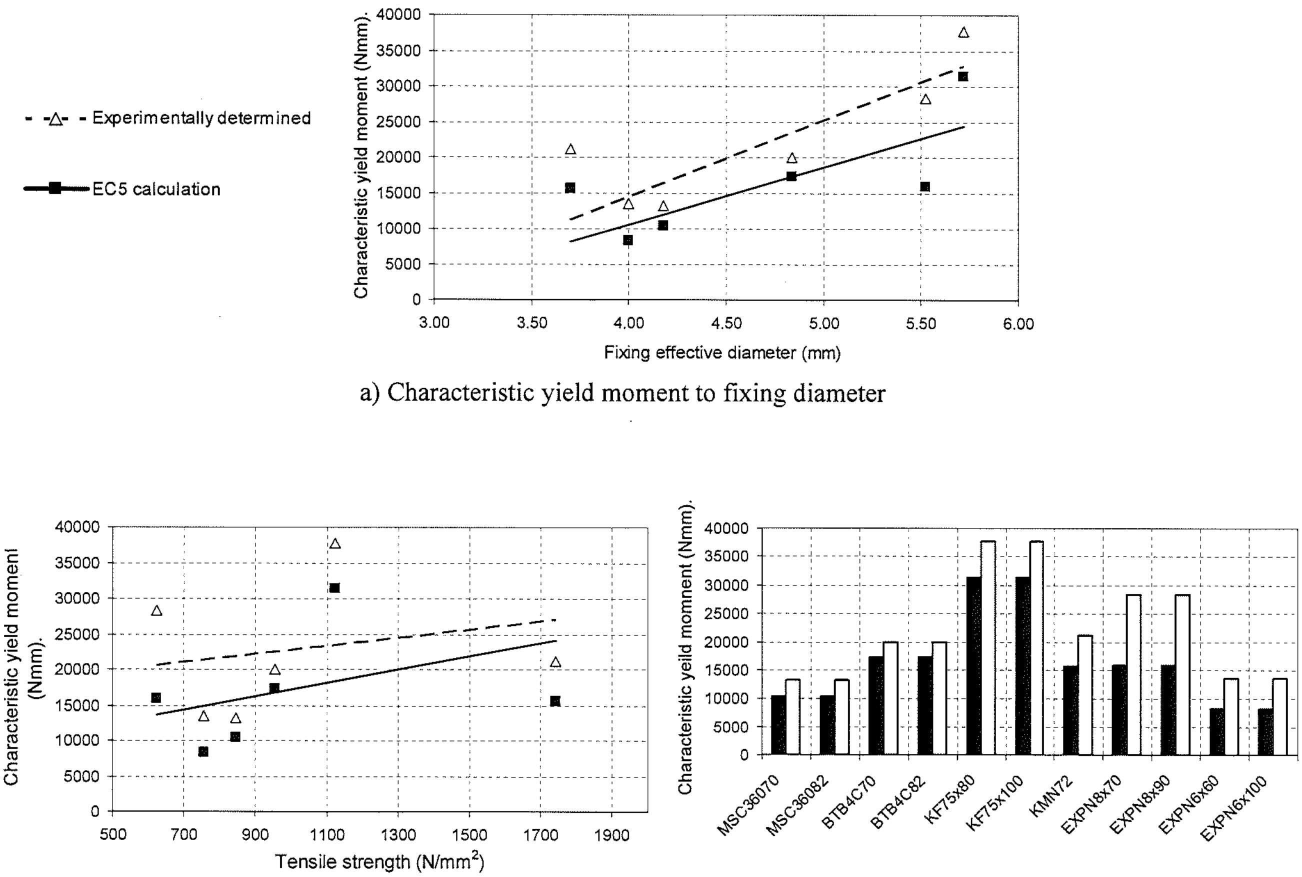
Where f_u is the tensile strength of the wire in N/mm^2 and d is the nail diameter in mm. The diameter of the fixing to be used is the effective diameter, for smooth shanked dowels such as the KMN72 (Figure 3.8) this is taken as the shank diameter. For threaded screws the effective diameter is taken as the root diameter multiplied by 1.1, in accordance with EC5 clause 8.7.1(3). For Express nails the effective diameter was taken as the equivalent diameter of a round fastener with the same cross sectional area.

It is known from Equation 3.3 that the magnitude of the yield moment of a fixing, $M_{y,Rk}$, is an interrelationship between diameter and tensile strength and that it is directly proportionate to both. Figure 3.11 shows that this statement is true of the shear fixings tested and it also shows that Equation 3.3 provides a conservative method of determining the yield moment of the fixings based on their tensile strength, effective diameter and the applied interpretations of the code.

Table 3.6 Fixing information and test results

Fixing Type	Test		Length	Diameter					Tensile Strength	Characteristic Yield Moment	
	Desig-nation	No		Head	Root	Thread	Shank	Effec-tive		EC5 Calc	Test Deter-mined
			mm	mm	mm	mm	mm	mm	N/mm²	Nmm	
Masonry Screw	MSC 36070	1	70.00	9.0	3.80	5.40	3.80	4.18	845.25	10451	13261
	MSC 36082	2	82.00	9.0	3.80	5.40	3.80	4.18	845.25	10451	13261
	BTB 4C70	3	70.00	12.0	4.40	6.40	4.70	4.84	954.24	17273	19936
	BTB 4C82	4	82.00	12.0	4.40	6.40	4.70	4.84	954.24	17273	19936
Masonry Anchor	KF 7.5×80	5	80.00	12.0	5.20	7.40	N/A	5.72	1122.30	31366	37777
	KF 7.5×100	6	100.00	12.0	5.20	7.40	N/A	5.72	1122.30	31366	37777
Shot Fired Dowel	KMN72	7	72.00	12.0	N/A	N/A	3.70	3.70	1742.45	15689	21171
Express Nail	EXPN 8×70	8	70.00	N/A	N/A	N/A	8.00	5.52	623.61	15917	28308
	EXPN 8×90	9	90.00	N/A	N/A	N/A	8.00	5.52	623.61	15917	28308
	EXPN 6×60	10	60.00	N/A	N/A	N/A	6.00	4.00	756.93	8324	13522
	EXPN 6×100	11	100.00	N/A	N/A	N/A	6.00	4.00	756.93	8324	13522

Notes:
- Characteristic values are based on a minimum of 5 tests.



a) Characteristic yield moment to fixing diameter

b) Characteristic yield moment to tensile strength

c) Characteristic yield of fixing types

Note: data points are scattered due to the effect of fixing diameter, however, the general trend holds true.

Figure 3.11 Relationship between characteristic yield moment and effective diameter, tensile strength and fixing type

3.6.2 Lateral Load Carrying Capacity

To evaluate the performance of the shear fixings in lateral shear three different industry standard substrates were used: common brick, 7N block and 20N block (Table 3.7). Initially the most commonly used fixings for the sole plate to foundation connection (MSC36070, MSC36082, BTB4C82, KF7.5×100, EXPN8×&0 and KMN72) were used in combination with all three substrates to evaluate the influence of substrate on connection strength.

The samples were formed in a manner representative of sole plate to foundation connection detailing (Figure 3.6). The samples were assembled following on-site practises; with the substrates and timber predrilled according to the fixing specifications, and damp proof coursing was placed at the interface of the timber and substrate. The addition of damp proof coursing limits the frictional resistance and therefore provides results which are representative of the application (sole plate to substrate connection). However, the results would also allow specification in other applications where there is no damp proof coursing at the interface and therefore increased connection strength due to frictional resistance.

Table 3.7 Industry standard substrates

Description	Average specific gravity	Average compressive strength (N/mm²)
Common brick (215x65x102.5)	2.15	27.56
7N Concrete block (440x215x100mm)	2.39	14.68
20N Concrete block (440x215x100mm)	2.34	31.72

To facilitate testing a symmetrical loading arrangement was used. The samples comprised two shear planes each with two fixings (Figure 3.12). Each test suite contained a minimum of 4 samples and the tests were conducted in accordance with BS EN 1380:1999 requirements.

During the fabrication of the samples with 72mm shot fired dowels difficulties were encountered. The shot fired dowels did not fully penetrate 20N/mm^2 concrete blocks and had a tendency to crack and split common brick during application. As a result the KMN72 fixings were only used to form test samples consisting of timber and 7N block.

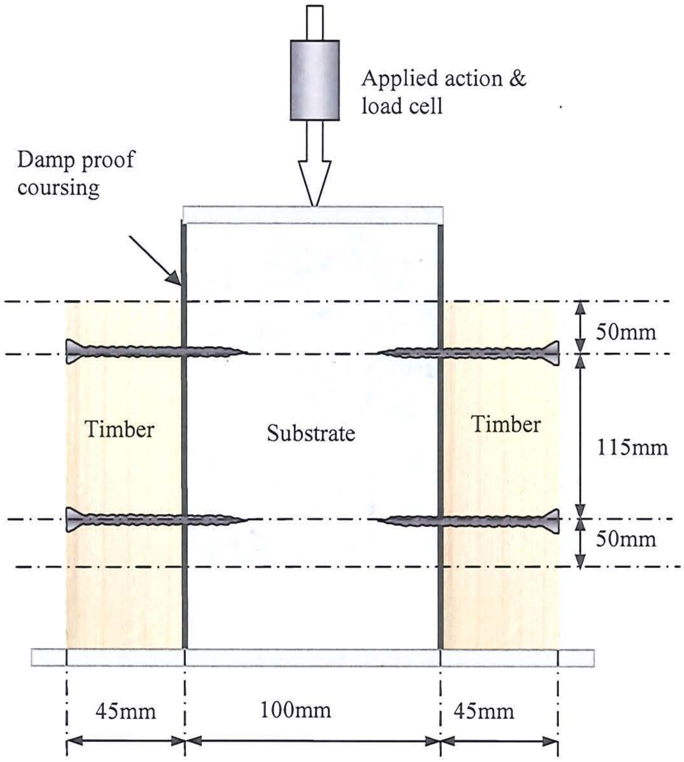
Shown in Figure 3.13 are the characteristic failure loads for the range of selected fixings in all three substrates, with the exception of the KMN72 as a result of fabrication problems. To account for varying timber density over the range of test pieces the failure loads have been normalised based on the average density (442kg/m^3).

For comparative purposes design calculations in accordance with EC5 have also been carried out, the results of which are shown relative to the test results in Figure 3.13. The design calculations have been carried out applying the following two methods:

1. Using the average timber density of the test samples and the yield moment of the fixing as determined from the tests on the fixings carried out.

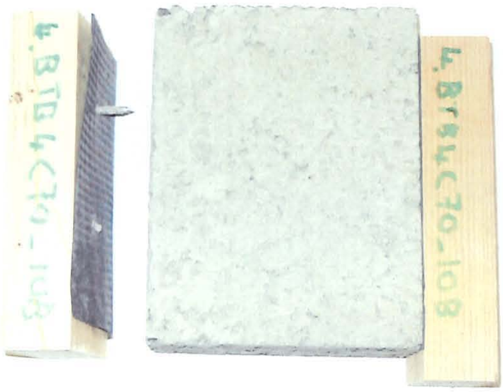
2. Using the characteristic density of C16 timber (normal sole plate material) as prescribed by BS EN 338(2003) and the tensile strength of the fixing determined from the tests conducted.

Frictional effects contribute to the lateral load carrying capacity of a nailed joint. As a connection yields friction between the members is caused by the pulling together of the members due to the axial load carrying capacity of the fixing or "withdrawal". It was considered that the inclusion of the damp proof coursing would limit the frictional resistance between the elements and as a result the axial withdrawal capacity of the connection has been conservatively taken as zero in both calculation methods.



a) Details of lateral shear sample

b) Sample being tested



c) Failed test specimen

Figure 3.12 Lateral load test sample and set-up

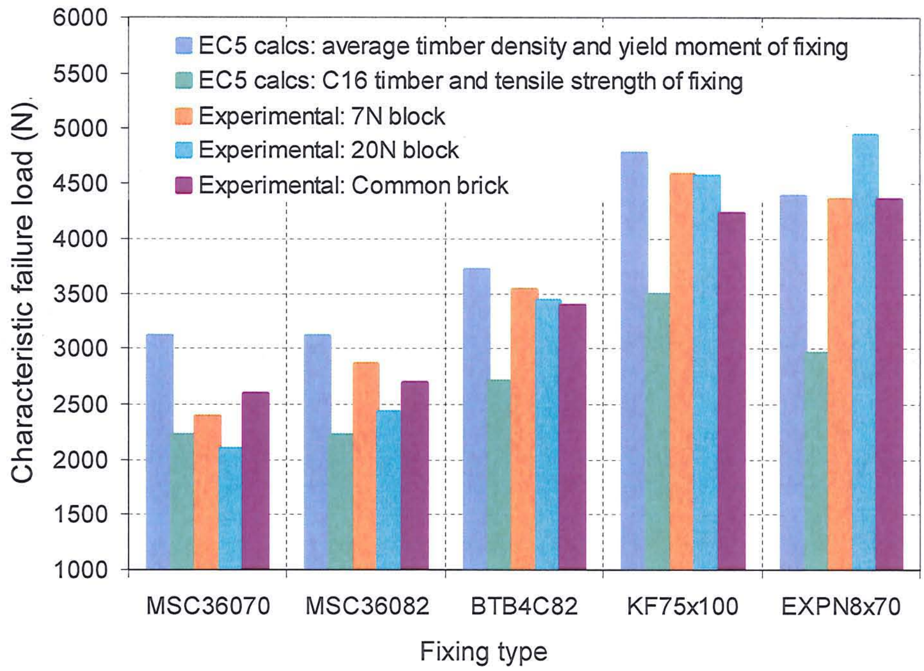


Figure 3.13 Variation in characteristic fixing failure load with substrate type

Correlation between the EC5 calculation method and test results is shown to be favourable although further investigation maybe required. Figure 3.13 demonstrates that for the range of substrates under consideration the nature of the substrate, as long as the fixing can be practically employed, has a negligible affect on connection strength relative to the influence of the properties of the fixing itself. Therefore, to evaluate the fixings relative to each other and the applicability of the EC5 design method the study was extended to encompass all fixings using 7N block as the substrate. The use of 7N block as the substrate is reflective of the substrate material most commonly used in domestic dwelling construction and also allowed the practical employment of all fixing types.

Shown in Table 3.8 are the results from the extended experimental program using 7N block. Figure 3.14 is the relationship between experimentally determined failure loads (normalised to account for timber density variations across the sample range) with EC5 calculated values against:

1. Using the average timber density of the test samples and the yield moment of the fixing as determined from the tests on the fixings carried out.
2. Using the characteristic density of C16 timber (normal sole plate material) as prescribed by BS EN 338(2003) and the tensile strength of the fixing determined from the tests conducted.

Table 3.8 Lateral load carrying capacity of fixings in 7N block

Fixings	Specification	Maximum load per fixing	Characteristic load per fixing	Normalised load per fixing	EC5 calculated load per fixing
		N	N	N	N
Masonry Screws	MSC36070	2470.94	2100.30	2389.51	2218.30
	MSC36082	2915.95	2478.56	2869.70	2218.30
	BTB4C70	3358.45	2854.68	3549.62	2706.68
	BTB4C82	3636.19	3090.76	3533.97	2706.68
Masonry Anchors	KF7.5×80	5441.37	4625.17	5813.23	3490.79
	KF7.5×100	4700.60	3995.51	4578.47	3490.79
Shot Fired Dowels	KMN72	1942.16	1650.83	2114.84	2161.88
Express Nail	EXPN8×70	4573.32	3887.32	4365.05	2958.91
	EXPN8×90	5871.31	4990.62	6285.29	2958.91
	EXPN6×60	2958.82	2515.00	3072.60	2072.23
	EXPN6×100	3836.45	3260.98	4113.73	2072.23

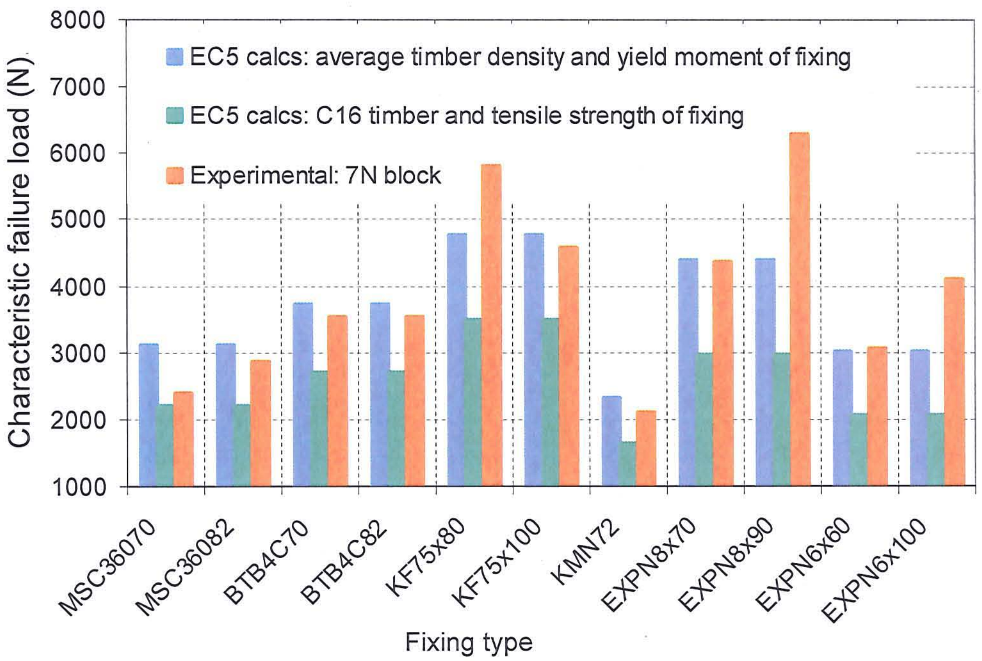


Figure 3.14 Lateral load carrying capacity of fixings in 7N block

Comparing the test results with those calculated in accordance with EC5 a relatively high level of correlation is shown with the level of correlation depending on the type of fixing being employed. For all cases with the exception of the KF7.5×80, EXPN8×90 and EXPN6×100 calculations considering the average density of the timber elements and the yield moment of the fixing produce marginally non-conservative results as shown Figure 3.14.

Two distinct modes of failure were observed during testing: ductile and relatively brittle failure modes (Figure 3.15). The masonry screws exhibited a rather brittle behaviour, with the fixings shearing off during the test resulting in a sudden loss of resistance. The express nails and KMN72 shot fired dowels exhibited a ductile behaviour, where large displacements were reached before any loss of resistance. The two masonry anchors exhibited different failure modes, with KF7.5×100 displaying a brittle behaviour, and KF7.5x80 failing in a ductile manner; which may explain why the KF7.5×80 achieved a higher lateral load carrying capacity.

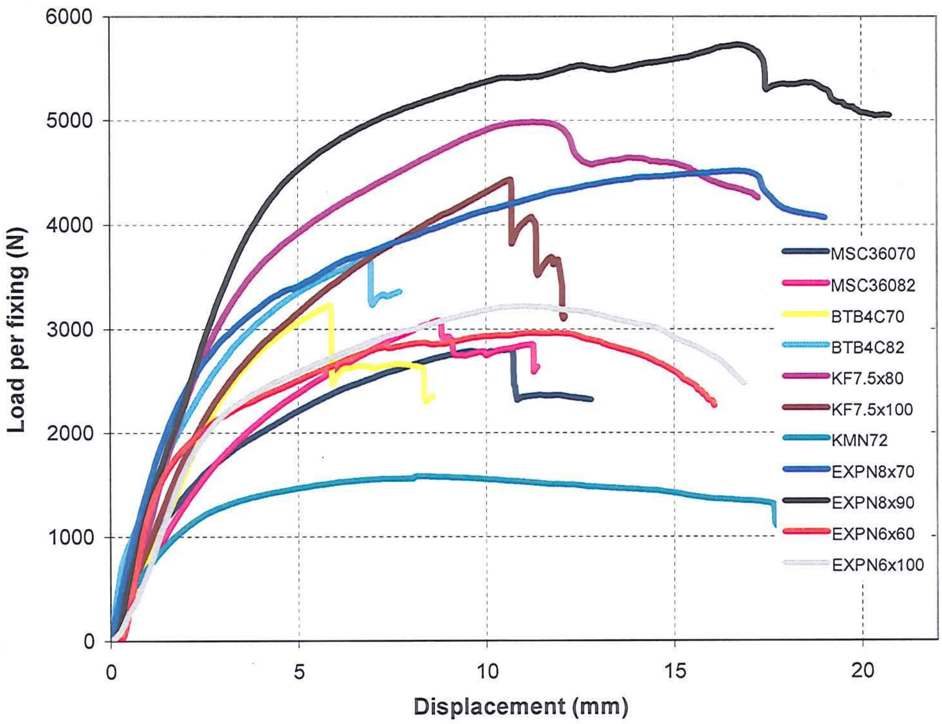


Figure 3.15 Load against displacement of fixings in 7N block

3.6.3 Specification

With a degree of precaution the design method of EC5 for timber to thick steel plate can be used for the specification of the given range of shear fixings (Table 3.6) when considering a sole plate to substrate connection. However, for safe and robust design, specification should be based on test values representative of the design case, as contained in Table 3.8, and factored accordingly. Shown in Figure 3.16 is the variation in design load carrying capacities of the shear fixing range per metre run for varying spacing between fixing centres. The results contained in Table 3.8 have been adjusted to be representative of C16 timber (characteristic density, ρ_{c16} = 310kg/m^3) and factored for an instantaneous load case in service class 1 or 2 (k_{mod} = 1.1) applying a material factor, γ_M, of 1.3 for solid section timber.

Previously it has been highlighted that regardless of the racking strength of the shear walls the connection between shear wall footer and sole plate is critical. Also shown in Figure 3.16 is the design lateral load carrying capacity of two number 45mm deep C16 grade timbers connected with standard 3.1×90mm galvanised wire nails at varying centre to centre spacings for the same design conditions.

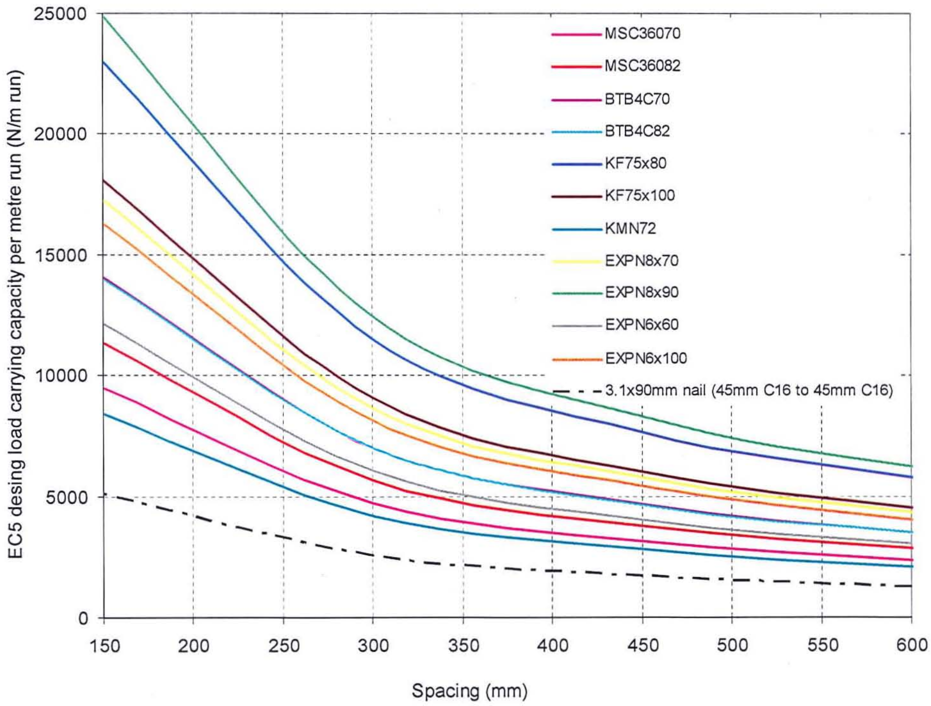


Figure 3.16 Design load carrying capacity per metre run for 45mm C16 timber to substrate

Each of the plots in Figure 3.16 corresponds to a trend line of a power function ($R^2 = 1$) the constants of which are contained in Table 3.9:

$$F = A \cdot sp^{-1} \qquad \text{Equation 3.4}$$

Where:
F is the lateral load carrying capacity in Newton's per m run.
A is as shown in Table 3.9.
sp is the spacing between the fixings in mm.

Table 3.9 Values of constants corresponding to Equation 3.4 and Figure 3.16

Designation	Fixing	Constant
		A
1	MSC36070	1.42E+06
2	MSC36082	1.70E+06
3	BTB4C70	2.11E+06
4	BTB4C82	2.10E+06
5	KF75×80	3.45E+06
6	KF75×100	2.72E+06
7	KMN72	1.25E+06
8	EXPN8×70	2.59E+06
9	EXPN8×90	3.73E+06
10	EXPN6×60	1.82E+06
11	EXPN6×100	2.44E+06
12	Galvanised wire nail (45mm C16 to 45mm C16)	7.68E+05

In terms of optimising resources the level of fixity between sole plate and substrate would be balanced with the level of fixity between the shear wall footer and sole plate. Applying the following equation achieves this balance:

$$sp_n = \frac{A_n \cdot sp_{12}}{A_{12}} \qquad \text{Equation 3.5}$$

From the experimental programme it was demonstrated that as a result of the shear fixings used to connect the sole plate to the substrate being formed from high strength steel they can have a tendency to fail in a brittle fashion. The standard 3.1×90mm galvanised wire nails used to connect the shear wall footer to the sole plate are formed from 600N/mm^2 strength wire which exhibits ductility and will therefore tend to fail in a plastic fashion which is preferable in design. So that a ductile failure of the connection takes place the connection between the sole plate and the substrate should be over specified relative to the footer to sole plate connection and also be a practical measurement for ease of application. Contained in Table 3.10 is specification information which has been derived applying Equation 3.5 with the spacing between the shear fixing connection (sp_n) rounded up to the nearest 10mm to enhance the probability of a ductile failure occurring should failure loads be reached.

Table 3.10 Optimised fixing specification and associated design capacity

Nail Spacing (3.1× 90mm)	Limit state design capacity	MSC		BTB		KF		KMN	EXPN			
		36070	36082	4C70	4C82	75× 80	75× 100	72	8× 70	8× 90	6× 60	6× 100
mm	N/m run	Spacing to the nearest 10mm										
50	15362	100	120	140	140	230	180	90	170	250	119	160
100	7681	190	230	280	270	450	360	170	340	490	237	320
150	5121	280	340	420	410	680	540	250	510	730	356	480
200	3841	370	450	550	550	900	710	330	680	980	475	640
250	3072	470	560	690	680	1130	890	410	850	1220	593	800
300	2560	560	670	830	820	1350	1070	490	1020	1460	712	960
350	2195	650	780	960	960	1580	1240	580	1180	1700	830	1120
400	1920	740	890	1100	1090	1800	1420	660	1350	1950	949	1280
450	1707	840	1000	1240	1230	2030	1600	740	1520	2190	1068	1430
500	1536	930	1110	1380	1360	2250	1770	820	1690	2430	1186	1590
600	1280	1110	1330	1650	1640	2700	2130	980	2030	2920	1424	1910
Note: Specification of the level of shear fixity would have to be enhanced if frictional resistance of the building is not sufficient to counteract sliding forces.												

Practical implications often play an important role in the specification of fixings to be used in timber to concrete connections. Speed of application, availability of equipment and whether the fixing can be easily employed (e.g. the use of shot fired dowels is limited as they tend to spall or crack high strength or brittle substrate material) all have to be given due consideration. However, in terms of the sole plate to substrate connections one of the governing criteria's is cost due to the high volume used.

Based on 1800 units per annum approximately 8400m (48m per unit) of sole plate is required to be connected to the foundation. Table 3.11 contains the unit cost of each fixing based on 2007 prices. Considering the information presented in Table 3.10 the cost per annum of employing each fixing relative to the wall footer to sole plate connection specification (2no × 45mm deep C16 strength grade timbers connected by 3.1×90mm galvanised wire nails) is illustrated. It is shown in Figure 3.17 that although the KF7.5×80 is the third most expensive fixing at £0.20 per unit it is in terms of lateral load carrying capacity more cost effective. Paradoxically the cheapest fixing, the MSC36070 priced at £0.10 per unit, is less cost effective in terms of lateral load carrying capacity.

Table 3.11 Cost of fixing

Fixing		Cost
No	Type	£/ fixing
1	MSC36070	0.10
2	MSC36082	0.12
3	BTB4C70	0.15
4	BTB4C82	0.16
5	KF75×80	0.20
6	KF75×100	0.24
7	KMN72	0.26
8	EXPN8×70	0.19
9	EXPN8x90	0.24
10	EXPN6×60	0.13
11	EXPN6×100	0.15
Note: KMN72 cost includes cost of fixing and charge.		

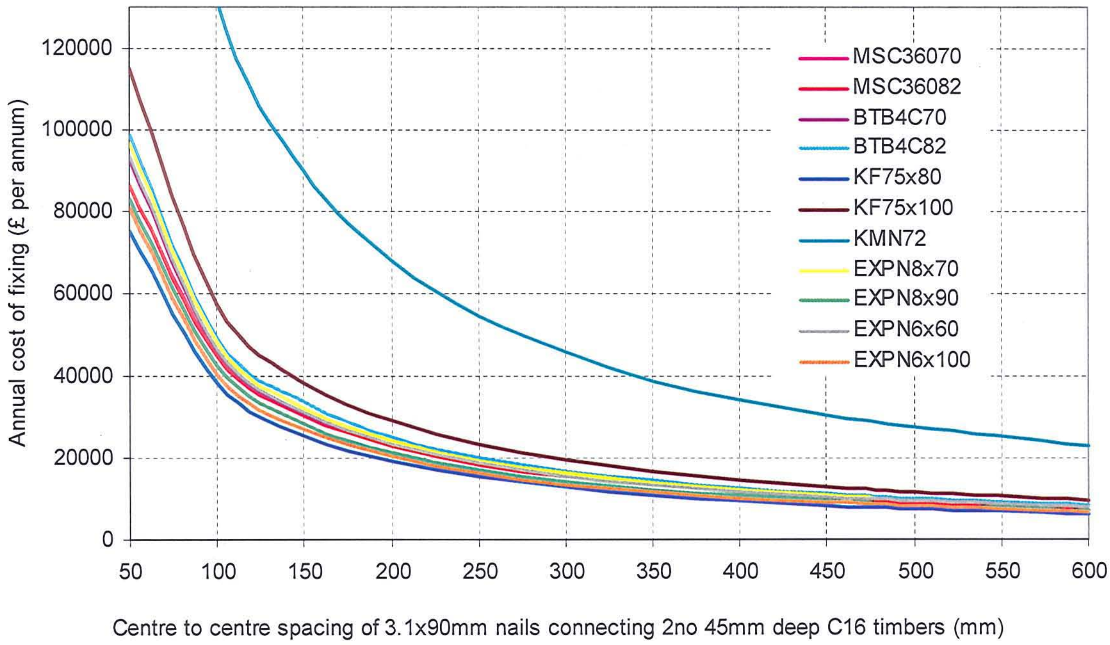


Figure 3.17 Annual cost of shear fixing specification relative to wall footer to sole plate connection

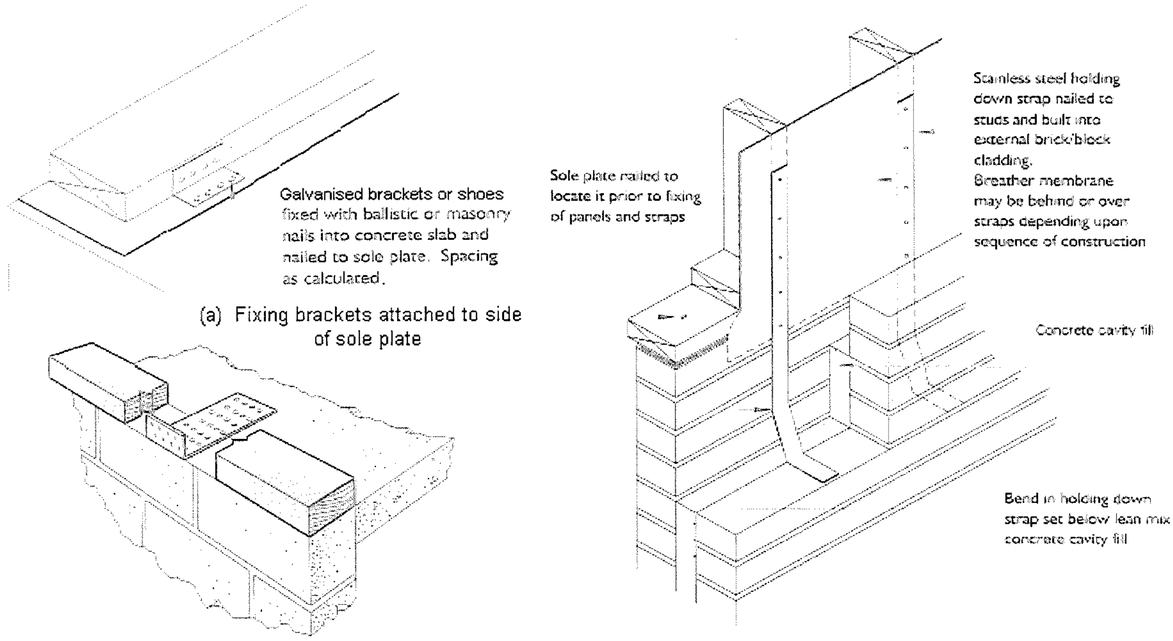
Considering the industry standard connection between the sole plate and footer, which is 3.1×90mm galvanised wire nails at 300mm centres, the optimum spacing of KF7.5×80 shear fixings is 1350mm and the optimum spacing of the KMN72 shear fixing (currently commonly used in practice) is 490mm. Based on this specification the KF7.5×80 would cost £12,800 and the KMN72 would cost £45,845 per annum based on 1800 units (see Appendix B for full tabulated results). Therefore, employing the KF7.5×80 offers a 72% reduction in annual shear fixing specification cost based on 1800 units.

3.6.4 Holding down & withdrawal

The applied shear force on a wall assembly results in an overturning moment which has to be counteracted by holding down anchorage. It is normal practice in the UK for holding down straps to be employed (Figure 3.18c). Holding down straps connect the vertical end stud to the foundation. They are normally attached to the end stud by means of 6 no 3.35×65mm ring shank nails or equivalent, the limit state connection strength (3.2kN) of which is the limiting criteria in design, and have their L-shaped end placed under the masonry cladding to create a holding down resistance.

According to Andreasson (2000) it is reasonable to assume that the dead load applied within the reach of the sheathing panel closest to the end is counteracting the uplift force. However, the uniformly distributed load along the top of the wall panel results in additional racking capacity in design, with this in mind the overall robustness of the system has to be considered i.e. the interrelationship between uniformly distributed load along the top of the wall panel, the subsequent additional racking resistance allowance and whether the uniformly distributed load can also provide a holding down resistance. Quantification of this interrelationship would require further testing before a level of redundancy could be confidently used in design to reduce holding down requirements.

Shear connections are not designed to transmit vertical forces to the foundation, although some capacity can be achieved. The interrelationship between shear and holding down resistance of the range of shear fixings considered is unknown. To ensure robust design the shear connections can therefore be specified to provide either a holding down or lateral resistance but, unless quantified by testing, not both in combination. The transfer of vertical forces from the sheathing to the sole plate would be via the bottom row of nails (instead of the vertical end stud) where the anchor bolts will further transmit the forces to the foundations. Because of the eccentric load transfer, transverse bending is created in the sill plate and splitting often occurs (Prion and Lam, 2003).



(a) Fixing brackets attached to side of sole plate

(b) Fixing brackets attached to side from beneath wall plate (Picture courtesy of Simpson Strong-Tie Ltd)

(c) Holding down straps showing correct installation method

Figure 3.18 Timber frame holding down methods (IstructE, 2007)

To quantify the holding down resistance of the shear connections which are most commonly used for sole plate to substrate connections, pull through (head side pulling through the timber) tests and pull out (point side pulling out of the substrate) tests of the fixings in the three substrates were conducted. Shown in Figure 3.19 are a batch of test pieces and the pull though test being conducted with examples of tested pull through failure modes given in Figure 3.20.

According to EC5 the pull through of screw type fixings should be determined by testing in accordance with EN1383 (EC5 clause 8.7.2(6)). However, for comparative purposes the pull through resistance of all the fixings have been calculated applying the equation for smooth nails in accordance with EC5 clause 8.3.2(4):

$$F_{ax,Rk} = f_{ax,k} \cdot d \cdot t + f_{head,k} \cdot d_h^{\,2} \qquad \text{Equation 3.6}$$

Where:

$f_{ax,k}$ is the characteristic withdrawal strength;

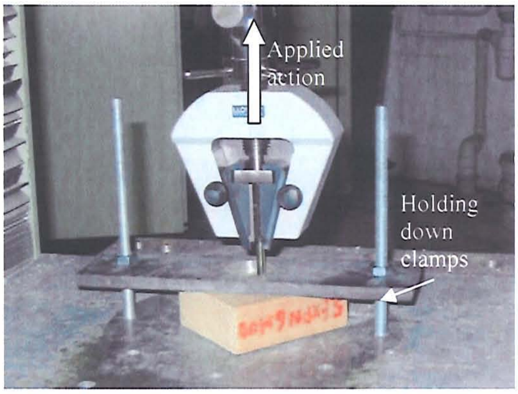
$f_{head,k}$ is the characteristic pull-through strength;

d is the effective nail diameter;

d_h is the effective nail diameter;

t is the thickness of the headside member;

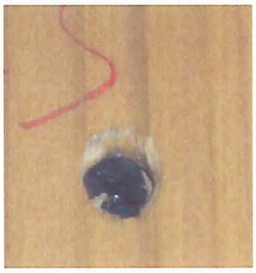
a) Batch of samples



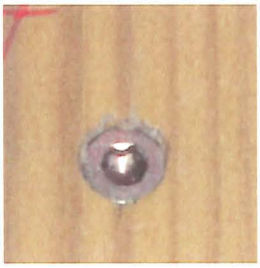
b) Pull through test

Figure 3.19 Pull through tests

The characteristic withdrawal strength of the masonry screws and anchors have been calculated applying the screw withdrawal equation of EC5 (equation 8.40) and the withdrawal strength of the express nails and shot fired dowels have been calculated applying the smooth nail equation of EC5 (equation 8.25).



a) Masonry screw



b) Masonry anchor

c) Express nail

d) Shot fired dowel

Figure 3.20 Pull through failure modes

Error! Reference source not found. compares the experimental results with the following (calculated results using the densities of the actual samples have been normalised to account for variations):

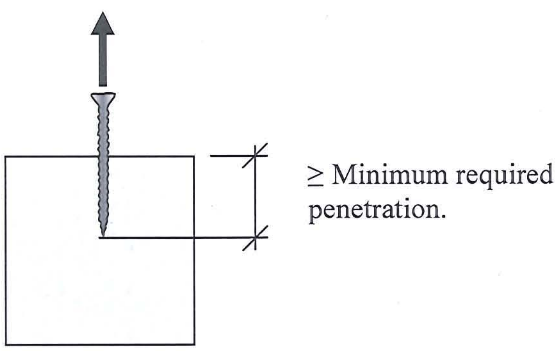
- Calculated head pull through only using the actual densities of the samples.
- Calculated head pull through only using the characteristic density of C16 timber.
- Calculated head pull through plus axial withdrawal using the actual densities of the samples.

It is shown in Table 3.12 that the head pull through only calculated results correlate conservatively with experimental results and that head pull through plus axial withdrawal calculations for all cases results in an overestimation of pull through resistance. It is postulated that the reason for the KF7.5x100 showing a higher degree of pull through resistance than the other fixing types is because the threaded part of the screw extends up to the screw head (Figure 3.8).

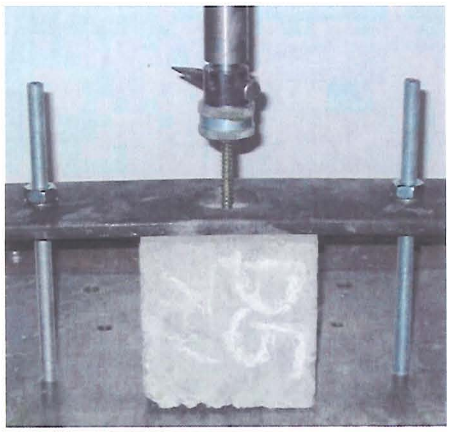
Table 3.12 Headside pull through experimental and calculated results

Fixing	Experimental results	EC5 calculated: Normalised head pull through only	EC5 calculated': Normalised head pull through + axial withdrawal	EC5 calculated: C16 head pull through only
	N	N	N	N
MSC36070	2343	1090	7291	551
MSC36082	2282	1169	7613	549
BTB4C82	3096	1908	9251	925
KF7.5x100	5623	2007	11057	894
EXPN8x70	2409	1898	2771	1038
KMN72	2542	1953	2970	950

For the pull out tests the fixings were inserted following the installation specification and on-site practise (Figure 3.21) and tested in accordance with BS EN 1382:1999 requirements.



a) Detail of pull out sample,



b) Sample being tested.

Figure 3.21 Pull-out test of fixings

Table 3.13 contains the mean and characteristic pull out loads and also for comparative purposes the characteristic load per unit penetration, this information is also shown graphically in Figure 3.22 and Figure 3.23 where it is also compared with the normalised characteristic pull through resistance of each fixing. The results show that the two fixings with a smooth shank, express nails and shot fired dowel, reached much lower pull out loads, compared to the threaded fixings tested. In all three substrates the masonry anchors reached the highest pull out loads. In 20N/mm^2 concrete block and in masonry brick they offered the highest pull out resistance, while in 7N/mm^2 concrete block masonry screws had similar or greater load per unit penetration.

Table 3.13 Pull-out test results

Fixings	Specification	7N/mm^2 Concrete			20N/mm^2 Concrete			Masonry Bricks		
		Max load	Characteristic		Max load	Characteristic		Max load	Characteristic	
			load	per unit penetration		load	per unit penetration		load	per unit penetration
		N	N	N/mm	N	N	N/mm	N	N	N/mm
Masonry Screws	MSC 36070	4809	4088	134	6332	5382	169	5558	4724	142
	MSC 36082	4598	3908	113	5989	5091	137	5278	4486	143
	BTB 4C82	3848	3271	102	4851	4124	129	7538	6407	154
Express Nail	EXPN 8x70	2791	2372	61	3213	2731	66	3219	2736	64
Masonry Anchors	KF7.5 x100	5965	5070	114	8561	7277	192	9532	8102	175
Shot Fired Dowel	KMN 72	2128	1808	28	N/A					

It is shown for all cases, with the exception of the shot fired dowel, that pull through governs the axial withdrawal resistance. To enhance the axial withdrawal resistance of all the fixings, with the exception of the shot fired dowel, washers could be specified to increase the bearing area. According to Prion and Lam (2003) the use of washers is advantageous. Large washers can reduce the effects of eccentric loading and prevent brittle splitting failure from occurring.

It may therefore be possible to provide the required holding down of racking walls through the appropriate specification of "shear fixings". Table 3.10 would be used to specify the required number for shear transfer and additional anchorage could be specified to provide holding down. The specification could be a combination of two types of fixing, for example the shot fired dowel could be specified for shear transfer and masonry anchors specified to provide holding down. For instance if an appropriate washer was specified the KF7.5x100 could provide up to 5kN anchorage which is 1.2kN more than the current method of a holding down strap. This merits further investigation and full scale racking tests to ensure robustness.

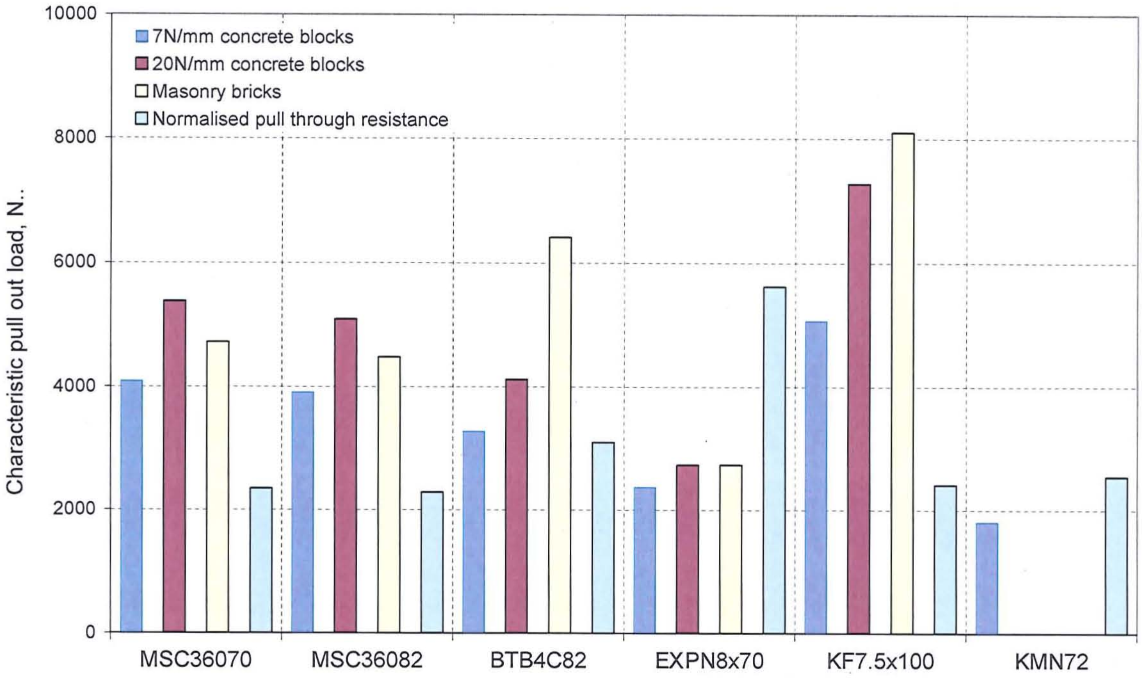


Figure 3.22 Characteristic pull out & pull through loads

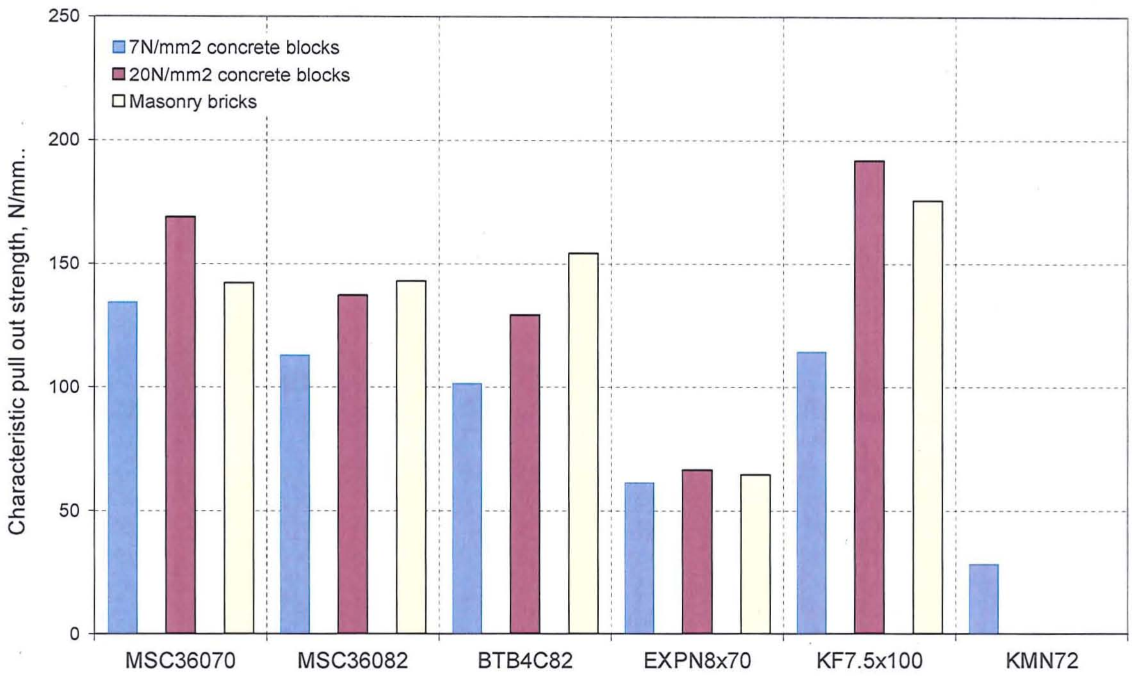


Figure 3.23 Characteristic load per unit penetration

3.6.5 Summary

As a result of the experimental programme carried out on a range of industry standard shear fixings the following conclusions are drawn:

- The use of EC5 calculation methods for determining the yield moment of high tensile strength dowel type fixings used for the connection of timber sole plates to concrete substrates is applicable. However, when the fixings are non-standard, such as in the case of the Express nail, calculated results can tend to be overly conservative.
- When determining the lateral load carrying capacity of timber sole plate to substrate connections the use of EC5 design calculations for timber to thick steel plate can with a degree of precaution be used. It has been demonstrated that the strength of the substrates considered is non-critical and that the nature of the fixing is more influential. Although EC5 design methods can be used it is recommended that to ensure robust, safe and serviceable design specification should be based on test results of representative samples.
- The practical application of what is being specified is important. The nature of the substrate material may dictate the fixing being used. As an example the use of shot fired dowels in high strength concrete would not be achievable due to the method of application.
- To optimise the specification of the sole plate to substrate connection the level of required fixity can be balanced with the level of required shear transfer. In design due consideration should be given to the failure mechanism with failure in a ductile mode favoured. As a result it is advantageous to over specify the level of sole plate to substrate connection relative to the nailed connection between the sole plate and shear wall footer. The sole plate to shear wall footer connection will tend to fail in a ductile fashion due to the method of fixity (600N/mm^2 tensile strength galvanised wire nails) compare to the majority of concrete shear fixings considered.
- The cheapest fixing is not necessarily the most cost effective option. The KF7.5×80, although the third most expensive fixing investigated, has the most value in terms of provision of lateral strength for the sole plate to substrate connection.
- For the range of substrates considered there is no practical restriction concerning the KF7.5×80 and in terms of use, application of the fixings is relatively affective although moderately slower than the application of shot fired dowels. Therefore, the KF7.5×80 is recommended in terms of cost effectiveness, practicality and performance.
- As a result of the self weight of the system a level of system redundancy could in theory be used to reduce the level of racking panel holding down required. The corresponding affect of

this on the overall racking resistance of the system has not be quantified and as a result is not recommended and requires further investigation.

- An alternative method of holding down which could be utilised is the withdrawal resistance of the shear fixings. A single shear fixing cannot, in accordance with good design practice, provide both holding down and shear resistance but the specification of shear fixings for the system could be carried out in a manner so that in total they provide both forms of resistance. To what extent this can be achieved requires to be quantified by full scale racking tests.
- The withdrawal resistance of all the fixings tested is governed by (with the exception of the KMN72 shot fired dowels) headside pull though which it has been demonstrated can be conservatively estimated using the EC5 method of design.

3.7 Conclusions

The following conclusions have been drawn and recommendations are made as a result of the analysis work on the stability of timber platform frame systems:

- Stiffness Proportionality is achieved by giving due consideration to the level of stiffness a wall diaphragm brings to the system as a result of its make-up, dimensions, level of opening and distance from the geometric centre of the system. Where possible, especially in systems where shear wall resistance is close in terms of magnitude to the applied shear force it is important to have stiffness proportionality.
- The strength of a connection can be critical when considering system continuity. In particular connections across party walls are highlighted, this connection can only be considered sufficient if the residual shear from the first block is less than the strength of the connection between the blocks.
- The connection between the wall plate and the sole plate is often critical in determining the racking resistance of a wall and should be over specified relative to the connection between the sole plate and the substrate to ensure that if failure does occur it is ductile.
- The design method of EC5 can be used with a degree of interpretation to provide the conservative estimation of yield moment, lateral load carrying capacity and withdrawal resistance for industry standard dowel type connections used for sole plate to substrate detailing. To improve specification and ensure safety testing of representative samples is recommended.
- The appropriate specification of "shear fixings" could provide both resistance to applied lateral loads and overturning forces. However, to ensure safety further testing is required including racking tests on full scale racking panels.

Published work

1. **Hairstans, R., Dodyk, R. and Kermani, A.** (2006) *"Stability of domestic dwellings"*. Proceedings of the 9th World Conference on Timber Engineering, 6-10 August 2006, Oregon USA

CHAPTER 4

DESIGN FOR STABILITY: DEVELOPMENT OF SEMI-EMPIRICAL MODELS

4.1 Introduction

Shown in Figure 4.1 is the development of 3 storey apartment blocks. It is notable from Figure 4.1 that a high level of opening is required in the front of the buildings with a negligible amount of opening required in the side due to architectural requirements. The architectural layout of a building affects its stability. However, the design procurement process of timber platform frame is structured in a form whereby the architect dictates the layout requirements (see Chapter 2 section 2.6 for further information). As a result the system is engineered to fit the layout with minimum balance between what is architecturally required and what can be structurally achieved. Demonstrated in this chapter, by means of developing and then combining semi-empirical models, is how the architecturally required layout impinges upon both the robustness and efficiency of the system.

a) 3 Storey timber platform frame

b) Masonry cladding of timber frame

c) Finished development

Figure 4.1 3 story apartment blocks

In the first instance a semi-empirical model is developed to determine the level of applied shear force on an orthogonal house up to three storeys high for a range of site (wind speed, altitude and distance from the sea) and building (height to ridge and eaves, length and width, aspect ratio and pitch angle) variables. Secondly an empirical model is developed which determines the level of racking resistance a timber frame wall can provide considering the sheathing arrangement and its level of fixity (make-up), the masonry cladding arrangement (wall type) and the level of opening it is to contain. The two models are then combined to determine the optimum level of opening which a timber frame wall of given make-up and type can allow relative to site and building conditions. The model is then used on industry standard design cases to demonstrate how it can be used to balance architectural requirements with what can be achieved structurally. The model is based on British Standard codes of practice as at the time of writing the UK National Annex for the European Code of Practice had not been finalised.

Finally a cost benefit analysis is conducted to quantify the cost effectiveness of attainable system racking performance. Based on 2006 prices the material cost of racking panel make-up is compared to the level of resistance it can provide. Cost of allowable opening is then considered and using the developed semi-empirical model the cost effectiveness of racking panel make-up and wall type are considered relative to building and site parameters.

4.2 Derivation of Required Racking Resistance Model

The formation of a semi-empirical model to determine the required racking resistance of masonry clad timber platform frame domestic dwellings was defined within the parameters as set out in Table 4.1.

A parametric study was conducted to determine the influence of the combination of building variables (Table 4.2) on the required racking resistance per metre run of the external walls. The calculated applied wind on the system was assumed to be distributed evenly between the external walls of the system parallel to the action of the wind (internal walls, at this stage, were not considered). As a result of even distribution between the external walls torsion of the system is negligible and therefore not considered in the analysis.

Table 4.1 and BS 6399-3:1997 Loading for buildings - Part 2: Code of practice for wind loads was used to determine the applied wind actions as a result of the non-availability of the UK National Annex for BS EN 1991-1-4:2005 Eurocode 1: Actions on Structures - Part 1-4: General actions – Wind.

The use of BS 6399-3:1998 was based on the following assumptions:

- The topography of the locations which the model is to be derived to encompass will not be significant as a result of the buildings the model is to consider forming part of a housing estate and therefore equation 2.2.2.2.2(9) of the code applies.
- The directional, seasonal and probability factors are all conservatively set as one.
- Only orthogonal buildings are to be covered by the derived model.

Table 4.1 Designation of variables and reasons for consideration

Site Variables	Symbol/ Units	Range of variables	Reason
Basic wind speed according to BS 6399	V_b (m/s)	23, 25, 27, 30	Covers the majority of wind cases in the UK.
Altitude	Δs (m)	0, 50, 100, 150, 200, 300, 400	Set out in a fashion which coincides with the topography of most housing developments
Distance from sea	D_{sea} (km)	<10; <100km; >100km.	Corresponds with the major bandings of BS 6399-3: 1998 Table 4
Building Variables	**Symbol/ Units**	**Range of variables**	**Reason**
Roof Type		Duo & Mono	Consistent with the majority of house types constructed in the UK.
Height to Ridge	H (m)	5.5, 10, 15	Taking into consideration building dimensions and roof type and pitch angle, worst case ridge heights have been specified to cover 1, 2 & 3 stories.
Height to Eaves	H_{ea} (m)	3, 7.4, 12.4	Maximum wall heights and floor depths have been considered to account for the majority of house types.
Front: Length of Building	L (m)	12, 9, 6, 3	Specified to cover the majority of cases and corresponding wind actions.
Gable: Width of Building	W (m)	7.5, 4.5, 3	
Aspect ratio	$\beta = L{:}W$	12:7.5 (1.6), 6:3 (2), 3:3 (1), 9: 3 (3), 9:4.5 (2); 12:3(4)	Combination of majority of cases and extremes considering the lengths and widths specified.
Pitch angle	ψ (deg)	35	Specified as the maximum pitch angle which covers the majority of house types under consideration.

Table 4.2 Combination of parameters

Type	Height to ridge, H (m)	Distance from sea, D_{sea} (km)	Altitude, Δs (m)	Wind Speed, V_b (m/s)	Aspect Ratio (β = Front: Gable)
Duo	5.5 10 15	D_{sea}<10; 10≤D_{sea}≤100; D_{sea}>100	0, 50, 100, 150, 200, 300, 400	23, 25, 27, 30.	12.0 : 3.0 (β = 4) 12.0 : 7.5 (β = 1.6) 6.0 : 3.0 (β = 2) 3.0 :3 .0 (β = 1) 9.0:3.0 (β = 3) 9.0:4.5 (β = 2)
Mono	5.5 10 15	D_{sea}<10; 10≤D_{sea}≤100; D_{sea}>100	0, 50, 100, 150, 200, 300, 400	23, 25, 27, 30.	12.0 : 3.0 (β = 4) 12.0 : 7.5 (β = 1.6) 6.0 : 3.0 (β = 2) 3.0 :3 .0 (β = 1) 9.0:3.0 (β = 3) 9.0:4.5 (β = 2)

Contained in Table 4.3 are arbitrary constants which were set based on the ratio of altitude to distance to sea of the building.

Table 4.3 Altitude to distance ratios

Altitude, Δs	Distance from sea. D_{sea} (km) ≤10	≤100	>100
	α		
0m	0.09	0.08	0.08
< 50m	0.1	0.09	0.09
< 100m	1.0	0.92	0.87
< 150m	1.8	1.66	1.57
< 200m	2.2	2.02	1.92
< 300m	5.0	4.60	4.37
< 400m	6.0	5.52	5.24

Shown in Figure 4.2 is the variation in required racking resistance of the system walls relative to the aspect ratio for a given roof type, distance from the sea and altitude. As a typical example, for each case shown in Figure 4.2 the wind speed has been set to the worst case scenario of 30m/s. The trend lines which correspond to each set of data are of a logarithmic type:

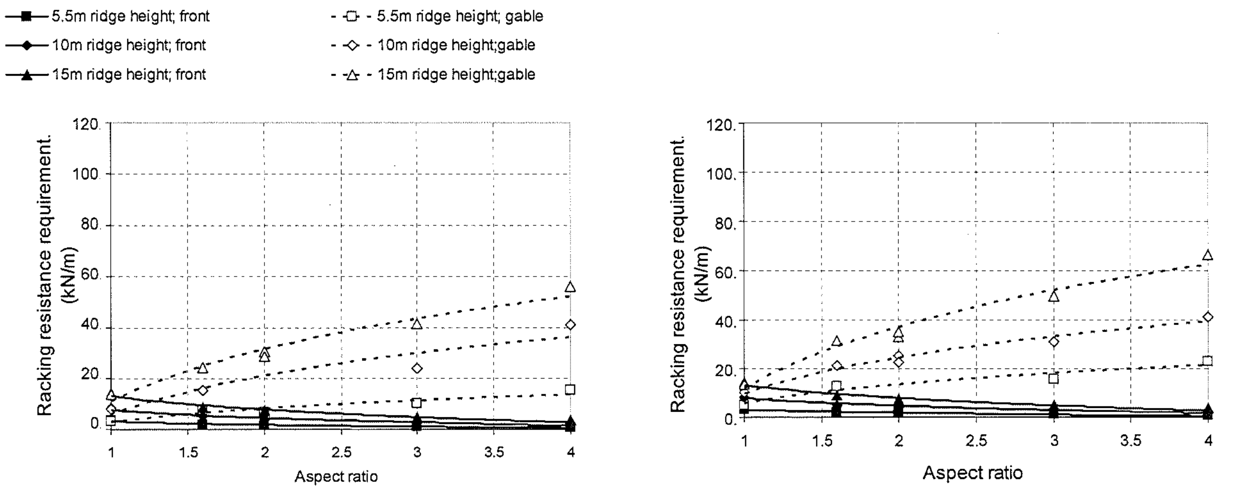
$$\textit{Racking Resitance Requirement} = A \cdot \ln(\beta) + B \qquad \text{Equation 4.1}$$

Where:

β is the aspect ratio

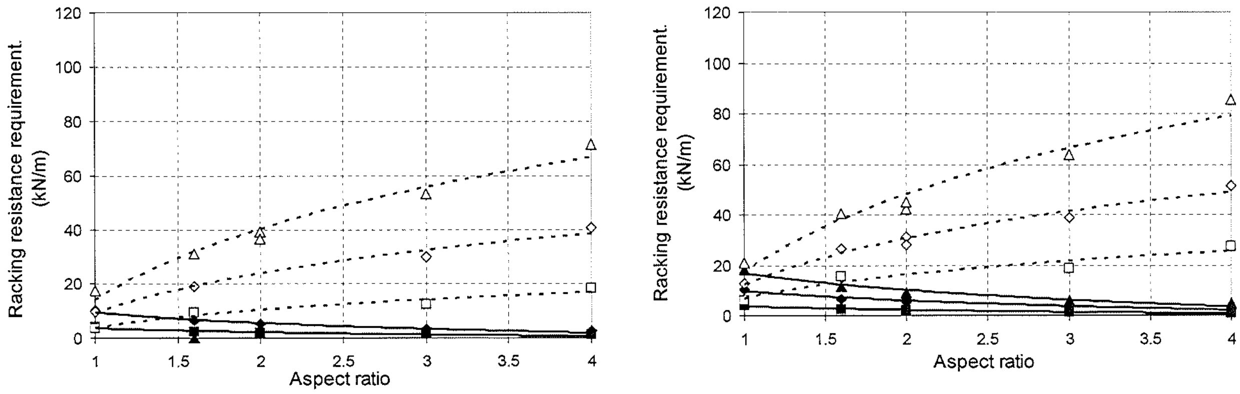
A & B are as defined in **Table 4.4** for each of the given trend lines presented.

Racking Resistance Requirement is in kN/m run.



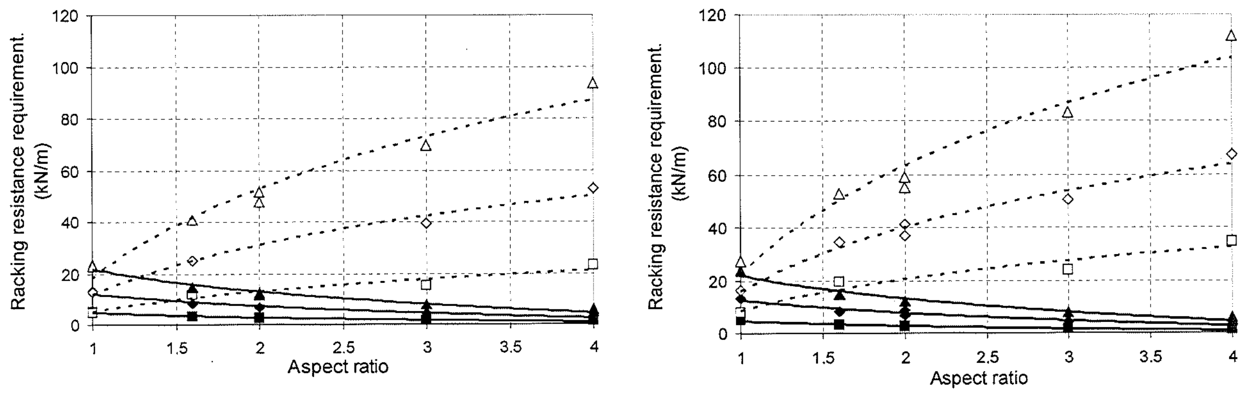
a) Duo; D_{sea}<10km; Δs = 0m; V_b = 30m/s

b) Mono; D_{sea}<10km; Δs = 0m; V_b = 30m/s



c) Duo; 10≤D_{sea}≤100; Δs = 150m; V_b = 30m/s

d) Mono; 10≤D_{sea}≤100; Δs = 150m; V_b = 30m/s



e) Duo; D_{sea} >100km; Δs = 400m; V_b = 30m/s

f) Mono; D_{sea} >100km; Δs = 400m; V_b = 30m/s

Figure 4.2 Racking resistance requirement against aspect ratio

Table 4.4 Values of constants corresponding to Equation 4.1 and Figure 4.2

Plot	Fixed Parameters	Ridge height	Side	Equation constant		R^2
		m		A	B	
a	Duo pitch roof ($\psi = 35^o$); D_{sea} <10km; Δs = 0m; V_b 30m/s	5.5	Front	-1.83	3.10	0.93
			Gable	7.74	3.28	0.94
		10	Front	-4.35	7.50	0.95
			Gable	21.92	5.93	0.90
		15	Front	7.45	12.95	0.95
			Gable	29.64	11.63	0.98
c	Duo pitch roof ($\psi = 35^o$); $10 \leq D_{sea} \leq 100$km; Δs = 150m; V_b = 30m/s	5.5	Front	-2.19	3.71	0.93
			Gable	9.26	3.93	0.94
		10	Front	-5.42	9.36	0.95
			Gable	21.26	9.17	0.98
		15	Front	-10.95	17.18	0.97
			Gable	32.62	16.87	1.00
e	Duo pitch roof ($\psi = 35^o$); D_{sea}>100km; Δs = 400m; V_b = 30m/s	5.5	Front	-2.77	4.69	0.93
			Front	11.70	4.98	0.94
		10	Gable	-7.05	12.16	0.95
			Front	27.63	11.92	0.98
		15	Gable	-12.48	21.68	0.95
			Front	49.62	19.47	0.98
b	Mono pitch roof ($\psi = 35^o$); D_{sea}<10km; Δs = 0m; V_b = 30m/s	5.5	Front	-1.83	3.10	0.93
			Gable	11.34	5.72	0.93
		10	Front	-4.43	7.79	0.94
			Gable	21.27	9.86	0.97
		15	Front	-7.61	13.16	0.95
			Gable	36.73	11.73	0.97
d	Mono pitch roof ($\psi = 35^o$); $10 \leq D_{sea} \leq 100$km; Δs = 150m; V_b = 30m/s	5.5	Front	-2.19	3.71	0.93
			Gable	-5.53	9.73	0.94
		10	Front	-5.53	9.73	0.94
			Gable	26.56	12.32	0.97
		15	Front	-9.73	16.84	0.95
			Gable	44.81	17.19	0.96
f	Mono pitch roof ($\psi = 35^o$); D_{sea}>100km; Δs = 400m; V_b = 30m/s	5.5	Front	-2.77	4.69	0.93
			Gable	17.17	8.66	0.93
		10	Front	-7.19	12.65	0.94
			Gable	30.48	17.74	0.97
		15	Front	-12.74	22.03	0.95
			Gable	58.63	22.49	0.96

From Figure 4.2 the following conclusions are drawn:

- The effect of roof shape, Duo or Mono, has a negligible effect on the required racking resistance of the system walls.
- Altitude and distance from the sea are shown to have an effect on the required racking resistance. However, the dominant factors which increase the required racking resistance are demonstrated to be a combination of aspect ratio and height to the ridge.
- The effect of ridge height on racking requirement becomes more prominent as the aspect ratio is increased. Therefore, aspect ratio is shown to be the governing factor in terms of increased

racking requirements. The closer the aspect ratio can be kept to 1 the more evenly distributed are the racking forces.

Equation 4.1 provides a relatively accurate estimation of required racking resistance for the front, back or gables of a building from the known aspect ratio if all the other parameters are known and the constants, A & B are therefore defined. To derive an equation which incorporates the variables of distance from the sea and altitude, the constants A & B from Equation 4.1 are plotted against the ratio of altitude to distance for a constant building height to ridge and wind velocity (Figure 4.3). The altitude to distance ratios were chosen to be within a range which was representative of the majority of locations where housing development sites would be, for example it would be uncommon to build at locations with an altitude over 400m. Therefore, a degree of judgement was used to reduce the range to be more representative.

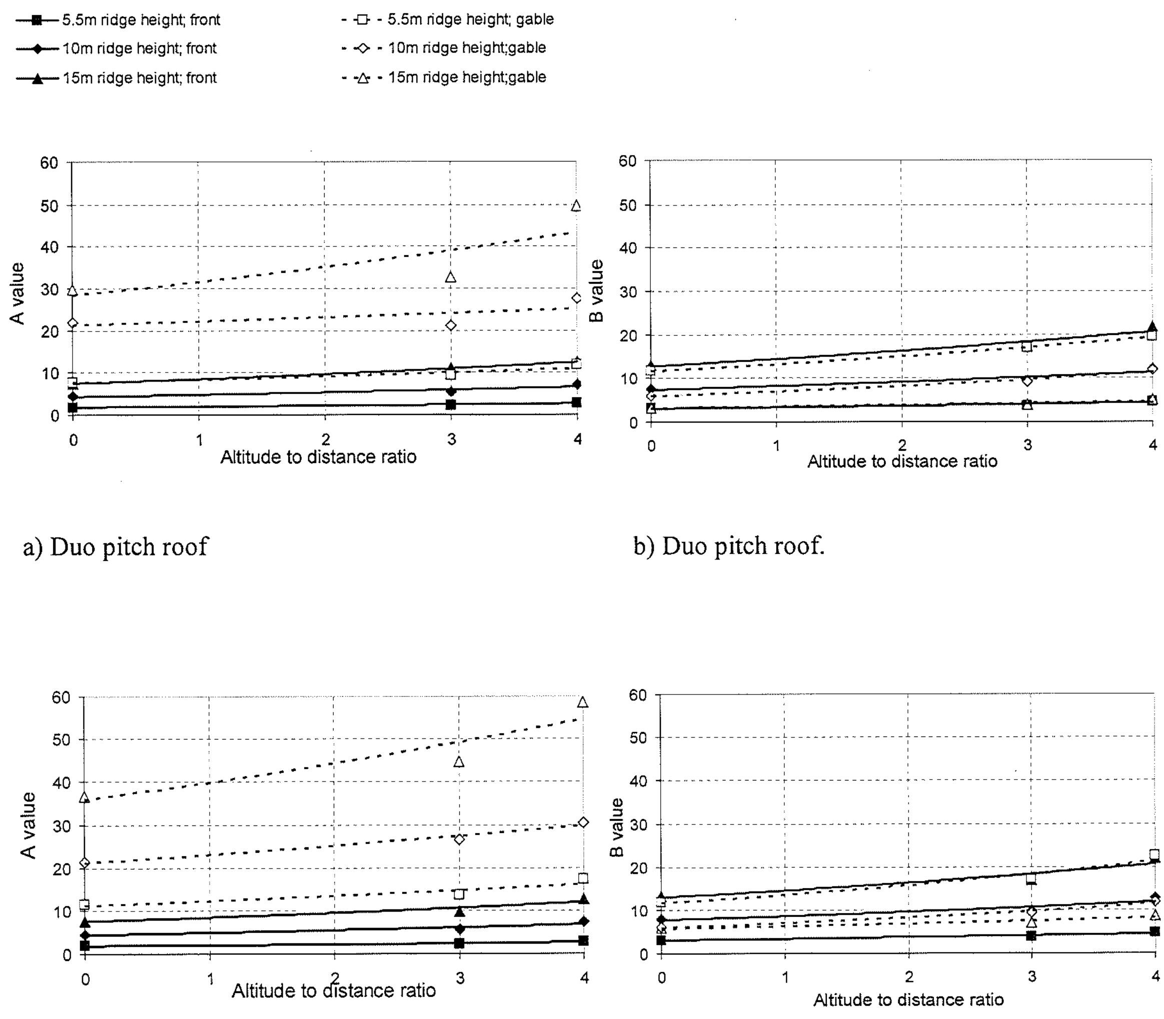
The trend lines which correspond to each set of data in Figure 4.3 are of exponential type:

$$Constants\ A\ \&\ B = P \cdot e^{Q \cdot \alpha} \qquad \text{Equation 4.2}$$

Where:

α is the site altitude to distance from the sea ratio.

P & Q are as defined in Table 4.5 for each of the given trend lines presented.



a) Duo pitch roof

b) Duo pitch roof.

c) Mono pitch roof

d) Mono pitch roof

Figure 4.3 Front & Gable wall panel racking constant *A* & *B* against the site altitude to distance from the sea ratio, α.

Table 4.5 Values of constants corresponding to Equation 4.2 and Figure 4.3

Fixed Parameters	Ridge height	Side	Racking constant, A			Racking constant, B		
			Equation constant		R^2	Equation constant		R^2
	m		P	Q		P	Q	
Duo pitch roof ($\psi = 35^o$); Front wall panels; $V_b = 30$m/s	5.5	Front	-1.80*	0.09	0.88	3.04	0.09	0.88
		Gable	7.58	0.09	0.88	3.12	0.09	0.88
	10	Front	-4.25*	0.11	0.90	7.34	0.11	0.90
		Gable	21.25	0.04	0.37	5.85	0.17	0.98
	15	Front	-7.45*	0.13	1.00	12.74	0.12	0.95
		Gable	28.34	0.11	0.65	11.60	0.13	1.00
Mono pitch roof ($\psi = 35^o$); Front wall panels; $V_b = 30$m/s	5.5	Front	-1.80*	0.094	0.88	3.04	0.094	0.88
		Gable	11.12	0.094	0.88	5.61	0.094	0.88
	10	Front	-4.33*	0.11	0.90	7.62	0.11	0.90
		Gable	21.11	0.086	0.98	9.53	0.13	0.83
	15	Front	-7.44*	0.12	0.91	12.88	0.12	0.91
		Gable	35.88	0.11	0.87	11.54	0.15	0.97

*Note: Converted to negative value to reflect information from Figure 4.2.

The variation in racking resistance requirement of the walls was also considered relative to the wind speed. Therefore, for varying building and location parameters, the required racking resistance against wind speed was plotted (Figure 4.4 & Figure 4.5) in order that a relationship could be defined.

The trend lines which correspond to each set of data in Figure 4.4 & Figure 4.5 are of a power type:

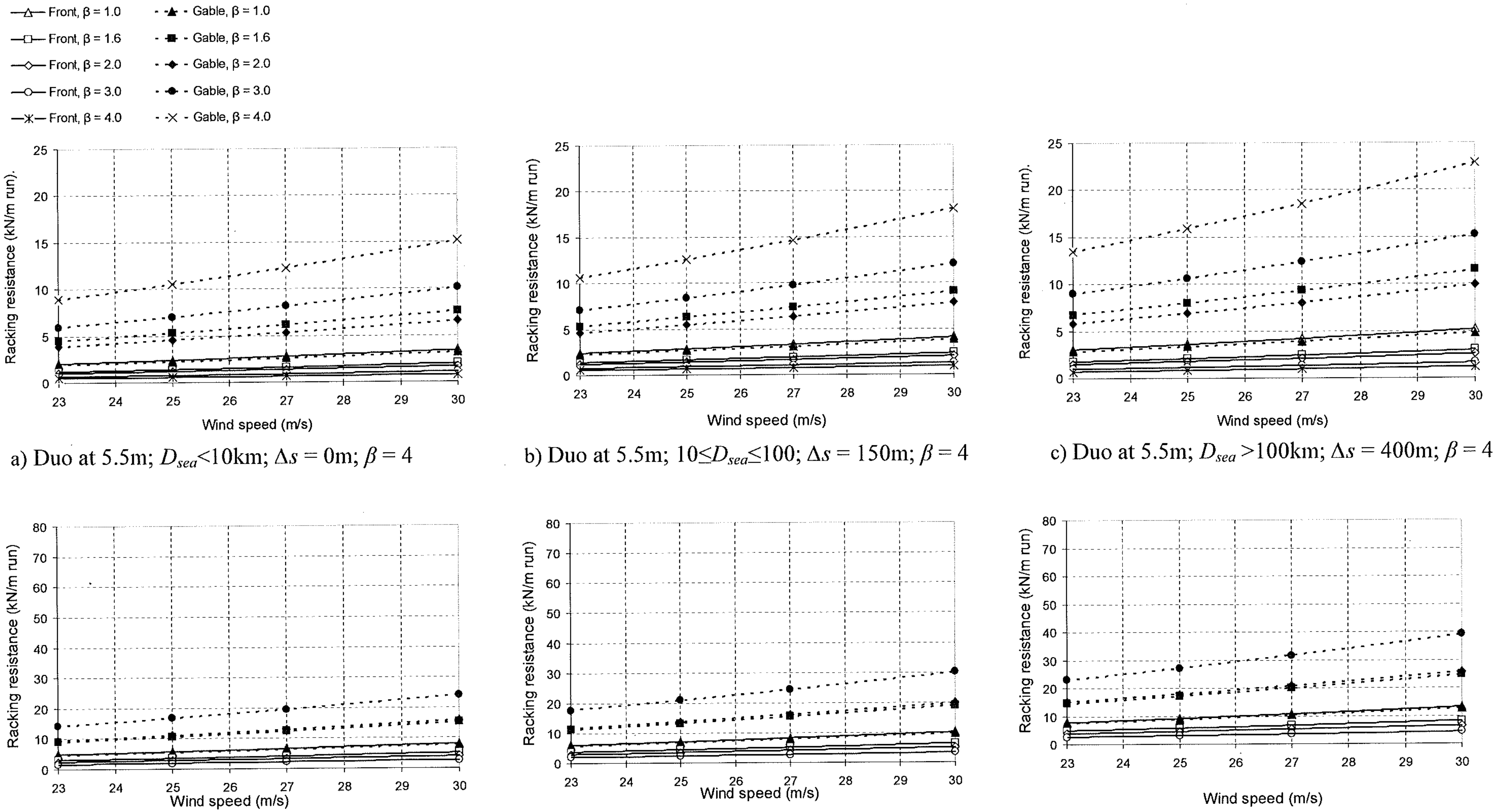
$$Racking\ Resistance\ Requireed = m \cdot V_b^{\ c} \qquad \text{Equation 4.3}$$

Where:

V_b is the wind speed in metres per second.

m & c are as defined in Table 4.6 for each of the given trend lines presented.

Racking Resistance Requirement is in kN/m run.



a) Duo at 5.5m; D_{sea}<10km; Δs = 0m; β = 4

b) Duo at 5.5m; 10≤D_{sea}≤100; Δs = 150m; β = 4

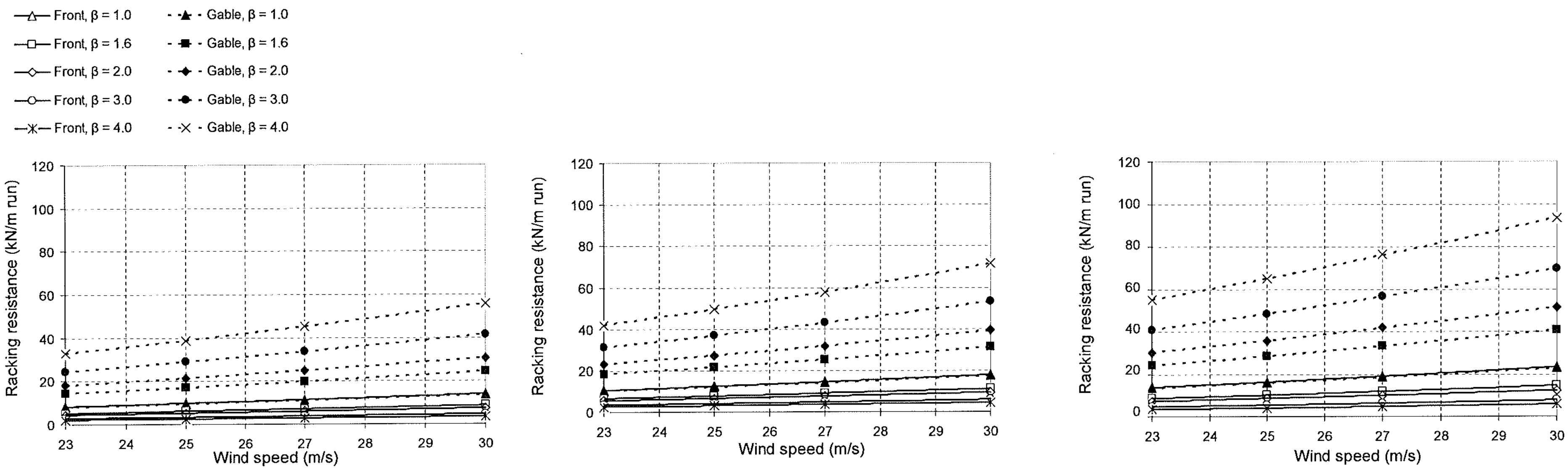
c) Duo at 5.5m; D_{sea} >100km; Δs = 400m; β = 4

a) Duo at 10m; D_{sea}<10km; Δs = 0m; β = 4

b) Duo at 10m; 10≤D_{sea}≤100; Δs = 150m; β = 4

c) Duo at 10m; D_{sea} >100km; Δs = 400m; β = 4

Figure 4.4 Racking resistance requirement for duo pitch roof (Ridge Height 5.5 & 10m)

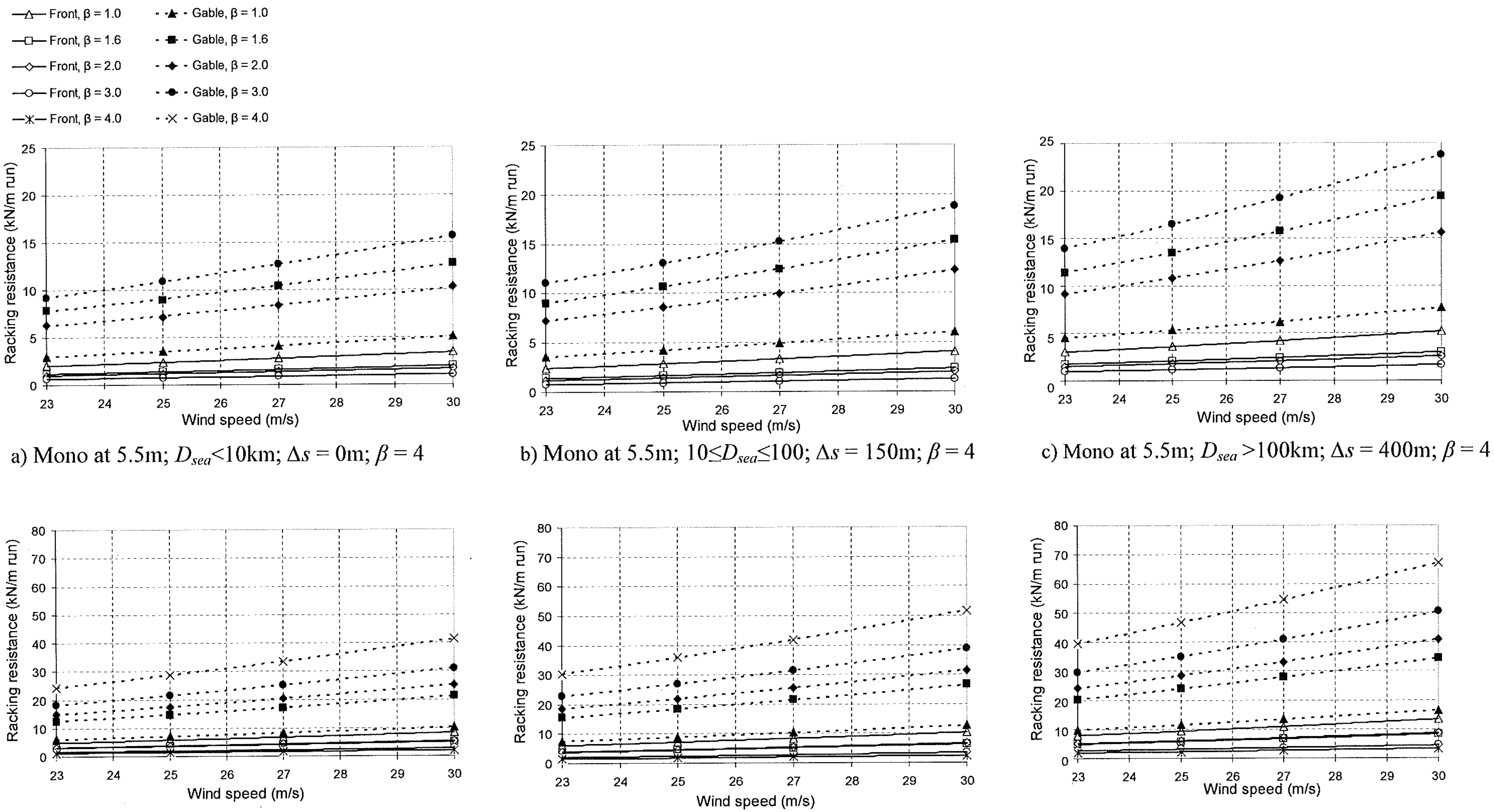


a) Duo at 15m; D_{sea}<10km; Δs = 0m; β = 4

b) Duo at 15m; 10≤D_{sea}≤100; Δs = 150m; β = 4

c) Duo at 15m; D_{sea} >100km; Δs = 400m; β = 4

Figure 4.4 Racking resistance requirement for duo pitch roof (Ridge Height 15m)



a) Mono at 5.5m; D_{sea}<10km; Δs = 0m; β = 4

b) Mono at 5.5m; 10≤D_{sea}≤100; Δs = 150m; β = 4

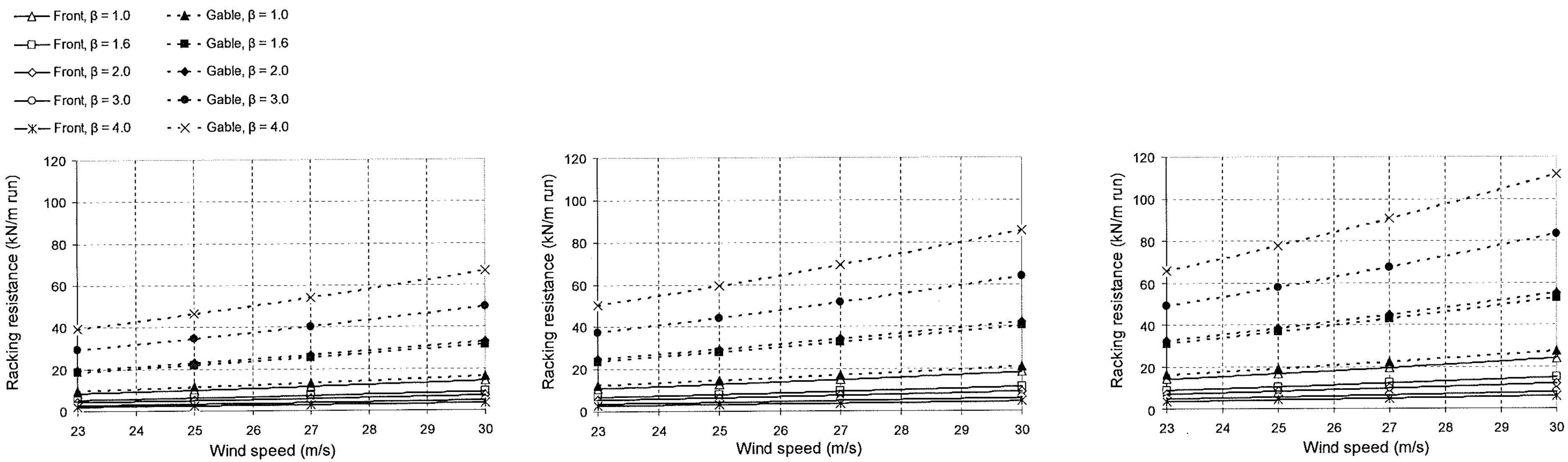
c) Mono at 5.5m; D_{sea} >100km; Δs = 400m; β = 4

a) Mono at 10m; D_{sea}<10km; Δs = 0m; β = 4

b) Mono at 10m; 10≤D_{sea}≤100; Δs = 150m; β = 4

c) Mono at 10m; D_{sea} >100km; Δs = 400m; β = 4

Figure 4.5 Racking resistance requirement for mono pitch roof (Ridge Height 5.5 & 10m)



a) Mono at 15m; D_{sea}<10km; Δs = 0m; $\beta = 4$

b) Mono at 15m; 10≤D_{sea}≤100; Δs = 150m; $\beta = 4$

c) Mono at 15m; D_{sea} >100km; Δs = 400m; $\beta = 4$

Figure 4.5 Racking resistance requirement for mono pitch roof (Ridge Height 15m)

Table 4.6 Values of constants corresponding to Equation 4.3 and Figure 4.4 (Duo Pitch)

Fixed Parameters	Aspect ratio, β	D_{sea}<10km			$10 \le D_{sea} \le 100$km			D_{sea}>100km		
		m	c	R^2	m	c	R^2	m	c	R^2
Gable walls; Duo pitch roof; Ridge Height, H = 5.5m;	1.0	0.004	1.997	1.0	0.004	1.999	1.0	0.006	2.000	1.0
	1.6	0.009	1.997	1.0	0.010	1.999	1.0	0.013	2.000	1.0
	2.0	0.007	1.998	1.0	0.009	1.999	1.0	0.011	2.000	1.0
	3.0	0.011	2.000	1.0	0.013	1.999	1.0	0.017	2.000	1.0
	4.0	0.168	2.000	1.0	0.020	1.999	1.0	0.026	1.996	1.0
Front walls; Duo pitch roof; Ridge Height, H = 5.5m;	1.0	0.004	1.999	1.0	0.005	2.000	1.0	0.005	2.000	1.0
	1.6	0.002	2.000	1.0	0.003	2.000	1.0	0.003	2.000	1.0
	2.0	0.002	2.000	1.0	0.002	2.000	1.0	0.003	2.001	1.0
	3.0	0.001	2.000	1.0	0.002	2.000	1.0	0.002	2.000	1.0
	4.0	0.001	1.999	1.0	0.001	2.000	1.0	0.001	2.000	1.0
Gable walls; Duo pitch roof Ridge Height, H = 10m;	1.0	0.009	2.000	1.0	0.011	2.000	1.0	0.044	2.000	1.0
	1.6	0.017	1.998	1.0	0.021	2.000	1.0	0.028	2.000	1.0
	2.0	0.018	1.996	1.0	0.022	2.000	1.0	0.123	1.963	1.0
	3.0	0.027	2.001	1.0	0.034	2.000	1.0	0.044	2.000	1.0
	4.0	0.036	1.996	1.0	0.045	2.001	1.0	0.059	1.999	1.0
Front walls; Duo pitch roof Ridge Height, H = 10m;	1.0	0.009	2.000	1.0	0.011	2.000	1.0	0.015	1.997	1.0
	1.6	0.006	2.001	1.0	0.007	2.000	1.0	0.009	1.999	1.0
	2.0	0.005	2.002	1.0	0.006	2.000	1.0	0.032	1.560	1.0
	3.0	0.003	2.000	1.0	0.004	2.000	1.0	0.005	1.997	1.0
	4.0	0.002	2.001	1.0	0.003	2.000	1.0	0.004	1.999	1.0
Gable walls; Duo pitch roof Ridge Height, H = 15m;	1.0	0.015	1.994	1.0	0.019	2.001	1.0	0.025	2.005	1.0
	1.6	0.027	2.000	1.0	0.035	2.001	1.0	0.045	2.005	1.0
	2.0	0.034	1.999	1.0	0.044	2.001	1.0	0.057	2.004	1.0
	3.0	0.016	2.001	1.0	0.059	2.001	1.0	0.076	2.001	1.0
	4.0	0.062	2.000	1.0	0.079	2.001	1.0	0.103	2.005	1.0
Front walls; Duo pitch roof Ridge Height, H = 15m;	1.0	0.052	2.000	1.0	0.007	2.001	1.0	0.009	2.066	1.0
	1.6	0.010	1.999	1.0	0.012	2.001	1.0	0.016	2.000	1.0
	2.0	0.009	1.986	1.0	0.020	2.001	1.0	0.014	1.997	1.0
	3.0	0.005	2.000	1.0	0.007	2.001	1.0	0.009	2.008	1.0
	4.0	0.004	1.999	1.0	0.005	2.000	1.0	0.007	2.001	1.0

Table 4.7 Values of constants corresponding to Equation 4.3 and Figure 4.5 (Mono Pitch)

Fixed Parameters	Aspect ratio, β	D_{sea}<10km			$10 \leq D_{sea} \leq 100$km			D_{sea}>100km		
		m	*c*	R^2	*m*	*c*	R^2	*m*	*c*	R^2
Gable walls; Mono pitch roof; Ridge Height, H = 5.5m;	1.0	0.056	2.000	1.0	0.007	2.005	1.0	0.009	2.000	1.0
	1.6	0.024	1.850	1.0	0.017	2.001	1.0	0.022	2.000	1.0
	2.0	0.019	1.850	1.0	0.014	2.001	1.0	0.017	2.000	1.0
	3.0	0.017	2.000	1.0	0.021	2.001	1.0	0.026	2.000	1.0
	4.0	0.041	1.851	1.0	0.030	2.000	1.0	0.039	1.999	1.0
Front walls; Mono pitch roof; Ridge Height, H = 5.5m;	1.0	0.004	1.999	1.0	0.005	2.000	1.0	0.006	2.000	1.0
	1.6	0.002	2.000	1.0	0.003	2.000	1.0	0.003	2.000	1.0
	2.0	0.002	2.000	1.0	0.002	2.000	1.0	0.003	2.000	1.0
	3.0	0.001	2.000	1.0	0.002	2.000	1.0	0.002	2.009	1.0
	4.0	0.001	1.999	1.0	0.001	2.000	1.0	0.001	2.000	1.0
Gable walls; Mono pitch roof Ridge Height, H = 10m;	1.0	0.01	2.000	1.0	0.01	2.00	1.0	0.02	2.00	1.0
	1.6	0.024	2.001	1.0	0.030	1.997	1.0	0.038	2.000	1.0
	2.0	0.028	2.001	1.0	0.035	1.999	1.0	0.045	2.000	1.0
	3.0	0.034	2.001	1.0	0.043	1.997	1.0	0.056	2.000	1.0
	4.0	0.046	2.001	1.0	0.058	1.997	1.0	0.074	2.004	1.0
Front walls; Mono pitch roof Ridge Height, H = 10m;	1.0	0.01	2.000	1.0	0.01	2.00	1.0	0.01	2.000	1.0
	1.6	0.006	2.001	1.0	0.007	2.000	1.0	0.009	1.999	1.0
	2.0	0.006	2.001	1.0	0.007	2.000	1.0	0.010	1.999	1.0
	3.0	0.003	2.000	1.0	0.004	2.000	1.0	0.005	1.999	1.0
	4.0	0.002	2.001	1.0	0.003	2.000	1.0	0.004	1.999	1.0
Gable walls; Mono pitch roof Ridge Height, H = 15m;	1.0	0.018	2.000	1.0	0.023	2.001	1.0	0.031	2.000	1.0
	1.6	0.035	2.000	1.0	0.045	2.001	1.0	0.059	2.000	1.0
	2.0	0.037	1.999	1.0	0.047	2.001	1.0	0.061	2.000	1.0
	3.0	0.055	1.999	1.0	0.071	2.001	1.0	0.093	2.000	1.0
	4.0	0.074	2.001	1.0	0.095	2.000	1.0	0.124	2.000	1.0
Front walls; Mono pitch roof Ridge Height, H = 15m;	1.0	0.016	1.988	1.0	0.020	1.999	1.0	0.027	1.999	1.0
	1.6	0.013	1.999	1.0	0.013	1.999	1.0	0.017	1.999	1.0
	2.0	0.008	1.999	1.0	0.010	1.999	1.0	0.013	1.999	1.0
	3.0	0.005	2.000	1.0	0.007	1.999	1.0	0.009	1.999	1.0
	4.0	0.004	1.998	1.0	0.005	1.997	1.0	0.007	2.000	1.0

From the information contained in Table 4.6 and Table 4.7 constant, c, is shown to be between 1.963 and 2.066 and hence has been taken to be 2 for all cases, however, as a result of the high range of variables given for, *m*, a further iterative step was deemed necessary. Therefore, the variables, m, from Equation 4.3 are plotted against the given aspect ratios, β, for both the gable and front of each roof type (Duo and Mono) at each of the given heights (5.5m, 10m & 15m), Figure 4.6.

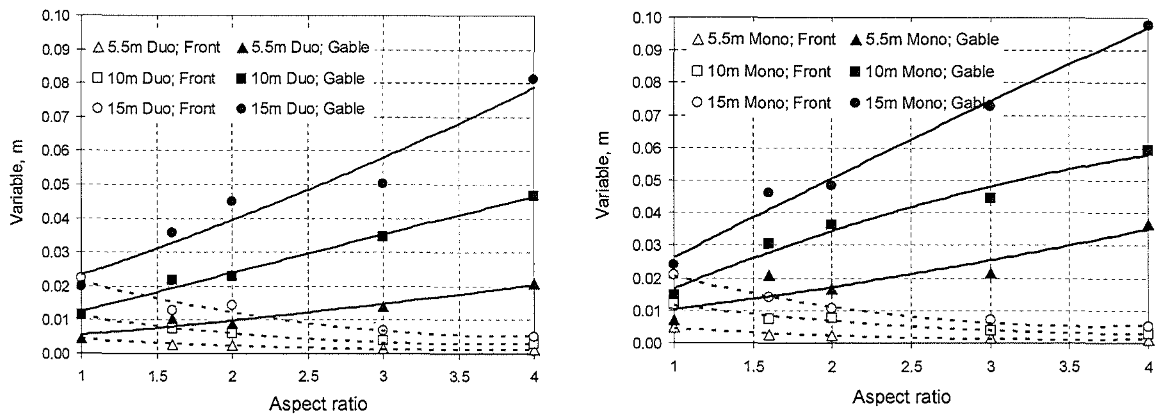
The trend lines which correspond to each set of data in Figure 4.6 are of a polynomial type:

$$m = r \cdot \beta^2 + s \cdot \beta + t \qquad \text{Equation 4.4}$$

Where:

β is the aspect ratio.

r, s & t are as defined in Table 4.8 for each of the given trend lines presented.



a) Duo pitch roof

b) Mono pitch roof

Figure 4.6 *m* against β for Duo & Mono roof types

Table 4.8 Values of constants corresponding to Equation 4.4 and Figure 4.6

Building Type and Ridge Height	Gable				Front			
	r	*s*	*t*	R^2	*r*	*s*	*t*	R^2
Duo at 5.5m	0.0004	0.0031	0.0022	0.94	0.0005	-0.0036	0.0075	0.97
Duo at 10m	0.0027	-0.0047	0.0227	0.99	0.0012	-0.0087	0.0186	0.98
Duo at 15m	-0.0063	0.0410	-0.0141	0.99	0.0023	-0.0167	0.0360	0.93
Mono at 5.5m	0.0054	-0.0228	0.0418	0.98	0.0005	-0.0038	0.0078	0.97
Mono 10m	-0.0055	0.03732	-0.0163	0.99	0.0012	-0.0084	0.0188	0.95
Mono at 15m	-0.0064	0.0486	-0.0182	0.97	0.0026	-0.0175	0.0356	1.00

To combine Equations 4.1 and 4.4 the following points were taken into account:

- Equation 4.1 determines the required racking resistance of an external wall relative to the aspect ratio, β, of the building and variables A and B which are defined by Equation 4.2.
- Equation 4.2 takes account of the Altitude to Distance Ratio, α, therefore, in its totality Equation 4.1 determines the racking resistance of an external wall depending on the aspect ratio, β, and Altitude to Distance Ratio, α, of the building.
- In a similar respect Equation 4.3 determines the required racking resistance of an external wall relative to Wind Velocity, V_b, where 30m/s is the worst case scenario, and also the variables m and c which are defined by Equation 4.4.
- Equation 4.4 takes account of the aspect ratio, β, therefore, in its totality Equation 4.3 determines the racking resistance of an external wall depending on the Wind Velocity, V_b, and aspect ratio, β.

To combine the two equations they are multiplied together and factored to take account of the wind speed. For the worst case wind scenario of 30m/s a factor of unity is considered and for lower wind speeds a reduction factor is applied, this reduction factor will be unity divided by a value which is a function of the required racking resistance of the external wall and aspect ratio, β. To determine this function, the racking resistance values (determined by applying Equation 4.4 for a set wind speed of 30m/s) are plotted against varying values of aspect ratio, β, as shown in Figure 4.7.

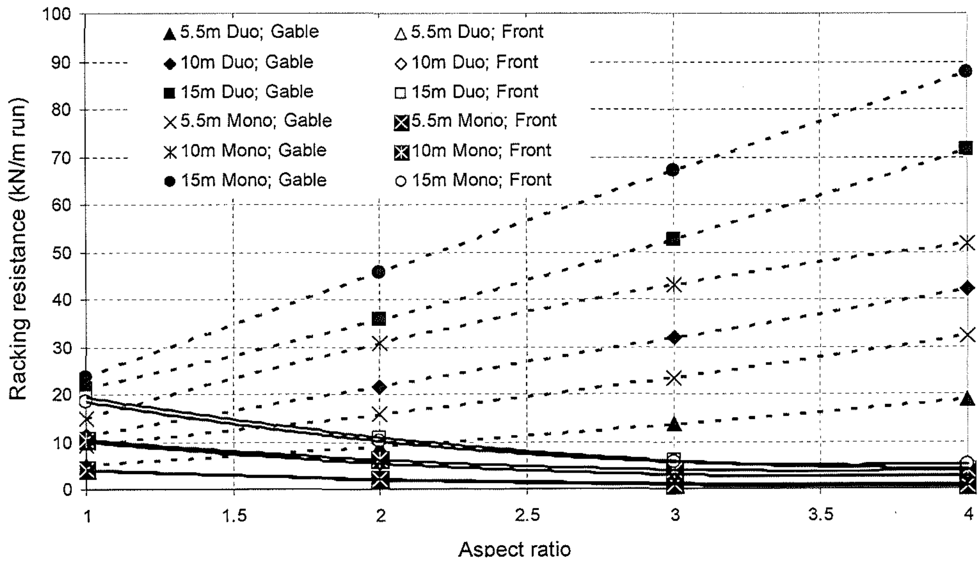


Figure 4.7 Racking resistance for varying values of aspect ratio, β (solid lines represent the front and dashed lines represent the gable ends)

The trend lines which correspond to each set of data in Figure 4.7 are of polynomial type:

$$\textit{Racking Resistance Requirement} = f \cdot \beta^2 + g \cdot \beta + h \qquad \text{Equation 4.5}$$

Where:

β is the aspect ratio.

f, g & h are as defined in Table 4.9 for each of the given trend lines presented.

Racking Resistance Requirement is in kN/m run.

Table 4.9 Values of constants corresponding to Equation 4.5 and Figure 4.7

Roof Type and Ridge Height	Gable Average				Front Average			
	f	g	h	R^2	f	g	h	R^2
Duo at 5.5m	0.36	2.79	1.98	1.00	0.45	-3.24	6.75	1.00
Duo at 10m	-0.03	10.35	0.99	1.00	1.08	-7.38	16.74	1.00
Duo at 15m	0.36	2.79	1.98	1.00	1.71	-13.59	31.23	1.00
Mono at 5.5m	0.63	4.50	4.23	1.00	0.45	-3.24	7.02	1.00
Mono 10m	-1.80	21.24	-4.50	1.00	1.08	-7.56	16.92	1.00
Mono at 15m	-0.36	23.22	0.63	1.00	1.98	-14.31	30.78	1.00

Combining Equations 4.1 & 4.4 the following equation is derived which allows a moderately conservative, but relatively accurate, estimation of racking resistance requirement of individual external walls:

$$\textit{Racking Resistance Requirement} = \left(\frac{r \cdot \beta^2 + s \cdot \beta + t}{f \cdot \beta^2 + g \cdot \beta + h} \right) \cdot V_b^{\,2} \cdot (P_A \cdot e^{Q_A \cdot \alpha} \cdot \ln(\beta) + P_B \cdot e^{Q_B \cdot \alpha})$$

Equation 4.6

Where:

β is the aspect ratio.

V_b is the basic wind speed in accordance with BS 6399-2:1997 in metres per second.

α is the site altitude to distance from the sea ratio.

Racking Resistance Requirement is in kN/m run.

It is noted that for all cases the outcome of the following part of the equation is consistent:

$$\left(\frac{r \cdot \beta^2 + s \cdot \beta + t}{f \cdot \beta^2 + g \cdot \beta + h}\right) = 0.0011$$

As a result Equation 4.6 can therefore be simplified to the following:

$$\textit{Racking Resistance Requirement} = 0.0011 V_b^{\ 2} \cdot (P_A \cdot e^{Q_A \cdot \alpha} \cdot \ln(\beta) + P_B \cdot e^{Q_B \cdot \alpha}) \qquad \text{Equation 4.7}$$

To enhance the model further relationships between P_A, Q_A, P_B and Q_B are developed over the given height range of 5.5m to 15m ridge height. This is done by plotting the variation of P_A, Q_A, P_B and Q_B over the given ridge height range as shown in Figure 4.8.

The trend lines which correspond to each set of data in Figure 4.8 are of a polynomial type:

$$P_A, Q_A, P_B \,\&\, Q_B = x \cdot H^2 + y \cdot H + z \qquad \text{Equation 4.8}$$

Where:

β is the aspect ratio.

H is the height to the ridge in metres.

x, y & z are as defined in Table 4.10 for each of the given trend lines presented.

Racking Resistance Requirement is in kN/m run.

Table 4.10 Equation constants from Figure 4.8

Variable	Duo			Mono		
P or *Q*	*x*	*y*	*z*	*x*	*y*	*z*
PA Front*	-0.0101	-0.3885	0.6412	-0.0063	-0.4647	0.9461
PA Gable	-0.1750	5.6806	-18.5050	0.0773	1.0220	3.1595
PB Front	0.0132	0.7506	-1.4911	0.0036	0.9619	-2.3596
PB Gable	0.0573	-0.2832	2.9468	-0.0494	1.6365	-1.8970
QA Front	0.0001	0.0028	0.0770	-0.0002	0.0061	0.0654
QA Gable	0.0026	-0.0520	0.2997	0.0007	-0.0125	0.1419
QB Front	-0.0002	0.0062	0.0646	-0.0002	0.0061	0.0654
QB Gable	-0.0026	0.0562	-0.1372	-0.0004	0.0145	0.0268

*Note: These constants have been converted to negatives to inverse the previous conversion required so that an exponential trend could be applied.

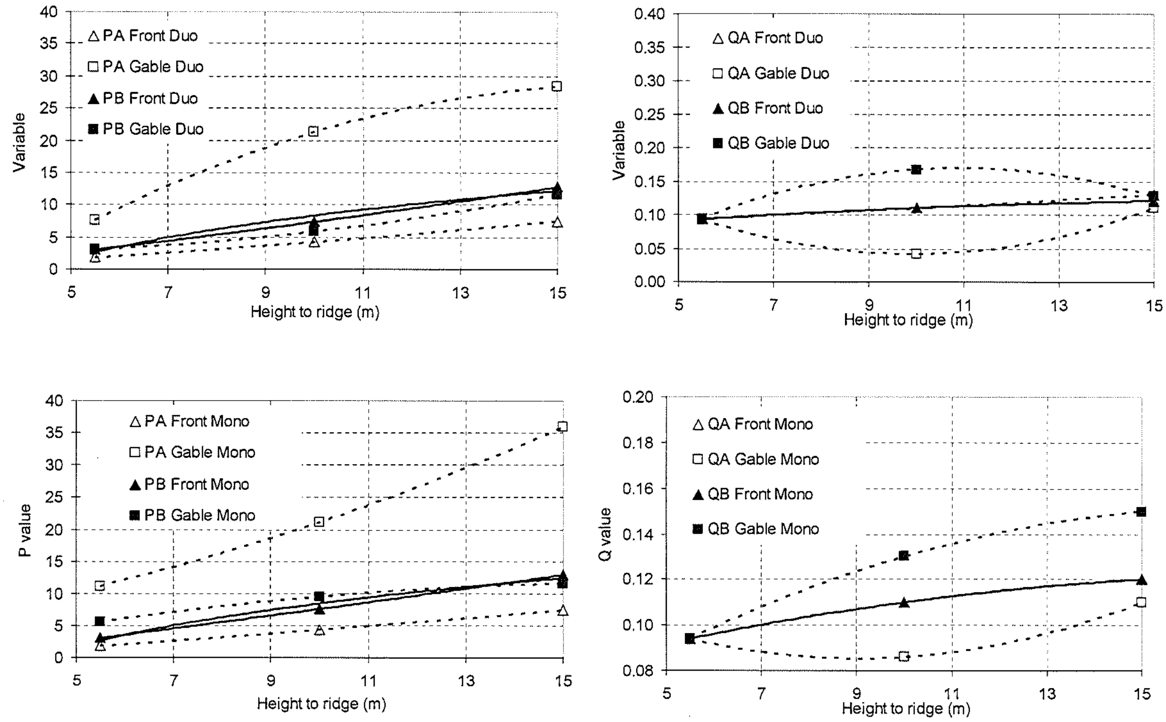


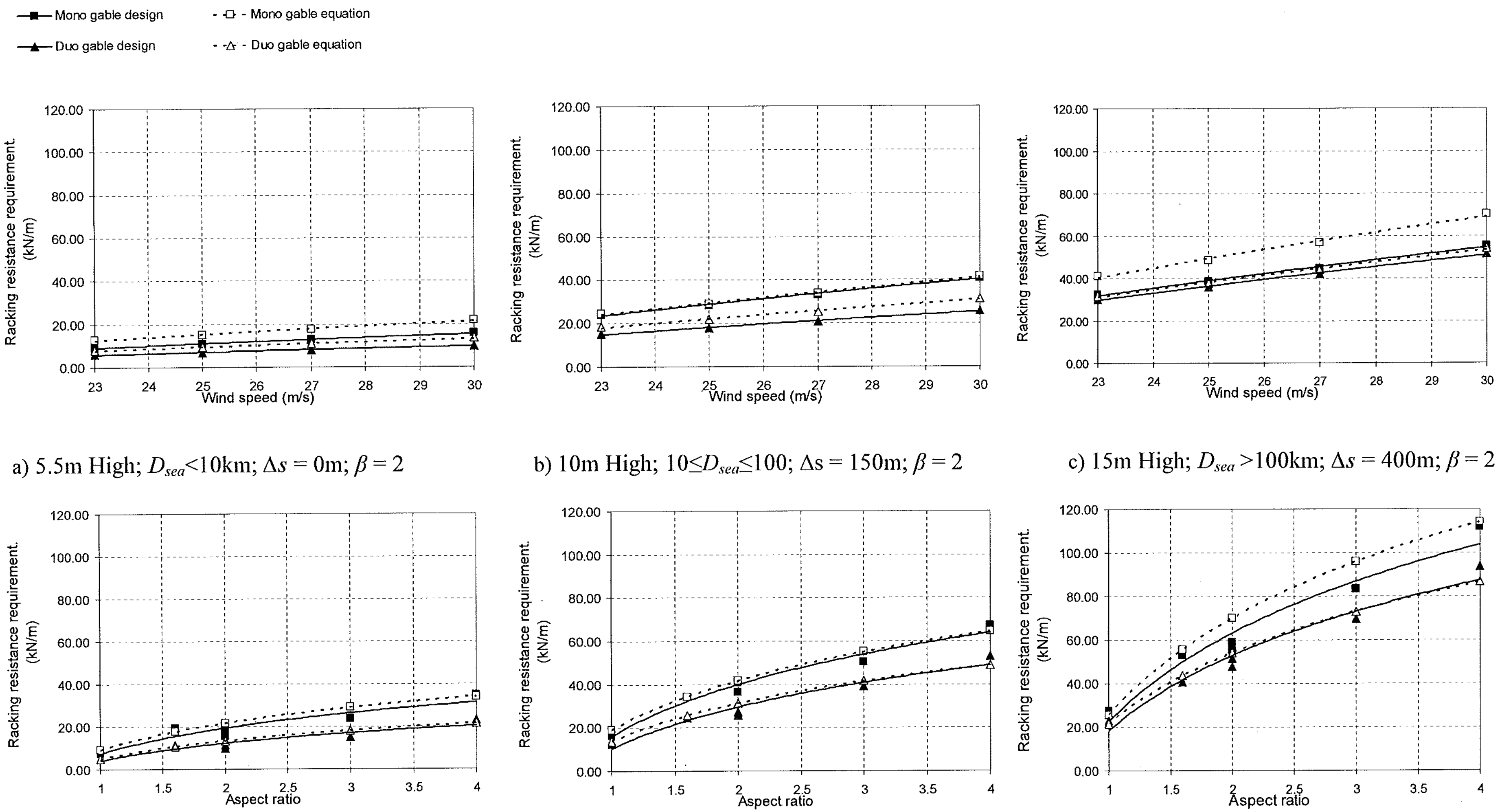
Figure 4.8 *P & Q* values relative to height to ridge for Duo & Mono pitch roofs

To verify that Equation 4.7 in combination with Equation 4.8 is working as intended the level of correlation between equation results and long hand design analysis results determined applying BS6399-3:1997 has been checked (Figure 4.9, Figure 4.10 and Figure 4.11). For the verification procedure gable panel racking results have been used as they consist of a larger range of values which provides a higher degree of scope for comparing the trends set and therefore identifying any cumulative errors.

From Figure 4.9 to Figure 4.11 the following conclusions are drawn:

- Good correlation of results is shown.
- The derived model provides a relatively accurate and tentative method of determining the required racking resistance of walls within the parameters set.
- The developed model would be suitable for initial design to provide a conservative estimate of racking requirements. However, for more accurate design, full design calculations maybe required.
- The developed model can be use to demonstrate what effect changing the variables as given in Table 4.1 has on the required racking resistance per metre run of the external walls. The calculated applied wind on the system was assumed to be distributed evenly between the

external walls of the system parallel to the action of the wind (internal walls, at this stage, were not considered). As a result of even distribution between the external walls torsion of this system is negligible and therefore not considered in the analysis.



a) 5.5m High; D_{sea}<10km; Δs = 0m; β = 2

b) 10m High; 10≤D_{sea}≤100; Δs = 150m; β = 2

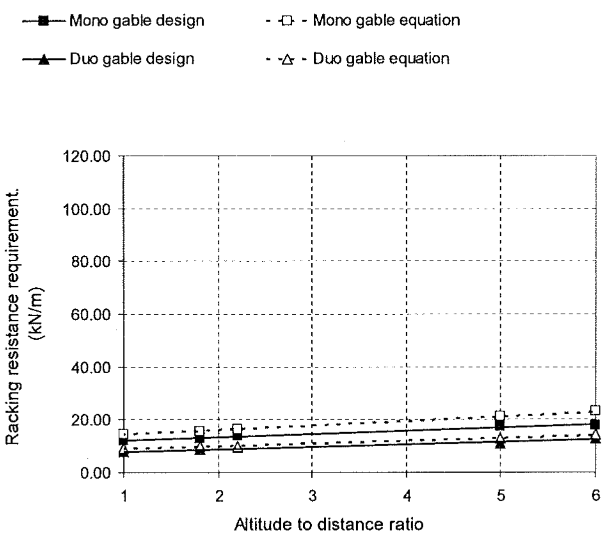
c) 15m High; D_{sea} >100km; Δs = 400m; β = 2

d) 5.5m High; D_{sea}<10km; Δs = 0m; V_b = 30m/s

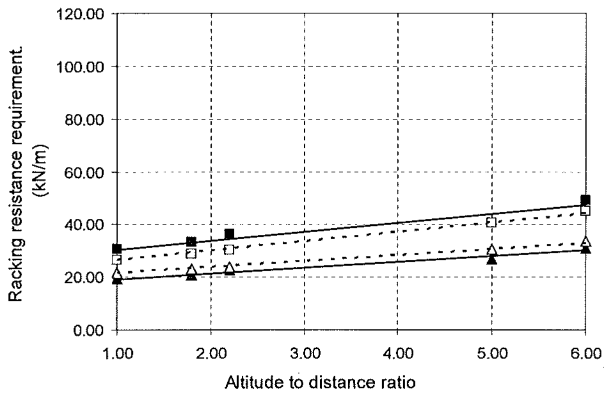
e) 10m High; 10≤D_{sea}≤100; Δs = 150m; V_b = 30m/s

f) 15m High; D_{sea} >100km; Δs = 400m; V_b = 30m/s

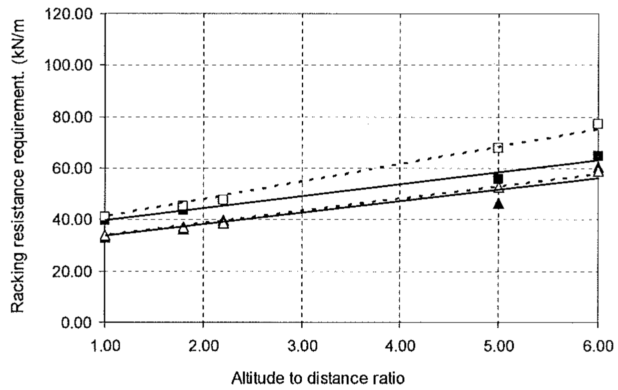
Figure 4.9 Level of correlation for varying wind speed, V_b & Aspect ratio, β.



a) 5.5m High; $\beta = 2$; $V_b = 30\text{m/s}$

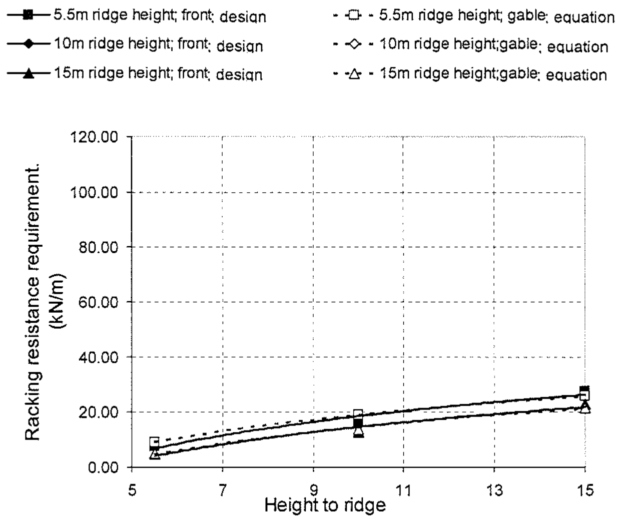


b) 10m High; $\beta = 2$; $V_b = 30\text{m/s}$

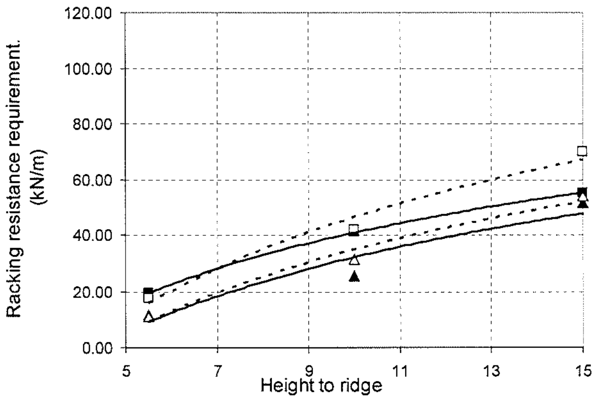


c) 15m High; $\beta = 2$; $V_b = 30\text{m/s}$

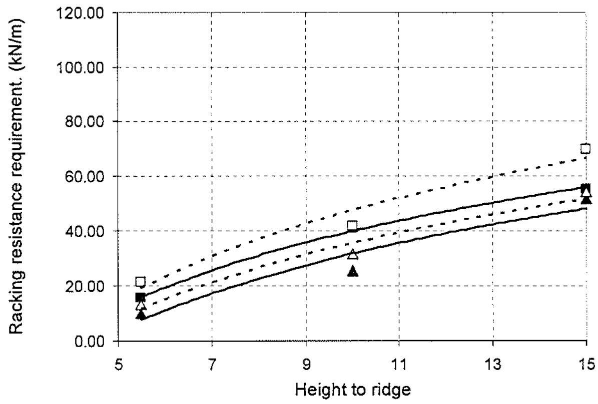
Figure 4.10 Level of correlation for varying altitude to distance ratio, α.



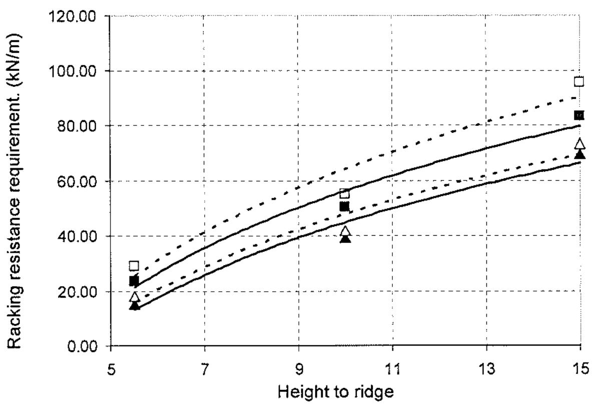
a) D_{sea}<10km; Δs = 0m; β = 1; V_b = 30m/s



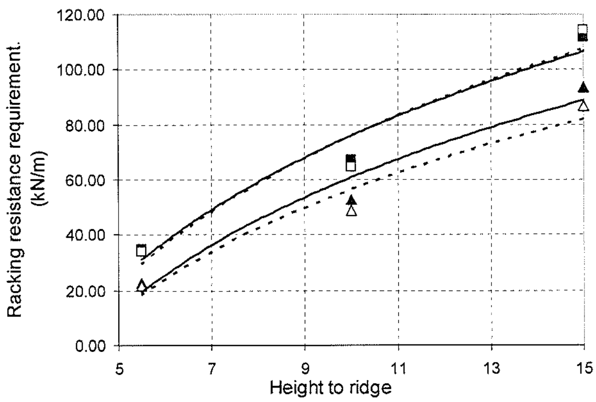
b) D_{sea}<10km; Δs = 0m; β = 1.6; V_b = 30m/s



c) D_{sea}<10km; Δs = 0m; β = 2; V_b = 30m/s



d) D_{sea}<10km; Δs = 0m; β = 3; V_b = 30m/s



e) D_{sea}<10km; Δs = 0m; β = 4; V_b = 30m/s

Figure 4.11 Level of correlation for varying height to ridge, *H*.

4.3 Derivation of Racking Wall Resistance Model

A semi-empirical model has been derived which, with a good degree of accuracy, provides a relatively conservative estimation of the required racking resistance of timber platform frame buildings covering the majority of design circumstances in the UK. In this section a further semi-empirical model is derived to determine the allowable level of racking resistance a timber frame wall can provide depending on the level of percentage opening it is to contain and the wall make-up and type. The use of BS 6399-2:1997 to derive the required racking resistance model dictates that the racking wall resistance model is derived applying the design rules of BS 5268-2: part 6.1: 1996 Structural use of timber - Part 6: Code of practice for timber frame walls - Section 6.1 Dwellings not exceeding four storeys.

BS 5268-2: part 6.1: 1996 was first published in 1988 and was regarded as innovative in its approach to design and testing for racking resistance. The design method contained in the code is restricted to timber frame walls in service class 1 & 2 conditions (the average equilibrium moisture content in the timber elements will not exceed 20% according to BS 5268-2:2002) not exceeding 2.7m high with studs spaced at a maximum 610mm centre to centre which have one or both faces partly or wholly connected to sheathing, lining, gusset plates or other forms of bracing. The methodology of the code known as the "*assessment method*" is one where the basic racking resistance of a range of materials and combinations of materials (BS 5268-2: part 6.1: 1996 Table 2, as shown in Appendix C) are modified by application of material modification factors (K_{101} fixing diameter, K_{102} nail spacing and K_{103} board thickness) and wall modification factors (K_{104} wall height, K_{105} wall length, K_{106} window, door and other fully framed openings in the wall and K_{107} variation in vertical load on the timber frame wall).

It is known that the factors used in BS 5268-2: part 6.1: 1996 were based on the findings of an extensive laboratory study by Robertson and Griffiths (1981) and from this study several important points are made:

- Within the normal range of design loadings the racking resistance of a panel increases as the vertical load increases.
- A holding down force on the leading stud (i.e. windward) acts as a stabilising force against overturning and improves racking performance.
- For panels up to 5m long the racking resistance increases as the length increases.
- The racking resistance of a panel is not directly proportional to the nailing centres and the relationship is different for different sheathing materials.
- The affect of reduced stud spacing below 600mm on racking resistance is negligible.

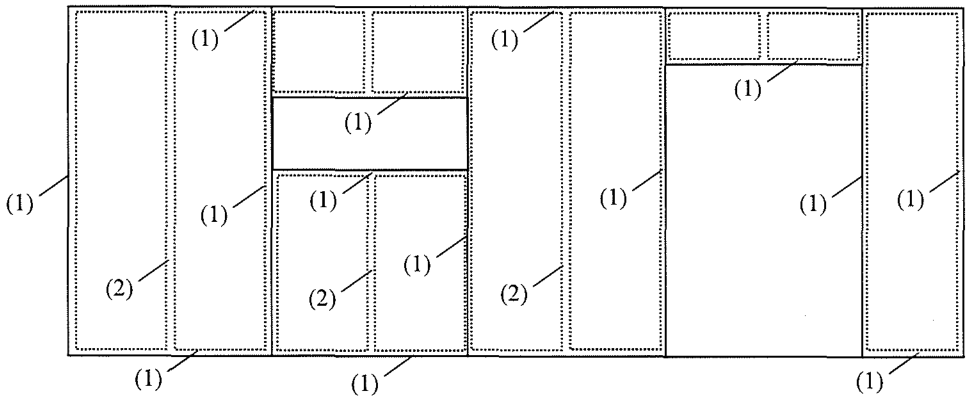
- Stud sizing does not appear to have a significant effect on racking resistance.
- Within certain limits masonry walls will reduce the wind load onto the timber frame of buildings.

The basic racking resistance values given in Table 2 of BS 5268-2: part 6.1: 1996 and modified as appropriate, by the derived modification factors K_{101} to K_{107}, according to the code give reasonably true assessments of the racking resistance of plain walls when subjected to test racking loads. When walls form part of completed dwellings the method of assessment according to the code underestimates the permissible racking resistance, since it does not take into account factors such as the stiffening effect of corners and the interaction of walls and floors through multiple fixings. Therefore, a further wall modification factor K_{108} provides a 10% increase in racking performance to account for a degree of system interaction. Further to this Robertson and Griffiths (1981) report that the 'Whole house' effect provides a significant stiffening contribution. A truss roof system can result in an increase in system stiffness of 24% when added and the addition of lining, cladding and internal partitions will serve to increase the stiffness of the system yet further.

For the derivation of the model a standard 2.4m high by 4.8m long wall panel was considered. The minimum length of wall is 3m and maximum is 12m according to Table 4.1, therefore 4.8m is representative and correspondent with standard 600mm dimensioning. The longest available timber section, unless finger jointing is used, is also 4.8m.

The structural components of the wall were based on industry standard materials and fixings. The timber framing was taken as minimum 38 × 89mm Grade C16 dimensional lumber (increased stud sizes would not influence the design racking resistance) sheathed externally, and when required internally, with 9mm OSB/3 which is a Category 1 sheathing material in accordance with BS 5268: part 6.1: 1996 Table 2 (see Appendix C for further information). A minimum layer of 12.5mm plasterboard is fixed internally as standard due to building performance requirements. The studs are at 600mm centres and are fixed to the top and bottom runners (header and footer). The sheathing material is fixed using 3.0×50mm galvanised wire nails for OSB (internal fixing centres are taken as twice the perimeter centres, Figure 4.12) and 3.9×55mm screws for plasterboard at 150mm centres.

Clause 4.9.5 of BS 5268: part 6.1: 1996 limits the maximum uniformly distributed load along the top of the wall to 10.5kN/m run and this maximum was taken for model derivation. The reason for taking the maximum allowance was to improve the overall balance when considering other aspects that would increase the racking resistance which have not been accounted for i.e. additional holding down resistance of the ends studs through interaction with perpendicular panels and holding down straps.



Note:
(1) Perimeter nailing
(2) Intermediate nailing (spacing for intermediate nailing should be a maximum of twice that of the perimeter nailing).

Figure 4.12 Wall diaphragm nailing

The range of panel make-ups which were considered are as defined in Table 4.11. The majority of timber frame houses in the UK are masonry clad and as a result masonry cladding has been assumed. Masonry cladding provides a degree of wind shielding depending on the configuration of the masonry and the number of storeys required to be shielded (wind shielding is limited to a maximum of 4 storeys) according to BS 5268-2:Part 6.1:1996. Shown in Table 4.12 are the three types of masonry cladding arrangements which were considered, Wall Type 1, 2 and 3 with each wall configuration proving a high, medium and low degree of shelter from the wind respectively.

Table 4.11 Wall diaphragm details

Wall Options	Wall configuration with perimeter nailing details
1	Double sheathed with 50mm nail centres
2	Double sheathed with 100mm nail centres
3	Double sheathed with 150mm nail centres
4	Double sheathed with 200mm nail centres
5	Single sheathed with 50mm nail centres
6	Single sheathed with 100mm nail centres
7	Single sheathed with 150mm nail centres
8	Single sheathed with 200mm nail centres

Note 1: Sheathing to be a minimum of 9mm OSB 3 or 9.5mm plywood.
Note 2: Where sheathing or linings are nailed to studs, the nails should be positioned so that the distance between the nail and the edge of the board or the face of the stud is not less than 7mm.
Note 3: The internal face of the wall panels are assumed to be lined with 12.5mm plasterboard which is connected to the wall panels with 2.65 mm diameter plasterboard nails at least 40mm long, maximum spacing150 mm.

Table 4.12 Masonry cladding arrangement type

Type 1	Type 2	Type 3
For masonry walls with buttresses or returns not less than 550mm length and not greater than 9m centre to centre.	For masonry walls with buttresses or returns at one end of wall not less than 550mm length with the other end without buttresses or returns less than 550mm length and wall length not greater than 4.5m	For masonry walls without buttresses or returns or with buttresses or returns of less than 550mm length.
		

In the first instance the variation in racking resistance of wall diaphragms relative to allowable percentage openings for given sheathing arrangements and nail spacings is analysed (Figure 4.13). The trend lines which correspond to each set of data in Figure 4.13 are of a polynomial type:

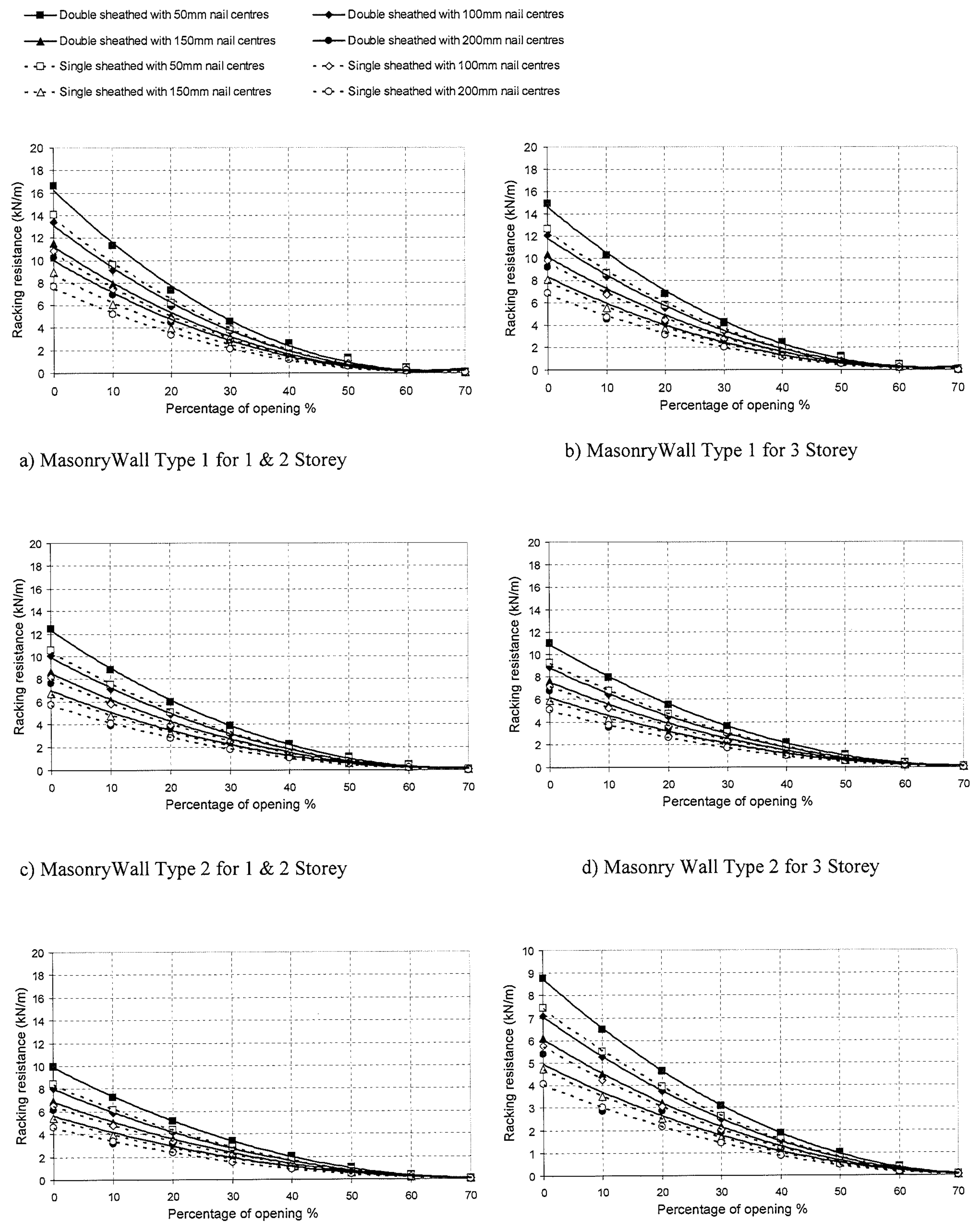
$$\textit{Racking Panel Resistance} = x \cdot Op^2 + y \cdot Op + z \qquad \text{Equation 4.9}$$

Where:

Op is the percentage of opening in the wall.

x, y & z are as defined in Table 4.13 for each of the given trend lines presented.

Racking Panel Resistance in kN/m run.



a) MasonryWall Type 1 for 1 & 2 Storey

b) MasonryWall Type 1 for 3 Storey

c) MasonryWall Type 2 for 1 & 2 Storey

d) Masonry Wall Type 2 for 3 Storey

e) MasonryWall Type 3 for 1 & 2 Storey

f) Msonry Wall Type 3 for 3 Storey

Figure 4.13 Racking resistance against percentage of opening in the wall

Table 4.13 Values of constants corresponding to Equation 4.9 and Figure 4.13

Sheathing	Nail Spacing	Wall Type 1 for 1 & 2 Storey Heights			Wall Type 2 for 1 & 2 Storey Heights			Wall Type 3 for 1 & 2 Storey Heights		
	mm	*x*	*y*	*z*	*x*	*y*	*z*	*x*	*y*	*z*
Double	50	0.0040	-0.50	16.23	0.0027	-0.36	12.29	0.0019	-0.27	9.88
Double	100	0.0032	-0.41	13.11	0.0022	-0.29	9.93	0.0016	-0.22	7.98
Double	150	0.0028	-0.35	11.23	0.0018	-0.25	8.51	0.0013	-0.19	6.84
Double	200	0.0025	-0.31	9.99	0.0015	-0.21	6.98	0.0011	-0.16	5.60
Single	50	0.0034	-0.43	13.73	0.0023	-0.30	10.40	0.0016	-0.23	8.36
Single	100	0.0026	-0.33	10.61	0.0017	-0.23	8.04	0.0013	-0.18	6.46
Single	150	0.0021	-0.27	8.74	0.0014	-0.19	6.62	0.0010	-0.15	5.32
Single	200	0.0018	-0.23	7.49	0.0012	-0.17	5.67	0.0009	-0.13	4.56
Sheathing	Nail Spacing	Wall Type 1 for 3 Storey Heights			Wall Type 2 for 3 Storey Heights			Wall Type 3 for 3 Storey Heights		
	mm	*x*	*y*	*z*	*x*	*y*	*z*	*x*	*y*	*z*
Double	50	0.0035	-0.45	14.64	0.0022	-0.31	10.90	0.0016	-0.24	8.75
Double	100	0.0028	-0.36	11.83	0.0018	-0.25	8.80	0.0013	-0.19	7.07
Double	150	0.0024	-0.26	8.33	0.0015	-0.21	7.55	0.0011	-0.16	6.06
Double	200	0.0020	-0.26	8.33	0.0013	-0.18	6.18	0.0009	-0.13	4.95
Single	50	0.0029	-0.38	12.39	0.0019	-0.26	9.22	0.0014	-0.20	7.40
Single	100	0.0023	-0.29	9.57	0.0015	-0.20	7.13	0.0011	-0.15	5.72
Single	150	0.0019	-0.24	7.89	0.0012	-0.17	5.87	0.0009	-0.13	4.71
Single	200	0.0016	-0.21	6.76	0.0010	-0.14	5.03	0.0007	-0.11	4.04

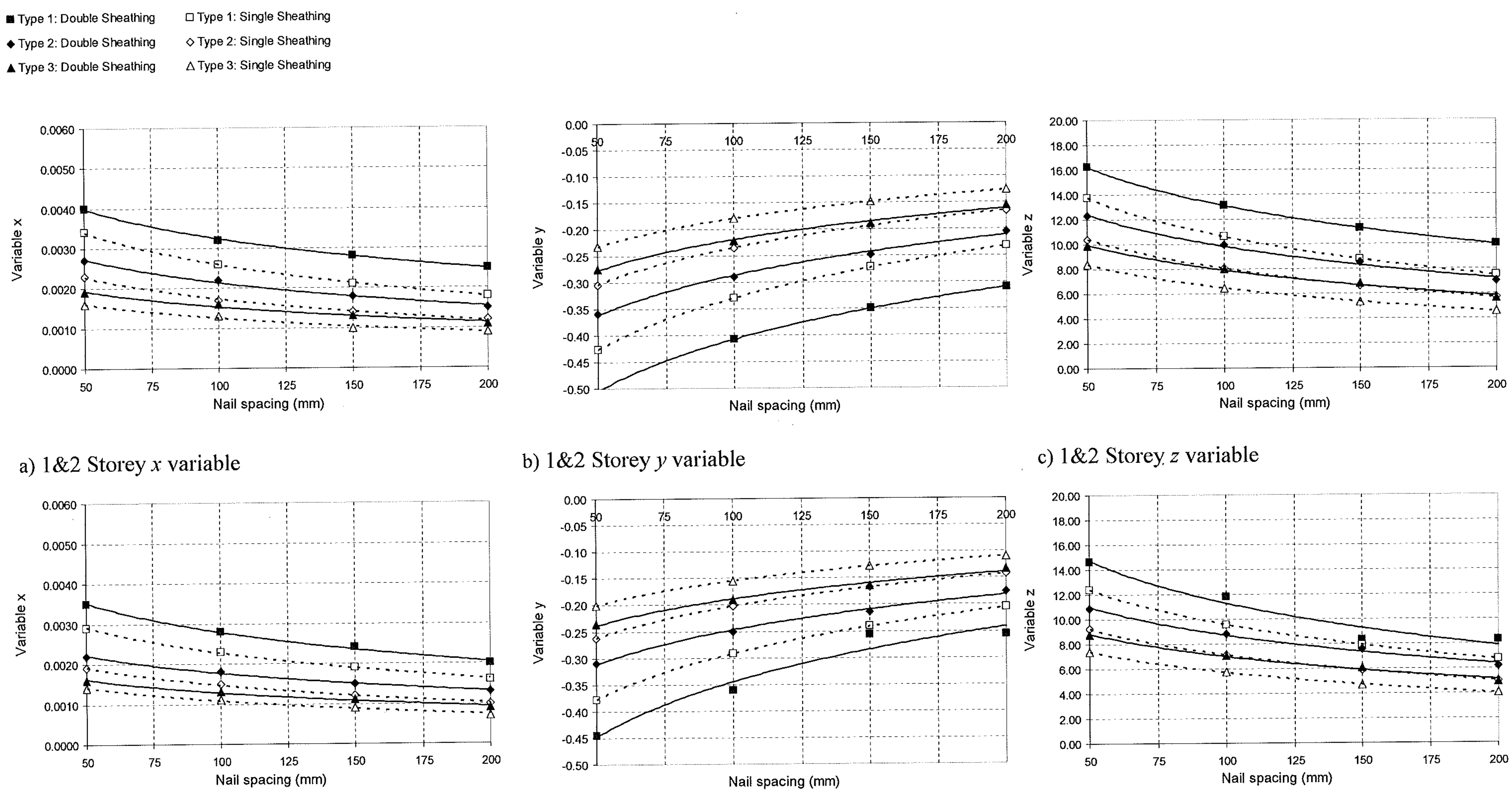
A further iterative step is required to define constants *x, y & z* depending upon the nail spacing. Therefore, the variables *x, y & z* are plotted against the nail spacing for each type of masonry wall arrangement as shown in Table 4.12 for 1, 2 & 3 storey heights, Figure 4.14. The trend lines which correspond to each set of data in Figure 4.14 are of a logarithmic type:

$$\text{x, y \& z} = A_n \cdot \ln(s) + B_n \qquad \text{Equation 4.10}$$

Where:

s is the perimeter nail spacing in millimetres.

A & B are as defined in Table 4.14 for each of the given trend lines presented.



a) 1&2 Storey x variable

b) 1&2 Storey y variable

c) 1&2 Storey z variable

d) 3 Storey x variable

e) 3 Storey y variable

f) 3 Storey z variable

Figure 4.14 x, y & z variables against nail spacing for 1, 2 & 3 storey heights

Table 4.14 Values of constants corresponding to Equation 4.10 and Figure 4.14

Type	Sheathing Arrangement	*x*		*y*		*z*	
		A_1	B_1	A_2	B_2	A_3	B_3
1 & 2 Storey							
1	Double	-0.0011	0.0082	0.14	-1.05	-4.51	33.89
	Single	-0.0012	0.0079	0.14	-0.98	-4.52	31.40
2	Double	-0.0009	0.0061	0.11	-0.79	-3.74	27.02
	Single	-0.0008	0.0054	0.10	-0.69	-3.42	23.78
3	Double	-0.0006	0.0042	0.08	-0.60	-3.01	21.75
	Single	-0.0005	0.0037	0.08	-0.53	-2.75	19.12
3 Storey							
1	Double	-0.0011	0.0077	0.15	-1.03	-4.95	34.10
	Single	-0.0009	0.0066	0.12	-0.86	-4.08	28.33
2	Double	-0:0007	0.0048	0.09	-0.68	-3.32	23.99
	Single	-0.0007	0.0045	0.09	-0.60	-3.03	21.09
3	Double	-0.0005	0.0036	0.07	-0.52	-2.67	19.27
	Single	-0.0005	0.0034	0.07	-0.46	-2.44	16.93

By combining Equations 4.9 and 4.10 the following equation is defined for determining the racking resistance of a wall for the given wall types and storey heights.

$$\textit{Racking Panel Resistance} = \left[A_1 \cdot \ln(s) + B_1\right] \cdot Op^2 + \left[A_2 \cdot \ln(s) + B_2\right] \cdot Op + A_3 \cdot \ln(s) + B_3$$

Equation 4.11

Where:

s is the perimeter nail spacing in millimetres.

A & B are as defined in Table 4.14.

Op is the percentage of opening in the wall.

Racking Panel Resistance in kN/m run.

To verify the derived model checks were carried out comparing Equation 4.11 with the long hand design analysis results determined applying BS 5268-2:Part 6.1:1996 as shown in Table 4.15. The average percentage error between design calculations and equation determined values was 5% which is considered to be relatively accurate.

Table 4.15 Percentage error of Equation 4.11 determined racking resistance requirements to long hand design calculation results

% Opening, Op	Nail spacing, *s*							
	50mm		100mm		150mm		200mm	
	Design	Equation	Design	Equation	Design	Equation	Design	Equation
	Double sheathing: Racking resistance (kN/m run)							
0	17	16	13	13	11	11	10	10
10	11	12	9	9	8	8	7	7
20	7	8	6	6	5	5	4	5
30	5	5	4	4	3	3	3	3
40	3	2	2	2	2	2	2	1
50	1	1	1	1	1	0	1	0
60	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0
Average % error		3		3		3		4
	Single sheathing: Racking resistance (kN/m run)							
0	14	14	11	11	9	9	8	7
10	10	10	7	8	6	6	5	5
20	6	6	5	5	4	4	3	3
30	4	4	3	3	2	2	2	2
40	2	2	2	1	1	1	1	1
50	1	0	1	0	1	0	1	0
60	0	0	0	0	0	0	0	0
70	0	0	0	0	0	0	0	0
Average % error		6		6		7		10
							All Inclusive Average Error	5

To further the verification process two typical design examples were carried out to BS 5268-6.1:1996 and checked against the derived model, these are detailed in Table 4.16. In these two cases a high degree of accuracy is exhibited although the results do appear to be marginally non-conservative. However, the added contribution of masonry cladding has not been taken into consideration in the BS 5268-6.1:1996 calculations and this would add an additional 0.4kN/m run resulting in the model solution being conservative.

Table 4.16 Types & corresponding racking (BS 5268-6.1 calculations & from derived model)

Wall panel type	External Sheathing			Internal Sheathing			Racking resistance (inclusive of masonry shielding factor K_{100})	
	Fixing type & arrangement			Fixing type & arrangement			BS 5268-6.1:1996 calculations	Derived model
	Type	Fixing type	External Centres	Type	Fixing Type	External Centres	kN/m	
100c/c Single	9mm OSB	3.0x50mm galvanised wire nails	100mm	Plaster-board	3.9×55mm Plasterboards screw	150mm	10.35	10.59
75c/c Double	9mm OSB	3.0x50mm galvanised wire nails	75mm	9mm OSB	3.0×50mm Galvanised wire nails	75mm	14.09	14.42

Note:

- Based on a 2.4×4.8m wall with no openings.
- Wall is masonry clad and corresponds to Wall Type 1 (BS 5268-6.1:1996, table 1) and the K_{100} factor has been used in calculation. To allow the K_{100} factor to be taken into consideration it has been employed inversely, namely increasing the racking resistance as opposed to reducing the applied wind action.
- Along the top rail a 10.5kN/m run uniformly distributed load (UDL) has been considered this enhances the racking resistance (BS 5268-6.1:1996, clause 4.9.5).
- Interaction of the system has been taken into account (BS 5268-6.1:1996, clause 4.9.6).
- Contribution of the masonry veneer (BS 5268-6.1:1996, clause 4.10) to racking resistance, which depending on the tie density, can contribute a minimum of 0.4kN/m run, has conservatively not been taken into account.

4.4 Optimising the Level of Opening

Two models have been derived, one which estimates the required racking resistance of a domestic dwelling and one which estimates the racking resistance of an external shear wall. By combining these models the optimum allowable level of opening can be determined depending on the building parameters (within the boundaries set in Table 4.1). Where the optimum is defined as when a balance is struck between the applied racking force and available racking resistance.

Required Racking Resistance (not considering the additional resistance of internal walls)

$$\textit{Racking Resistance Requirement} = 0.0011V_b^{\,2} \cdot (P_A \cdot e^{Q_A \cdot \alpha} \cdot \ln(\beta) + P_B \cdot e^{Q_B \cdot \alpha}) \qquad \text{Equation 4.7}$$

Where:

β is the aspect ratio.

V_b is the basic wind speed in metres per second.

α is as defined in Table 4.3.

P_n & Q_n are as defined by Equation 4.8.

H is the height to the ridge in metres.

x, y & z are as defined in Table 4.10.

Racking resistance requirement in kN/m run

Equation 4.7 determines the required racking resistance of each of the external walls acting in the same orientation as the action of the wind. To make an allowance for internal racking walls a degree of interpolation is required as shown:

$$Racking\ Resitance\ Requirement = 0.0011V_b^{\,2} \cdot (P_A \cdot e^{Q_A \cdot \alpha} \cdot \ln(\beta) + P_B \cdot e^{Q_B \cdot \alpha}) - \frac{\Sigma L_i \cdot r_i}{2 \cdot L}$$

Equation 4.12

Where:

ΣL_i is the total length of the internal racking walls in metres.

L is the length of the external wall the internal wall is parallel to in metres.

r_i is the racking resistance of the internal racking walls in kilo Newtons per metre run.

Racking resistance requirement in kN/m run

Racking Panel Resistance:

$$Racking\ Panel\ Resistance = [A_1 \cdot \ln(s) + B_1] \cdot Op^2 + [A_2 \cdot \ln(s) + B_2] \cdot Op + A_3 \cdot \ln(s) + B_3$$

Equation 4.11

Where:

s is the perimeter nail spacing in millimetres.

A & B are as defined in Table 4.4.

Op is the percentage of opening in the wall.

The maximum allowable level of opening will therefore correspond to when both equations are balanced:

$$Racking\ Resitance\ Requirement = Racking\ Panel\ Resistance$$ Equation 4.13

Therefore:

$$[A_1 \cdot \ln(s) + B_1] \cdot Op^2 + [A_2 \cdot \ln(s) + B_2] \cdot Op + A_3 \cdot \ln(s) + B_3 - y(\beta) = 0$$

This polynomial expression can be solved for *Op* as follows:

$$Op = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$ Equation 4.14

Where

$$a = A_1 \cdot \ln(s) + B_1$$ Equation 4.15

$$b = A_2 \cdot \ln(s) + B_2$$ Equation 4.16

$$c = \ln(s) + B_3 - y(\beta)$$ Equation 4.17

Solving the polynomial provides two out-puts as a result of the ± expression. However, the result of the polynomial when "+" is taken is greater than 100% which of course is unfeasible in terms of allowable percentage opening. Therefore, the allowable percentage of opening is that when "–" is taken.

4.5 Applying the Model

Using the model the influence of building parameters and racking panel sheathing arrangement and level of fixity were analysed. To limit the analysis work to areas which were regarded as more critical the following points were taken into consideration:

- It has been demonstrated that the effects of wind speed and altitude to distance ratio on required racking resistance are, relative to other factors such as building height and aspect ratio, non-critical overall.
- The majority of design cases in UK are situated such that the design wind speed is equal to or less than 25m/s, therefore 25m/s has been used for analysis.
- In terms of altitude to distance ratios extreme cases do not normally occur, for example it is highly unlikely that a design case will have a distance from the sea of zero corresponding to an altitude greater that 400m. Therefore, a medium case is set which corresponds to an alpha value of 2 according to Table 4.3 this will approximately cover altitudes between 100 and 200m at any distance from the sea.
- Primarily the required racking resistance will be transferred by the external walls of the system to the substructure. It is beneficial for the external walls only to be required to resist

the applied wind action because in so doing the internal wall layout of the system has a higher degree of flexibility.

With due consideration of the above points the developed model has been applied to produce Figure 4.15 & Figure 4.16 which demonstrate the allowable level of percentage of opening relative to:

- Aspect ratio 1 to 4 (for buildings up to 12m in length).
- Building heights up to 15m (5.5m considered as 1 storey; 10m as 2 storey and 15m as 3 storeys).
- Wall make-up:
    - C16 framing material with minimum 38×89mm studs at 600m centres.
    - Single or double Category 1 sheathing material.
    - Perimeter nail spacing of Sheathing (50, 100, 150 & 200mm).
    - Minimum 12.5mm internal plasterboard lining.
- Masonry cladding arrangement: Wall Type 1, 2 or 3 (Table 4.12).
- The additional racking resistance provided by internal racking walls has not been considered.

It is to be noted that where the plots return to zero and form a straight line failure of the system has occurred as zero percentage of opening can be incorporated in the wall.

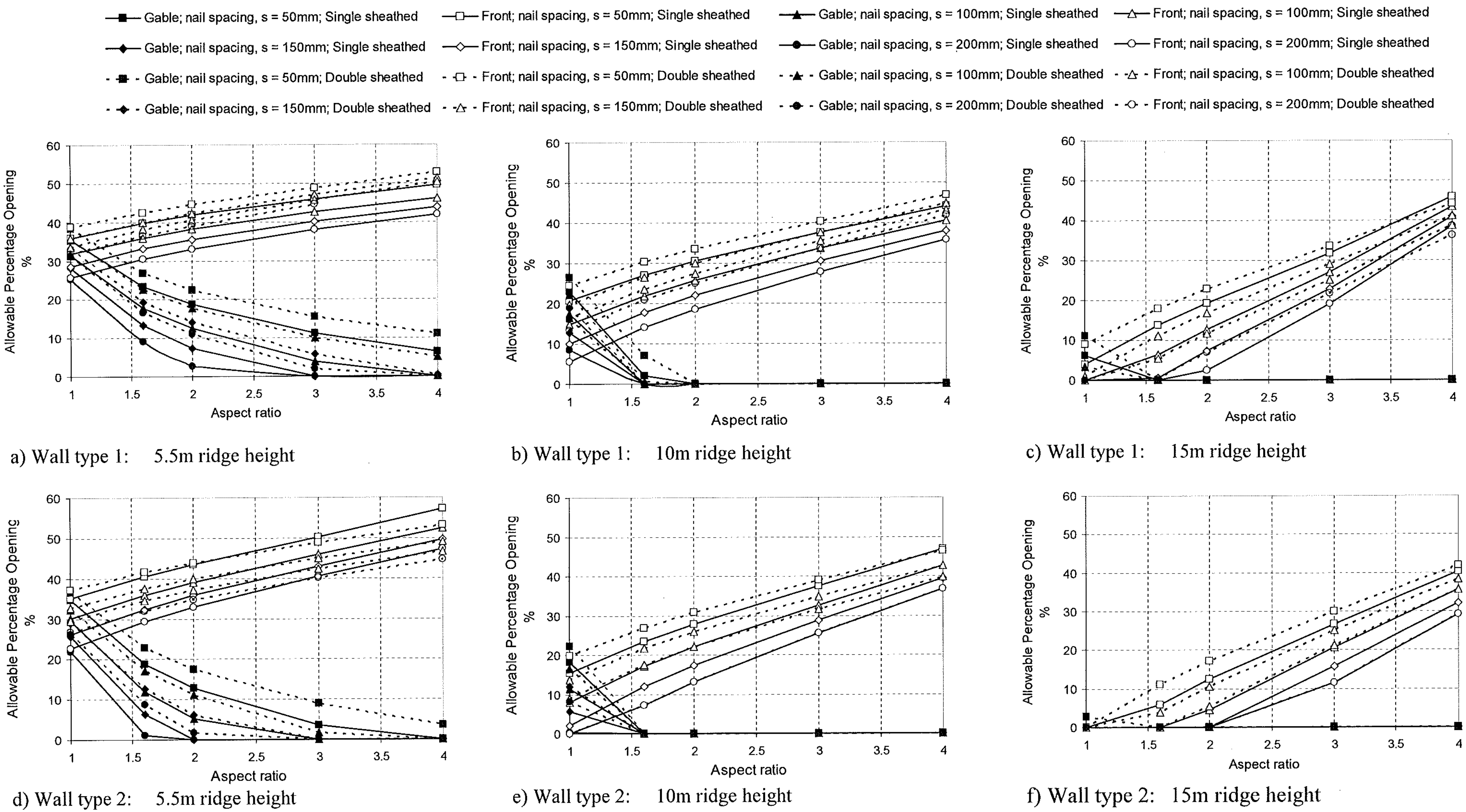
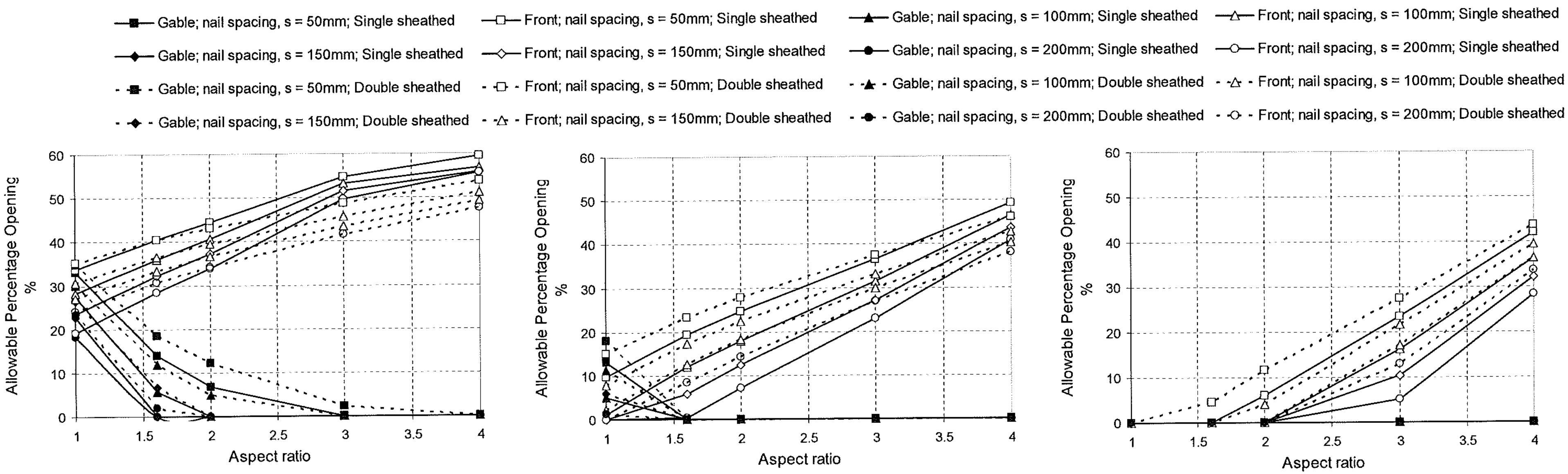


Figure 4.15a Duo pitch roof: Allowable percentage opening relative to aspect ratio, building height, wall make-up and type.

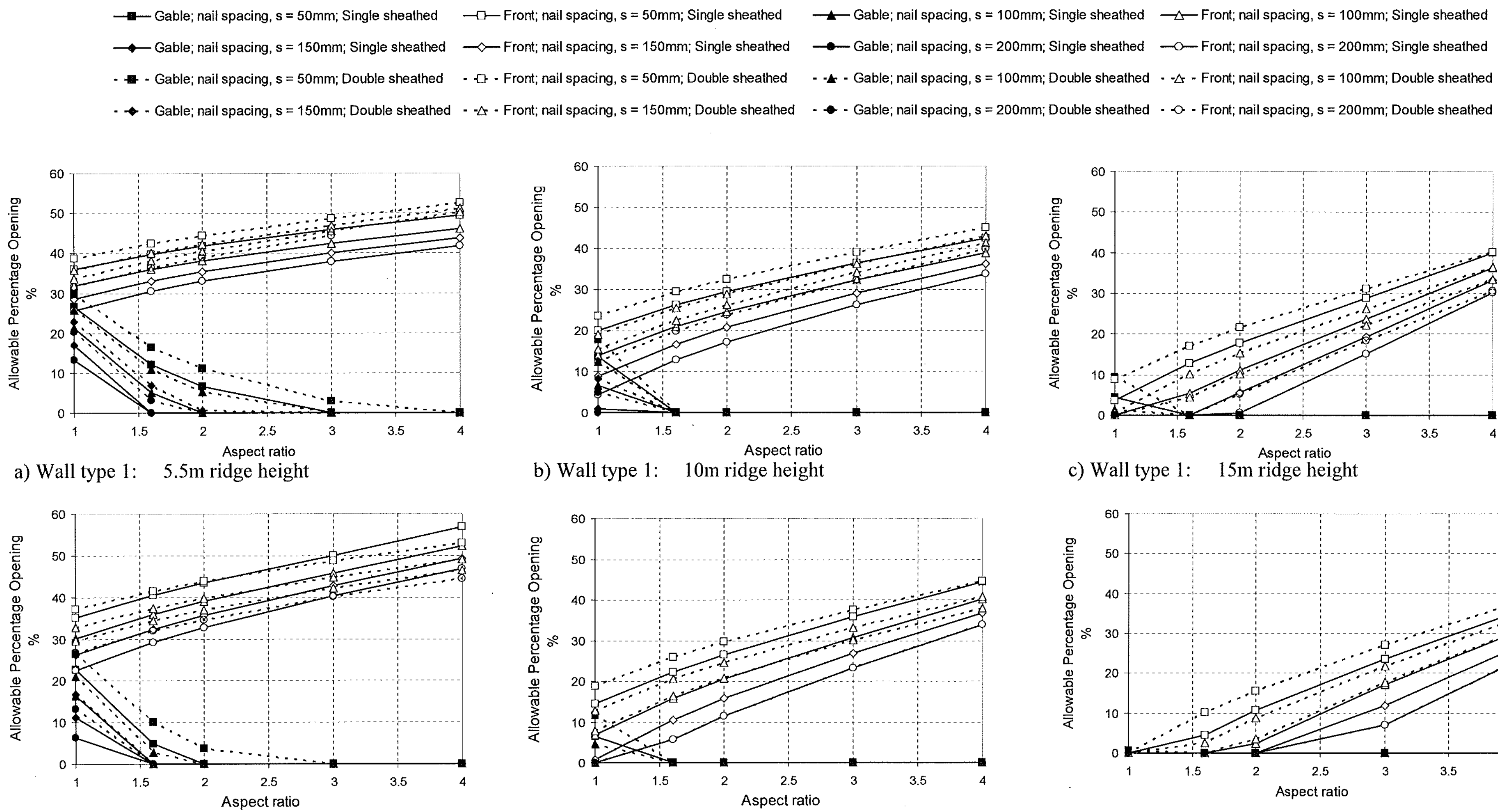


g) Wall type 3: 5.5m ridge height

h) Wall type 3: 10m ridge height

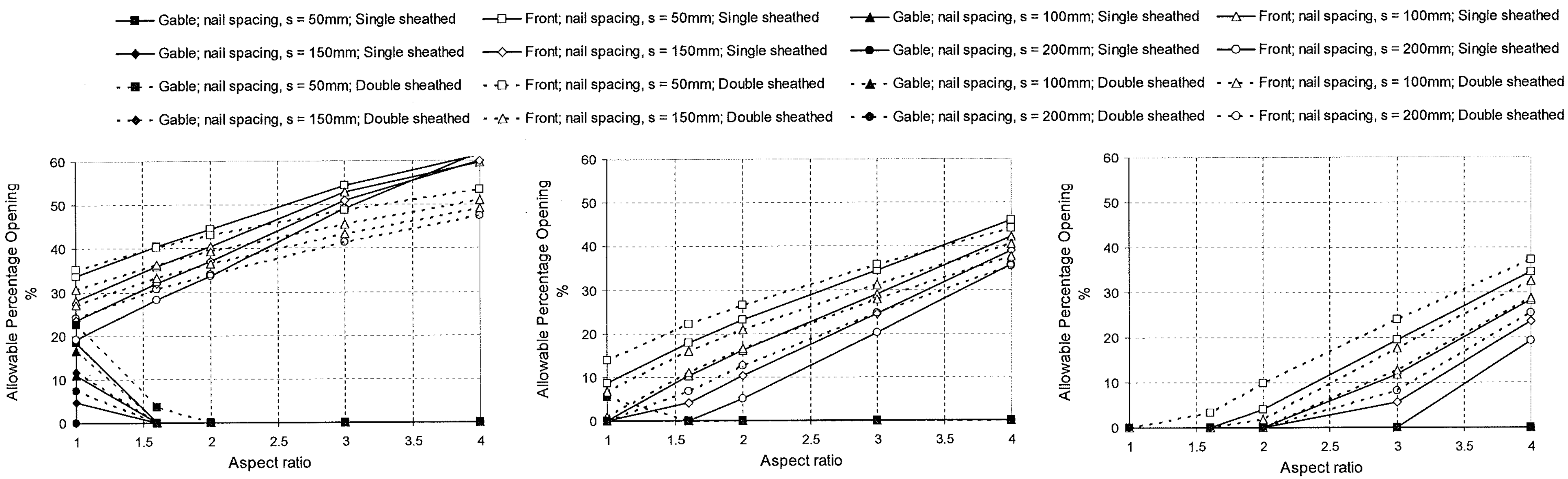
i) Wall type 3: 15m ridge height

Figure 4.15b Duo pitch roof: Allowable percentage opening relative to aspect ratio, building height, wall make-up and type.



a) Wall type 1: 5.5m ridge height
b) Wall type 1: 10m ridge height
c) Wall type 1: 15m ridge height
d) Wall type 2: 5.5m ridge height
e) Wall type 2: 10m ridge height
f) Wall type 2: 15m ridge height

Figure 4.16a Mono pitch roof: Allowable percentage opening relative to aspect ratio, building height, wall make-up and type.



g) Wall type 3: 5.5m ridge height

h) Wall type 3: 10m ridge height

i) Wall type 3: 15m ridge height

Figure 4.16b Mono pitch roof: Allowable percentage opening relative to aspect ratio, building height, wall make-up and type.

From Figure 4.15 to Figure 4.16 the following conclusions are drawn:

- If the level of allowable opening in either of the walls goes to zero the whole system has failed. Consider case b in Figure 4.15 when the aspect ratio, β, is equal to 2 all front walls, regardless of sheathing arrangement and nail spacing, are allowed a percentage of opening. However, all the gable wall make-ups, regardless of the type, have zero allowable percentage of opening therefore the system as a whole would not function as the gable walls would fail.
- The following relationships are interpolated from Figure 4.15 to Figure 4.16:

$$Op_f \propto \beta; \quad Op_f \propto \frac{1}{H} \quad \& \quad Op_g \propto \frac{1}{\beta} \;; \; Op_g \propto \frac{1}{H}$$

  Where:
  Op_f is the level of allowable percentage openings in the front.
  Op_g is the level of allowable percentage openings in the front.
  β is the aspect ratio.
- An aspect ratio, $\beta = 1$, would allow approximately an even level of openings in all sides of the building if wall type, sheathing arrangement and nail spacing are consistent.
- It is normally the request of the house builder to have a high degree of opening in the front (or back) of the house, with minimum openings in the gables. If this is the case it is advantageous to have a higher aspect ratio.
- The effect of wind shelter from the masonry wall is beneficial, reducing the racking requirement and therefore allowing a higher level of percentage opening. As a result Wall Type 1, because it provides a higher degree of shelter, is more advantageous.
- It is demonstrated that the percentage improvement due to increased sheathing level is not as effective as increasing nail spacing. An extra layer of Category 1 sheathing material on the internal face provides an extra 0.84kN/m run. However, plasterboard, a Category 2 material, which is required to be fitted on the internal face to satisfy building performance criteria, provides an additional 0.28kN/m run. According to Note 6 of BS5268-2:1996 Table 2, the additional contribution from a secondary layer of Category 1, 2 & 3 materials should only be included once in the determination of basic racking resistance, no matter how many additional layers may be attached to the wall panel. As a result the actual increase in racking resistance from fitting an additional layer of Category 1 material is only 0.56kN/m run if the minimum level of fixity is specified.

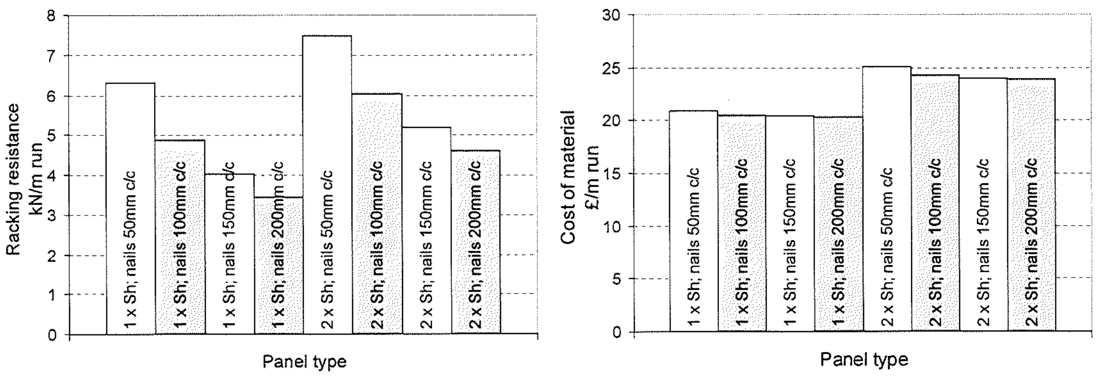
4.6 Cost Benefit Analysis

Shown in Figure 4.17 is the racking resistance and associated cost (based on 2006 figures) of a range of commonly specified racking panels. It is noted from Table 4.17 that if a ‘standard racking panel’ (single sheathed externally with Category 1 material fixed with nails at 100mm centres and internally faced with plasterboard), is considered, a 22.6% increase in racking performance can be achieved by reducing the nail spacing to 50mm. By comparison an 18.8% increase in racking performance is achieved if the panel is double sheathed with Category 1 material. There is therefore an imbalance with a 2% increase in cost achieving a 22.6% increased design racking performance compared to a 15% increase in cost achieving a 18.8% increased design racking performance. As a result of this “Cost per percentage opening” is considered with respect to panel make-up (sheathing arrangement and nail spacing).

Figure 4.15 to Figure 4.16 show the level of allowable percentage opening relative to both the aspect ratio, β, value and the wall make-up and type, using this information and that of Table 4.17, the “Cost per percentage of opening” is determined:

Cost per % of opening = Cost per metre run of wall / total allowable % of opening

The “Cost per percentage of opening” is based on the ‘optimum’ area of opening for the given building parameters and wall panel type. Whereby, the optimisation is specified as achieving the most value from the panel make-up in terms of allowable level of opening for the cost incurred. Figure 4.18 to Figure 4.21 demonstrate the effect aspect ratio, β, wall type and panel make-up have on the “Cost per allowable percentage of opening”.



a) Racking panel resistances

b) Racking panel cost

Figure 4.17 Resistance and cost for given make-ups (see Table 4.17 for details)

Table 4.17 Wall panel type, cost and racking resistance

Panel type	Abbreviation	Cost £/m run	Racking Resistance kN/m
Single sheathed with external nails at 50mm centres	1 x Sh; nails 50mm c/c	£20.91	6.32
Single sheathed with external nails at 100mm centres	1 x Sh; nails 100mm c/c	£20.53	4.89
Single sheathed with external nails at 150mm centres	1 x Sh; nails 150mm c/c	£20.40	4.02
Single sheathed with external nails at 200mm centres	1 x Sh; nails 200mm c/c	£20.34	3.45
Double sheathed with external nails at 50mm centres	2 x Sh; nails 50mm c/c	£25.03	7.47
Double sheathed with external nails at 100mm centres	2 x Sh; nails 100mm c/c	£24.27	6.03
Double sheathed with external nails at 150mm centres	2 x Sh; nails 150mm c/c	£24.02	5.17
Double sheathed with external nails at 200mm centres	2 x Sh; nails 200mm c/c	£23.89	4.60

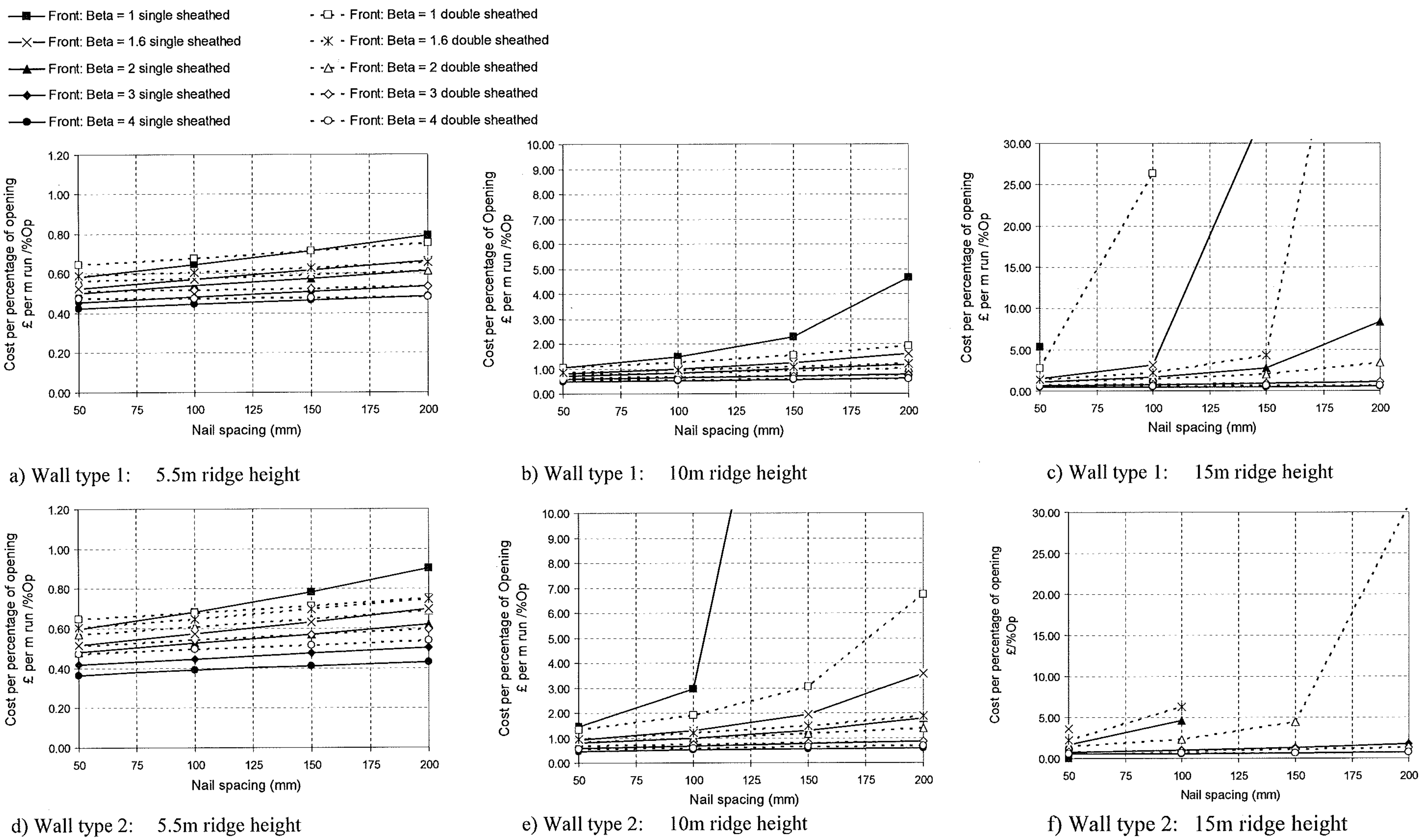
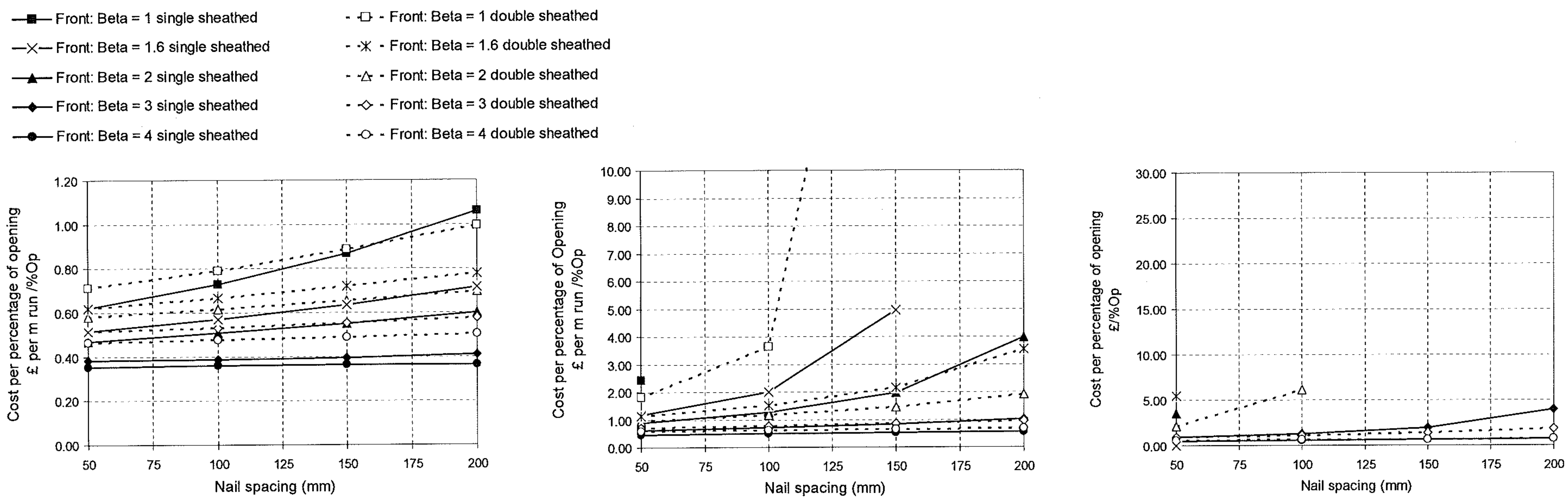


Figure 4.18 Duo pitch roof: Allowable percentage opening cost relative to aspect ratio, building height, wall make-up for wall type 1 & 2



g) Wall type 3: 5.5m ridge height

h) Wall type 3: 10m ridge height

i) Wall type 3: 15m ridge height

Figure 4.19 Duo pitch roof: Allowable percentage opening cost relative to aspect ratio, building height, wall make-up for wall type 3

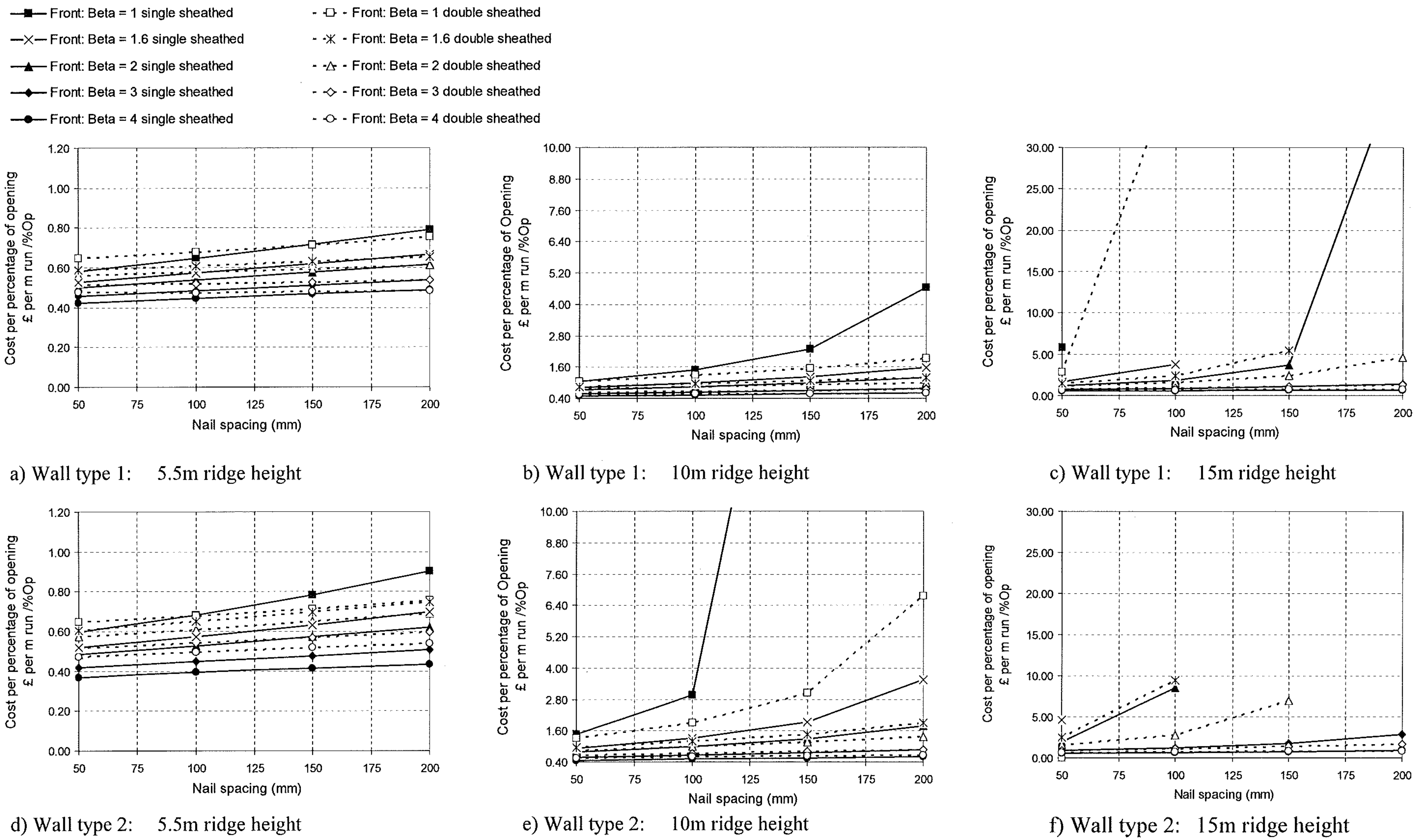
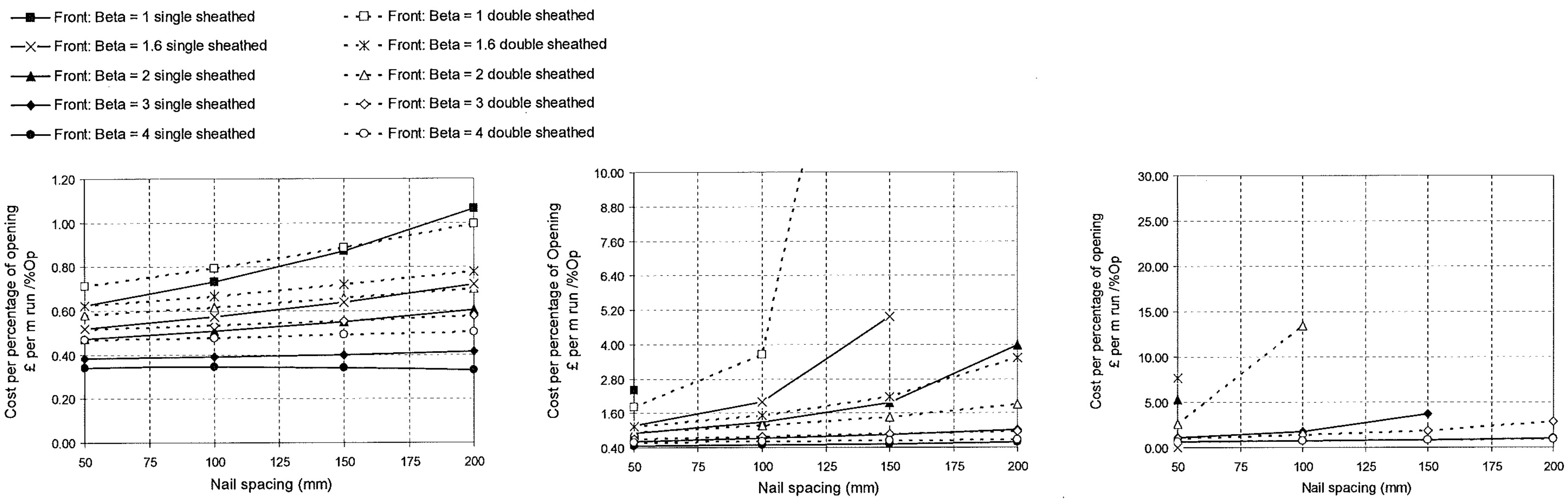


Figure 4.20 Mono pitch roof: Allowable percentage opening cost relative to aspect ratio, building height, wall make-up for wall type 1 & 2



g) Wall type 3: 5.5m ridge height

h) Wall type 3: 10m ridge height

i) Wall type 3: 15m ridge height

Figure 4.21 Mono pitch roof: Allowable percentage opening cost relative to aspect ratio, building height, wall make-up for wall type 3.

From Figure 4.18 to Figure 4.21 the following conclusions are drawn:

- It is shown that because Wall Type 1 (Table 4.12) provides the most added shelter more economical design is achieved.
- It is more cost effective to reduce the spacing between nails than add a secondary layer of sheathing and it is shown that at reduced nail spacing single sheathed panels provide a higher degree of value in terms of level of opening. However, as nail spacing is increased there are occasions where a double sheathed panel is more cost effective than a comparable single sheathed panel.
- It is demonstrated that as aspect ratio, β, is increased the cost effectiveness of the openings is improved.
- Reducing the wind catchment area of the roof allows a higher level of percentage opening to be achieved. If the pitch angle is consistent it is beneficial to have a Duo Pitch rather than a Mono Pitch as the contact area in the worst case direction would be reduced.
- In the study conducted only one roof pitch angle has been looked at (35°), it can therefore be deduced that reducing the pitch of the roof, which would also reduce the wind contact area of the roof, would allow an increased percentage of opening at no further cost.

From the information contained in Figure 4.13, Table 4.18 has been produced. Table 4.18 contains the most economical wall options for the following criteria:

- Duo pitch roof at 35°.
- Wall type 1.
- Gable walls are considered to have zero required opening.
- No racking is provided by internal walls.

Table 4.18 Economical wall options

Height to ridge	Aspect ratio, β	Front					Gable		
		Sheathing	Nail spacing	Allowable % *Op*	Cost	Cost per % *Op*	Sheathing	Nail spacing	Cost
m			mm	%	£/ m run	£/ %*Op*		mm	£/ m run
5.5	1	Single	50	36	20.91	0.58	Single	200	20.34
5.5	1.6	Single	50	40	20.91	0.53	Single	200	20.34
5.5	2	Single	50	42	20.91	0.50	Single	200	20.34
5.5	3	Single	50	46	20.91	0.45	Single	200	20.34
5.5	4	Single	50	50	20.91	0.42	Single	200	20.34
5.5	1	Double	50	39	25.03	0.65	Single	200	20.34
5.5	1.6	Double	50	42	25.03	0.59	Single	200	20.34
5.5	2	Double	50	45	25.03	0.56	Single	200	20.34
5.5	3	Double	50	49	25.03	0.51	Single	100	20.53
5.5	4	Double	50	53	25.03	0.42	Single	50	20.91
10	1	Single	50	21	20.91	0.91	Single	200	20.34
10	1.6	Single	50	27	20.91	0.77	Single	200	20.34
10	1	Double	50	26	25.03	1.03	Single	200	20.34
10	1.6	Double	50	30	25.03	0.83	Single	200	20.34
15	1	Single	50	4	20.91	5.38	Single	50	20.91
15	1	Double	50	9	25.03	2.79	Single	50	20.91
15	1	Double	100	1	24.27	26.32	Single	50	20.91

It is shown in Table 4.18 that if there are no openings in the gables, which is normal for the majority of design cases, and the racking resistance required from the applied wind action can indeed be resisted by the external gable walls, single sheathed walls with 200mm spaced nails is normally sufficient and indeed an over specification. The cost of the gable walls, if the aspect ratio, β, value allows the system to work, can therefore be considered to be for most cases a constant of £20.34 per m run. If this is the case what dictates the overall cost of the system is the level of required percentage opening in the front.

Considering a 5.5m height to ridge system the most economical arrangement, in terms of achieving a high level of percentage opening in the front and back would be an aspect ratio, $\beta = 4$. If the walls are single sheathed with 50mm spaced nails and the aspect ratio, $\beta = 4$, a 50% opening can be achieved in the front and back, this is compared to an allowable level of opening of 36, 40, 42 and 46% if the aspect ratio, $\beta = 1$, 1.6, 2, and 3 respectively. Therefore, when the aspect ratio, $\beta = 4$, the material cost of the front and gable racking walls would be £20.91 & £20.34 per metre run per storey height respectively for these arrangements.

Again considering a 5.5m height to ridge system, if double sheathed racking walls with 50mm nail spacing are used in the front and back a 39% level of opening can be achieved for an aspect ratio, β = 1 compared to 36% level of opening in the walls when single sheathed. However, the cost of achieving this increased 3% in opening in the front and back is an additional £4.12 per metre length of wall or rather a 20% increase in material cost. The same level of opening could have been achieved in the front and back walls using single sheathed walls for approximately the same area of dwelling if the dimensions of the system were such that the aspect ratio, β = 1.6. Shown in Table 4.19 are examples of this.

Table 4.19 5.5m Ridge height options & wall costs

Wall Make-Up	Wall lengths		Total dwelling area	Aspect ratio, β	Front & Back		Cost[2]		
	Front & Back	Gables			% of Opening[1]	Actual opening area	Front & Back	Gables	Total
	m	m	m^2		%	m^2	£		
Nail spacing, s = 50mm; Double sheathed	9.5	9.5	90.25	1	39	9	476	386	862
Nail spacing, s = 50mm; Single sheathed	9.5	9.5	90.25	1	36	8	397	386	784
Nail spacing, s = 50mm; Single sheathed	12	7.5	90.00	1.6	40	11	502	305	807
Nail spacing, s = 50mm; Double sheathed	9	4.5	40.50	2	45	10	451	183	634
Nail spacing, s = 50mm; Single sheathed	9	4.5	40.50	2	42	9	376	183	559
Nail spacing, s = 50mm; Single sheathed	11.5	3.5	40.25	3	46	13	481	142	623

Note:
1. % Opening is the allowable level of opening in both the front and back per level.
2. Cost is considering one level which equals one racking wall per side i.e. 2 gable racking walls.

In Table 4.19 it is demonstrated that by increasing the opening requirement from 36 to 39% the financial cost is £78, if 500 houses were to be built then this would result in an additional material cost of £39,000.

Considering a 10m height to ridge system, only aspect ratios, β = 1 & 1.6, can be achieved without having to introduce internal racking walls at additional cost. Examples of design options are shown in Table 4.20.

Table 4.20 10m Ridge height options & wall costs

Wall Make-Up	Wall lengths		Total Area	Aspect ratio, β	Front & Back		Cost[2]		
	Front & Back	Gables			% of Opening[1]	Actual Area	Front & Back	Gables	Total
	m	m	m^2		%	m^2	£		
Nail spacing, s = 50mm; Double sheathed	9.5	9.5	90.25	1	26	6	951	795	1746
Nail spacing, s = 50mm; Single sheathed	9.5	9.5	90.25	1	21	5	795	773	1568
Nail spacing, s = 50mm; Double sheathed	12	7.5	90.00	1.6	30	9	1201	610	1812
Nail spacing, s = 50mm; Single sheathed	12	7.5	90.00	1.6	27	8	976	610	1587

Note:
1. % Opening is the allowable level of opening in both the front and back per level.
2. Cost is considering one level which equals one racking wall per side i.e. 2 gable racking walls.

It is demonstrated in Table 4.20 that for the case of β = 1 increasing the opening requirement from 21% to 26% results in a financial cost of £178. If 500 houses were to be built then this would result in an additional material cost of £89,000. However, the same level of percentage opening in the front and back could have been gained, with a negligible reduction in area (0.25m^2), by increasing the aspect ratio to 1.6. The material saving from altering the aspect ratio to 1.6 is £159, again considering 500 houses this is a material cost saving of £79,500.

Considering a 15m height to ridge system, only an aspect ratio of, β = 1, can be achieved without having to introduce internal racking panels. However, the level of permitted opening would not be sufficient and therefore added internal racking walls would have to be provided. Although this is the case examples of design options are shown in Table 4.21 which provides an indication of the cost variation between different systems.

Table 4.21 15m Ridge height options & wall costs

Wall Make-Up	Wall lengths		Total Area	Aspect ratio, β	Front & Back		Cost[2]		
	Front & Back	Gables			% of Opening[1]	Actual Area	Front & Back	Gables	Total
	m	m	m^2		%	m^2	£		
Nail spacing, s = 50mm; Double sheathed	9.5	9.5	90.25	1	9	2	1427	1192	2619
Nail spacing, s = 50mm; Single sheathed	9.5	9.5	90.25	1	4	1	1192	1192	2384
Nail spacing, s = 100mm; Double sheathed	9.5	9.5	90.25	1	1	0.2	1383	1192	2575

Note:
1. % Opening is the allowable level of opening in both the front and back per level.
2. Cost is considering one level which equals one racking wall per side i.e. 2 gable racking walls.

It is demonstrated from the study carried out that the most cost effective method of gaining added racking resistance, and as a result increasing the level of allowable opening, from a timber frame system, is to increase the aspect ratio. Increasing the aspect ratio need not reduce the internal area of the dwelling but is only an acceptable method if the external gable walls are capable of carrying the additional wind load acting on the front (or back) due to the increased catchment area. If the gable walls are not sufficient to resist the wind action internal racking walls can be introduced although this will tend to increase cost due to added material and foundation requirements.

Where the required area of opening in the front is marginally more than can be achieved from single sheathed walls with 50mm spacing (the lowest spacing which can be specified), added sheathing is required. The improvement in racking performance is disproportionate to the added cost and therefore proves to be uneconomical as the true value of the added material is not being gained. However, there are two options available which reduce cost:

1. The level of opening can be reduced to a level which is acceptable to negate the requirement for extra sheathing; this is the most cost effecitve method.
2. The aspect ratio of the system can be increased without reducing the internal area, as long as the gables are capable of carrying the increased load, and this can as shown allow additional opening to be achieved with a reduced level of material cost.

4.7 Applying the model to actual design cases

The derived model for determining the percentage of allowable opening is applied to a range of industry standard house types the details of which are given in Table 4.22 and shown in Figure 4.22 are the front and gable elevations. These particular house types have been chosen due to the required level of opening in the front and rear of the buildings, which were specified for architectural purposes, being close to or on the allowable limit.

Table 4.22 Building information

Parameter	Symbol	Unit	Dee	Don	Spey*	Tay	Tweed
			Value				
Altitude to distance ratio	α	N/A	1	1	1	1	1
Gable wall type (BS 5268-6.1:1996, Table 1)	N/A	N/A	1	1	1	1	1
Front wall type (BS 5268-6.1:1996, Table 1)	N/A	N/A	1	1	2	1	1
Length of building	L	m	9.2	9.5	11.8	8.9	8.5
Width of building	W	m	6.5	6.6	8.4	8.4	7.8
Aspect ratio (L/W)	β	N/A	1.4	1.4	1.4	1.1	1.1
Basic wind speed	V_b	m/s	24	24	24	24	24
Height to eaves	H_{ea}	m	5.2	5.2	5.2	5.2	5.2
Height to the ridge	H	m	7.1	7.5	7.4	7.5	7.5
Roof pitch	ψ	Degrees	30	30	30	30	30

Note:
- *The Spey house type has a Wall Type 1 at the front and Wall Type 2 at the rear. As a result Wall Type 2 has been conservatively adopted.
- Altitude to distance ratio has been considered as 1, this equates to:
    - 100m < Altitude, Δs, ≤ 150m
    - Distance from the sea, D_{sea} ≤ 10km
- The external wall dimensions were used to determine building length and width, a degree of interpolation has been used where additional elements protrude out from the building.
- Length was taken as the larger dimension of the building and width is taken as the smaller dimension.
- The equation is based on a roof pitch of 35°.
- Height to the ridge is calculated using the given width of building and roof angle.

The range of houses shown in Figure 4.22 were originally designed using CP3 Chapter V (now obsolete) to determine the wind loading. The developed model is in accordance with BS 6399-3:1997, therefore to ensure that the model has been developed interpreting the code correctly the applied wind load on the range of houses using both codes is compared (Table 4.23). This form of check is recommended by Cook (1998c).

According to Cook (1998a) in the majority of cases where BS 6399-3:1997 results in being overly conservative it has been as a result of misinterpretation of the rules for roughness categories. In terms of roughness 'in town' criteria was generically taken during model development as the houses being considered will form part of a large scale development.

The use of BS 6399-3:1997 may result in more conservative wind loading due to the modification of the 'division of parts' rule – clause 5.5.2 of CP3 and clause 2.2.3.2 of BS 6399:Part 2. The removal of the 'division of parts' could result in an increase in racking requirement by up to 15% for two storey buildings but this is generally off-set in areas of low exposure by reduced dynamic pressures (Cook, 1998b).

Table 4.23 Corresponding wind action to applied method of determination

Wind action		Code of practice		% Difference
		CP3 Chapter V	BS 6399-1:1997	
		kN		
Dee	On front	32	28	-16
	On gable	33	37	12
Don	On front	33	29	-15
	On gable	35	39	10
Spey	On front	58	51	-13
	On gable	35	33	-6
Tay	On front	33	38	13
	On gable	32	34	4
Tweed	On front	37	33	-12
	On gable	32	34	6
Average % difference				3

Wind on the gable of the building is more critical due to required openings in the front of the building limiting the level of racking resistance to gable wind action. Therefore, the application of BS 6399-1:1997 is on average marginally more conservative but the level of correlation between the codes, given the differences between them, is favourable.

In Table 4.24 to 4.26 the racking resistance for the given wall parameters applying the design rules of BS 5268-6.1:1996 are compared with those from the derived model, Equation 4.11. In accordance with clause 4.9.5 of BS 5258-6.1:1996 the racking resistance of the walls has been determined in conjunction with the uniformly distributed load information detailed in Table 4.27.

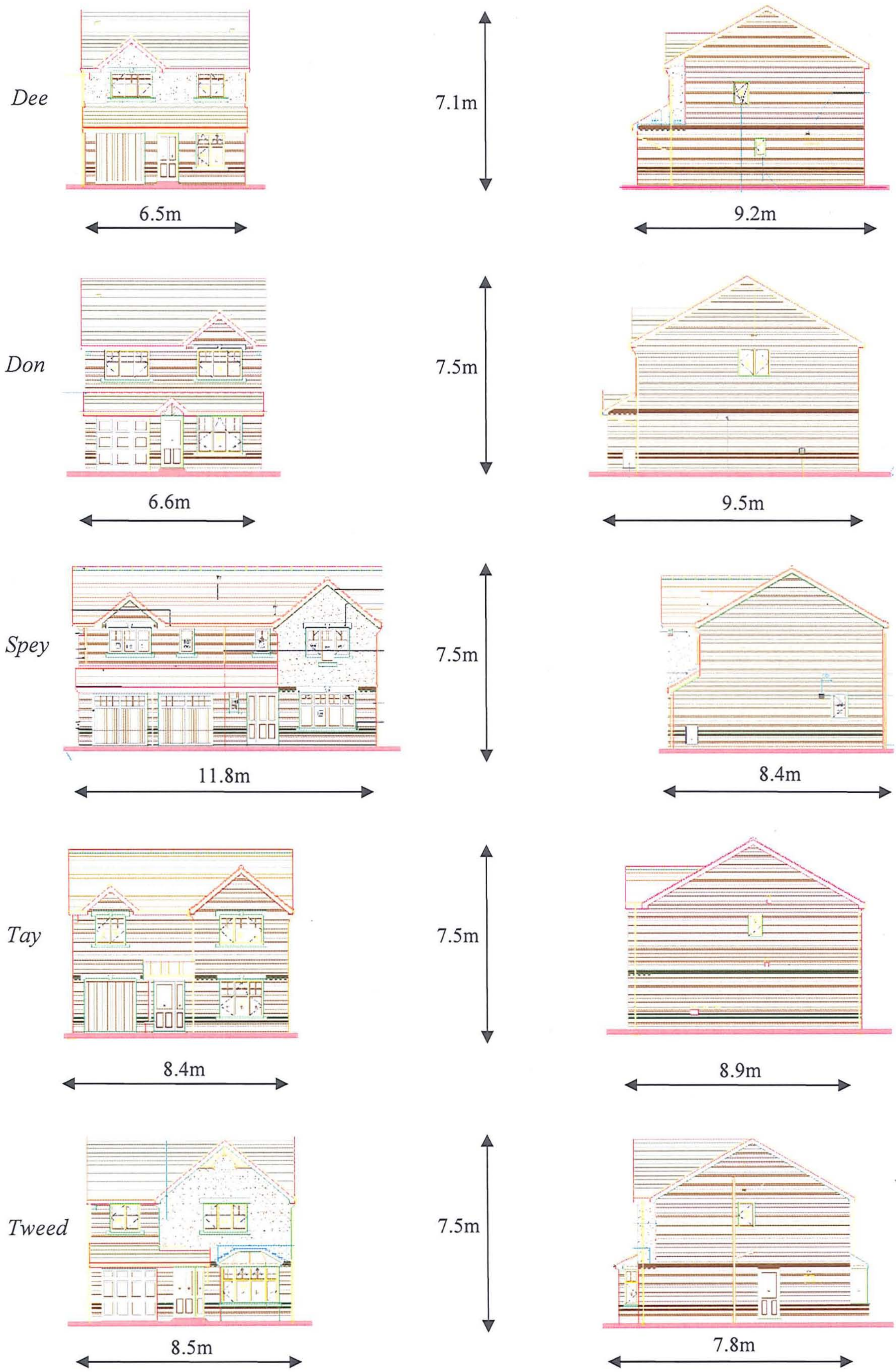


Figure 4.22 Front & gable elevations of house types

Table 4.24 Racking resistance of Dee and Don House Types

Wall designation	Wall Type	Length (m)	Nail spacing in mm & Sheathing arrangement	% of opening in wall	Total racking resistance of wall in accordance with BS 5268-6.1:1996 (kN)	Racking resistance: BS 5268 Design (kN/m)	Racking resistance: Model (kN/m)
Dee House Type: Wind on side							
Ground floor front wall	1	4.10	75 / Double	42	6.53	2.41	1.79
Ground floor rear wall	1	6.50	75 / Double	40	9.61	2.24	2.12
Ground floor internal wall1	1	4.20	100 / Single	0	11.00	5.82	5.82*
				Total	27.14	10.47	9.73
Dee House Type: Wind on front							
Ground floor gable wall 1	1	9.20	100 / Single	2	34.97	8.45	9.92
Ground floor gable wall 2	1	9.20	100 / Single	2	34.97	8.45	9.92
Ground floor internal wall 1	1	5.10	100 / Single	0	13.35	5.82	5.82*
				Total	83.28	22.71	25.66
Don House Type: Wind on side							
Ground floor front wall	1	4.00	75 / Double	50	2.37	0.83	0.77
Ground floor rear wall	1	6.60	75 / Double	40	9.47	2.17	2.12
Ground floor internal wall1	1	4.30	100 / Single	0	11.26	5.82	5.82*
				Total	23.10	8.83	8.70
Don House Type: Wind on front							
Ground floor gable wall 1	1	9.50	100 / Single	2	36.04	8.43	9.92
Ground floor gable wall 2	1	9.50	100 / Single	2	36.04	8.43	9.92
Ground floor internal wall 1	1	5.00	100 / Single	0	13.09	5.82	5.82*
				Total	85.17	22.68	25.66

Note:

- *The racking resistance values of internal racking walls have been calculated, in accordance with BS 5268-6.1:1996 for all cases and a conservative approach has been taken with the vertical load conditions not considered ($K_{107} = 1$).
- It is to be noted that the K_{100} factor (5268-6.1:1996, table 1) has been used to increase racking resistance rather than reduce wind loading.
- An additional racking allowance from the masonry cladding of 0.4kN/m run could in certain instances be added to the racking resistance of the external gable walls calculated in accordance with BS 5268-6.1:1996.

Table 4.25 Racking resistance of Spey and Tay House Types

Wall designation	Wall Type	Length	Nail spacing in mm & Sheathing arrangement	% of opening in wall	Total racking resistance of wall in accordance with BS 5268-6.1:1996	Racking resistance	
						BS 5268 Design	Model
		m			kN	kN/m	
Spey House Type: Wind on side							
Ground floor front wall	2	6.10	75 / Double	48	4.14	0.96	0.85
Ground floor rear wall	1	11.80	100 / Double	35	15.36	2.05	1.76
Ground floor internal wall1	1	6.40	100 / Single	0	16.76	5.82	5.82*
				Total	36.26	8.83	9.45
Spey House Type: Wind on front							
Ground floor gable wall 1	1	8.40	100 / Single	4	34.97	9.25	9.28
Ground floor gable wall 2	1	8.40	100 / Single	4	34.97	9.25	9.28
Ground floor internal wall 1	1	7.40	100 / Single	0	19.37	5.82	5.82*
				Total	89.30	24.32	24.38
Tay House Type: Wind on side							
Ground floor front wall	1	8.90	75 / Double	50	6.22	0.98	0.77
Ground floor rear wall	1	8.90	100 / Single	33	12.17	2.24	3.16
Ground floor internal wall	1	6.00	100 / Single	0	15.71	5.82	5.82*
				Total	34.10	9.04	9.75
Tay House Type: Wind on front							
Ground floor gable wall 1	1	8.40	100 / Single	0	35.72	9.45	10.58
Ground floor gable wall 2	1	8.40	100 / Single	0	35.72	9.45	10.58
Ground floor internal wall 1	1	4.90	100 / Single	0	12.83	5.82	5.82*
				Total	84.27	24.72	26.99

Note:

- *The racking resistance values of internal racking walls have been calculated, in accordance with BS 5268-6.1:1996 for all cases and a conservative approach has been taken with the vertical load conditions not considered ($K_{107} = 1$).
- It is to be noted that the K_{100} factor (5268-6.1:1996, table 1) has been used to increase racking resistance rather than reduce wind loading.
- An additional racking allowance from the masonry cladding of 0.4kN/m run could in certain instances be added to the racking resistance of the external gable walls calculated in accordance with BS 5268-6.1:1996.

Table 4.26 Racking resistance of Tweed House Type

Wall designation	Wall Type	Length (m)	Nail spacing in mm & Sheathing arrangement	% of opening in wall	Total racking resistance of wall in accordance with BS 5268-6.1:1996 (kN)	Racking resistance (kN/m) BS 5268 Design	Model
Tweed House Type: Wind on side							
Ground floor front wall	1	7.80	75 / Double	55	2.55	0.46	0.35
Ground floor rear wall	1	7.80	100 / Single	40	7.98	1.55	1.92
Ground floor internal wall	1	3.70	100 / Single	0	9.69	5.82	5.82*
Total					20.22	7.83	8.10
Tweed House Type: Wind on front							
Ground floor gable wall 1	1	8.50	100 / Single	10	25.89	6.77	7.47
Ground floor gable wall 2	1	8.50	100 / Single	10	25.89	6.77	7.47
Ground floor internal wall 1	1	3.80	100 / Single	0	9.95	5.82	5.82*
Total					61.73	19.36	20.76

Note:
- *The racking resistance values of internal racking walls have been calculated, in accordance with BS 5268-6.1:1996 for all cases and a conservative approach has been taken with the vertical load conditions not considered ($K_{107} = 1$).
- It is to be noted that the K_{100} factor (5268-6.1:1996, table 1) has been used to increase racking resistance rather than reduce wind loading.
- An additional racking allowance from the masonry cladding of 0.4kN/m run could in certain instances be added to the racking resistance of the external gable walls calculated in accordance with BS 5268-6.1:1996.

Table 4.27 UDL information from original calculations

Wall designation	Dee	Don	Spey	Tay	Tweed
	Uniformly distributed load (UDL) kN/m				
Ground floor front walls	1.10	1.10	1.10	5.80	5.60
Ground floor rear wall	6.80	6.00	6.30	5.80	5.60
Ground floor internal wall	0.00	0.00	0.00	0.00	0.00
Ground floor gable wall 1	3.20	3.20	3.50	3.20	3.20
Ground floor gable wall 2	3.20	3.20	3.50	3.20	3.20
Ground floor internal wall 1	0.00	0.00	0.00	0.00	0.00

Contained in Table 4.28 is the following information (based on the wall information detailed in Table 4.24 to Table 4.26):

- The level of required openings in the walls of the given house type, marginally achieved from carrying out design calculations in accordance with BS 5268-6.1:1996 and CP3 Chapter V.
- The allowable level of openings in the walls, considering only the external walls to be providing racking applying Equation 4.7.
- The allowable level of openings in the walls, with additional allowance made for the internal racking walls applying Equation 4.12 with the resistance of the internal racking walls calculated in accordance with BS 5268-6.1:1996.

Table 4.28 Design required & allowable percentage opening at ground floor level

House type	Wall	Level of percentage opening					
		Actual required opening, %		Model determined allowable % opening: Considering external walls only		Model determined allowable % opening: Inclusive of internal racking walls*	
		Individual	Total	Individual	Total	Individual	Total
Dee	Gable	2	54	14	51	21	70
	Front & Rear	52		37		49	
Don	Gable	2	57	13	50	18	72
	Front & Rear	55		37		53	
Spey	Gable	4	59	15	49	24	71
	Front & Rear	55		34		47	
Tay	Gable	10	58	26	60	37	80
	Front & Rear	48		34		43	
Tweed	Gable	10	66	25	59	31	71
	Front & Rear	56		34		40	

*Internal racking wall resistance calculated in accordance with BS 5268-6.1:1996

Shown in Table 4.29 are the percentage differences between the allowable level of opening determined by the model and what is required by the house type. The developed model has been applied considering no internal contribution from additional racking walls and also allowing for an internal racking contribution.

Table 4.29 Percentage differences between results

House type	Gable		Front & Rear		Combined Front & Rear + Gable	
	External Only	Including internal walls	External Only	Including internal walls	External Only	Including internal walls
	% Difference					
Dee	86	90	-29	-5	5	23
Don	85	89	-33	-3	12	21
Spey	73	83	-38	-15	18	17
Tay	61	73	-29	-10	-4	28
Tweed	59	68	-42	-29	14	7
Average	73	81	-34	-12	9	19

From Table 4.29 the following conclusions can be drawn:

- In the cases studied the percentage of opening required in the gable wall is less than what is achievable when compared with the model results. This is as expected, for the majority of cases it is the level of opening required in the front and rear which is critical.
- If the proposed model is applied, not considering the additional racking resistance provided by the internal walls, then the allowable level of opening in the front & rear of the House Types examined will be on average 34% less than what is required. If the model is applied and an allowance to allow for the additional resistance provided by internal walls is provided (which would be the case in full design) then the allowable level of opening in the front & rear of the House Types examined will be on average 12% less than what is required.
- If full design is carried out the systems, although marginally, do work. Therefore, although the model is conservative by approximately 12% it is accurate given the range of variables to be considered as provided by Table 4.29.
- In terms of both the results contained in Table 4.29 the Tweed House Type shows the poorest correlation between the architecturally required and model attainable level of opening. However, it is to be noted that the Tweed House Type requires a relatively high level of opening in the front and rear given the dimensions of the building and low level of racking resistance provided by internal walls.
- It is demonstrated that the full level of allowable opening in the gables is not being utilised by an average of 81% when comparing the architectural requirement to the model determined allowable level (Table 4.29). Therefore, the openings could be more evenly proportioned and this would result in a more efficient system i.e. the material and fixing specification of the gable walls is in excess of what is required, therefore they could be better utilised.

Examination of the application and accuracy of the proposed model on a variety of standard or typical house types has demonstrated that the model is relatively conservative. With this in mind and considering the number of variables involved in determining the racking resistance of a building, it is concluded that the proposed model provides a powerful tool for tentative analysis and determination of racking requirements of timber platform frame buildings with a large combination of parameters; and hence providing a range of possible alternative solutions.

For the range of houses shown in Figure 4.22 the proposed model has been used to determine the optimum level opening for varying racking panel and wall types (Figure 4.23 to Figure 4.26). Inspection of Figure 4.25 shows that the inclusion of additional racking resistance due to internal walls in the model tends not to provide valid results when considering Wall Types 2 & 3, Wall Type 2 tends to allow a higher level of opening than Wall Type 1. It was also noted that for Wall Types 2 & 3, when internal racking walls are considered, the quadratic equation can on occasion not be solved resulting in the allowable opening in the wall returning to zero at a premature stage. Consequently Figure 4.24 and Figure 4.26 have been produced containing the results for Wall Type 1 only and it is these results which are confidently used for the comparative study.

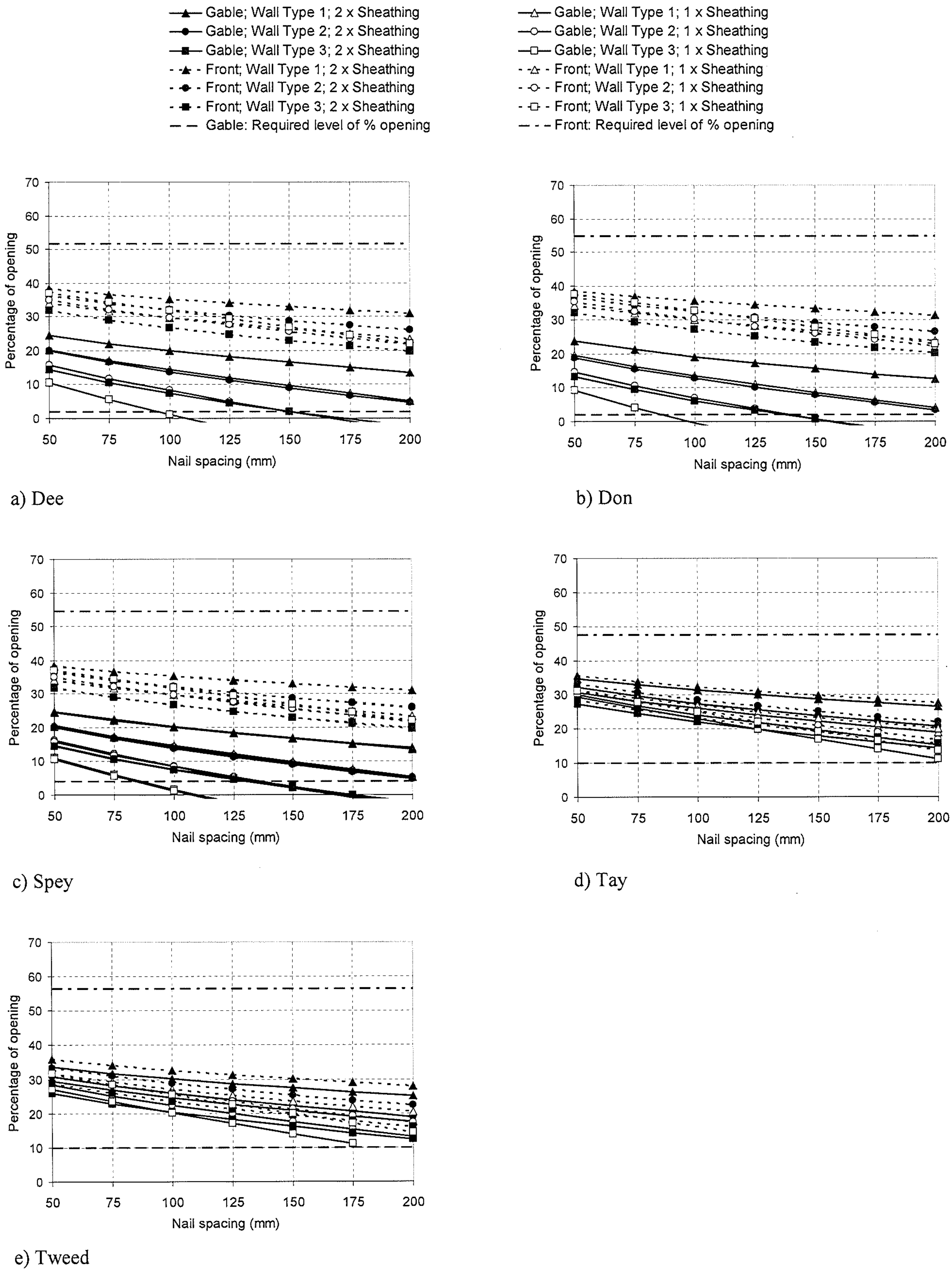


Figure 4.23 Model determined allowable percentage opening for varying nail spacing and wall types with the racking contribution from the external walls only considered.

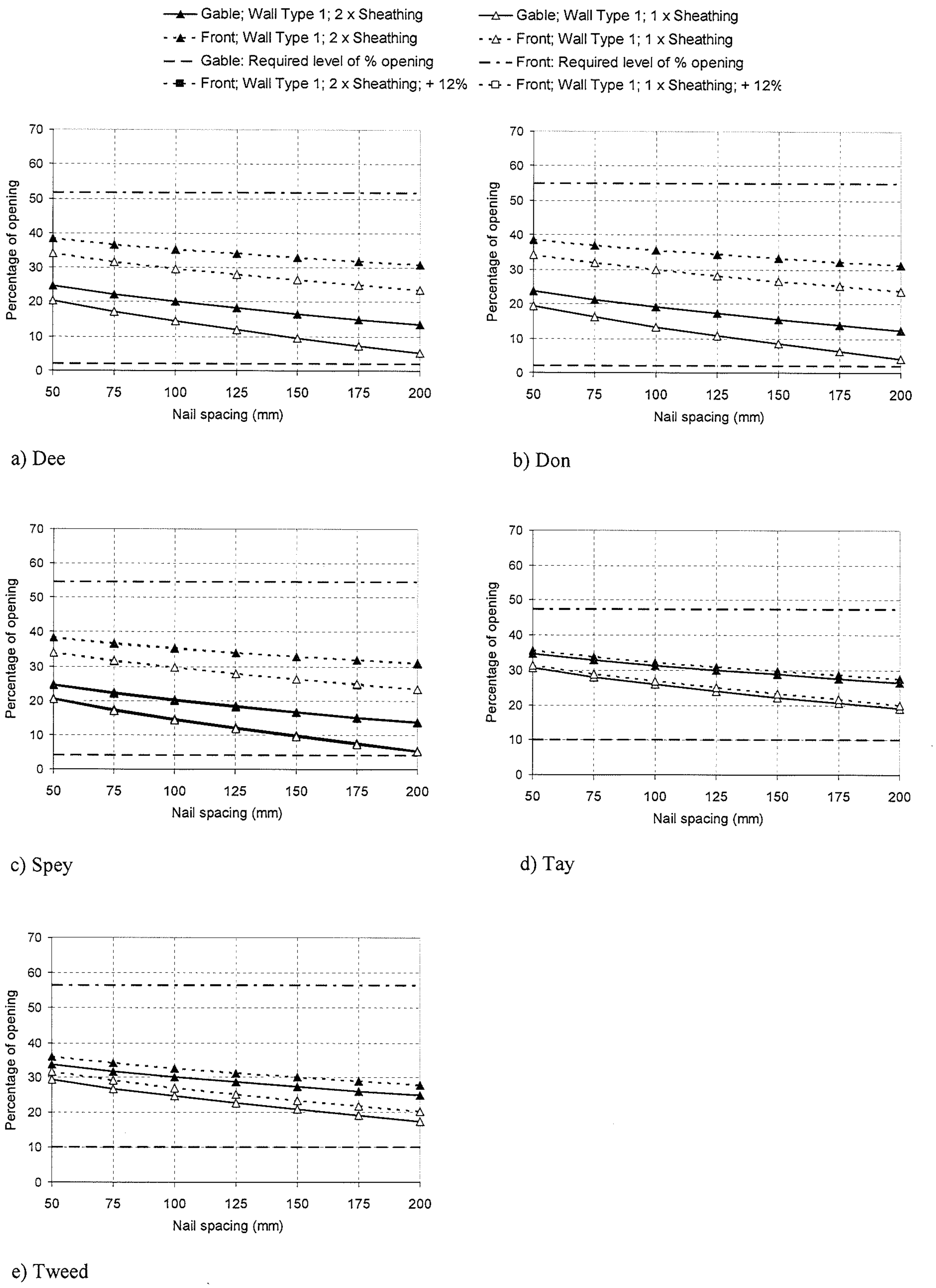


Figure 4.24 Model determined allowable percentage opening considering Wall Type 1 for varying nail spacing with the racking contribution from the external walls only considered.

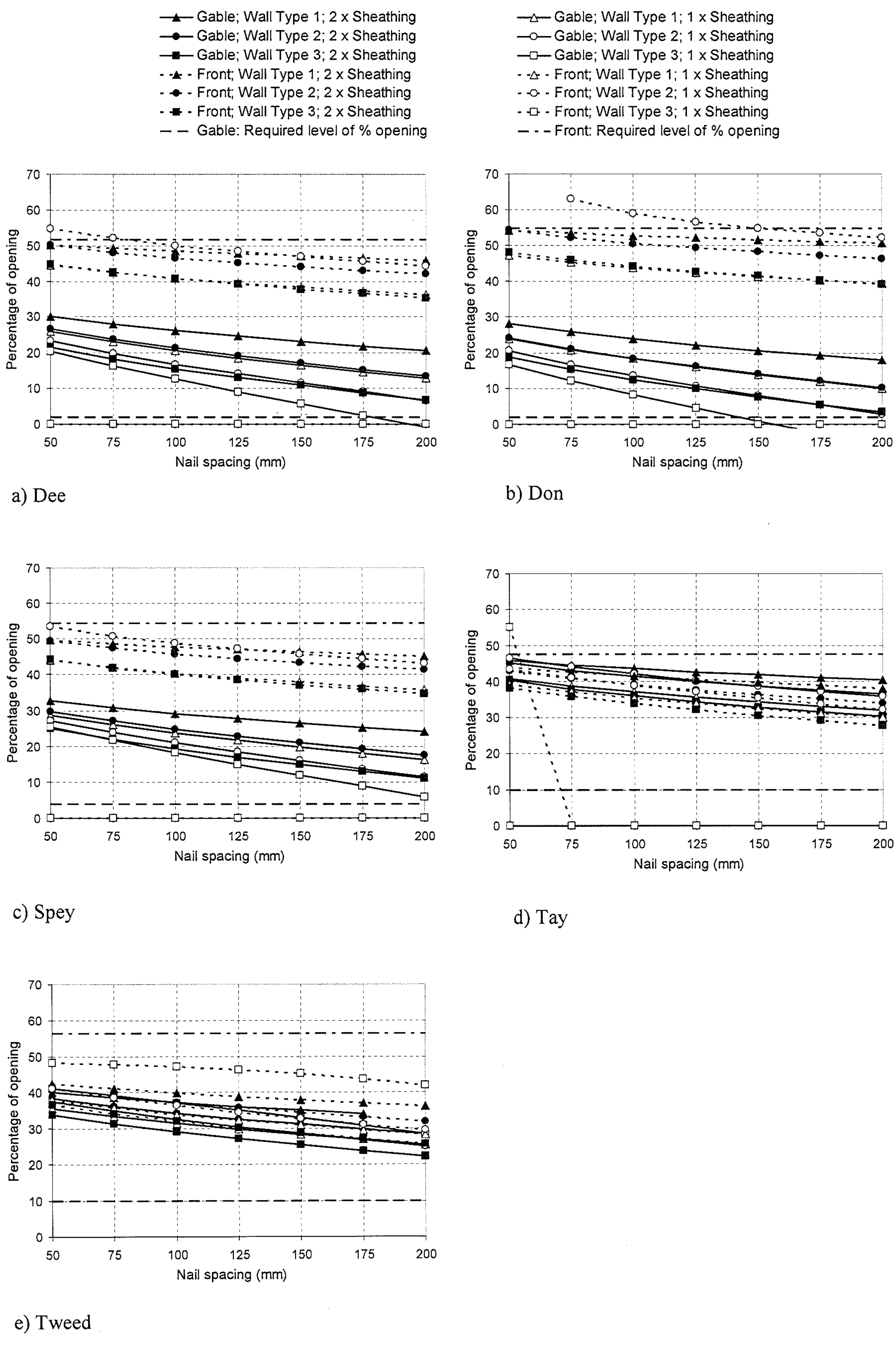
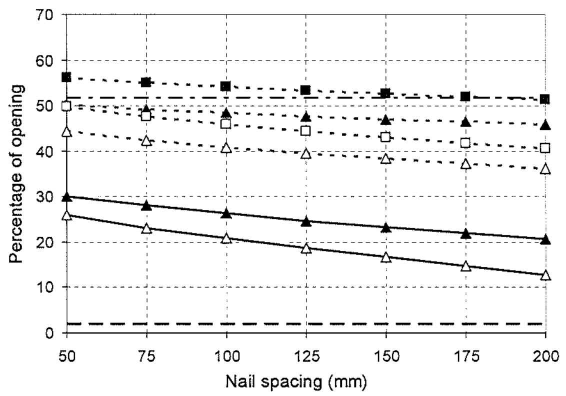
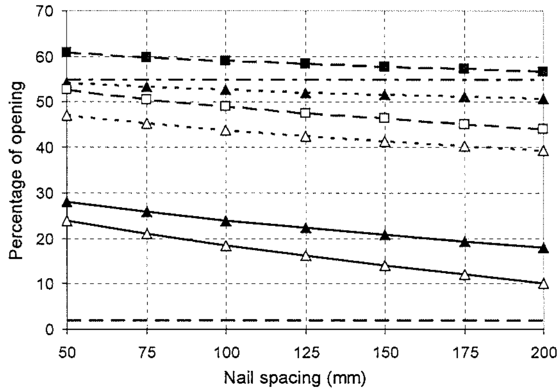


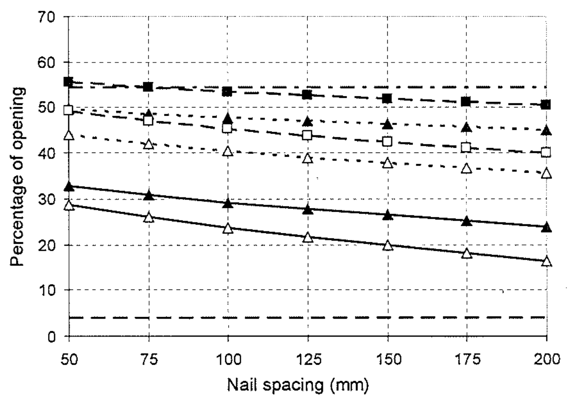
Figure 4.25 Model determined allowable percentage opening for varying nail spacing and wall types with the racking contribution from internal walls additionally considered.



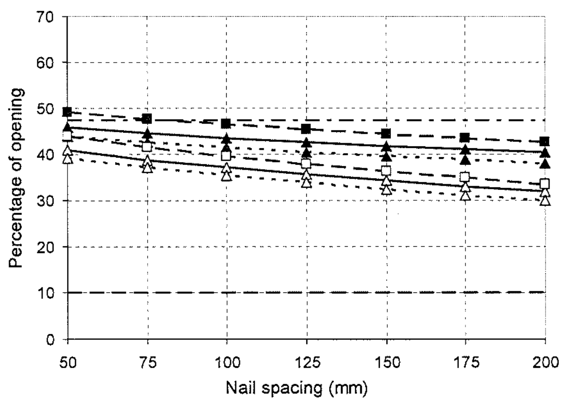
a) Dee



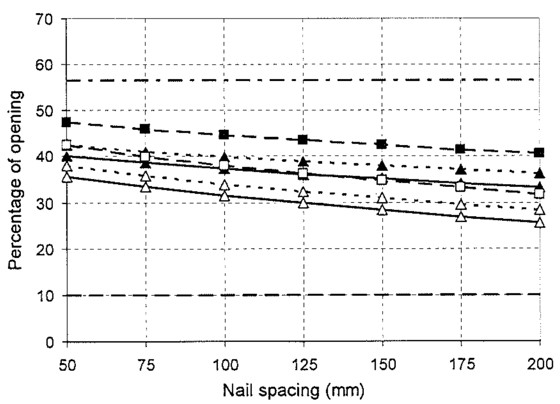
b) Don



c) Spey



d) Tay



e) Tweed

Figure 4.26 Model determined allowable percentage opening considering Wall Type 1 for varying nail spacing with the racking contribution from internal walls additionally considered.

From Figure 4.23 to Figure 4.26 the following conclusions are drawn:

- It is shown in Figure 4.23 that the level of opening architecturally required cannot be achieved by any wall make-up if additional racking resistance from internal walls is not considered. The wall type (1, 2 or 3) does not significantly alter the level of allowable opening and, as a result Figure 4.24 has been produced to show the result considering Wall Type 1 only.
- It is shown in Figure 4.24 that if the House Types detailed in Figure 4.22 are to be designed such that the external walls only are to be used to provide racking resistance then the level of opening in the front and back should be limited to a maximum of 40%. Further to this, if economical design is to be achieved (single sheathed walls), using only the external walls to provide racking resistance, then the level of opening in the front and rear would have to be limited to a maximum of 35%.
- For the majority of circumstance the model provides relatively accurate results which are conservative by a margin of approximately 12% when the additional racking resistance of the internals is considered.
- It has been demonstrated that the proposed model can be used in initial design to determine what is achievable in terms of allowable openings.
- In the case of the Dee and Don house types it is shown that, when adjusted to allow for a 12% conservatism single sheathing with 50mm nail spacing may be a viable option which is more cost effective than the use of additional sheathing.

4.8 Conclusions

It has been demonstrated that the proposed model is capable of predicting, with a relatively high degree of accuracy given the number of variables involved, the optimum level of percentage opening (level of opening which can be obtained given the applied racking force and available racking resistance without the need for additional system bracing) in typical timber platform frame domestic dwellings constructed in the UK within the preset boundaries contained in Table 4.1.

The developed model has been used to look at the financial and structural implications of the architectural layout of buildings, which in normal UK house construction requires a high level of percentage opening in the front and back of houses and a negligible amount in the gable walls. From a financial perspective it has been demonstrated that in order that the required level of opening in the front of a house does not impinge on system cost there are two viable options available:

1. The percentage of opening can be reduced to a level which is acceptable to negate the requirement for extra sheathing but can be achieved by means of reduced nail spacing.
2. The aspect ratio of the system is increased without reducing the internal area, as long as the gables are capable of carrying the increased load, allowing additional opening to be achieved without an increase in cost.

It is known that manufacturing and material costs are directly related, additional material to be added requires additional man hours. Therefore, by endorsing the above points an overall financial gain would be made in terms of material and labour costs and, if a closed panel system were to be adopted, manufacturing throughput would increase.

From the study of a range of design cases the model has been further verified. However, it has been demonstrated that inclusion of internal walls within the model can tend to cause an overestimation when considering Wall Types 2 and 3. Although this is the case looking at actual design cases has enhanced the study and demonstrated that the proposed model can be used with confidence for initial design and costing purposes. By applying the model to design cases, although the model is understood to be on the conservative side and have limitations as previously stated, it has been shown that for the range of house types reviewed the architectural features required are both financially penalising in terms of manufacturing costs and also difficult to acheive structurally.

It is understood that a large level of opening is required in the front and rear of houses and negligible in the gable walls due to site restrictions, houses are built within close proximity and the software used by developers optimises the number and orientation of plots to maximise the use of available land and allow the ease of access and egress of services to the plots. This being the case it requires the procurement process to be explored so that points 1 and 2 above can be fully or even partially endorsed.

The intention is to present the findings of this research to house builders to demonstrate the impact that architectural layout has on cost effectiveness and structural robustness with a view to implementing an improved balance between the affected factions. The developed models are to be used as part of the cost estimation procedure for proposed contracts and also for initial structural design.

One of the main objectives of the research project was to simplify the design process whilst maintaining a level of transparency such that a designer or engineer employing the methods developed understands the influencing factors. This has been achieved, evidence of which is the successful use of the models to derive a simplified design technique for determining racking resistance requirements published by the Scottish Buildings Standards Agency (2007) in "*Structural Guidance for Small Buildings: Technical Handbook*".

CHAPTER 5

WALL DIAPHRAGMS

5.1 Introduction

The EU Directive on Energy Performance of Buildings, which has the aim of promoting energy performance within the EU, will impact upon the timber platform frame industry. To achieve the requirements of the directive the U-value of wall details will have to be improved. This chapter begins with the development of a semi-empirical model which can be used to estimate the U-value of a timber frame wall detail. The initial objective of developing the model was to provide a readily available method of providing wall options for typical timber platform frame systems without the need for specialist software. The derived model is then used to evaluate a range of wall options which would be capable of meeting the target U-value and these options are then compared relative to each other in terms of Life Cycle Assessment (LCA) and monetary cost.

Structural Insulated Panels (SIPs) are considered as an alternative to traditional timber frame walls mainly due to their improved energy efficiency. The latter section of this chapter provides an overview of the benefits of SIPs and also presents the findings from a research programme investigating their structural performance. The investigation was an extension of research work carried out by Kermani (2005) into the performance of SIPs when subjected to bending and axial compression to encompass the racking performance of SIPs and the effects of size and position of openings for doors and windows on racking performance.

5.2 Development of a Sustainable Wall Detail

5.2.1 General

As a material timber is generally considered to have excellent environmental credentials as it is naturally renewable, easily worked and non-toxic. As a renewable resource, its main attribute is that it absorbs and thus reduces the amount of CO_2 in the atmosphere, which is only released if it decays or is burnt. In essence every cubic metre of timber used in place of other building materials saves the release of 0.8t of CO_2. Considering an average detached timber frame house this equates to around 4 to 5 tonnes of CO_2 (Harris, 2005).

Timber platform frame is also environmentally efficient when considering the building envelope and falls comfortably within the UK Governments priorities of reducing climate change and providing a low carbon economy with sustainable production and consumption; all with duty of care towards natural resources. In endorsing the EU Directive 2002/91/EC on energy performance of buildings the recent introduction of the revised Part L of the Building Regulations (ODPM, 2006) will lead to an improvement in the energy efficiency of buildings by around 20%.

The Standard Assessment Procedure (SAP) rates the energy efficiency of dwellings and is required for new homes and conversions under the Building Regulations. The assessment indicators of the energy performance are, according to DEFRA (2005), energy consumption per unit floor area, an energy cost rating (the SAP rating), an environmental impact rating (based on CO_2 emissions) and a Dwelling CO_2 Emission Rate (DER).

The Environmental Impact Rating is based on the annual CO_2 emissions associated with space heating, water heating, ventilation and lighting, less the emissions saved by energy generation technologies. It is adjusted for floor area so that it is essentially independent of dwelling size for a given built form.

The Environmental Impact Rating is expressed on a scale of 1 to 100, the higher the number the better the standard with 100 representing zero energy cost.

The Dwelling CO_2 Emission Rate is a similar indicator to the Environmental Impact Rating, which is used for the purposes of compliance with the Building Regulations. It is equal to the annual CO_2 emissions per unit floor area for space heating, water heating, ventilation and lighting, less the emissions saved by energy generation technologies, expressed in kg/m^2/year.

For new buildings compliance is assessed via a whole-house calculation using SAP 2005 software approved by BRE on behalf of the Department for Environment, Food and Rural Affairs; the Office of the Deputy Prime Minister; the Scottish Executive; the National Assembly for Wales; and the Department of Finance and Personnel.

The revised regulations implemented are markedly different in approach from previous regulations in their criteria for compliance, by making a requirement in terms of overall $C0_2$ emissions in addition to performance requirements on individual elements. In relation to the timber platform frame industry the revised regulations will, in conjunction with other requirements, result in wall U-values in domestic dwellings to be reduced to between 0.27 and 0.30W/m^{2}K, with the target U-value to ensure overall SAP rating compliance, based on current timber platform frame systems, 0.27W/m^{2}K. A U-value is described by Doran (2006) as the quantity of heat that will flow through unit area in unit time, per unit difference in temperature between the external and internal environment.

5.2.2 Timber Frame Wall

Shown in Figure 5.1 is a traditional timber platform frame wall detail in UK construction with a 50mm outside cavity and external masonry skin, the U-value of which is 0.40W/m^{2}K. Therefore, the thermal rating of timber frame walls will have to improve. However, timber frame is at an advantage when considering other forms of construction as a result of being able to comply through a number of available options.

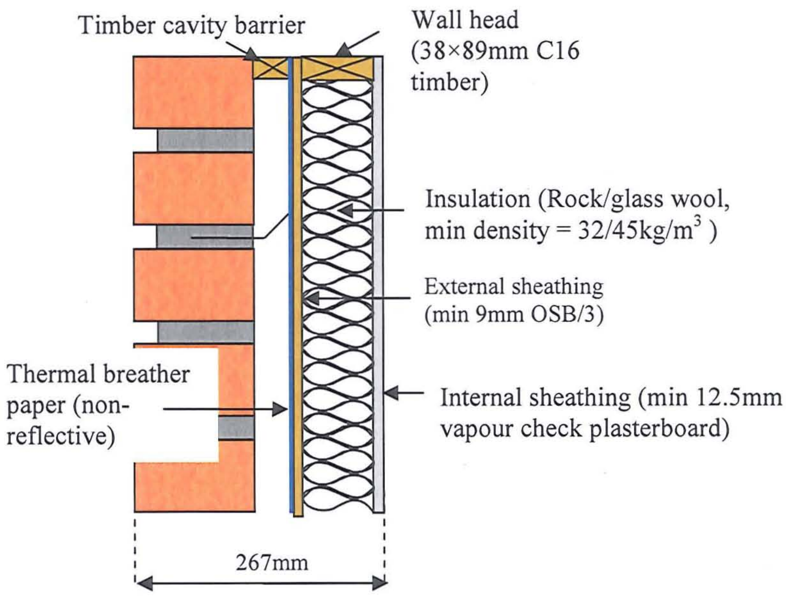


Figure 5.1 Standard timber frame wall detail

The amount of thermal bridging can be reduced. Thermal bridging in timber frame walls is normally caused by gaps in insulation layers within the fabric, structural elements, especially lintels and frames, joints between elements and joints around windows and doors. In relation to this the incorporation of 'Robust Detailing' in the form of a fibre cavity barrier as a replacement to timber is beneficial.

Use of a low emissivity surface in the form of reflective breather paper can reduce the radiation transfer across an airspace, so that the airspace has a higher thermal resistance which results in a constant U-value rating reduction of approximately 0.02W/m^{2}K compared with one bounded by surfaces of normal (high) emissivity. It is to be noted that low emissivity cannot be considered to have an effect on the U-value if the surface is not adjacent to an airspace of at least 22mm wide in the construction (Ward, 2001).

Internal or external sheathing with improved thermal conductivity can be used. However, this is limited as the primary function of the external sheathing is to provide racking resistance to the wall diaphragm and as a result is required to be a Category 1 primary board material (BS 5268: Section 6.1: 1996), examples of which are 9.5mm plywood, 9.0mm medium board, 6.0mm tempered hardboard or 9.0mm oriented strand board grade 3 (OSB/3) which normally have a thermal conductivity, λ value, of 0.13W/mK. The thermal conductivity performance of external sheathing can be improved by processes such as bitumen impregnation but this is limited to 0.05W/mK (Hunton Fibre, 1994).

In relation to the internal sheathing a 12.5mm minimum thickness of plasterboard (λ = 0.29 W/mK) is required to be fixed to the inside face of external walls in domestic dwellings so that fire and sound transfer regulations are met. In instances where added racking resistance is required an internal sheathing layer of Category 1 primary board material would be added although the added benefit in terms of thermal performance is limited (it also was shown in Chapter 4 Table 4.17 that increased nailing is a more cost effective method of achieving improved racking performance).

Using Elmhurst SAP Energy Rating Software a parametric study was conducted to determine the relationship between U-value and sheathing thickness for a range of λ values when considering the wall detail in Figure 5.1 incorporating a fibre cavity barrier and a low emissivity cavity. It is to be noted that the U-value calculations carried out are inclusive of an allowance for cold bridging due to the timber elements of the wall in the form of a 0.15 timber fraction (15% cold bridging) in accordance with Anderson (2006).

The results of this study are illustrated in Figure 5.2 and it is to be noted that the sheathing thickness given could be an accumulative thickness, i.e. 9mm internal and external sheathing of the same λ value would result in a total thickness of 18mm.

The trend lines which correspond to each set of data are of a logarithmic type:

$$y(t_{sh}) = A \cdot \ln(t_{sh}) + B \qquad \text{Equation 5.1}$$

Where:

t_{sh} is the thickness of the sheathing material in mm.

A & B are as defined in Table 5.1 or each of the given trend lines presented.

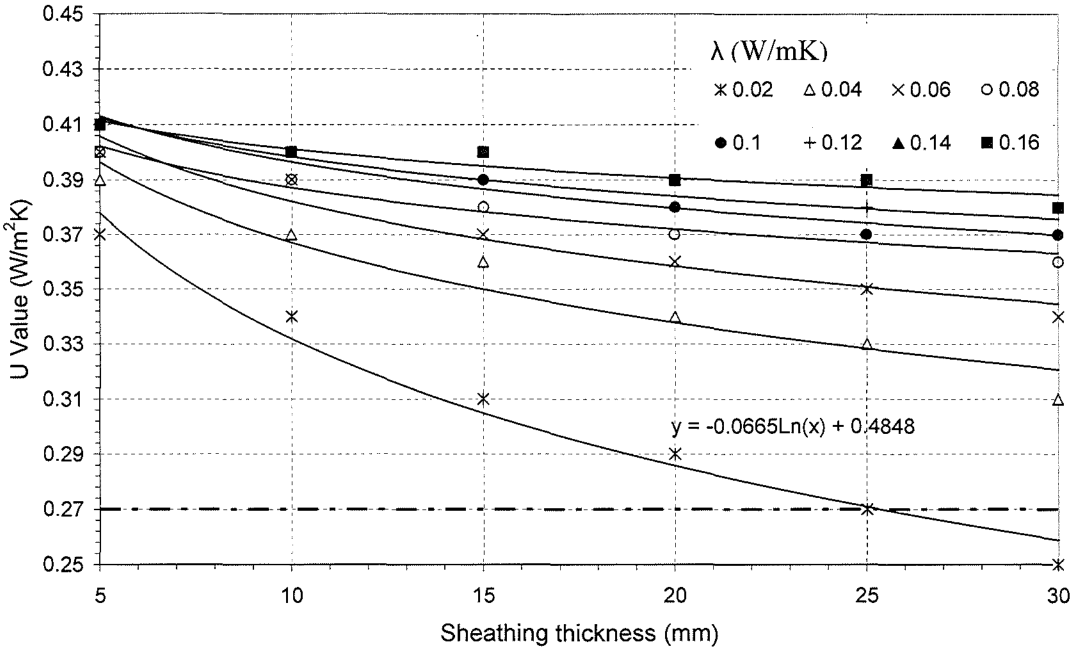


Figure 5.2 Relationship of wall detail U-value with changing sheathing thickness and λ value (broken line represents target U-value)

Table 5.1 Values of constants corresponding to Equation 5.1and Figure 5.2

λ	Constant	
(W/mK)	*A*	*B*
0.02	-0.07	0.49
0.04	-0.05	0.48
0.06	-0.04	0.47
0.08	-0.03	0.44
0.10	-0.03	0.45
0.12	-0.03	0.45
0.14	-0.02	0.44
0.16	-0.02	0.43

To achieve the target U-value of 0.27W/m^{2}K it is demonstrated in Figure 5.2 that a sheathing material of low thermal conductivity would have to be specified, approximately λ = 0.02 W/mK at a thickness of 25mm which would correspond to a thermal resistance, *R*, value of 0.8 m^{2}K/W (thermal resistance, *R*, is equal to the thermal conductivity, λ, divided by the thickness of the material). Illustrated in Figure 5.3 is the relationship between cost and thermal resistance for a range of readily available and commonly used sheathing products. In Figure 5.3 two types of plasterboard are given, normal plasterboard and vapour check plasterboard. The specification of a vapour check plasterboard can negate the requirement to specify and attach a polyethene barrier behind the plasterboard to prevent moisture ingress which results in a time saving.

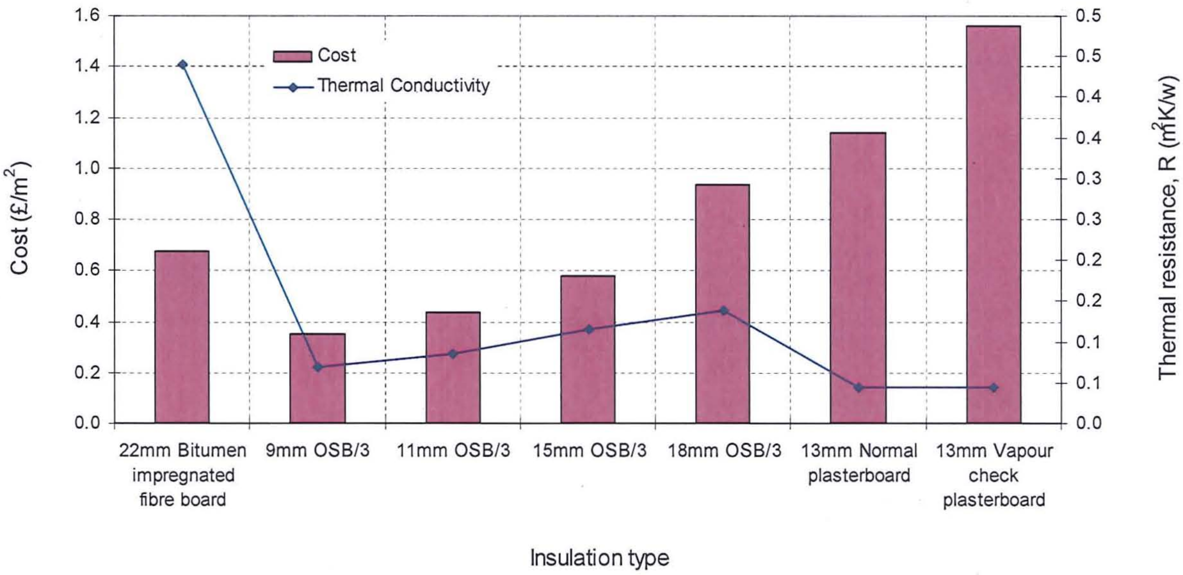


Figure 5.3 Material cost (2007 figures) and thermal resistance of sheathing products

It is demonstrated in Figure 5.3 that a sheathing material which can provide the structural racking performance and thermal resistance required to enhance the original standard wall detail is not available. However, from Figure 5.3 it is concluded that of the products considered 22mm bitumen impregnated fibre board and 9mm OSB/3 offer the most value in terms of cost and thermal performance.

A further parametric study was conducted considering the relationship between internal insulation (between studs) thickness for a range of λ values. The results of this investigation are as illustrated in Figure 5.4 with the trend lines shown corresponding to each set of data of a linear type:

$$y(t_{ii}) = C \times t_{ii} + D \qquad \text{Equation 5.2}$$

Where:

t_{ii} is the thickness of the internal insulation in mm

C & D are as defined in Table 5.2 for each of the given trend lines presented.

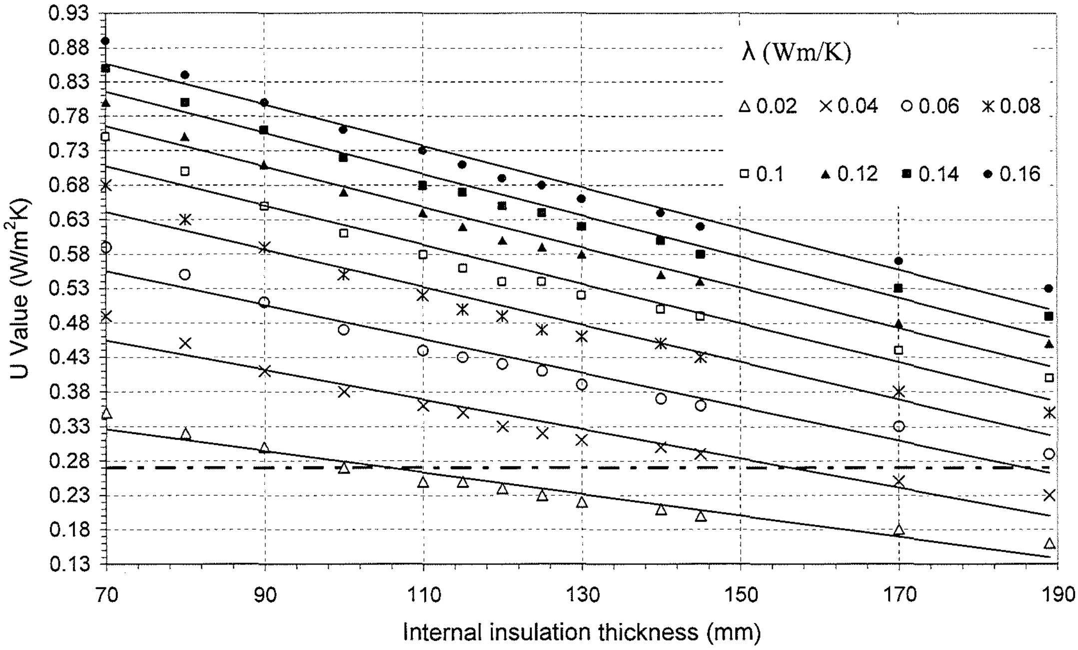


Figure 5.4 Relationship of wall detail U-value with internal insulation (between studs) thickness and λ value (broken line represents target U-value)

Table 5.2 Values of constants corresponding to Equation 5.2 and Figure 5.4

λ	Constant	
(W/mK)	*C*	*D*
0.02	-0.0016	0.44
0.04	-0.0021	0.60
0.06	-0.0025	0.73
0.08	-0.0027	0.83
0.10	-0.0028	0.91
0.12	-0.0029	0.97
0.14	-0.0030	1.02
0.16	-0.0030	1.07

The insulation contained within the wall can be a variety of materials. Contained in Table 5.3 is the approximate range of thermal conductivity and corresponding cost for a range of insulation materials. Also contained in Table 5.3 are Life Cycle Analysis (LCA) ratings based on a 60-year building design life (Anderson and Howard, 2000). LCA provides precise information on the overall impact a product has on the environment from the time the raw materials are extracted through to the end of its life, including transport, production and use.

The level of thermal resistance, *R*, an insulation can provide to a wall detail is governed by its thickness which is determined by the stud width. External timber frame wall studs are limited to a minimum size of 38×72mm by BS 5268: Section 6.1:1996, but normal practice is to use either a 38 or

45mm thick by 89, 95, 115 or 140mm wide C16 timber section although other available stud widths include 97, 114, 120, 145, 170, 184 and 195mm (TRADA, 2005).

Current practice for the majority of timber platform frame manufacturers is to use a 38×89mm stud due to availability of section and cost. It is demonstrated that to achieve the target U-value of 0.27W/m^{2}K, whilst maintaining 38×89mm stud, the insulation between the studs would have to have a thermal conductivity, λ, value lower than 0.02 W/mK and according to Table 5.3 the only insulation capable of offering this would be a high performing polyurethane, although the minimum value stated is 0.022W/mK which would require a thickness of approximately 110mm. To reduce the required thickness from 110mm a low emissivity service void can be introduced on the inside wall face as a method of reducing the level of thermal bridging. A service void would normally be constructed using 25×38mm timber battens running longitudinally along the header and footer of the wall panel and at 600mm centres vertically to allow the fixing of the internal layers of plasterboard. The low emissivity cavity would be created by placing a reflective polythene vapour barrier over the insulation and because this layer would prevent water ingress vapour check plasterboard is not required and normal plasterboard would be specified.

Increasing the stud size is a further option. For instance the use of a 140mm thick stud would allow the use of an insulation product with an approximate thermal conductivity, λ, of between 0.030 & 0.040W/mK. As a means of measuring the cost efficiency of available and commonly used insulation products Figure 5.5 was produced allowing cost and thermal conductivity to be compared relative to each other.

Table 5.3 Insulation materials and their associated ratings

Insulation Type	Life Cycle Assessment	Thermal conductivity, λ	Cost
		W/mK	£/m^2
Corkboard insulation with density 120kg/m^3	Medium	0.050 – 0.040	£7 - £11
Expanded polystyrene (EPS)	Low	0.040 – 0.032	£5 - £7
Extruded polystyrene (XPS) (HCFC free) with density less than 40kg/m^3	High	0.036 – 0.027	£10 -£12
Foamed glass insulation	Medium	0.042	£14 - £17
Glass wool insulation with a density of 10 - 32kg/m^3	Low	0.040 – 0.033	£2 - £10
Rock wool insulation with a density of 23 - 45kg/m^3	Low	0.040 – 0.033	£1 - £15
Polyurethane insulation (PU) (HCFC free)	Medium	0.028 – 0.022	£7 - £8
Recycled cellulose insulation	Low	0.044 – 0.038	£2 - £4
Based on a Life Cycle Assessment (LCA) considering a 60-year building design life, the costs are indicative as built costs inclusive of materials, labour and plant (Anderson and Howard, 2000) with thermal conductivity based on information from Elmhurst SAP Energy Rating Software.			

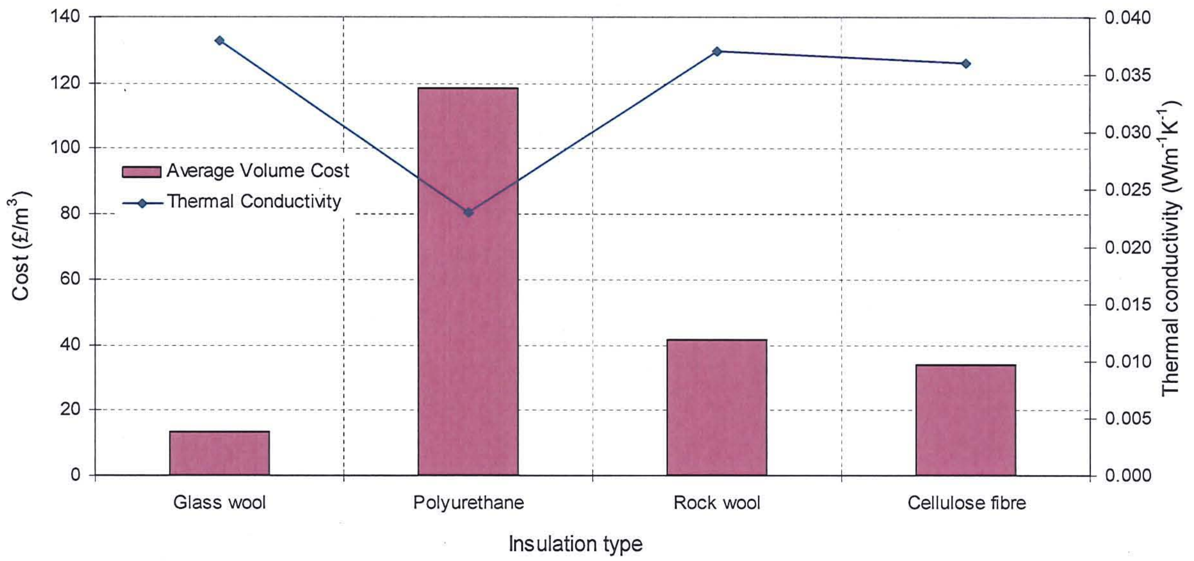


Figure 5.5 Material cost (2007 figures) and thermal conductivity of internal insulation products

From Figure 5.5 it is demonstrated that glass wool is the most cost effective method of providing insulation compared to the other readily available products. However, the use of polyurethane due to its low level of thermal conductivity may in combination with other materials provide a cost effective alternative. Glass wool according to Anderson and Howard (2000) is considered in terms of Life Cycle Assessment (LCA) to have low environmental impact (Table 5.3). Glass wool is produced with materials that are plentiful, such as sand and limestone, and increasingly more recycled glass ("cullet") is being used. Saint-Gobain Isover (2007) report that more than 40% of the raw material used in their glass wool product is accounted for by recycled glass.

To improve the U-value rating of a wall detail another option is to apply an internal (inside sheathing face) or external (in the cavity) thermal laminate. An external thermal laminate will normally be fixed to the external sheathing board by stainless steel nails at specified centres up to a maximum thickness of 50mm due to on-site practicality. Internally thermal laminates can be fixed to the studs or internal sheathing material beneath the plasterboard. Alternatively the internal thermal laminate will form part of the wallboard whereby it is bonded to the plasterboard prior to fixing and this could be placed upon battens to form a low emissivity service void which reduces thermal bridging.

To conclude the parametric study based on the wall detail shown in Figure 5.1 (incorporating a fibre cavity barrier as well as a low emissivity cavity) the relationship between the U-value of the wall detail for a range of thermal laminate thicknesses and λ values was considered. The results of this study are illustrated in Figure 5.6 with the trend lines shown corresponding to each set of data of a logarithmic type:

$$y(t_{il}, t_{el}) = E \cdot \ln(t_{il}, t_{el}) + F \qquad \text{Equation 5.3}$$

Where:

t_{il} or t_{el} is the thickness of the thermal laminate in mm

E &F are as defined in Table 5.4 for each of the given trend lines presented.

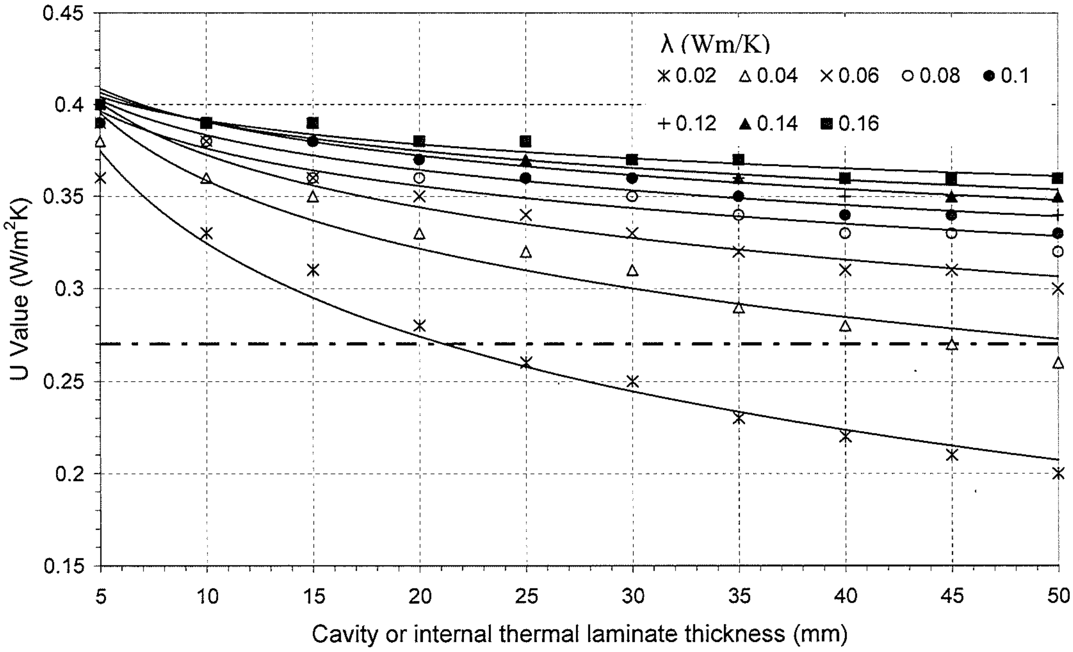


Figure 5.6 Relationship of wall detail U-value with cavity or internal thermal laminate thickness and λ value (broken line represents target U-value)

Table 5.4 Values of constants corresponding to Equation 5.3 and Figure 5.6

λ	Constant	
W/mK	*E*	*F*
0.02	-0.07	0.48
0.04	-0.04	0.46
0.06	-0.03	0.46
0.08	-0.02	0.44
0.10	-0.02	0.45
0.12	-0.02	0.45
0.14	-0.02	0.44
0.16	-0.02	0.44

It is shown in Figure 5.6 that to achieve a U-value of $0.27 W/m^2K$ whilst maintaining a 38×89mm stud the use of a 20mm thick thermal laminate of low thermal conductivity (0.02W/mK) would be an option. Figure 5.7 shows the cost relative to thermal conductivity for a range of readily available thermal laminates and from this information it is considered that the use of a high performing polyurethane or closed cell phenolic foam are options with polyurethane being more cost effective.

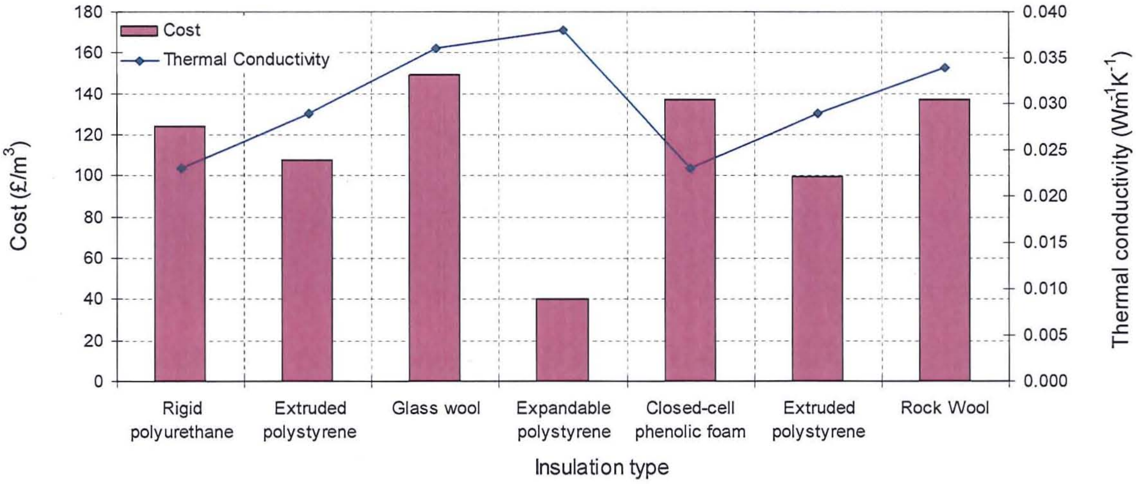


Figure 5.7 Material cost (2007 figures) and thermal conductivity of thermal laminates

The information presented in Tables 5.1, 5.3 & 5.4 can be used in combination with Equations 5.1 to 5.3 respectively to determine the U-value of a standard wall detail, as shown in Figure 5.1 (incorporating a fibre cavity barrier as well as a low emissivity cavity), for a range of sheathing materials, internal insulation (between the studs) and thermal laminates of varying thermal conductivity, λ, and thickness. To derive an all encompassing equation the λ values are plotted against the equation constants (A, B, C, D, E & F) for each case as illustrated in Figure 5.8 the trend lines of which are for each case of logarithmic type (Equation 5.3) with the exception of constant C (External sheathing material) which is linear (Equation 5.4):

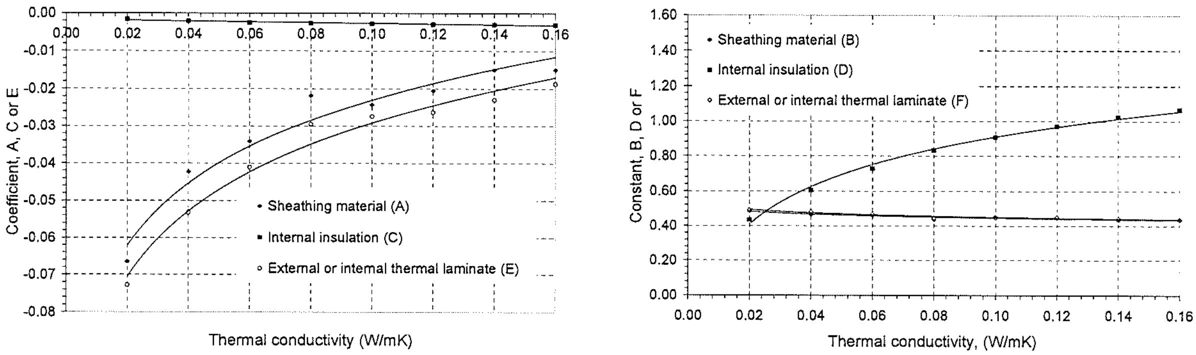
$$y(A,B,D,E,F) = P \cdot \ln(A,B,D,E,F) + Q \qquad \text{Equation 5.4}$$

$$y(C) = P \cdot C + Q \qquad \text{Equation 5.5}$$

Where:

A *to* F are as defined in Tables 5.1, 5.3 & 5.4.

P & Q are as defined in Table 5.5.



a) Constants A, C & E against thermal conductivity, λ

b) Constants B, D & F against thermal conductivity, λ

Figure 5.8 Equations 5.1 to 5.3 constants against thermal conductivity

Table 5.5 Values of constants corresponding to Equation 5.4 & 5.5 and Figure 5.8

	Constant	
	P	*Q*
A	0.0243	0.0330
B	-0.0231	0.3923
C	-0.0093	-0.0017
D	0.3106	1.6241
E	0.0258	0.0302
F	-0.0277	0.3859

Combining Equations 5.1 to 5.3 with Equations 5.4 & 5.5 provides the following equations to determine the U-value relative to variations in:

Sheathing thickness and λ value:

$$y_{sh}(\lambda_{sh}, t_{sh}) = [P_A \cdot \ln(\lambda_{sh}) + Q_A] \cdot \ln(t_{sh}) + [P_B \cdot \ln(\lambda_{sh}) + Q_B] \quad \text{Equation 5.6}$$

Internal insulation (between studs) thickness and λ value:

$$y_{ii}(\lambda_{ii}, t_{ii}) = [P_C \cdot \lambda_{ii} + Q_C] \cdot \ln(t_{ii}) + [P_D \cdot \ln(\lambda_{ii}) + Q_D] \quad \text{Equation 5.7}$$

External (cavity) thermal laminate thickness and λ value:

$$y_{el}(\lambda_{el}, t_{el}) = [P_E \cdot \ln(\lambda_{el}) + Q_E] \cdot \ln(t_{el}) + [P_F \cdot \ln(\lambda_{el}) + Q_F] \quad \text{Equation 5.8}$$

Internal thermal laminate thickness and λ value:

$$y_{il}(\lambda_{il}, t_{il}) = [P_E \cdot \ln(\lambda_{il}) + Q_E] \cdot \ln(t_{il}) + [P_F \cdot \ln(\lambda_{il}) + Q_F] \qquad \text{Equation 5.9}$$

Where:

λ_{el} is the thermal conductivity of the external (cavity) thermal laminate in W/mK ($0.02 \le \lambda_{el} \le 0.16$)
λ_{sh} is the thermal conductivity of the sheathing material in W/mK ($0.06 \le \lambda_{sh} \le 0.16$)
λ_{il} is the thermal conductivity of the internal thermal laminate in W/mK ($0.02 \le \lambda_{il} \le 0.16$)
λ_{ii} is the thermal conductivity of the internal insulation (between studs) in W/mK ($0.02 \le \lambda_{ii} \le 0.06$)
t_{el} is the thickness of the external (cavity) thermal laminate in mm ($5 \le t_{el} \le 40$)
t_{sh} is the thickness of the sheathing material in mm ($5 \le t_{sh} \le 30$)
t_{il} is the thickness of the internal thermal laminate in mm ($5 \le t_{il} \le 40$)
t_{ii} is the thickness of the internal insulation (between studs) in mm ($80 \le t_{ii} \le 190$)

To estimate the U-value of a timber frame wall detail as shown in Figure 5.1 (incorporating a fibre cavity barrier and a low emissivity cavity) the following equation has been derived combining Equations 5.6 to 5.9 and applying a degree of interpolation:

$$U = 4 \times 10^{-4}\{[25.8Ln(K) + 30.2]Ln(\textstyle\sum t_i) - 27.7Ln(K) - (9.3\lambda_{ii} + 1.7)t_{ii} + (310.6Ln(\lambda_{ii}) + 2010)\}$$

Equation 5.10

Where:

$$K = \frac{t_{el}}{\sum t_i}\lambda_{el} + \frac{t_{sh}}{\sum t_i}\lambda_{sh} + \frac{t_{il}}{\sum t_i}\lambda_{il}$$

$$\sum t_i = t_{el} + t_{sh} + t_{il}$$

Using the derived equation and with due consideration to the findings of the research conducted various wall make-ups were considered of which the nine contained in Figure 5.9 & Figure 5.10 were taken forward and checked using SAP software for U-value compliance. Shown in Figure 5.11 is the correlation between the SAP software results and those determined from applying the semi-empirical model and it is demonstrated that a relatively high degree of correlation is achieved. The semi-empirical model therefore offers a method of estimating the U-value of a wall detail prior to ascertaining full compliance using the software.

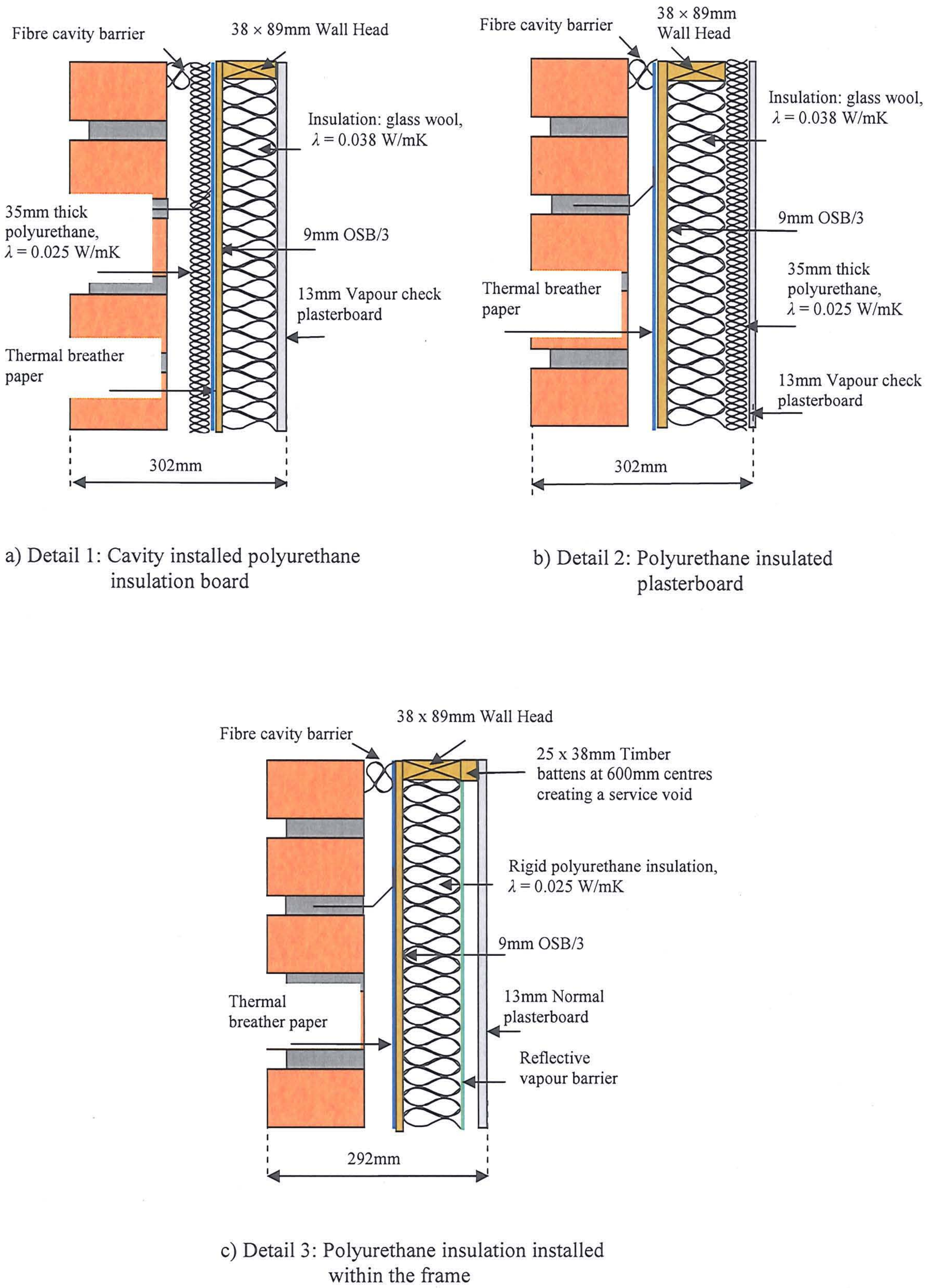
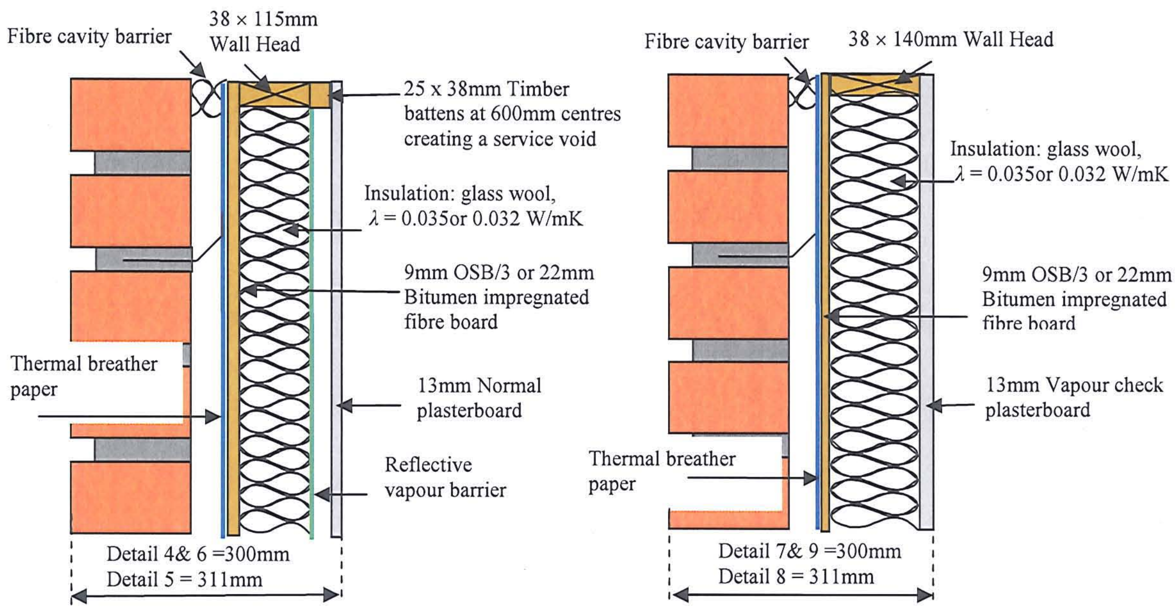


Figure 5.9 Timber frame wall options maintaining a 38×89mm stud



a) Detail 4, 5 &6: 115 stud with glass wool
Note:
Detail 4: 9mm OSB; Glass wool $\lambda = 0.035$ W/mK
Detail 5: 22mm Bitumen impregnated fibre board; Glass wool $\lambda = 0.035$ W/mK
Detail 6: 9mm OSB; Glass wool $\lambda = 0.032$ W/mK

b) Detail 7, 8 &9: 140 stud with glass wool
Note:
Detail 7: 9mm OSB; Glass wool $\lambda = 0.035$ W/mK
Detail 8: 22mm Bitumen impregnated fibre board; Glass wool $\lambda = 0.035$ W/mK
Detail 9: 9mm OSB; Glass wool $\lambda = 0.032$ W/mK

Figure 5.10 Timber frame wall options with 115 and 140 thick studs

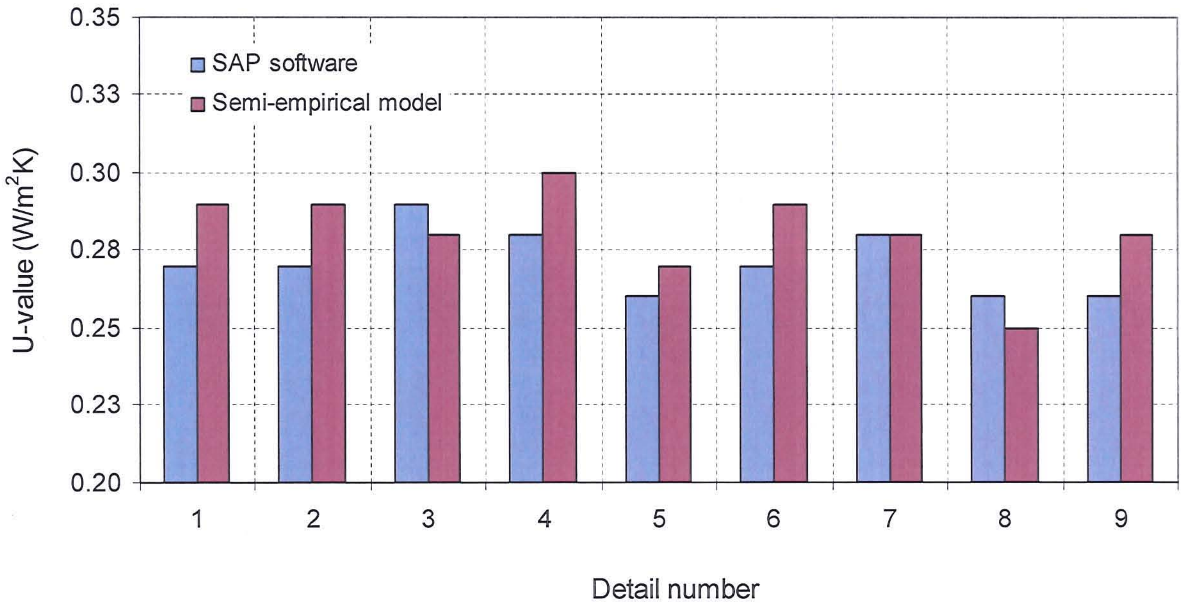


Figure 5.11 Correlation between SAP software out-put and empirical model

In addition to the U-value calculations the wall details were rated in relation to sustainability using the insulation LCA ratings of Table 5.3 as a result of the other materials being relatively consistent. Further to this a full material cost based on 2007 figures was also calculated and this information is contained in Table 5.6 (for a full break-down of material cost please refer to Appendix D). To compare the wall details in relation to both cost and U-value as determined using the SAP software Figure 5.12 was produced.

Table 5.6 Wall detail ratings

Detail designation	Life Cycle Assessment	Material Cost	U-Value W/m^2K	
		£/m run	SAP software determined	Equation 5.9 determined
Standard (Figure 5.10)	Medium	17.24	0.40	N/A
1	Medium	37.85	0.27	0.29
2	Medium	39.89	0.27	0.29
3	Medium	45.21	0.29	0.28
4	Low	29.31	0.28	0.30
5	Low	32.41	0.26	0.27
6	Low	33.15	0.27	0.29
7	Low	32.77	0.28	0.28
8	Low	35.87	0.26	0.25
9	Low	36.57	0.26	0.28

Note:
For studs a timber fraction of 0.15(15%) has been used in accordance with the guidelines of BR 443 (Anderson, 2006).
Material costs are based on 2007 figures.

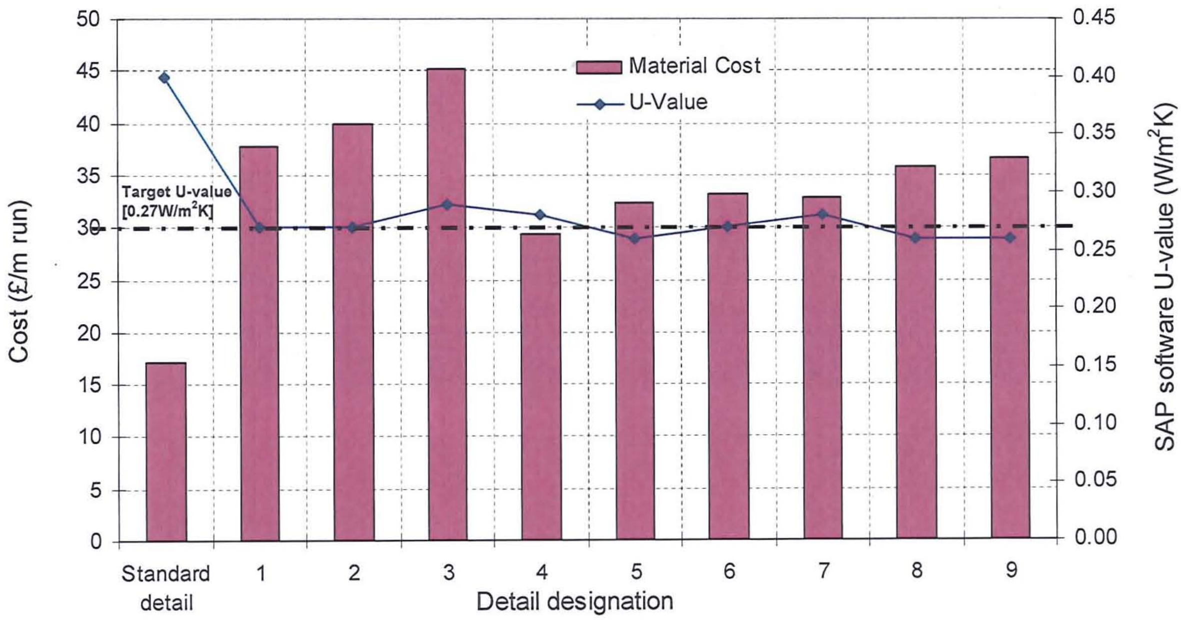


Figure 5.12 Comparison of material cost relative to wall detail U-value

The implications of each detail have not been measured in terms of impact to on-site erection. However, it can be predicted without true measurement that Details 1, 3, 4, 5 & 6 will take longer to construct as a result of additional work. Detail 1 requires the installation of a thermal laminate in the cavity and Details 3 to 6 require the creation of a service void. Considering Detail 2 the thermal laminate is bonded to the plasterboard which would be fitted as normal resulting in no extra work.

Of the options considered the target U-value of 0.27W/m^{2}K is met, whilst maintaining a 38×89mm stud by Details 1 & 2. Maintaining a 38×89mm stud is advantageous as it would allow existing frame designs for house types to be transferred and the section size is readily available. However, in terms of both the cost and LCA rating Details 1 to 3 are at a disadvantage to Details 4 to 9 which use either a 38×115mm or a 38×140mm stud. Details 4 & 7 have a U-value of 0.28W/m^{2}K which is greater than the target value of 0.27W/m^{2}K. However, the use of a deeper section would reduce the requirement for cripple studs (studs supporting lintels etc) (Figure 5.13) which corresponds to a reduction in thermal bridging and would therefore improve the thermal efficiency of the wall assisting full envelope compliance. A reduction in cold bridging as a result of a reduced level of cripple studs would depend on the frame design and the level of reduction, if any, would be related to the nature of the system and can therefore not be relied upon for all cases. Considering Details 4 & 7 only a small reduction (0.01W/m^{2}K) in U-value rating is required, to achieve this a further alternative which was considered was the use of a 22mm thick bitumen impregnated fibre board for sheathing. Bitumen impregnated fibre board offers an improved level of thermal resistance, λ = 0.05W/mK, and its use improves the U-value rating of Details 5 & 8 to 0.26 W/m^{2}K, which is below the target value, and corresponds to a marginal cost increase of £0.78 per m run.

The use of an increased stud section would require the re-design of existing house types but in terms of material cost are at an advantage to Details 1, 2 & 3. Comparing Detail 4 with Detail 7 and Detail 5 with Detail 8 increasing the stud section to 140mm results in an additional cost of £3.46 per m run. However, the deeper section of 140mm would correspond to a further reduction in framing material and therefore reduced thermal bridging. Details 4 & 7 would require additional work on-site due to the introduction of a service void which would increase erection time and cost.

Although currently the highest performing, readily available glass wool product which can be used in a wall application between studs over 90mm deep, has a thermal conductivity, λ = 0.035 W/mK, Details 6 & 9 consider a glass wool with λ = 0.032 W/mK and a degree of interpolation has been carried out to determine the cost of the insulation. The corresponding U-value of Details 6 & 9 is 0.27 & 0.26 W/m^{2}K respectively. However, both options are shown in Figure 5.12 not to be cost effective.

Figure 5.13 Examples of cripple studs supporting lintels

5.2.3 Conclusions

As a result of the EU Directive on Energy Performance of Buildings and the corresponding revisions to the Building Regulations being implemented, the energy efficiency of dwellings are to be rated applying the Standard Assessment Procedure (SAP) which requires the use of Building Research Establishment (BRE) approved computer software. For the energy efficiency of timber platform frame systems to comply with the revised regulations the required U-value of walls will have to be reduced to between 0.27 and 0.30W/m^{2}K with the target, to ensure overall SAP rating compliance, 0.27W/m^{2}K.

To examine the affect of sheathing, internal insulation and thermal laminate thickness and thermal conductivity on U-value rating a series of parametric studies were conducted. From the parametric studies conducted a semi-empirical model was developed which, with a relatively high degree of accuracy, provides a simplified method of estimating the U-value of masonry clad timber frame walls. The developed model was used to determine a range of wall detail solutions which were then checked for full compliance using BRE accredited software. Of the solutions considered it has been concluded that the most cost effective method of attaining the reduced U-value requirement, whilst having a low Life Cycle Assessment (LCA), is to increase the stud size to either 115mm (introducing a low emmisivity service zone) or 140mm and in both cases incorporating a glass wool with lower thermal conductivity (λ = 0.035 W/mK). However, these wall details only attain a U-value of 0.28 W/m^{2}K which would require other aspects of the whole building to be considered to achieve envelope compliance. If the target U-value of 0.27 W/m^{2}K is required to be met then the most cost effective way of enhancing the U-value rating of a 115 or 140mm stud wall to meet the target U-value is to introduce a bitumen impregnated sheathing board rather than using a glass wool of reduced thermal conductivity. The specification of either a 115mm stud wall option or 140mm stud wall option will depend on a balance between:

- Availability of timber section: 38×115mm is less readily available.
- Cost: a 140mm stud wall is £3.46 per m run more expensive.
- On-site issues: a 115mm stud wall requires additional on-site work to create a service void which would correspond to an increase in erection time and cost.

Published work

1. **Hairstans, R., Dodyk, R. and Kermani, A.** (2006) *"Development of the optimum sustainable wall detail"*, 9th World Conference on Timber Engineering, Portland, USA.
2. **Hairstans, R., Dodyk, R. and Kermani, A.** (2007) *"Sustainable timber frame wall diaphragm development"*, accepted for publication in SUE MoT International Conference., Glasgow, UK.

5.3 Structural Insulated Panels used as Wall Diaphragms

5.3.1 General

For 150 years wood studs have gone unchallenged as the dominant structural system in low-rise, wood framed construction, (Cathcart 1998). However, Structural Insulated Panels (SIPs), Figure 5.14, provide a viable alternative. They offer structure, sheathing, insulation and airtightness in a single product.

Compared with standard frame construction, SIPs can be inherently more energy efficient. Indeed the performance of walls formed using SIPs are normally well in excess of what the proposed U-Value base case performance values are to be and in fact can achieve the perceived 2010+ values.

Figure 5.14 Structural Insulated Panels (SIPs) during construction

However, there is limited available information on the structural performance of SIPs. In this section an evaluation of the performance of SIPs with regard to the three major load components which SIPs are predominately subjected to is given:

1. vertical loads (direct compression)
2. transverse wind loads (combined bending and axial compression)
3. in-plane lateral forces imposed by wind and or seismic loading (racking loads).

5.3.2 Background Information

SIPs are structural composites with two outside skins normally of OSB/3 and a bonded internal core of insulation. The most common insulating foam materials are:

1. Expanded polystyrene (EPS)
2. Extruded polystyrene (XPS)

3. Polyurethane.

These are typically oil based and from an environmental perspective can be melted down and reused. The insulations are also very low density and as a result only small masses of the primary resource are needed to produce high levels of insulation. OSB is produced from fast-growing trees and forest thinnings and recycled timber. In conjunction with this if the OSB boards are autohesively bonded to the rigid urethane insulation core during manufacturing the requirement for potentially environmentally harmful adhesives is eliminated.

In terms of the building envelope SIPs out perform traditional timber frame walls. Typical external wall constructions comprising an inner leaf of TEK Haus SIPs, produced by Kingspan Tek Haus Building Systems (BBA, 2002), finished with 12.5 mm plasterboard on timber battens and a brick outer leaf with a 50 mm vented cavity, will achieve an estimated U-value of between 0.19 and 0.22 $Wm^{-2}K^{-1}$, depending on the number and type of panels.

Part of the efficiency improvement is attributed to the insulating properties of the foam. A substantial improvement is also associated with the reduced need for framing members, which can operate as "thermal bridging", (Lee, 1997; Waters, 2003).

SIPs are also typically lightweight, although this is dependent on the outer-skins, which facilitates on-site installation. As a result of the insulation being preinstalled off-site SIPs tend not to suffer from:

- Sagging insulation.
- Wet insulation due to exposure on-site which could reduce thermal performance.
- Gaps and voids in insulation coverage left by poor site workmanship.

There are two main fabrication techniques: (a) an industrial adhesive is applied to a pre-cut foam core and then the core is cold pressed between two pieces of facing (panel boards) until the adhesive is cured; and (b) the foam is poured into pre-spaced facings and the foam cures to bond to the facings (Lee, 1997). Either method produces a single solid building element that provides both structural and insulation qualities. These panels can be produced in varying sizes and thicknesses depending on application and thermal/structural requirements.

SIPs generally cost 2 to 10 percent more than an insulated and sheathed wood frame, but provide 20 to 50 percent more insulation, (Cathcart, 1998). With wood framing, additional insulation requires deeper lumber dimensions or double framing, SIP insulation, by contrast, gets less expensive per unit volume as the panels get thicker, since the skins and manufacturing and installation process remain the same.

With regard to fire, manufacturers across North America have proven the performance of SIP systems through some of the most extensive fire assembly testing in the construction industry. The results of this destructive testing allow documentation of SIP performance under rigorous test standards. American national standards like ASTM-E119 and ASTM-E84 have been met by protecting SIPs in a similar fashion to other wood-based structures. For example residential structures are typically required to meet a 15-minute standard and they can meet that by fitting 12.5mm common gypsum over SIPs (Tracy, 2000). When considering SIP construction in residential dwellings for the UK the internal linings of the structure will require a class 0 (non-combustible) or class 1 (semi-combustible) lining depending on the size and occupancy of the building relative to the required fire protection. This can be achieved by applying 1 layer of 12.5mm gypsum plasterboard to obtain class 1 and 2 layers to obtain a class 0 fire rating.

SIPs utilise a stressed-skin principle where the overall strength of the panel is much greater than the strength of the components hence the reduced need for structural framing. For a SIP to function robustly there must be no slip between the outer skins and the core material. To achieve this adhesive technology is used. The adhesive used must be capable of transferring shear and tensile forces across the interface and not deteriorate over time or under the effect of moisture (Milner, 2003). A series of tests to evaluate the strength of a glue bonded polystyrene insulating core to OSB manufactured under normal conditions showed that when subjected to tensile loading (perpendicular to the plane of a panel) and also skewed/eccentric loading (in-plane shear) all failures occurred in the polystyrene and the glue-lines remained intact demonstrating that suitably robust bonding techniques are available (Kermani, 2005).

With regard to durability no long term test programmes are recorded. However, there are examples of SIP buildings in the USA that have been in service for 50 years. It is also reasonable to expect that as a result of the component parts that make up a SIPs product that a quality-manufactured panel itself should not deteriorate or degrade unless it is incorrectly built, exposed to ultra violet light, rodents or insects (Milner, 2003).

Although SIPs have been used extensively as an alternative structural system to conventional framing for residential and light commercial buildings, to date little independent data is available on their structural performance and behaviour. The American Plywood Association Supplement No.4 (APA, 1983) is the only standard dealing with sandwich panels and provides some limited design information on the uniform transverse or the combined loading cases.

5.3.3 Resistance of SIPs to Vertical and Transverse Loads

Studies at Napier University have evaluated the resistance of SIPs to direct compression and transverse wind loads (Kermani, 2005). From the direct compression loading tests it was concluded that when constructing panels using method (a), previously described, improvements in strength are gained when the polystyrene core blocks are suitably bonded at any joints. It was found, from the tests carried out, that failures were initiated at unglued joints in instances where the joint was situated at the mid-height horizontal plane; indeed a 20% reduction in strength was noted. The information from the direct compression tests was used to produce a chart to assist design (Figure 5.15).

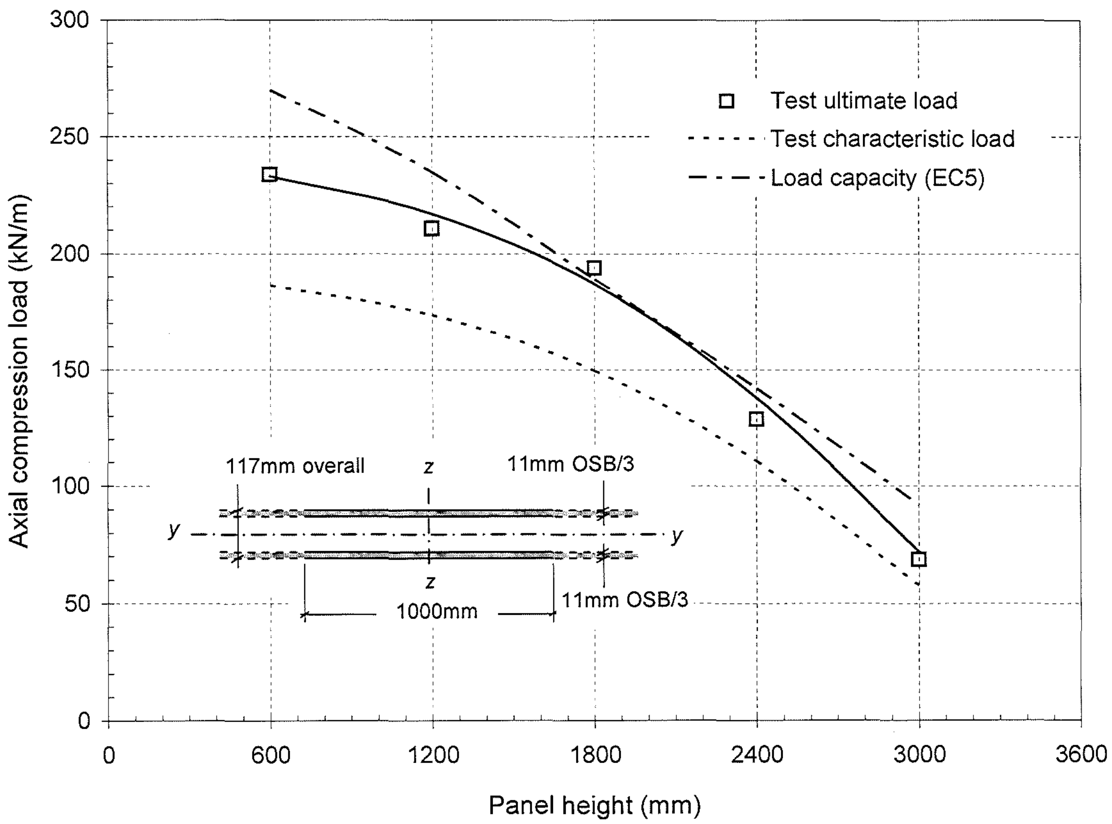


Figure 5.15 Chart for estimating direct compression capacity (Kermani, 2005)

The transverse wind loading resistance of SIPs panels was evaluated from combined bending and axial compression tests. The panels tested were 2.4m high with an overall thickness of 117mm using 11mm thick grade 3 OSB side boards. From the test information a further chart to assist design was produced (Figure 5.16).

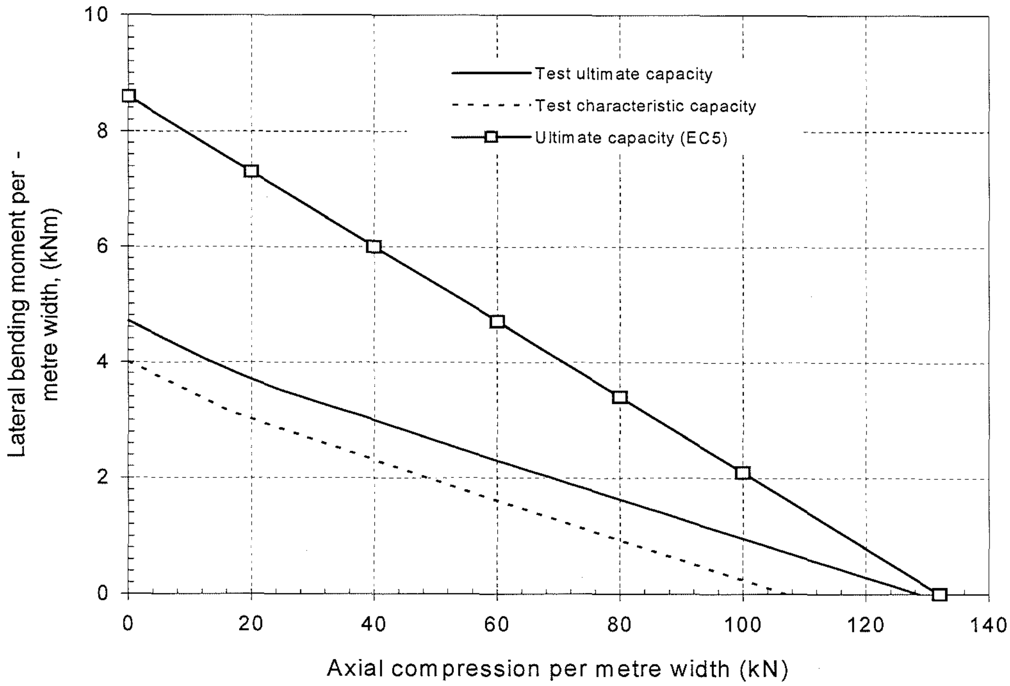


Figure 5.16 Comparison of combined bending and axial compression capacity of SIP wall panels of 2.4 m high with 117 mm overall thickness and 11mm thick OSB facings (Kermani, 2005)

On both occasions the test performance of the panels were also compared to load capacities based on EC5. It was concluded from these comparisons that calculation of the load capacities to EC5, with the assumption of full composite action and shear transfer between the elements of SIPs, particularly under loading combinations where bending is dominant, leads to an overestimation of their strength capacities, as illustrated in Figure 5.15 and Figure 5.16. It is therefore important that, in the absence of a detailed analysis (e.g. a non-linear finite element method), correlated/adjusted test results are used for determination of the design properties.

5.3.4 Racking Strength of SIP Walls

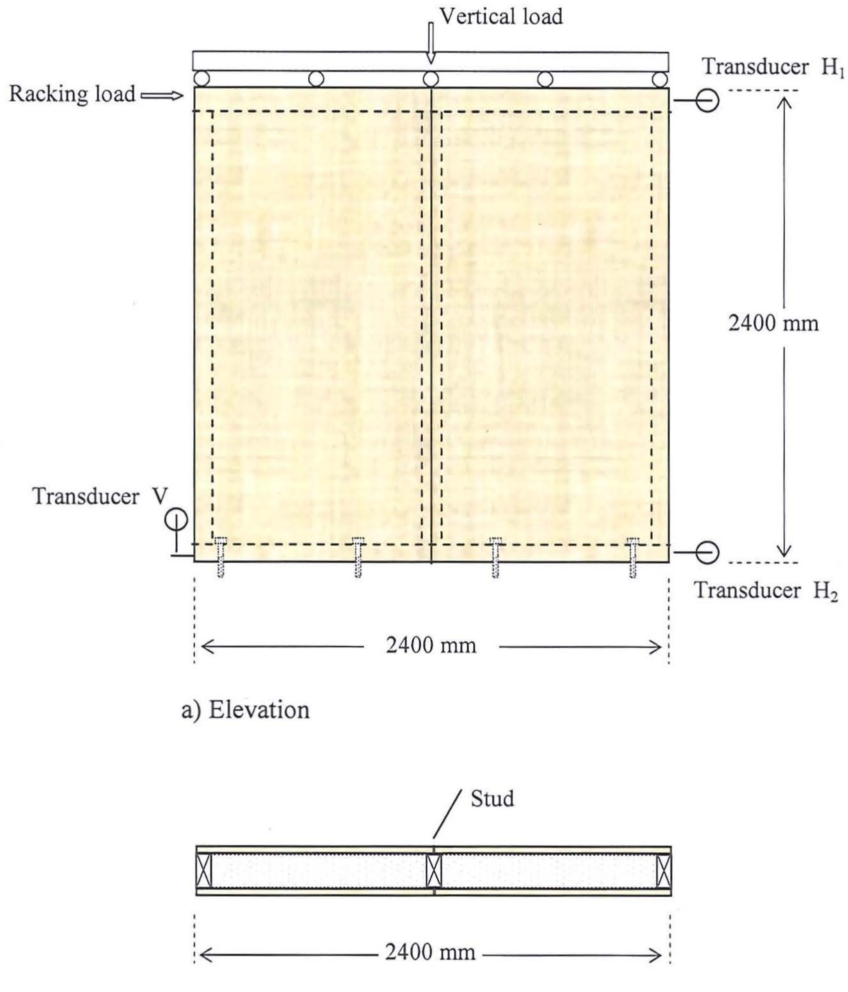
As part of this PhD research programme, the investigation by Kermani (2005) was extended to determine the racking performance of SIPs and the effects of position and size of the openings for doors and windows on the structural performance of SIPs.

Racking load tests were carried-out on SIP walls in accordance with BS EN 594:1996 and BS 5268: Section 6.1: 1996. The panels were of the same make-up as in the tests reported by Kermani (2005) with an overall thickness of 117mm consisting of a 95mm insulating core and 11mm grade 3 OSB side boards. The wall configuration tested combined two panels of 2400mm high × 1200mm long, the header, footer and end studs of 47 × 95mm C16 timber sections. The two panels were joined at the middle by lapping 23.5mm of the OSB side boards over an intermediate wall stud and the connection was made by 2.65mm diameter screws, 35mm long at 250mm centres. The connection strength at the intermediate stud exceeded the recommendations of EC5 and was therefore considered appropriate. The footer was bolted to the test floor using 9mm diameter holding down bolts at approximately 600mm centres and the wall was then connected to the footer using 2.65mm diameter screws, 35mm long at 200mm centres. Details of a typical SIP wall configuration during testing are shown schematically in Figure 5.17.

Results

Nineteen walls of 2400mm × 2400mm were constructed and tested to evaluate the racking resistance of the SIP walls for a series of applied vertical loading conditions along the header and also to determine the effects of size and position of opening (for windows and doors) on the racking strength and stiffness of SIP walls.

Seven solid SIP walls were tested for horizontal racking resistance under vertical applied loads of 0, 12.5 and 25kN. The horizontal (racking) loads were applied via a compression jacking unit, operated by a hand pump and measured by a load cell. In accordance with BS EN 594:1996 the racking load was applied at a constant rate of movement related to the displacement transducer H_1 (Figure 5.17).



b) Cross-section

Figure 5.17 Wall panel and loading details.

The overall horizontal displacement of the panel was monitored by transducers, H_1, H_2 and in accordance with the code reported as the difference between the two. The vertical movement of the panel was monitored by transducer V_1 and reported separately. From the measured horizontal displacement of the panel and corresponding racking load applied the racking stiffness of the panel was calculated in accordance with BS EN 594:1996. The maximum racking load the panel can support was also measured and this load corresponds to prescribed failure criteria:

1. When the panel collapses.
2. When the reported displacement of the panel reaches 100mm.

In accordance with BS 5268:1996 Section 6.1 both the racking stiffness and the maximum racking load are used to determine the basic test racking resistance (as load/m of wall length) separately and the lower of the two values is taken. The results of the tests are detailed in Table 5.7.

Table 5.7 Racking strength of solid walls (with no openings)

Wall reference	Wall details	Vertical load constant	Test ultimate load, F_{max}	Test racking strength load, F_{ult}	Failure mode	Test racking design load to BS 5268, R_d	Governing Criteria to BS 5268	Characteristic test racking resistance to BS 5268, R_k
		kN	kN	kN		kN/m		kN/m
Wall 1	As shown in Figure 5.17	25.0	26.3	---	OSB panels were disjointed from the soleplate	6.37	Strength	---
Wall 2	ditto	25.0	25.8	---	ditto	5.18	Stiffness	---
Wall 3	ditto	25.0	27.8	---	ditto	6.72	Strength	---
Summary		25.0	Min. value = 25.8	23.99		6.25		10.00
Wall 4	ditto	12.5	22.2	17.76	ditto	3.75	Strength	7.40
Wall 5	ditto	0.0	11.5	---	ditto	2.79	Strength	---
Wall 6	ditto	0.0	12.5	---	ditto	3.03	Strength	---
Wall 7	ditto	0.0	12.8	---	ditto	3.1	Strength	---
Summary		0.0	Min. value = 11.5	10.7		2.79		4.46

5.3.5 Comparison of SIP wall racking performance with traditional timber frame stud wall

The racking resistance of the SIP walls from the tests were compared to the design racking values of a comparable wood stud shear wall with 47 × 95mm C16 studs at 600mm centres sheathed on both sides with 11mm thick grade 3 OSB fixed to the internal and external framing elements by 2.65mm diameter screws, 35mm long at 250mm centres. The design calculations were carried out in accordance with BS 5268:1996 for basic racking resistance and also EC5 (section 9.2.4.3 Simplified analysis of wall diaphragms - Method B adopted in UK) for characteristic strength, and the results are shown in Figure 5.18 and Figure 5.19 respectively. In the case of designing to both BS 5268:1996 (Figure 5.18) and EC5 (Figure 5.19) a SIP wall out-performs the comparable stud wall diaphragm and in both cases the general trend of results is the same with increased racking resistance of the SIP wall proportional to increased vertical applied loads.

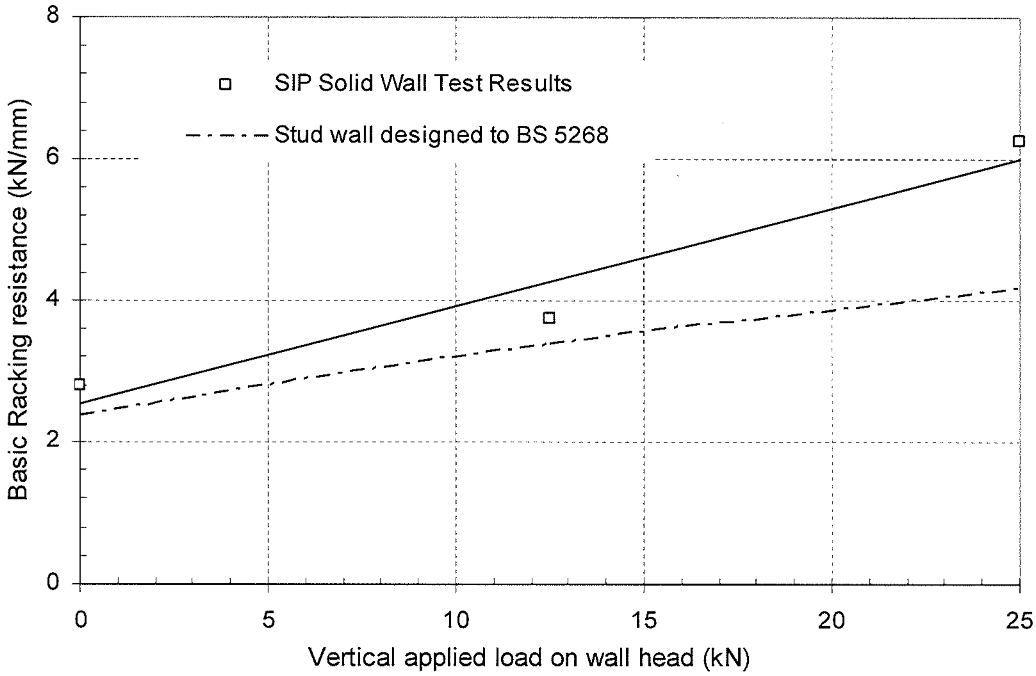


Figure 5.18 Racking design load of tested SIPs and of a stud wall of comparative framing material designed to BS 5268:1996 Section 6.1

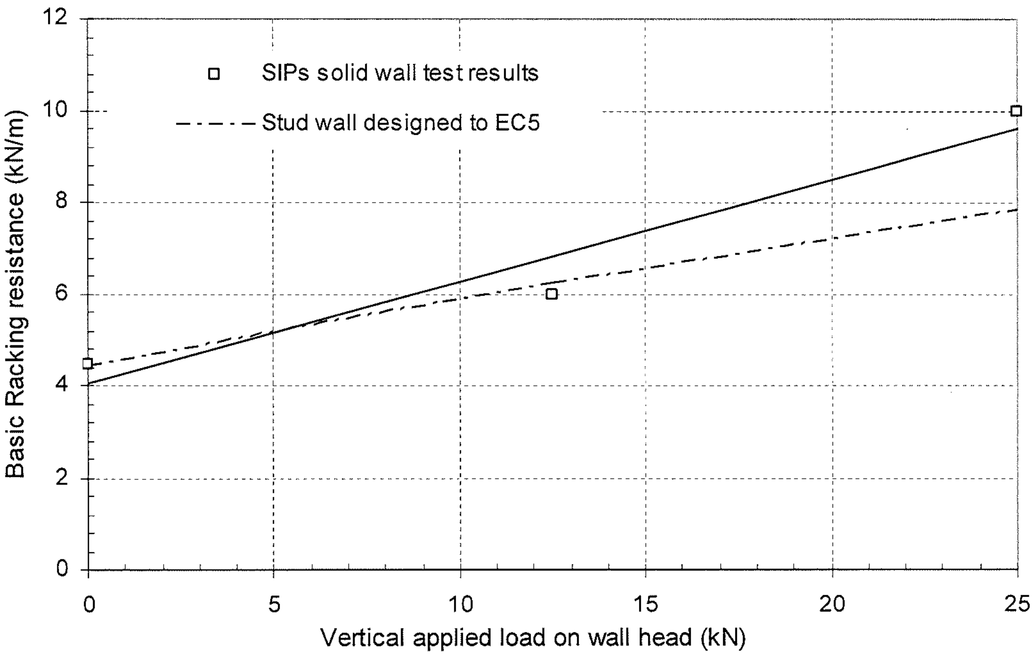


Figure 5.19 Characteristic racking strength of tested SIPs and of a stud wall of comparative framing material designed to EC5 Method B

Effects of openings

The effects of openings for windows and doors on the racking strength and stiffness of SIP walls were examined by testing a further 12 walls. The size of the openings were determined by standard window and door sizes, Table 5.8, and placed in a range of positions as shown in Figure 5.20. For each opening type and size two replicate walls were tested, the first under 0kN and the second under 25kN constant vertical load. The test set-up and measurement methods were the same as previously described for walls with no openings and the results of the tests are detailed in Table 5.9.

Table 5.8 Details of the openings for windows and doors

Test details	Replicate	Opening for window
25kN load: (4 tests) 0kN load: (4 tests)	2 of: 2 of: 2 of 2 of:	$a \times b$ = 600×600 $a \times b$ = 900×600 $a \times b$ = 1800×600 $a \times b$ = 1800×1800
25kN load: (2 tests) 0kN load: (2 tests)	2 of: 2 of:	$c \times d$ = 900×2100 * $c \times d$ = 1800×2100 *
Note: *See Figure 5.20 for details.		

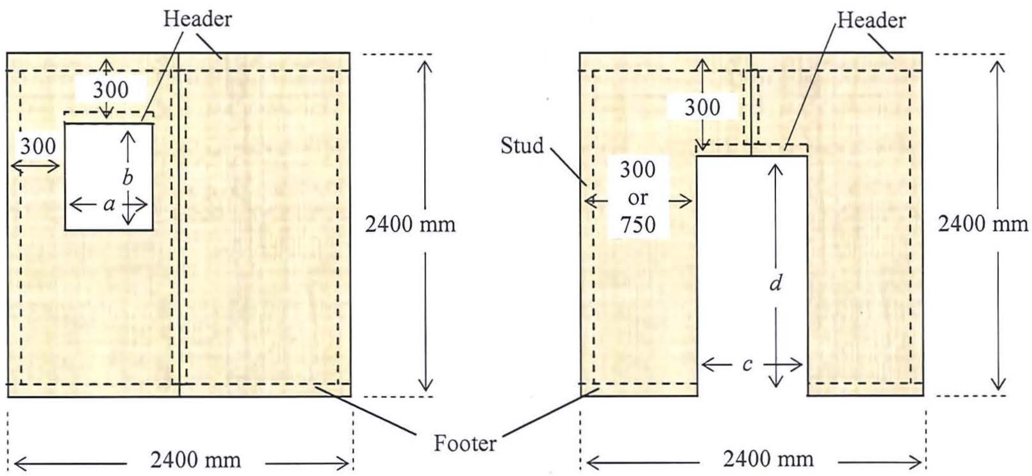


Figure 5.20 Wall panel opening details

Again the test racking resistance of the SIP walls were compared to design calculations, in accordance with BS 5268:1996 and EC5 Method B, of a comparable wood stud shear wall, constructed of the same framing material and fastener specification as before, allowing for the area of opening. The effects of openings for the test and calculated results to BS 5268:1996 are compared in Figure 5.21. In this figure the basic test racking strength values are normalised in accordance with BS EN 594:1996 to provide a single trend line which demonstrates the variation in racking resistance to percentage openings for zero applied vertical loading.

Figure 5.21 illustrates a correlation in results between basic test racking capacity and those calculated in accordance with BS 5268:1996 for a stud wall of comparative make-up. It is demonstrated that as the percentage of area of opening is increased the resistance of the SIP wall to racking is reduced and that the general trend corresponds with that of the stud wall of comparative framing material. This reduction in racking strength and stiffness with respect to the level of opening has also been reported in previous studies on long stud shear walls of 2.4 ×12m (Johnson and Dolan 1996) and shear walls constructed with oversized OSB panels (4.8 x 4.8m) (Enjily and Griffiths 1996).

Research on stud walls by Patton-Mallory et al. (1985) and Enjily and Griffiths (1996), has also highlighted that as the area of the openings is increased, the governing design criterion may more likely be serviceability (stiffness) rather than ultimate load (strength). The findings from the study on SIPs demonstrated a similar effect occurred (Table 5.9), with all failures being as a result of stiffness with the exception of Wall 16 and Wall 17, the walls with the smallest percentage of opening (both 6%).

Table 5.9 Racking strength of walls with openings

Wall reference	Wall details	Opening %	Vertical Load, constant,	Test ultimate load, F_{max},	Test racking strength load, F_{ult},	Basic test racking resistance to BS 5268, R_b,	Characteristic test racking resistance to BS 5268, R_k,	Failure Mode & Governing Criteria to BS 5268
			kN	kN	(kN	kN/m	kN/m	
Opening for doors (see Figure 5.20):								
Wall 8	Opening: 1800×2100	65%	25.0	6.3	5.04	0.30	2.1	Stiffness Panel tore at the top corners of the opening
Wall 9	ditto	ditto	0.0	3.85	3.08	0.28	1.6	ditto ditto
Wall 10	Opening: 900×2100	33%	25.0	15.78	12.62	1.36	5.26	ditto ditto
Wall 11	ditto	ditto	0.0	8.9	7.12	1.34	2.97	Stiffness Above + disjointing from the soleplate
Opening for windows (see Figure 5.20):								
Wall 12	Opening: 1800×600	19%	25.0	18.11	14.49	2.05	6.04	Stiffness ditto
Wall 13	ditto	ditto	0.0	11.9	9.52	2.48	3.97	Strength ditto
Wall 14	Opening: 900×600	9%	25.0	31.92	25.30	3.75	10.54	Stiffness Panels were disjointed from the soleplate
Wall 15	ditto	ditto	0.0	15.24	12.19	2.82	5.08	Stiffness ditto
Wall 16	Opening: 600×600	6%	25.0	14.24	11.39	1.67	4.75	Strength Panel tore at the top corners of the opening
Wall 17	ditto	ditto	0.0	15.49	12.39	3.12	5.16	Strength ditto
Wall 18	Opening: 1800×1800	56%	25.0	8.19	6.55	0.5	2.73	Stiffness Panel tore at the top corners of the opening
Wall 19	ditto	ditto	0.0	7.45	5.96	0.67	2.48	ditto ditto

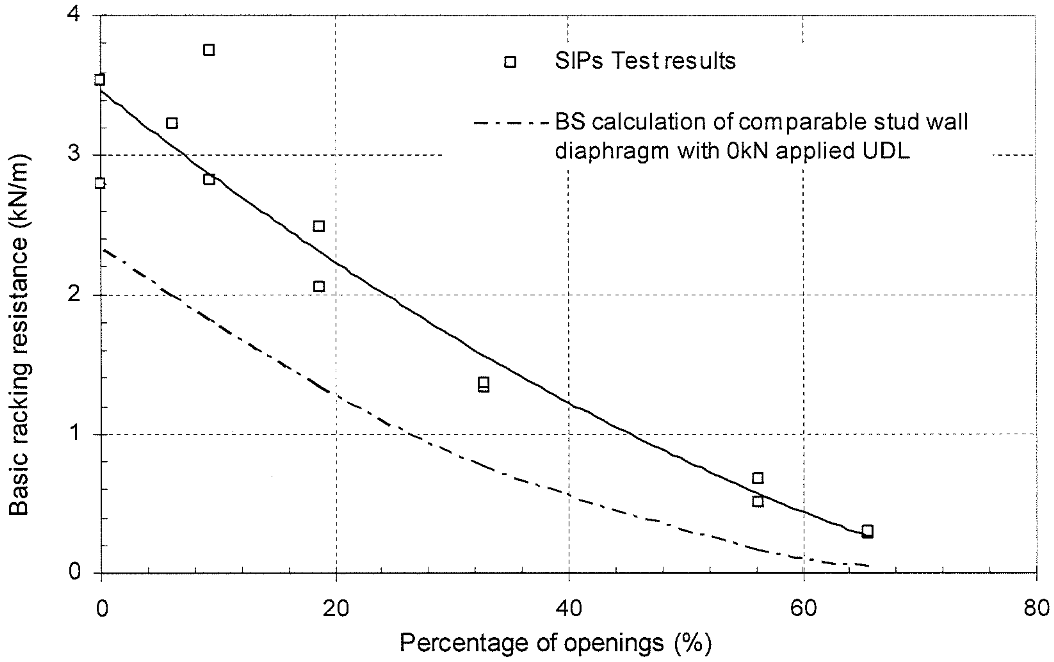


Figure 5.21 Effects of opening size on racking strength of walls: Normalised basic test racking strength of SIPs compared with similar stud wall designed to BS 5268:1996

In Figure 5.22 a comparison is made between the characteristic test racking strength and the characteristic strength of stud walls calculated in accordance with EC5 method B under a constant vertical load of 25kN with openings of the same size and in the same position. The variations in design values (to EC5) shown in Figure 5.22 are as a result of the following rules set by EC5 to account for the effects of the size and position of an opening:

1. For a panel to contribute to the in-plane (racking) strength of a wall the width of the panel should be at least the panel height divided by 4.
2. Where an opening is formed in a panel, the length of panel each side of the opening should be considered as separate panels.

Consider two cases which result in fluctuations in the calculated values to EC5.

An opening 1800mm long and 600mm high positioned centrally in a 2400mm long wall results in a 19% opening level. However, the racking resistance of the wall, as a result of the two conditions stated above, is zero.

Whereas in the case with 33% opening, where the opening is 900mm long and 2100mm high, again positioned centrally, the combination of the two conditions above result in the panel having a relatively high degree of racking resistance. Therefore, although the percentage of panel available for racking in this case is less the racking resistance is higher due to the orientation and positioning of the opening.

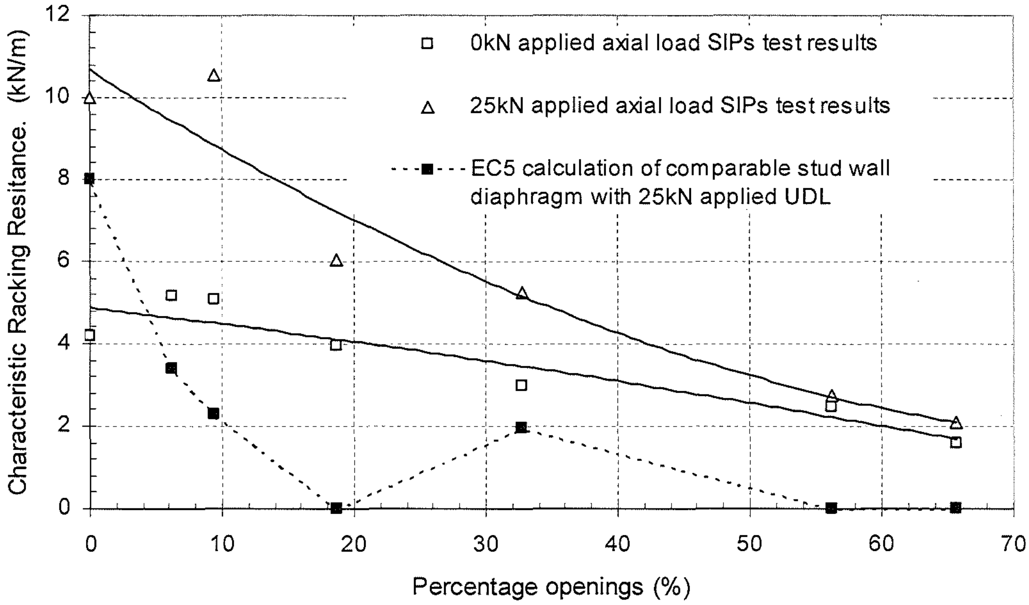


Figure 5.22 Characteristic racking strength test results of SIPs compared with comparable stud wall designed to EC5

To explore the above points further, a stud wall of comparative framing material was designed in accordance with EC5 method B with an opening of 900mm wide at accumulative 300mm distances along the panel, starting at a position on the left hand edge of the wall. Figure 5.23 shows the results of the parametric study (for a case with no vertical loading) illustrating the points made above and demonstrating that based on EC5 Method B, the position of the opening can significantly affect the racking strength of the panel. The figure also highlights that a nominal change in the position of opening can increase or decrease the design racking strength by some 50%.

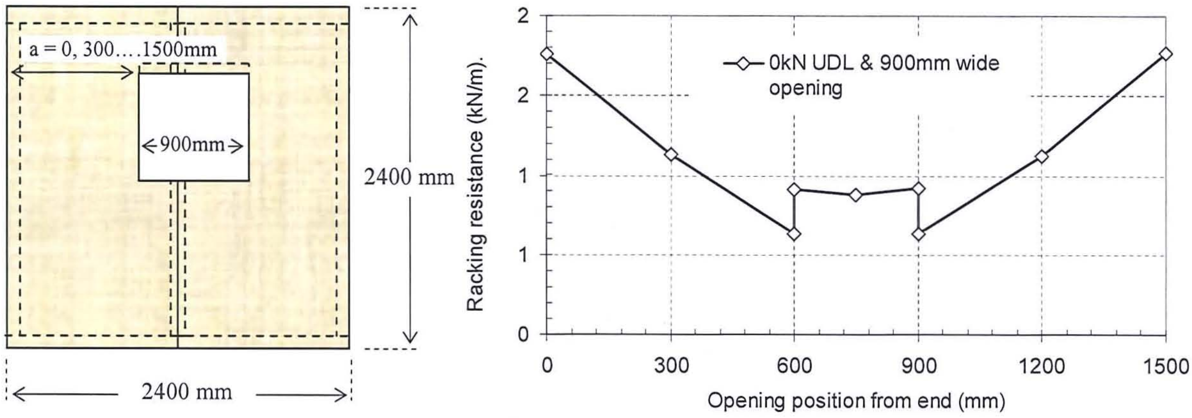


Figure 5.23 Racking resistance of panel with varying opening position designed to EC5 Method B

5.3.6 Conclusion

SIPs are a sustainable and cost efficient alternative to traditional stud wall diaphragms for domestic dwelling construction if the whole life cycle cost of the house is considered. SIPs have improved insulation qualities due to a reduction in cold bridging, they satisfy all other building regulations and are known to be durable if a stringent manufacturing procedure is implemented.

Walls constructed of SIPs provide a superior racking resistance to a comparable traditional stud wall designed to BS 5268:1996 or EC5 (when taking the effects of openings into account) and, as expected, the racking strength increases with increasing applied vertical loads. The racking strength of SIP walls is also directly related to the size of the openings; with an increase in opening size reducing racking resistance sharply.

The comparative study carried out on the effect of size and position of openings has illustrated that:

- Walls with openings constructed of SIPs are structurally more efficient than stud walls of comparative framing material and fastener spacings designed in accordance with the requirements of BS 5268:1996.
- The design methodology of BS 5268:1996, allowing for the effect of percentage openings, is in line with the behaviour of SIPs with openings.
- The characteristic racking resistance of SIPs without openings can be conservatively estimated using EC5 method B and an equivalent stud wall.
- When openings are formed in a wall, EC5 rules can provide overly conservative design values for racking resistance.

Published work

1. **Kermani, A. and Hairstans, R.** (2006) *"Racking performance of structural insulated panels"*, American Society of Civil Engineers (ASCE) Journal of Structural Engineering. Vol.132, No. 11, pp 1806-1812, ISSN 07339445.

CHAPTER 6

SHOT FIRED DOWEL FLITCH BEAMS

6.1 Introduction

In Chapter 2 the concept of lean manufacturing was defined according to Bergstrom and Stehn (2005) as *"a holistic management philosophy, with product quality as the primary goal, which underlines the critical importance of employees, customers, improvements of the two main conversion processes, design and production, and elimination of all other activities, to achieve customisation of high volume products (Crowley, 1998; London and Kenley, 2001)"*. In this chapter a more efficient method of flitch beam fabrication using a shot fired dowel connection is presented the endorsement of which demonstrates the implementation of a lean manufacturing technique:

- The needs of the employee are shown to be understood (training, equipment maintenance, practicality etc).
- The requirements of the customer were taken into account by giving due consideration to product value.
- The production method was improved by means of optimising the fabrication procedure.
- Quality of product is demonstrated to be assured by means of conducting a laboratory programme which allowed the safe and robust specification of shot fired dowel flitch beams in timber platform frame systems.

The main objective of this part of the research programme was to develop an in-depth understanding of the structural behaviour and performance of shot fired dowel flitch beams. As a result the essential elements of the configuration were identified and a study was conducted on the following parameters:

- Pull-out, pull through and lateral load bearing capacity of the connections.
- Influence of number of nails employed on shot fired dowel flitch beam stiffness and strength.
- Bending strength and stiffness of flitch beams representative of those which would be used in timber platform frame construction fabricated using the shot fired dowel method of connection.

- The effects of shear forces and methods of determining the shear modulus of shot fired dowel flitch beams.
- The behaviour of the elements of a shot fired dowel flitch beam in strain for a range of load span conditions.

The purpose of the laboratory testing and analysis was to provide consultant engineers designing for timber platform frame manufactures with information to allow the safe specification of shot fired dowel flitch beams manufactured from a range of industry standard products.

6.2 General

In timber design there are instances when not only large spans and heavy loads predominate but also the available depth of the section is restricted in some way (Carmichael, 1984). In timber platform frame domestic dwelling construction such cases are common, especially so at ground floor level, examples of which are garage door openings and bay windows (Figure 6.1).

In these instance it as advantageous to specify a timber based product as it easier to connect to ancillary components and can normally be installed in the factory as part of a floor cassette or wall panel. The fitting of a steel section is a site operation which is awkward, time consuming and can pose a health and safety problem.

a) Garage door opening

b) Bay window opening

Figure 6.1 Examples of onerous load span conditions

It is normally serviceability criteria which dictate the specification with deflection the governing factor. In terms of specification the availability of timber is normally restricted to C24 sections at a maximum depth of 240mm and the thickness of the beam is restricted by the width of the bearing area (89mm if it is a standard cripple stud). Therefore, specifying a larger timber section or higher strength grade is not normally an option.

There are other options available rather than using a steel section. The use of a timber composite such as Intrallam (Laminated Strand Lumber or LSL), Parallam (Parallel Strand Lumber or PSL) or LVL, are readily available with deeper sections. However, these products are at a cost premium (normally 2 to 3 times more expensive than C24 strength grade timber) and the mean E value, used to determine stiffness EI in design, is not generally an improvement on C24. Intrallam, Parallam and LVL have a mean E value of 10300, 12750 and 13500Nmm^{-2} respectively compared to 11000Nmm^{-2} for C24 timber. Therefore, in terms of design these products are only at an advantage, considering the additional cost, if a deep section can be specified.

Depth of section is normally restricted in design. If the beam is to form part of the floor system then it has to conform to the depth of joist being used, which in the majority of cases is limited to a 241mm deep I-joist, although there are occasions where deeper joists may be used for large spanning floors but normally the preference is to reduce joist spacing and maintain a shallow floor.

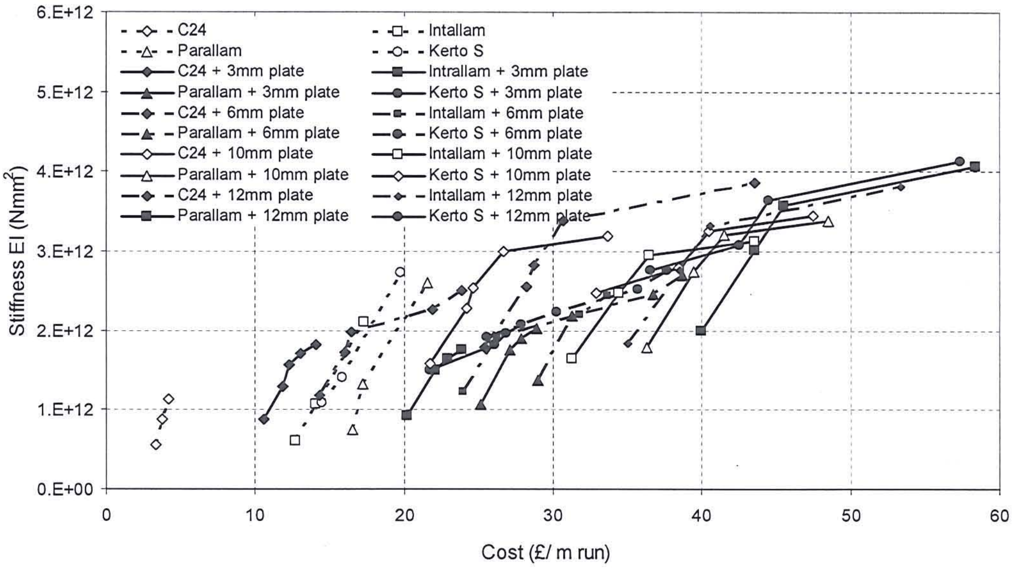
A further option is to use a flitch beam which is a timber-steel-timber sandwich beam traditionally formed with a bolted connection. An example of a bolted flitch beam being fabricated can be seen in Figure 6.2. Flitch beams combine the benefits of timber construction (ease of working, readily available resource, simple connection of ancillary components) with the strength and stiffness of structural steelwork (Bainbridge, et al, 2001).

What governs specification will be the stiffness of beam required, allowable depth of section and cost. Shown in Figure 6.3 is a comparison of available beam options in terms of stiffness and material cost. It is shown that there are occasions where the specification of a flitch beam is advantageous in terms of material cost. As an example a flitch beam consisting of 190 and 220mm deep C24 grade timber and 180 and 200mm deep 6mm steel plate respectively is more cost effective than Intrallam, Parallam and Kerto S LVL up to depths of 241mm.

Flitch beams can also be fabricated using timber composites where additional stiffness is required and allowable depth is limited. The most prevalent use of timber composites in domestic dwelling construction is as rim board material for floor cassettes. The specification of the rim board is at the discretion of the floor designer and supplier as they guarantee the flooring system. The use of composite timber materials as rim board material is mainly due to its stability as an end product. The manufacturing process of timber composites result in a product with low moisture content (6 – 12%), which is strong and consistent and less prone to shrinking, warping, cupping, bowing or splitting which result in solid grade timber due to seasoning stresses (Lam and Prion, 2003). Figure 6.3 also contains examples of different configurations of flitch beam consisting of timber composites which provide improved value.

Figure 6.2 Fabrication of a bolted flitch beam

Research on traditional bolted flitch beams dates back to 1859 (Desai, 2003). The bolting together of the constituent parts is slow and inefficient requiring the pre-drilling of holes in the timber and steel elements to be bolted together. In 1973 Stern, G. E. and Kumar, V. K. reported the use of hammer or machine-driven nails or gun-driven staples as an innovative method of connecting the flitch beam elements together that would alleviate the problems associated with fabrication. More recently the use of baintically hardened nails ballistically fired using a SPIT P200 cartridge gun (Figure 6.4) has been investigated (Larsen and Mettem, 2001 and Alam, 2004). This alternative method of flitch beam fabrication employs readily available equipment which requires minimal training and maintenance and is more efficient due to speed of application. The information available from previous studies was, although valuable, not sufficient enough to allow confident specification and replacement of the existing traditionally bolted product. As a result a laboratory test programme was carried out and the results of this work are documented herein and, where appropriate, reference is made to previous studies.



Note: information is based on actual available material sizes & costs and each point is related to the following information:

- All options are based on a total timber width of 90mm.
- Each C24 grade timber point corresponds to a depth of 190, 220 & 240mm.
- Each Intrallam and Parrallam point corresponds to a depths of 200, 241 & 302mm
- Each Kerto S LVL point corresponds to a depth of 220, 241 & 300mm
- Each flitch beam point is based on the following material depths in mm (timber + steel):
  - C24: 190 + 180; 220 + 200; 240 + 200, 220 & 235mm respectively.
  - Intrallam & Parallam: 200 + 180; 241 + 200, 220 & 235mm respectively.
  - Kerto S LVL: 220 + 180; 240 + 200, 220 & 235mm respectively.

Figure 6.3 Comparison of readily available beam options

a) Nails & cartridge

b) Machined end of nail

c) SPIT P200 Disc Cartridge Tool

Figure 6.4 Bainitically hardened nails & equipment

6.3 Strength of Connection

6.3.1 Introduction

In a traditional bolted flitch beam the specification of bolts is based on the strength of the connection. According to IstructE (2007) the load transfer to the steel plate per unit length is constant if the bolts are spaced equidistantly, so the load per bolt = $F_{UDL,s}/n$ where F_{UDL} is the uniformly distributed load along the beam and n is the number of bolts. Such that shot fired dowels can be specified for a flitch beam application it is important the connection method is understood. As a result an experimental programme was carried out to investigate the strength properties of both the shot fired dowel fixing and the flitch connection to be formed.

Flitch beams used in timber platform frame systems are formed from a range of steel thicknesses and timber element types normally, C24 grade timber, Kerto S Laminated Veneer Lumber (LVL) and Timberstrand Laminated Strand Lumber (LSL). Therefore, the test programme was set-out in a manner which investigated the range of products used to form flitch beams. The results from the experimental programme are compared with appropriate design methods and recommendations are made to allow safe and robust design to take place.

6.3.2 General

The serviceability and the durability of timber structures mainly depend on the design of the joints between the elements; the reason for this is that the connections in a timber system are normally the weakest link (Racher, 1995). Bolts and nails are both dowel type connections. The traditional bolted connection used for flitch beam fabrication would require the pre-drilling of holes 1mm larger than the bolt diameter for ease of installation. Davis and Claisse (2000) report that laterally loaded timber joints constructed from dowels experience an initial slip whereby, as a result of the bolt hole clearance, load transfer across the joint is only achieved after an initial slip of the joint which brings the bolt into bearing contact with the wood. There is also a 'bedding in' stage where the initial load results in localised crushing of the cut wood surface.

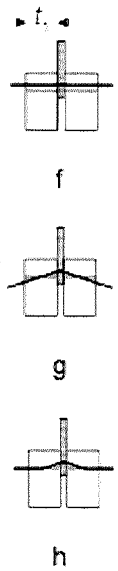
Nails are the most commonly used fastener in timber construction and are available in a variety of lengths, cross-sectional areas and surface treatments (Hilson, B. O, 1995). For nail fixing pre-drilling is required in EC5 if the thickness of the timber element is less than approximately seven times the diameter of the nail and also if the density of the timber is greater than 550kg/m^3 to prevent splitting. Pre-drilled holes are normally restricted to 80% of the nail diameter. The nails used in shot fired nailed flitch beams are 60mm long, have a diameter of 3.6mm and are formed from high strength steel, hardened through the lower banite reaction within the range of 250-400°C (Alam and Ansell, 2003). The nails are shot using an explosive charge, therefore pre-drilling is not an option, but splitting due to high impact and cleavage of the timber fibres was at early stages envisaged to be a design issue.

Splitting decreases the load-bearing capacity of multiple fastener joints and it is for this reason EC5 stipulates minimum nail spacing to prevent over splitting of timbers. The larger the spacing, the smaller the tension stresses perpendicular to the grain caused by the wedge effect of the fasteners.

Large spacing therefore contributes to plastic connection behaviour and consequently increases the capacity of multiple fastener joints according to Blass (1995).

The number of shot fired nails specified will depend on the lateral load carrying capacity of the fixing. According to EC5 the characteristic load carrying capacity of a connection consisting of a steel plate of any thickness as the central member of a double shear connector can be calculated applying the following equations which were first developed by Johansen (1949):

Failure Modes:



$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d \\ f_{h,k} \cdot t_1 \cdot d \cdot \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,k} \cdot d \cdot t_1^2}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} \\ 2.3\sqrt{M_{y,Rk} \cdot f_{h,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} \end{cases}$$

Equation 6.1

Where

$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener.

$f_{h,k}$ is the characteristic embedment strength in the timber member.

t_1 is the smaller of the thickness of the timber side member or the penetration depth.

d is the fastener diameter.

$M_{y,Rk}$ is the characteristic fastener yield moment.

$F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener.

6.3.3 Tensile & Yield Moment Capacity

To determine the yield moment capacity of the nails so that the calculation methods of EC5 for determining the lateral load carrying capacity of the joint could be used tests were conducted in accordance with BS EN 409:1993 the test set-up of which is shown in Figure 6.5. From the tests conducted the characteristic yield moment capacity, $M_{y,Rk}$, of the fixing was determined to be 17956Nmm (compared to 16558Nmm found by Alam, 2004).

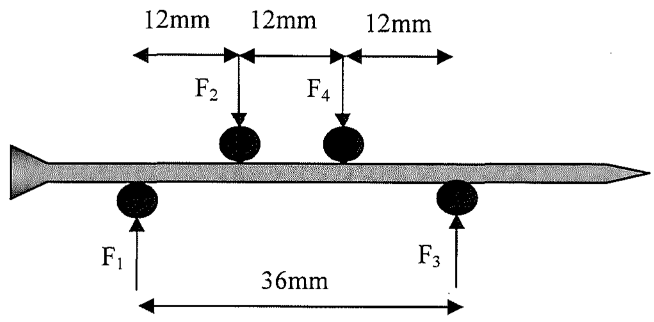


Figure 6.5 Yield moment test set-up

To determine the characteristic yield moment, $M_{y,Rk}$, of the range of fixings which formed this study by calculation the following equation can be used in accordance with EC5 clauses 8.3.1.1:

$$M_{y,Rk} = 0.3 \cdot f_u \cdot d^{2.6} \qquad \text{Equation 6.2}$$

Where f_u is the tensile strength (2000N/mm^2) of the wire and d is the nail diameter in mm. The diameter of the fixing to be used is the effective diameter, for smooth shanked dowels this is taken as the shank diameter (3.6mm). Therefore, by calculation the yield moment, $M_{y,Rk}$, is 16770Nmm. The test determined yield moment of this study is 7% more than that determined by EC5 calculation which is an acceptable level of percentage difference.

6.3.4 Axial Load Carrying Capacity

Frictional effects contribute to the lateral load carrying capacity of a nailed joint. As a connection yields friction between the members is caused by the pulling together of the members due to the axial load carrying or "withdrawal" capacity of the fixing. The method of treatment of a nail will have an affect on its withdrawal resistance. Galvanising a nail, which protects the metal (normally steel), with a coating of zinc does not help the resistance of a nail from pull out as it creates a smooth surface. However, sheradised nails or those with cement coatings will have an improved level of pull out as the coating method will provided additional frictional resistance.

Considering the connection strength calculations of EC5, withdrawal strength of the fixing is an important parameter if failure mode "*g*" or "*h*" is critical. In accordance with EC5 the following expressions are used to determine the characteristic withdrawal capacity of smooth nails for nailing perpendicular to the grain:

$$F_{ax,Rk} = \min\begin{cases} f_{ax,k} \cdot d \cdot t_{pen} \\ f_{ax,k} \cdot d \cdot t + f_{head,k} + d_h^2 \end{cases} \qquad \text{Equation 6.3}$$

Where

$f_{ax,k}$ is the characteristic pointside withdrawal capacity.

$f_{head,k}$ is the characteristic headside pull through strength.

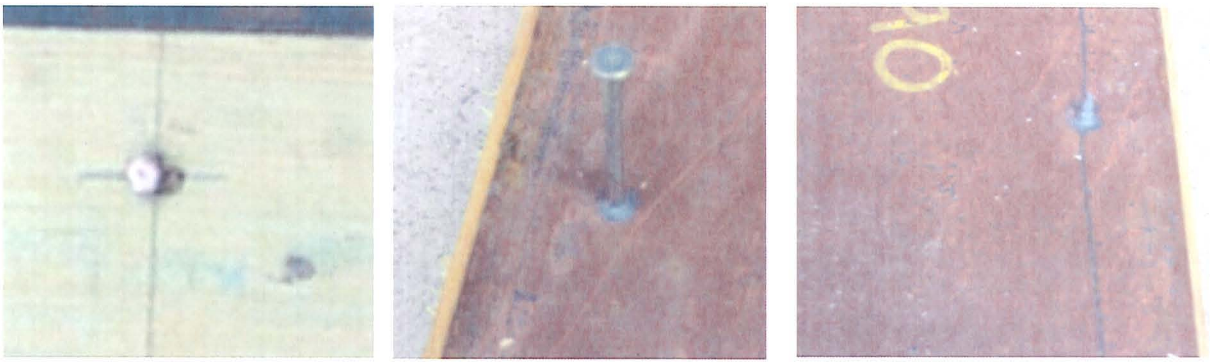
d is the nail diameter as defined in EN 14592.

t_{pen} is the pointside penetration length.

t is the thickness of the headside member.

d_h is the nail head diameter.

Considering the range of steel plates available the maximum pointside penetration will be when 3mm steel is used. The use of 3mm steel plate will result in approximately 12mm pointside penetration depending on the level of embedment of the nail head (Figure 6.6a). In accordance with Eurocode 5 (clause 8.3.2(7)) the pointside withdrawal cannot be considered as it is less than 8d, and as a result the withdrawal capacity should be taken as zero. However, due to the nature of the shot fired nail connection a cold weld forms between the fixing and the steel element (Figure 6.6b & c) and this enhances the pull out resistance of the nail.



a) Embedment of nail head in timber

b) Headside of nail after removal of timber showing cold weld

c) Pointside of nail after removal of timber showing cold weld

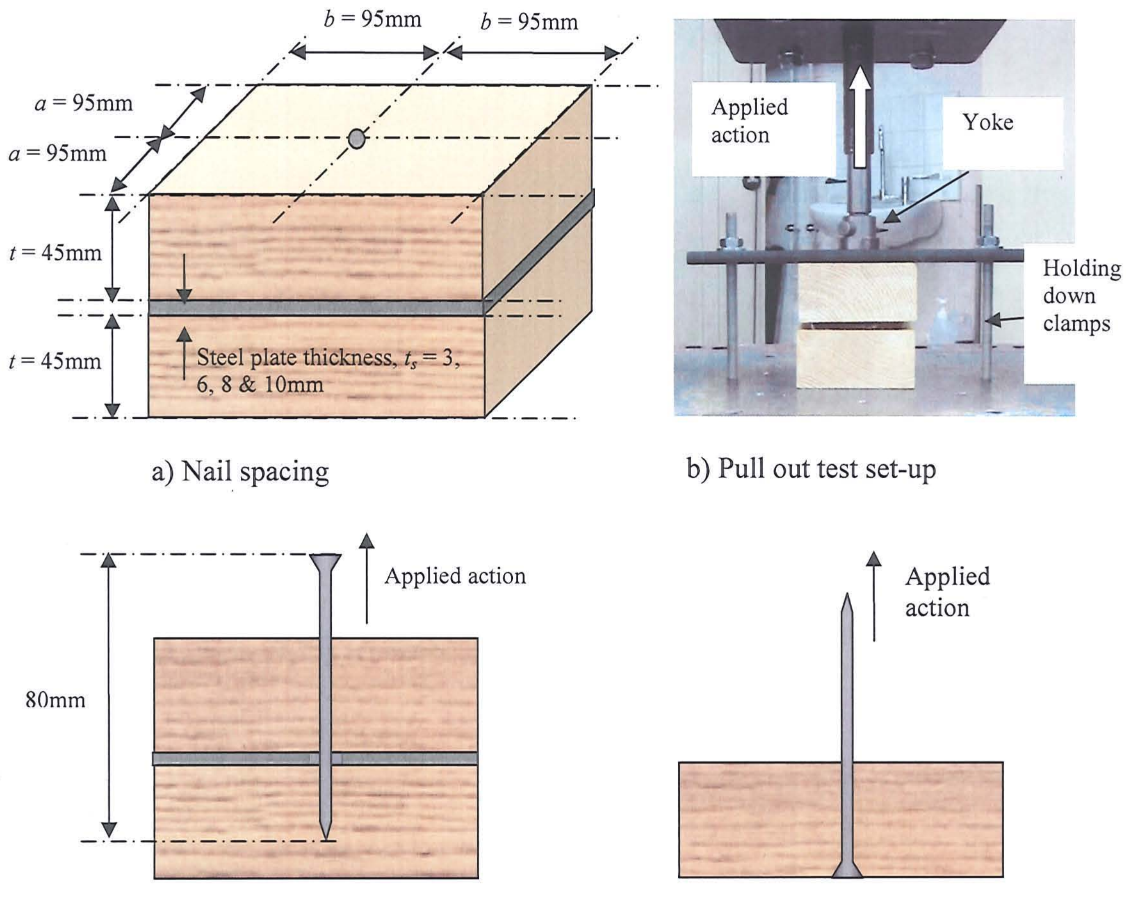
Figure 6.6 Shot fired dowel nail connection

To quantify the pull out (nail is pulled out of the steel weld and timber element) and pull through (nail is pulled through the timber element) strength of the connection and the influence of the timber element (solid section or composite), density and steel thickness, tests were conducted in accordance with BS EN 1382:1999.

The pull out and pull through tests were set-out so that the spacing requirements were in excess of the requirements of BS EN 1382:1999 as shown in Table 6.1and Figure 6.7. For both the pull out and pull through tests 80mm nails were used as this would result in a larger pointside or headside to grip. The pull out tests were carried out on each timber element type (C24, LVL and Timberstrand LSL) for each steel thickness and the pull through tests were carried out on each timber element type.

Table 6.1 Fastener spacing for standard withdrawal test specimens

	Fastener Spacing, mm		
	a	*b*	width
BS EN 1382:1999	$\geq 5d$	$\geq 10d$	$\geq l_p + 5d$
	18	72	98
Test specimens	95	95	190



a) Nail spacing

b) Pull out test set-up

c) Pull out test specimen

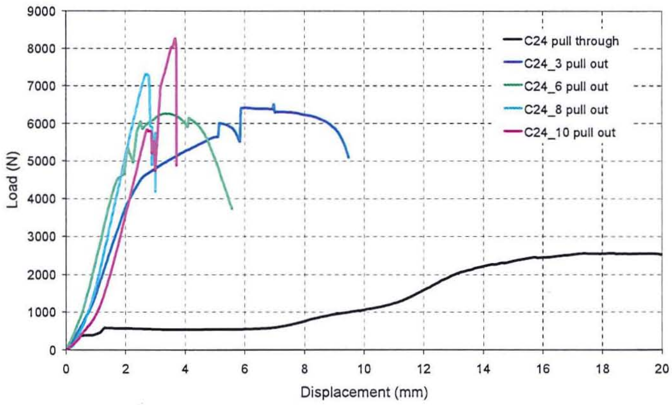
d) Pull through test specimen

Figure 6.7 Pull out and pull through test specimen

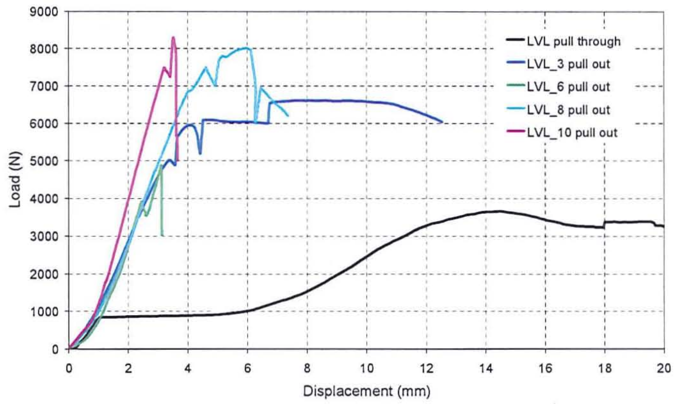
During the pull out and pull through tests the applied load and corresponding displacement was measured using a data logger for each test set which are as designated:

- C24, LVL & TS pull through
- C24_3, 6, 8 &10 pull out – C24 grade timber with 3, 6, 8 or 10mm steel.
- LVL_3, 6, 8 &10 pull out – LVL timber element with 3, 6, 8 or 10mm steel.
- TS_3, 6, 8 &10 pull out – Timberstrand LSL timber element with 3, 6, 8 or 10mm steel.

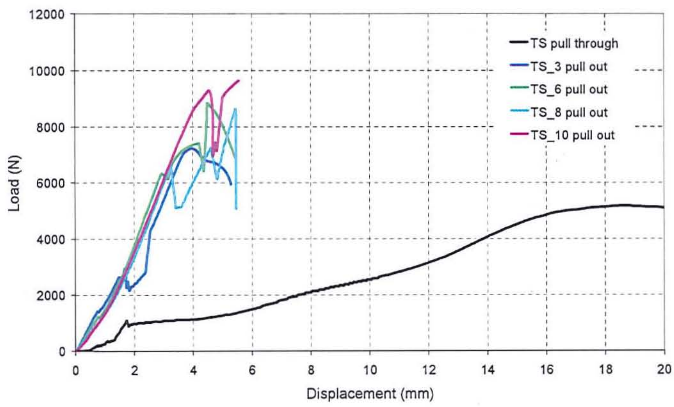
Each test set comprised of four samples and in Figure 6.8 the average load displacement curves of each test set are shown. From Figure 6.8 it can be seen that pull out is for the majority of cases greater than the pull through force. It is also shown in Figure 6.8 that pull out failure, when the steel is thicker than 3mm, tends to be brittle and this is because the thinner steel will deform under lower levels of load whereas the thicker steel will not deform and the cold weld will eventually fail in a brittle manner.



a) C24



b) LVL

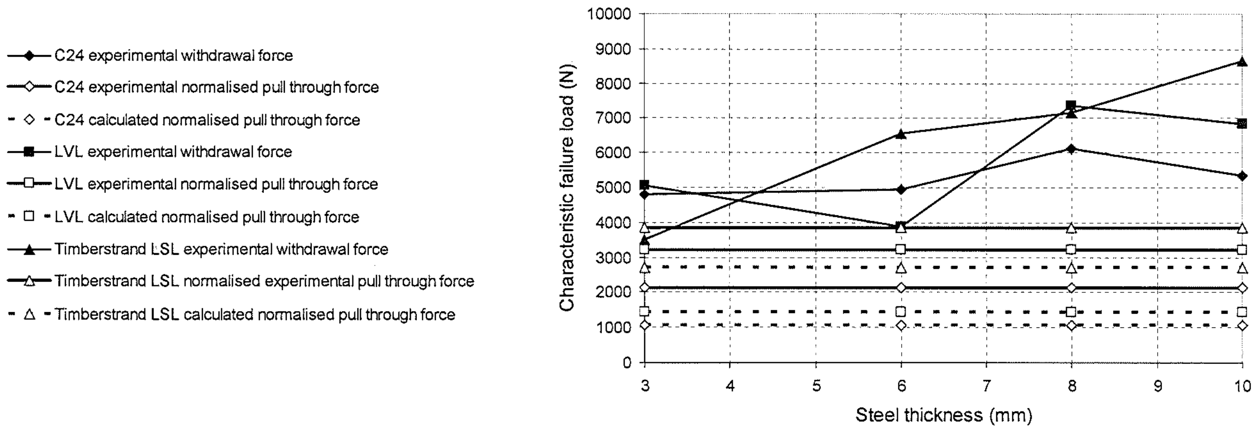


c) Timberstrand LSL

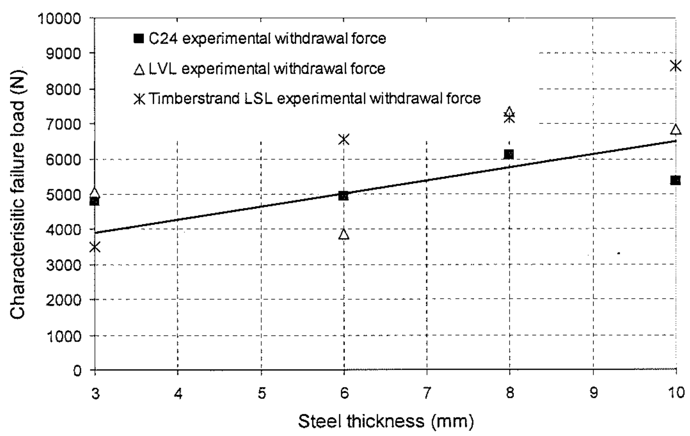
Figure 6.8 Pull out and pull through load against displacement curves

From the experimental work carried out the withdrawal force, with respect to steel thickness and timber element type, and head side pull through force, with respect to density and timber element type, were measured. The experimental results and calculated headside pull through force are compared in Figure 6.9 and to compensate for variations in density across the samples the failure loads, which are averaged from the 4 test pieces, have been normalised relative to the average density of the set.

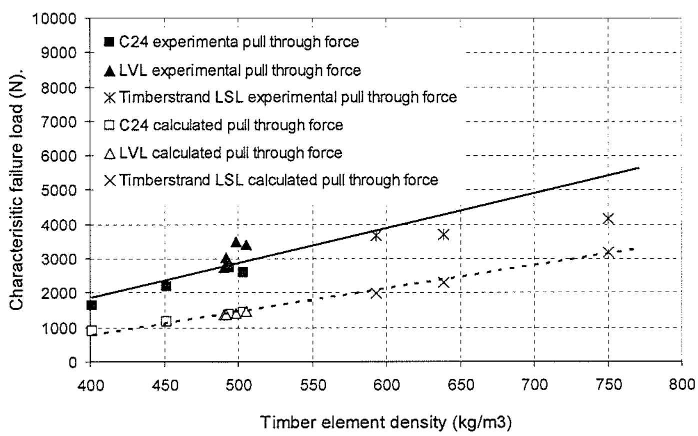
The general trend of all withdrawal test results with respect to steel thickness is also shown in Figure 6.9. In this instance variations in timber element density are not compensated for because the full range of timber element types are used. From the pull through force tests the relationship between timber element density and failure load, again averaged from 4 test pieces, was also investigated for each timber element type and compared to the calculated headside pull through force (Figure 6.9).



a) Characteristic withdrawal and headside pull through force relationship with steel thickness (normalised for density)



b) Characteristic withdrawal strength relationship with steel thickness



c) Characteristic withdrawal strength relationship with timber density

Figure 6.9 Comparison of experimental and calculated axial load carrying capacity

From Figure 6.9 the following conclusions are drawn:

- The experimental withdrawal strength of nails in a flitch beam connection is, as a result of the cold weld which is formed with the steel element, in the majority of cases greater than the experimental headside pull through force.
- The only case where the experimental withdrawal strength is marginally lower than the experimental pull through force is in the Timberstrand LSL connection with 3mm plate as a result of the high density of timber element and relatively thin steel plate.
- It is noted from the plots that the general trend is an increase in withdrawal strength with plate thickness. Increased steel thickness results in a larger weld contact area improving the connection strength.
- In all cases the experimental withdrawal strength of the connection is greater than the calculated headside pull through strength and it can therefore be recommended that it is safe to use headside pull through strength to determine the withdrawal capacity of the fixings when carrying out lateral load carrying capacity calculations in accordance with Eurocode 5.
- Further clarification of the above point is shown in Figure 6.9c where it is demonstrated that the relationship between increased timber element density and failure load is directly proportional and the calculated pull through force trend for the given density range, determined in accordance with Eurocode 5, conservatively correlates with the experimental trend.

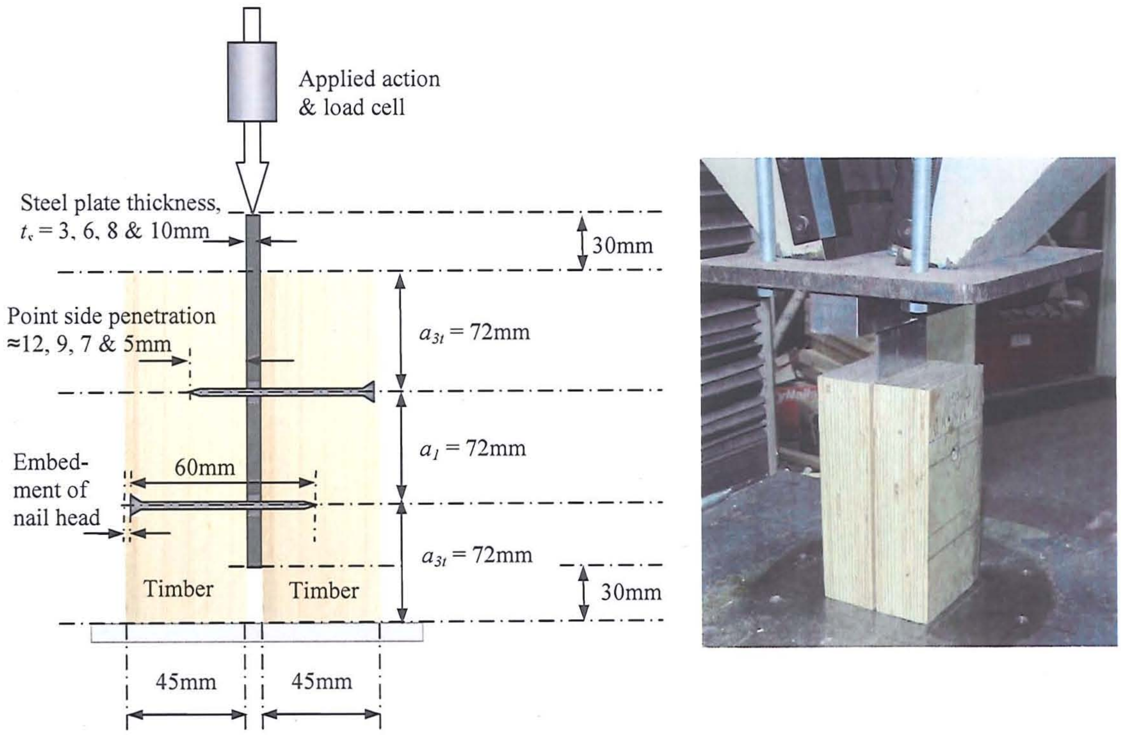
6.3.5 Lateral Load Carrying Capacity

The specification of nails in the design of a shot fired dowel flitch beam will depend on the strength of the connection. Therefore, to evaluate the strength of the shot fired nailed flitch joint when subjected to lateral loading double shear tests were carried out. The test specimens were fabricated in accordance with the detailing contained in Figure 6.10 and loaded laterally as shown. The tests were conducted in such a manner that the influence of timber element type (C24 grade timber, LVL and Timberstrand LSL), density and steel thickness (3, 6, 8 & 10mm) were evaluated. The reason for doing this was to provide information which could be used in the design of industry standard flitch beams which use the timber element types listed and have varying steel thickness depending on the load span conditions.

The spacing of the nails in the connection, as contained in Table 6.2, was in excess of the recommended minimum spacing as stipulated by EC5 to alleviate potential interaction of localised stresses between the nails in the wood (Alam, 2004) and also to alleviate the effects of splitting. Splitting of the timber is heightened in this form of connection as a result of the high impact force of nail application. It was also noted that splitting was more prevalent in solid timber sections, especially so in high density timbers as a result of closeness of grain and therefore a higher instance of cleavage planes. However, the engineered products showed a reduction in splitting and in particular Timberstrand LSL showed negligible signs of splitting due to the inherent properties of the material. The random orientation of strands in Timberstrand LSL results in the dissipation of splitting energies in all directions which corresponded to a reduction in splitting.

Table 6.2 Nail spacing for lateral load test

	Spacing and edge distances, mm				
	Spacing, a_1 (parallel to grain)	Spacing, a_2 (perpendicular to grain)	Distance, $a_{3,t}$ (loaded end)	Distance, $a_{3,c}$ (unloaded end)	Distance, $a_{4,t}$ (loaded edge)
EC5 requirement	$\geq(7+8\cos\alpha)d$	$\geq 7d$	$\geq(15+5\cos\alpha)d$	$\geq 15d$	$\geq(7+5\sin\alpha)d$
	54	25.2	72	54	25.2
Test Pieces	$20d$	$20d$	$20d$	$20d$	$20d$
	72	72	72	72	72



a) Details of lateral shear sample

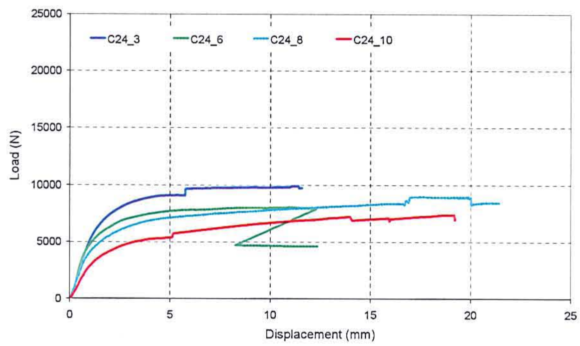
b) Sample being tested

Figure 6.10 Lateral load test sample and set-up

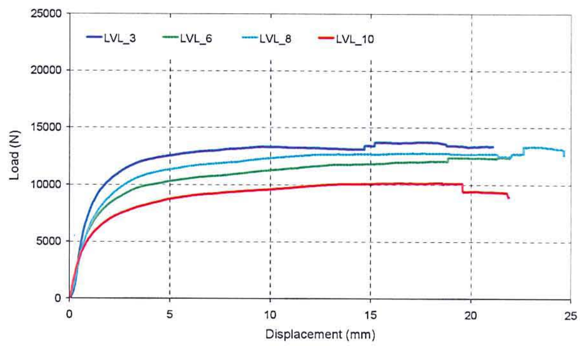
During the lateral load tests the applied load and corresponding displacement was measured using a data logger for each test set which are as designated:

- C24_3, 6, 8 &10 – C24 grade timber with 3, 6, 8 and 10mm steel.
- LVL_3, 6, 8 &10 – LVL timber element with 3, 6, 8 and 10mm steel.
- TS_3, 6, 8 &10 – Timberstrand LSL timber element with 3, 6, 8 and 10mm steel.

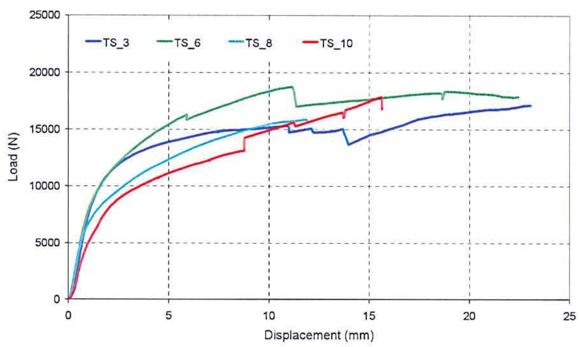
Each test set comprised of 4 test samples and contained in Figure 6.11 are the average load against displacement plots of the sets.



a) C24



a) LVL



c) Timberstrand LSL

Figure 6.11 Lateral load against displacement curves

From Figure 6.11 it is shown that the stiffest connections in order of timber element type are; Timberstrand LSL, LVL and C24 grade timber. This is as expected due to the relative increase in density and the fact that timber composites are less prone to splitting during nail application. Splitting of the timber will reduce its strength at the nail timber interface resulting in increased embedment upon load application.

The general trend of the results contained in Figure 6.12 is an increase in steel thickness corresponds to a reduction in connection stiffness. An increase in steel thickness reduces the cross sectional area of the nail in contact with the timber element. In accordance with EC5 characteristic embedment strength for nails up to a diameter of 8mm without predrilled holes is calculated as follows:

$$f_{h,k} = 0.082 \cdot \rho_k \cdot d^{-0.3} \qquad \text{Equation 6.4}$$

Where

$f_{h,k}$ is the characteristic embedment strength in the timber member.

ρ_k is the characteristic timber density.

d is the nail diameter.

To determine the load carrying capacity of the connection for failure modes "f" and "g" of Equation 6.1 embedment strength is multiplied by the cross sectional area and forms a component of the overall lateral resistance. Therefore, a reduction in embedment strength due to a reduction of cross sectional area in contact with the timber element will reduce the load carrying capacity of the connection.

The load against displacement curves which are contained in Figure 6.12 for the shot fired dowel connections tested demonstrate that the connection is ductile. However, it was noted during the experimental programme that nails within the connection, particularly when the connection was formed from a timber composite product, would on occasion fail in shear after a relatively high degree of displacement.

The information from the laboratory programme was processed to determine the characteristic lateral resistance of the shot fired dowel connection and this information is presented in Table 6.3. The results presented have been averaged from the four test pieces and then normalised, to compensate for variations in sample density across the range of specimens to allow a truer comparison. Also contain in Table 6.3 are the calculated results applying Equation 6.1 based on two methods:

1. *Test yield moment & full fixing embedment*: Yield moment (17956Nmm) is as quantified from the fixing tests and t_1 is the sum of penetration depths in both elements which equates to the length of the nail minus the thickness of the steel plate.
2. *Tensile strength yield moment & average fixing embedment*: Yield moment (16770Nmm) is as calculated in accordance with Eurocode 5 from the known tensile strength of the fixing (2000N/mm^2) and t_1 is the mean penetration depth (Bainbridge, et al., 2001).

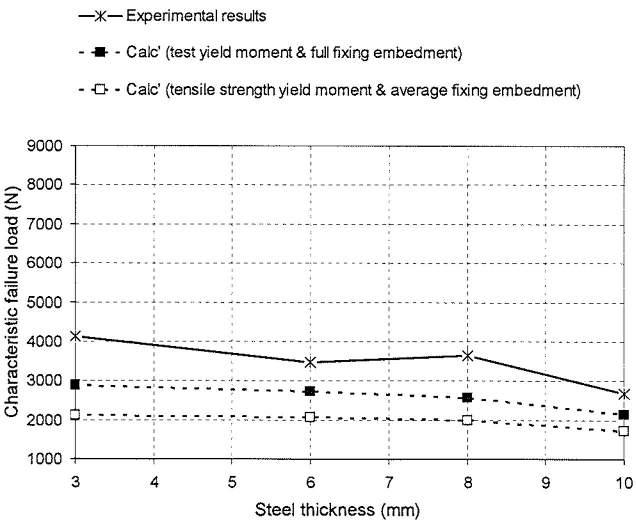
As a result of previous findings the withdrawal capacity of the fastener in both cases has been taken as headside pull through calculated in accordance with EC5.

Table 6.3 Comparison of lateral load experimental results to calculated results

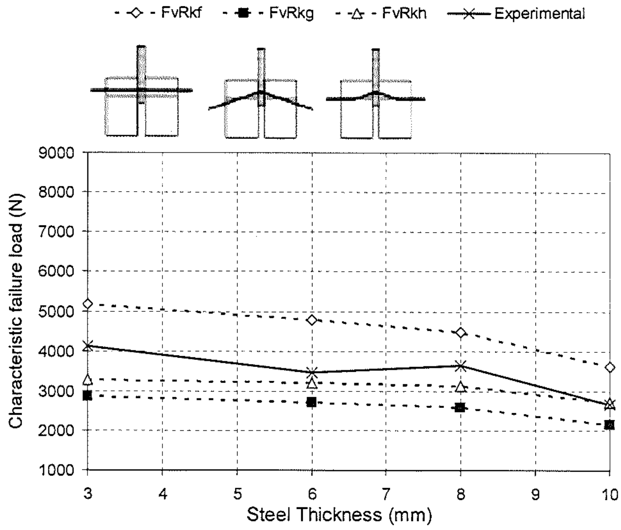
Test designation	Characteristic experimental lateral resistance	Calculated Characteristic Lateral Resistance			
		Test yield moment & full fixing embedment	% Difference with experimental results	Tensile strength yield moment & average fixing embedment	% Difference with experimental results
	N	N		N	
C24_3	4130	2868	31	2129	48
C24_6	3458	2715	21	2064	40
C24_8	3629	2584	29	1999	45
C24_10	2670	2146	20	1712	36
Average	3472	2579	26	1976	43
LVL_3	5441	2729	50	2060	62
LVL_6	5074	2663	48	2052	60
LVL_8	5700	2895	49	2247	61
LVL_10	4351	2625	40	2080	52
Average	5141	2728	47	2110	59
TS_3	6691	3478	48	2545	62
TS_6	8172	3819	53	2835	65
TS_8	6486	3267	50	2473	62
TS_10	7029	4274	39	3240	54
Average	7094	3709	48	2773	61

Shown in Figure 6.12 a, c, & e is the relationship between steel thickness and connection shear strength for each timber element type (C24 grade timber, LVL and Timberstrand LSL) compared with both methods of calculation. It is shown that both methods of calculation conservatively correlate with the trend set by the experimental results with the *Test yield moment & full fixing embedment* trend demonstrating improved correlation. Therefore, considering *Test yield moment & full fixing embedment* the experimental characteristic failure loads are compared to the failure modes of Equation 6.1 in Figure 6.12 b, d & f.

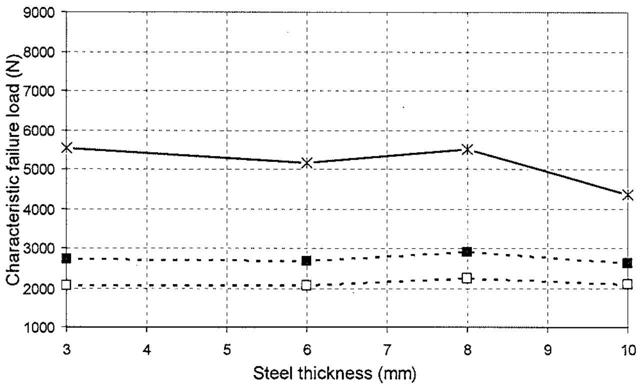
The effect of timber element density is also considered. Figure 6.13 demonstrates the variation in characteristic failure load with timber element density (C24 = 419kg/m^3; LVL = 505kg/m^3 & Timberstrand LSL = 688kg/m^3) from both the experimental results and as calculated in accordance with EC5 using Equation 6.1, again considering *Test yield moment & full fixing embedment.* Finally the correlation of experimental results with calculated EC5 failure modes are compared in Figure 6.14.



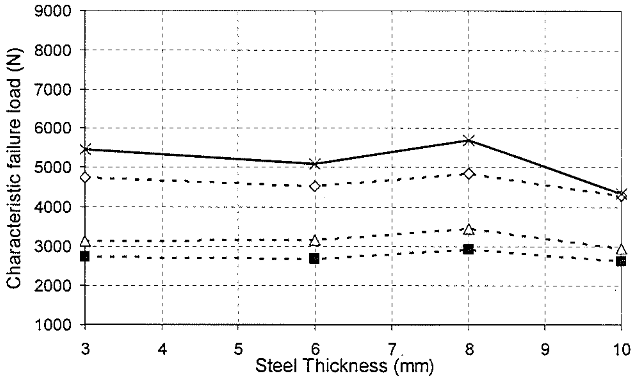
a) C24: Characteristic failure loads for varying steel thicknesses.



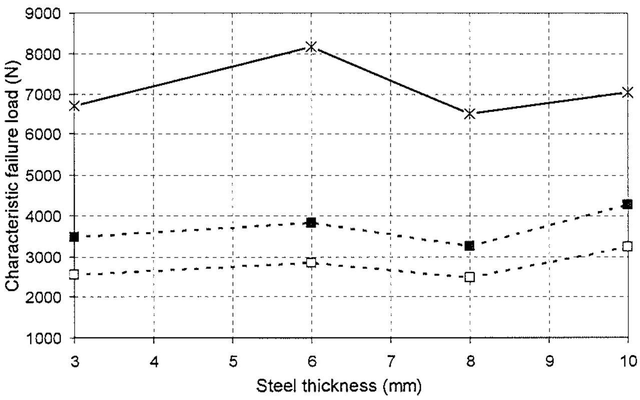
b) C24: Characteristic experimental results compared to Equation 6.1 failure modes



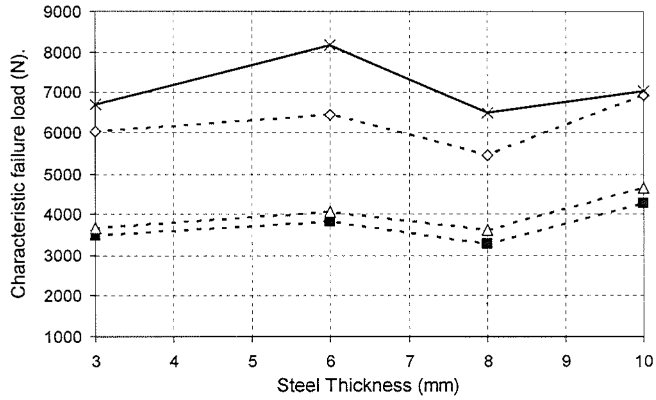
c) LVL: Characteristic failure loads for varying steel thicknesses.



d) LVL: Characteristic experimental results compared to Equation 6.1 failure modes



e) TS LSLS: Characteristic failure loads for varying steel thicknesses.



f) TS LSL: Characteristic experimental results compared to Equation 6.1 failure modes

Figure 6.12 Comparison of EC5 calculated results to experimental results for varying steel thickness (Note: Test yield moment & full fixing embedment have been used for the calculated results)

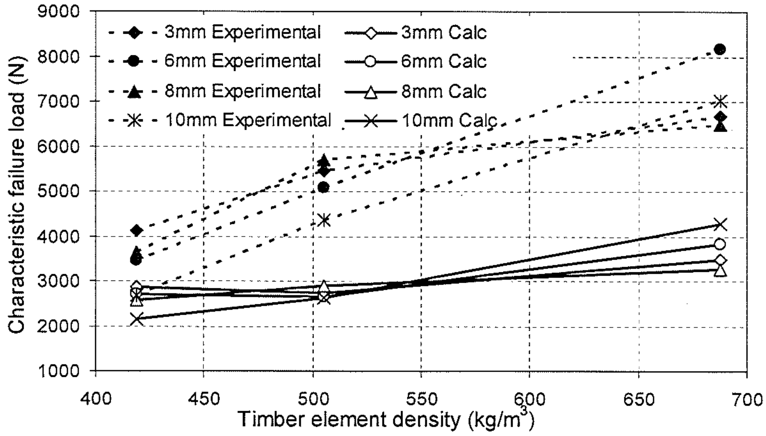


Figure 6.13 Variation in characteristic failure load with timber element density

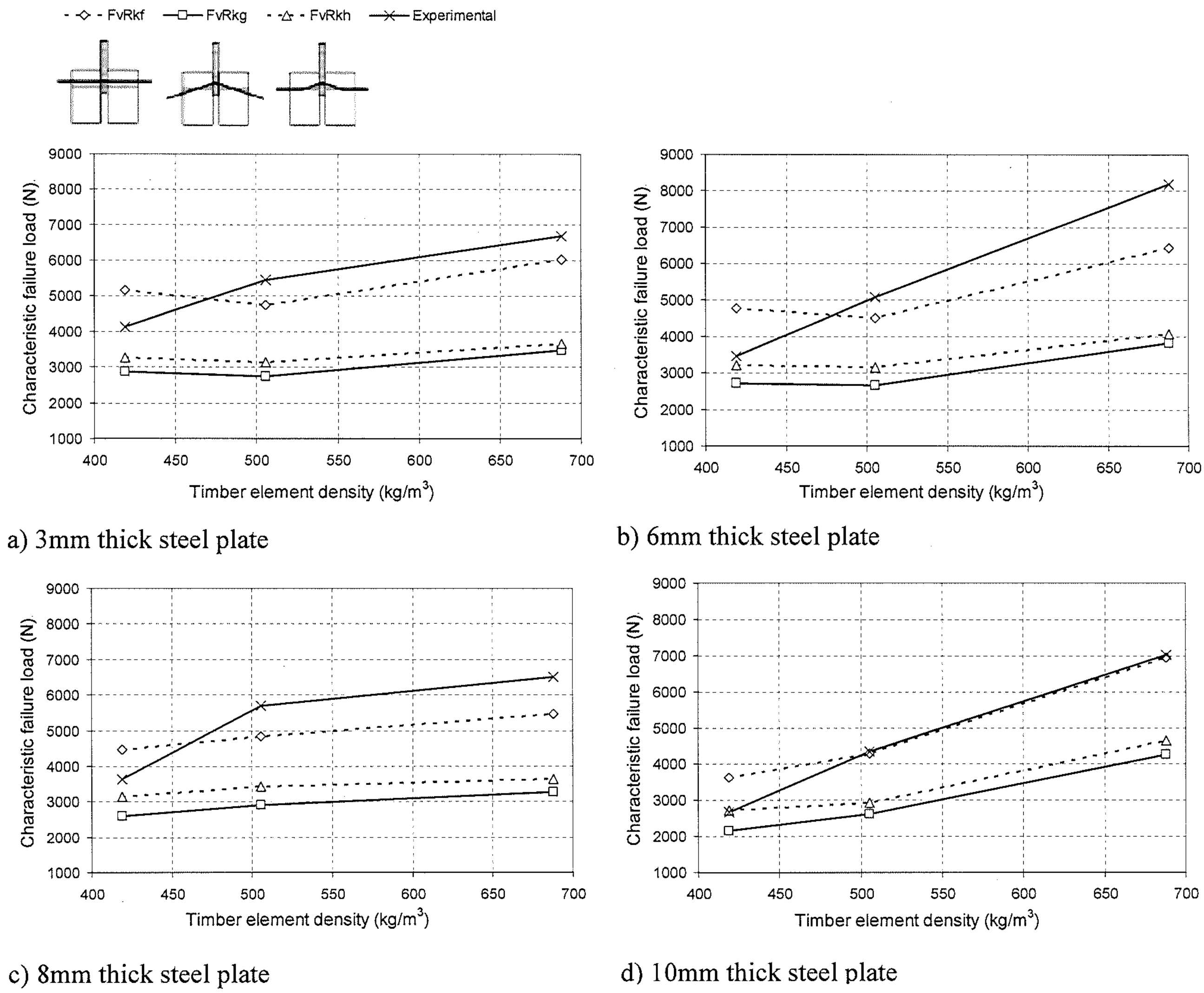


Figure 6.14 Comparison of EC5 calculated results to experimental results for varying timber density (Test yield moment & full fixing embedment have been used for the calculated results)

6.3.6 Summary

From the experimental work carried out the following is concluded:

- It is demonstrated that calculations to determine the lateral load carrying capacity of a shot fired dowel flitch connection considering the test determined yield moment of the fixing and taking the full fixing embedment show improved correlation with experimental results. Calculations carried out considering the yield moment determined from the tensile strength of the fixing and taking the average fixing embedment tend to be more conservative.
- In the order of highest to lowest connection strength, shot fired dowel flitch beam connections are listed as follows; Timberstrand LSL, LVL and C24 grade timber. The improved strength of Timberstrand LSL connections is as a result of two key factors, the higher density of Timberstrand LSL which relates directly to improved embedment strength and also the reduced level of splitting due the nature of the material.
- The results from the C24 grade timber section flitch connection show a high level of correlation with the calculated failure mode "h" which corresponds to the formation of a plastic hinge at the steel timber interface. Due to the relatively low embedment strength of the timber element yielding of the fastener will take place resulting in a plastic hinge forming in the fastener. Splitting of the timber, which corresponds to a reduction in embedment strength, will tend to enhance this failure mode. Evidence of this type of failure in a flitch connection formed using C24 grade timber is shown in Figure 6.15.
- Comparison of the results from the engineered wood composites (LVL and Timberstrand LSL) tests to the calculated failure mechanisms provides further indication of the level of conservatism. According to Equation 6.1 mode "g" is the critical failure mode. However, in both engineered wood cases the experimental results have a higher degree of correlation with mode "f", which is the highest predicted failure mode. Failure in mode "f" corresponds to a bearing failure of the timber. If the on-set of bearing failure in the timber element is at a relatively high load, creating a stiffer connection as shown in Figure 6.8, then this may cause the fixing to shear because the embedment strength of the timber is preventing it from yielding and this corresponds to experimental evidence, Figure 6.15.

a) Bending failure of fixing in C24 flitch connection

b) Shear failure of fixing in LVL flitch connection

c) Shear failure of fixing in Timberstrand LSL flitch connection

Figure 6.15 Examples of laterally loaded shot fired dowel connection failures

6.4 Laboratory Study – Effect of Nailing Pattern on Strength and Stiffness

6.4.1 Introduction

One of the disadvantages of the traditional bolted flitch beams is the time required for fabrication. The use of a shot fired dowel connection offers the opportunity to make a significant time saving if the number of nails used is optimised. The purpose of the research work documented in this section was to determine the influence of shot fired dowel nails on the strength and stiffness of flitch beams and make recommendations for a standardised nailing specification. Standardising the nailing schedule would result in a simplification of design procedures and reduce the fabrication time by means of implementing repetition.

6.4.2 Experimental Programme & Results

To determine the influence that the nailing pattern has on flitch beam stiffness and strength, 45 flitch beams were tested to failure in bending using a variety of nailing patterns. The beams constructed comprised of a sandwich configuration with two C24 grade timbers or Kerto S LVL timber elements sandwiching a 3mm steel plate, details of which are contained in Figure 6.16. The beams tested formed three sets:

1. C24_1.8 – 100mm deep C24 grade timber flitch beam over 1.8m effective span.
2. C24_2.1 – 117mm deep C24 grade timber flitch beam over 2.1m effective span.
3. LVL_1.8 – 100mm deep LVL flitch beam over 1.8m effective span.

For each set 4 different nailing patterns (Figure 6.16) were tested as well as beams formed with no nails which were held together with finger tightened clamps. The nail patterns were determined based on the following points:

1. The minimum nailing requirement was based on the calculated load carrying capacity of the beam and the resulting lateral load carrying capacity per shear plane per fastener required.
2. The minimum spacing requirements as set-out in EC5 were adhered to.
3. Edge and end distances and also the distance between two parallel nails on the same face was a minimum of 150mm to reduce splitting.

The beams were tested in 4 point bending in accordance with BS EN 408(2003) with load and displacement measured via a data logger. The displacement measurements were taken by 3 sets of two transducers placed either side of the beam at the designated positions (A, B & C) shown in Figure 6.17. As a result of the loading conditions the bending moment over the mid span of the beam is theoretically constant as illustrated in Figure 6.17. The average displacement between the 4 transducers at A & C was calculated and the difference between this value and that of the average of the two transducers at B is used to produce the load (total applied load) against displacement curves contained in Figure 6.18. The curves contained are the average curves of the 3 beams tested in each set and presented in Table 6.4 are the results from the experimental programme which have been normalised to account for variations in density so that a truer comparison can be made.

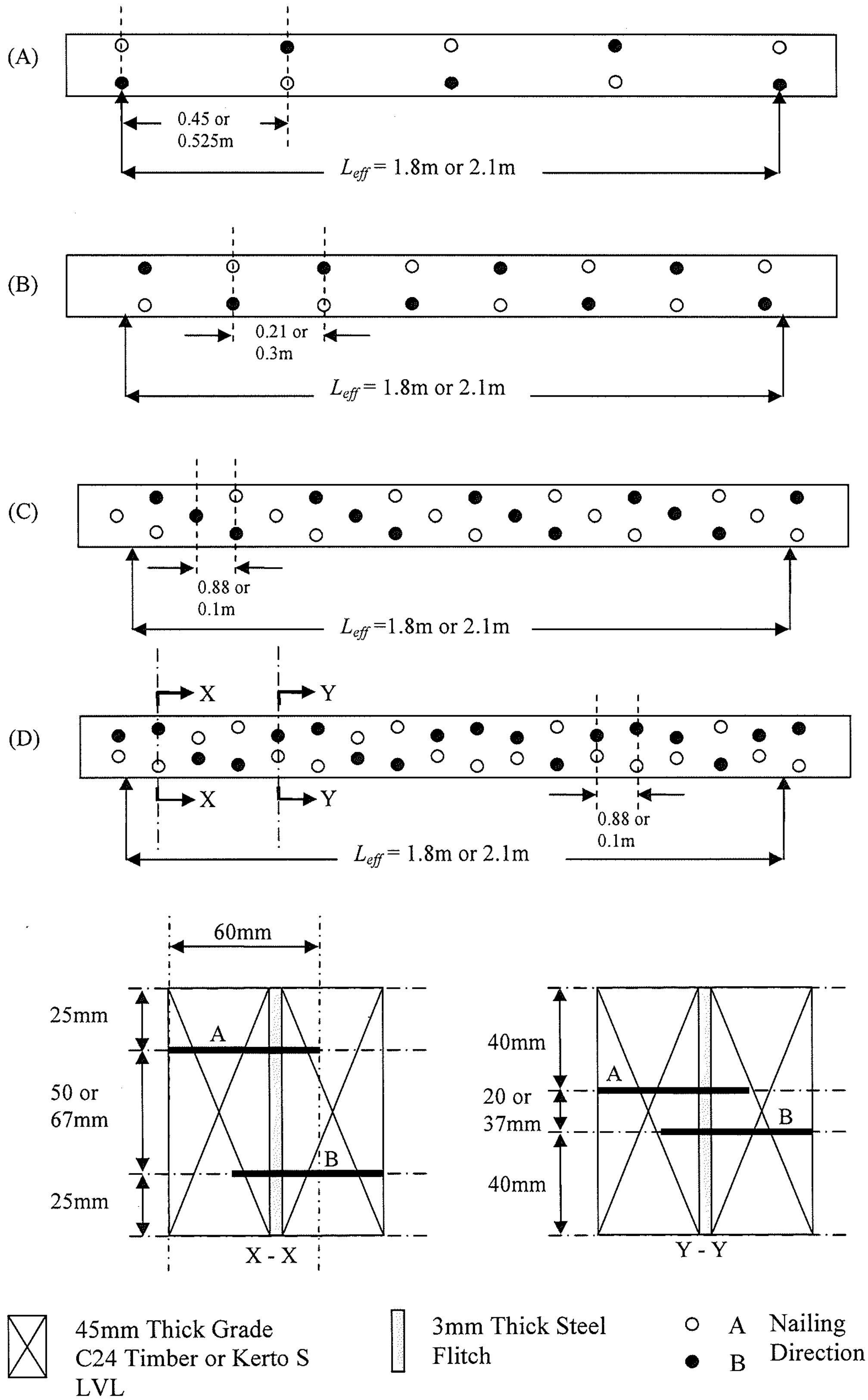
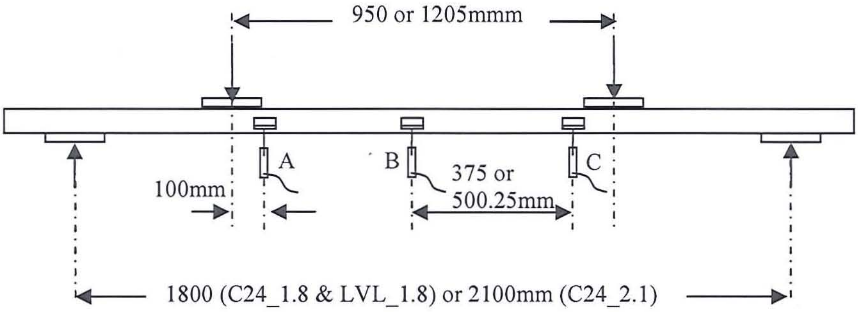
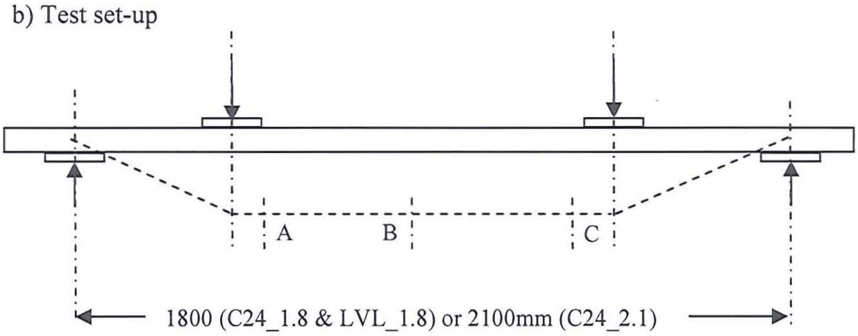


Figure 6.16 Nailing patterns: 5 nails per side (A); 8 nails per side (B); 13 nails on one side & 14 on the other (C) and 18 nails per side (D)

a) Fabrication and testing of flitch beams

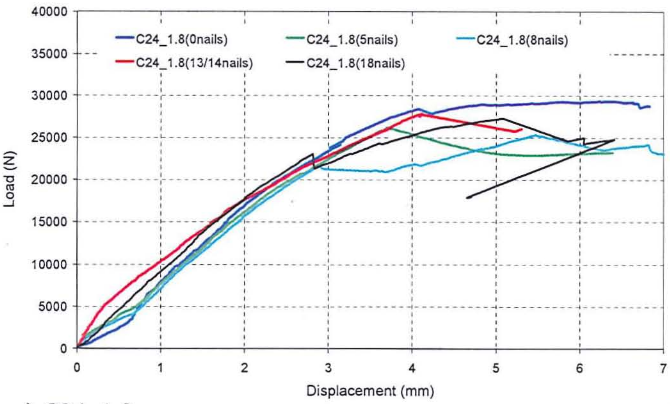


b) Test set-up

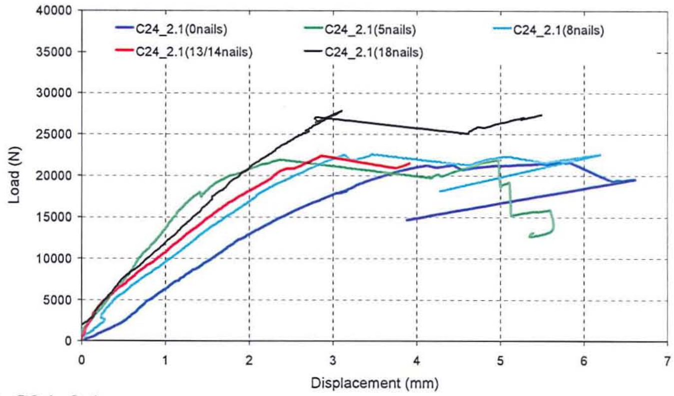


c) Bending moment

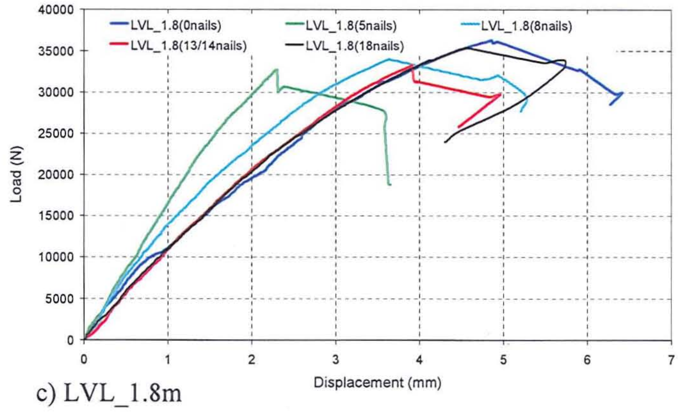
Figure 6.17 Fabrication of flitch beams and test set up



a) C24_1.8m



b) C24_2.1m



c) LVL_1.8m

Figure 6.18 4 point bending load against displacement curves
(Displacement is taken as the difference between the average of the 2 transducers at B less the average displacement of the 4 transducers at A and C)

Table 6.4 Experimental programme results

Beam Designation	Number of nails	Characteristic & Normalised	
		Stiffness	Failure Load
		EI	P_{max}
		Nmm^2	kN
C24_1.8	0	9.99E+07	24.40
	5	9.38E+07	22.69
	8	9.84E+07	16.90
	13/14	8.75E+07	19.40
	18	1.08E+08	21.50
C24_2.1	0	1.66E+08	24.32
	5	1.54E+08	18.85
	8	1.40E+08	17.05
	13/14	1.28E+08	22.12
	18	1.51E+08	25.29
LVL_1.8	0	1.41E+08	31.08
	5	1.25E+08	30.80
	8	1.02E+08	23.68
	13/14	1.23E+08	28.91
	18	1.13E+08	29.98
Note: Normailisation accounts for density variation			

The results contained in Table 6.4 were used to compare the bending moment capacity and stiffness (EI) of flitch beams relative to number of nails used. To do this the characteristic bending moment capacity of the beams, calculated from the given load span conditions (Figure 6.17) and failure loads (Table 6.4), were plotted against the number of nails used, the results of which are presented in Figure 6.19. Variation in stiffness relative to number of nails was also considered and this relationship, for the beams tested, is contained in Figure 6.20. Finally the relationship between stiffness and bending moment capacity was considered and this relationship, for the beams tested, is shown in Figure 6.21.

From the relationships presented in to Figure 6.19 to 6.21 it is demonstrated that in general there is both a reduction in strength and stiffness of the beams with increased number of nails. The application of nails tends to split the timber elements and as a result reduces both the bending moment capacity and stiffness of the flitch beam. In terms of failure modes there was evidence from the tests conducted that if the nailing pattern was such that the position of a nail corresponded with an area of high stress concentration failure of the beam would be exacerbated; examples of this are contained in Figure 6.22. Also, noted from the failed beams was the tendency for the steel plate to buckle out of plane due to compression in the top chord, again examples of this are shown in Figure 6.22.

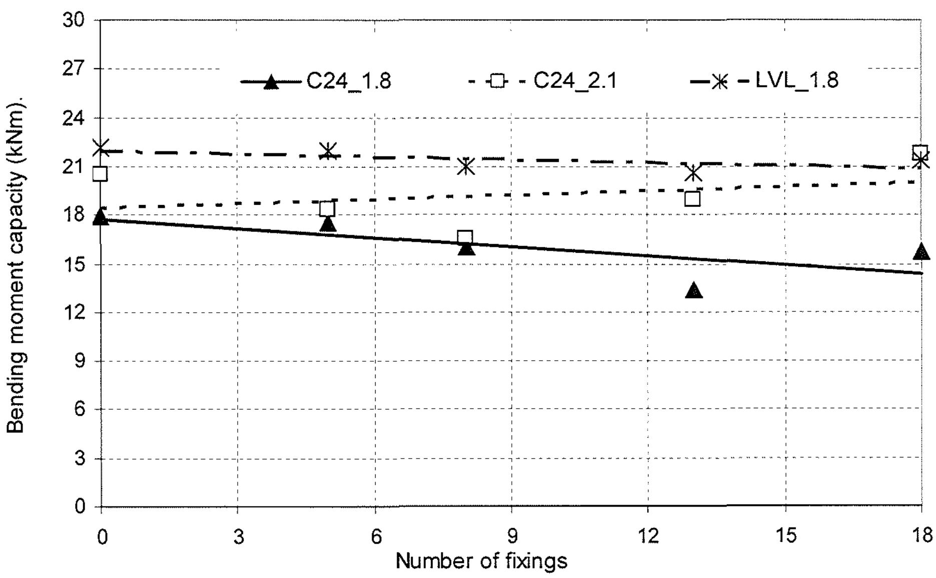


Figure 6.19 Bending capacity against number of nails

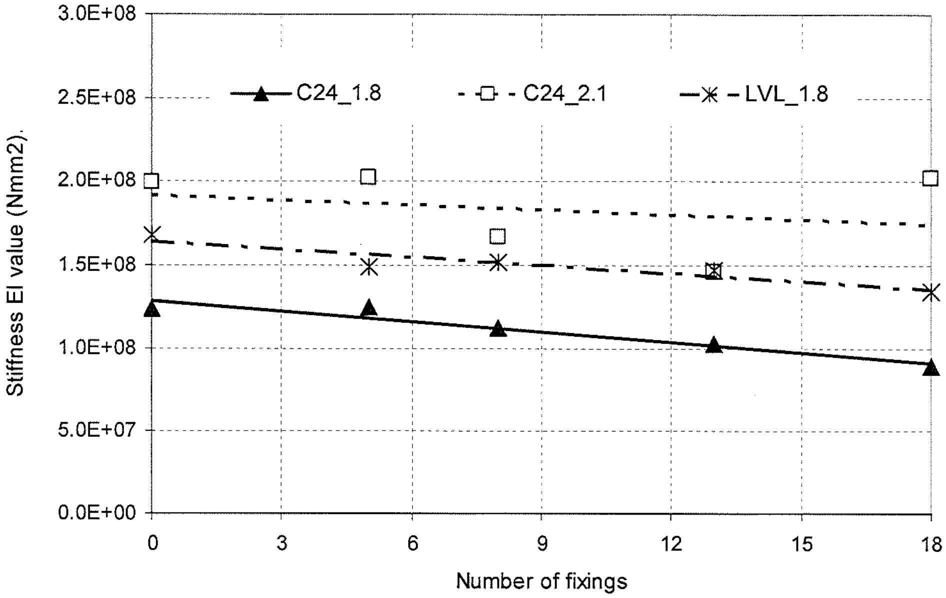


Figure 6.20 Stiffness against number of nails

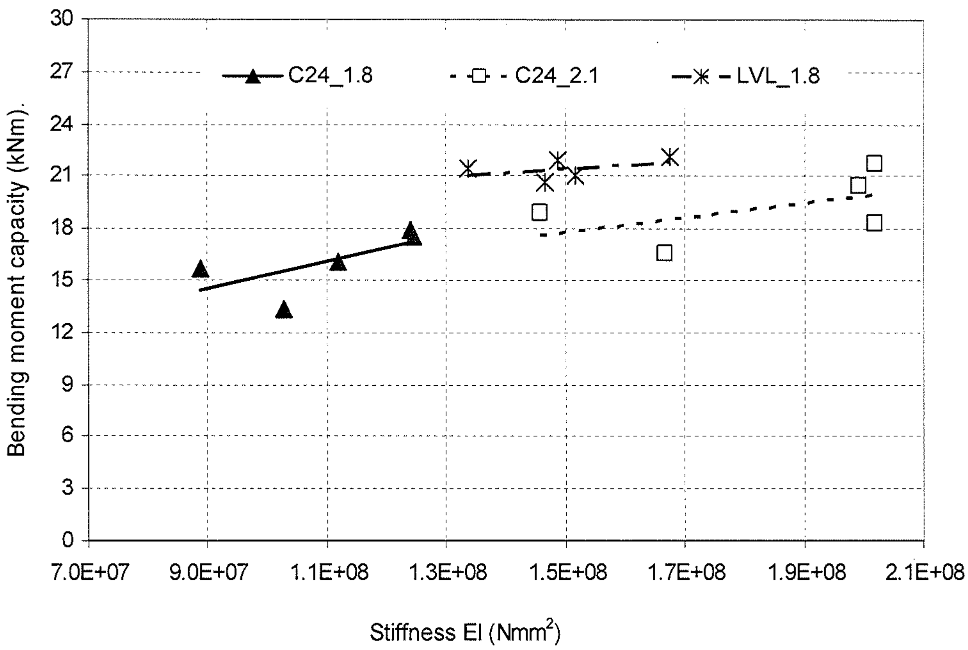
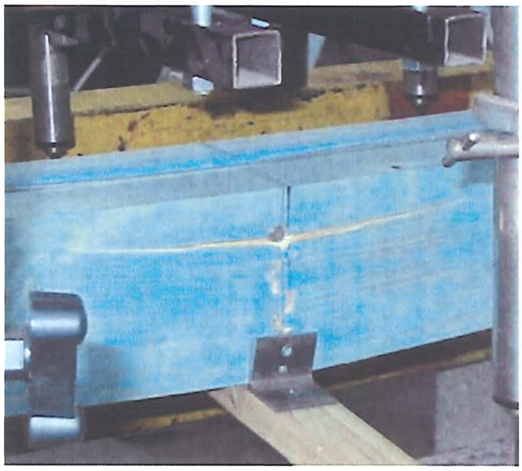


Figure 6.21 Bending capacity against stiffness

a) Split propagating from an area of high stress concentration in the bottom chord



b) Split propagating from an area of high stress concentration in the top chord

c) Buckling of steel in the top chord of an LVL flitch beam

d) Buckling of steel in the top chord of an C24 strength timber flitch beam

Figure 6.22 Examples of flitch beam failure conditions

Based on the findings of the experimental work the following conclusions are made:

1. As a result of increased number of nails tending to reduce both the strength and stiffness of flitch beams the number of nails specified in design should be the minimum required to transfer load to the steel plate per unit length.
2. Nails should be set-out such that their position does not correspond to areas of high stress concentration.
3. At high load out of plane buckling of the steel was observed in the top chord of the flitch beams due to compression forces. However, when the beams form part of a system additional restraint will be provided due to the connection with ancillary parts.

6.4.3 Forming a Standardised Nailing Pattern

To improve the two main conversion processes, design and production, a standardised nailing pattern generic to all flitch beams constructed was deemed as a requirement. A generic nailing pattern would reduce design time and improve the fabrication procedure by means of repetition. A review of industry standard house types which contained flitch beams was conducted and from the given load span information the required number of shot fired dowels was specified as the minimum required to transfer load to the steel plate per unit length. Contained in Table 6.5 is the revised specification and it is demonstrated in Figure 6.23 that the revised specification of shot fired dowels does not result in increased cost.

Table 6.5 Industry standard flitch beams and revised shot fired dowel specification

House type & beam number	Desig-nation	Timber element type C24	Beam Dimensions				Steel dimensions		Fixing	Nail spacing	
			Span (m)	Length (m)	Height (mm)	Breadth (mm)	Height (mm)	Thick-ness (mm)	Type	No	mm
Braemar 1	1	C24	2.43	2.73	190	45	180	6	SC9 60*	14	348
									Bolts	8	
Culzean 1	2	C24	2.30	2.60	190	45	180	6	SC9 60	14	329
									Bolts	8	
Holyrood 1	3	Intrallam	3.20	3.20	241	45	200	10	SC9 60	14	414
									Bolts	10	
Hopetoun 2	4	C24	1.85	2.08	190	45	180	6	SC9 60	12	300
									Bolts	6	
Hopetoun 3	5	Intrallam	2.60	2.60	241	45	200	10	SC9 60	16	300
									Bolts	8	
Rowan 1	6	C24	2.80	3.18	190	45	180	10	SC9 60	16	360
									Bolts	9	
Rowan 1	7	C24	4.40	4.63	190	45	180	10	SC9 60	14	618
									Bolts	14	
Maple 1	8	C24	2.50	2.80	190	45	180	10	SC9 60	14	358
									Bolts	8	
Cramond 1	9	C24	2.30	2.68	190	45	180	10	SC9 60	16	300
									Bolts	7	
Grange 1	10	C24	2.70	3.08	190	45	180	10	SC9 60	14	397
									Bolts	9	
Tamar 1	11	Intrallam	3.03	3.03	241	45	200	12	SC9 60	18	304
									Bolts	10	
Kielder 1	12	C24	1.89	2.19	190	45	180	6	SC9 60	10	379
									Bolts	7	
Kielder 2	13	C24	3.50	3.65	190	45	180	10	SC9 60	12	559
									Bolts	12	
MS3 Leithen 1	14	C24	2.79	3.17	190	45	180	12	SC9 60	12	478
									Bolts	9	
MS3 Leithen 2	15	Intrallam	2.95	2.95	241	45	200	6	SC9 60	12	442
									Bolts	10	

*Note: SC9 60 are 3mm diatmere 60mm long Spit nails

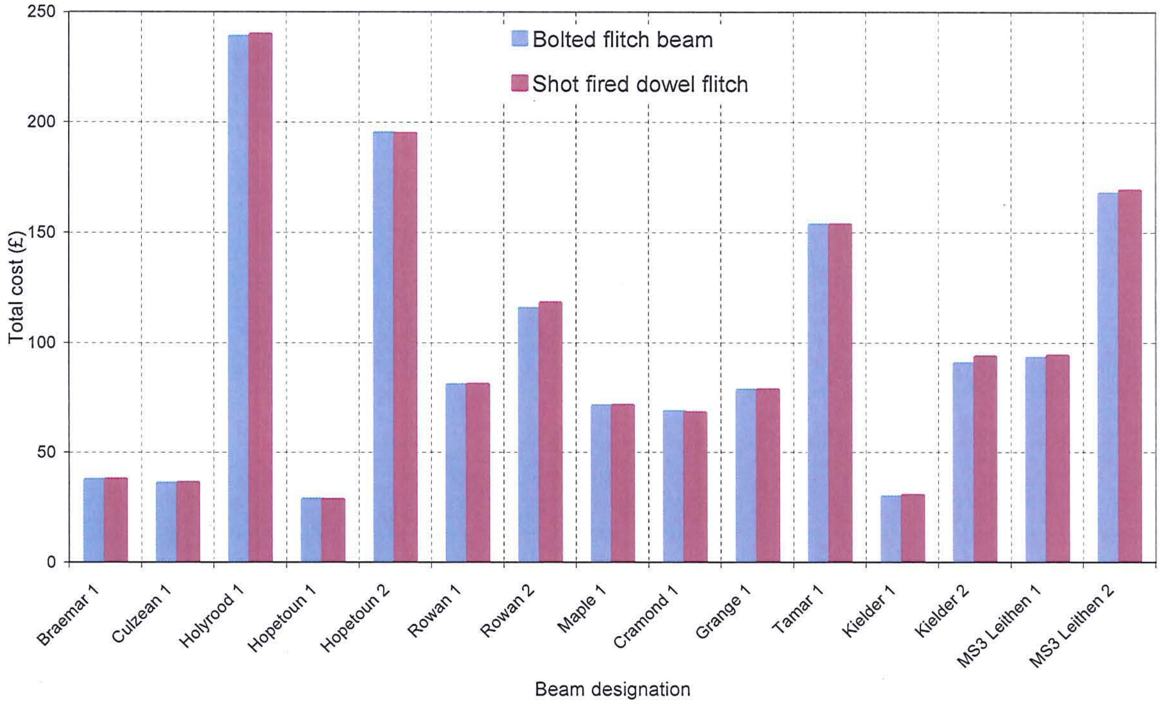


Figure 6.23 Flitch beam cost comparison (based on 2005 prices) (Refer to Table 6.5 for beam information)

As a result of the information contained in Table 6.5 the generic nailing pattern shown in Figure 6.24 was implemented and on each drawing the minimum number of nails to be applied would be specified. To improve the quality of the product, employees fabricating flitch beams would be instructed to try if practically possible and apply nails in a manner which did not coincide with areas of high stress concentration i.e. near knots at the top and bottom chord of the mid-span of a beam.

To allow for fabrication tolerances the steel plate element of the flitch beam will in normal circumstance not be the same height as the timber element but a distance α will be allowed between the elements. This tolerance ensures that the steel does not stand proud of the timber elements due to poor fabrication or shrinkage of the timber.

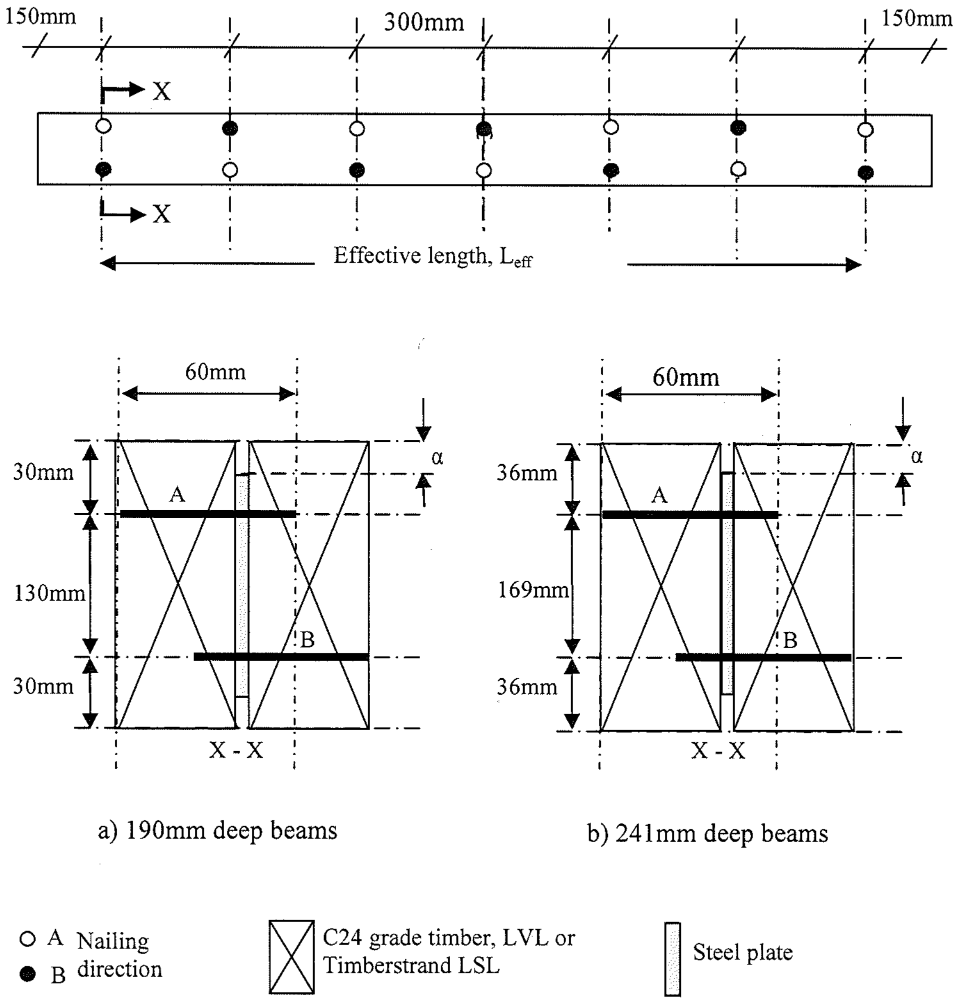


Figure 6.24 Standardised nailing specification

6.5 Laboratory study – Stiffness, Bending Strength and the effects of Shear

6.5.1 Introduction

The initial study considered flitch beams with steel and timber elements of the same height so that the effect of splitting of the timber due to nail application could be monitored. The purpose of the laboratory study documented in this section was to ensure the quality of flitch beams fabricated using shot fired dowels applying the optimised nailing specification.

A range of traditional flitch beams used in domestic dwelling were selected. The beams were selected such that they exhibited a range of steel plate and timber component sizes and consisted of both strength graded (C24) and engineered timber (Timberstrand LSL). These beams were then manufactured using the shot fired nail connection and an experimental programme was conducted.

The experimental programme consisted of testing the flitch beams in three and four point bending in accordance with BS EN 408:2003 to determine the following parameters:

1. Modulus of Elasticity in bending (MoE)
2. Bending Strength
3. Shear Modulus

The test determined values are compared with calculated results and recommendations are made to ensure the safe and robust design of shot fired dowel flitch beams for use in timber platform frame systems.

6.5.2 Flitch Beam Properties

Presented in Table 6.6 are the flitch beam sets which were tested, the corresponding timber and steel dimensions and nailing specification. Shown in Figure 6.25 are the nailing patterns employed and the cross sectional details.

Table 6.6 Flitch beam information

Set	No	Timber element					Steel		Nails	
		Type	Length (m)	Height (mm)	Breadth (mm)	No	Height (mm)	Thickness (mm)	No	Spacing mm
C24_6	3	C24	2.53	185	4	2	180	6	14	371
C24_10	3	C24	2.54	185	45	2	180	10	16	320
TS_10	4	LSL	3.00	241	45	2	220	10	16	386

Prior to the fabrication of the flitch beams the strength and stiffness properties of the elements used were quantified by means of testing. Each timber element was individually tested in 4 point bending within the elastic range in accordance with BS EN 408(2003) to determine the modulus of elasticity in bending of the beam in all 4 orientations. Figure 6.26 contains the set-up used and also demonstrates the 4 orientations (A, B C and D) about which the beams were tested. The processed results are shown in Figure 6.27 and these results have been used to produce Figure 6.28 and 6.29.

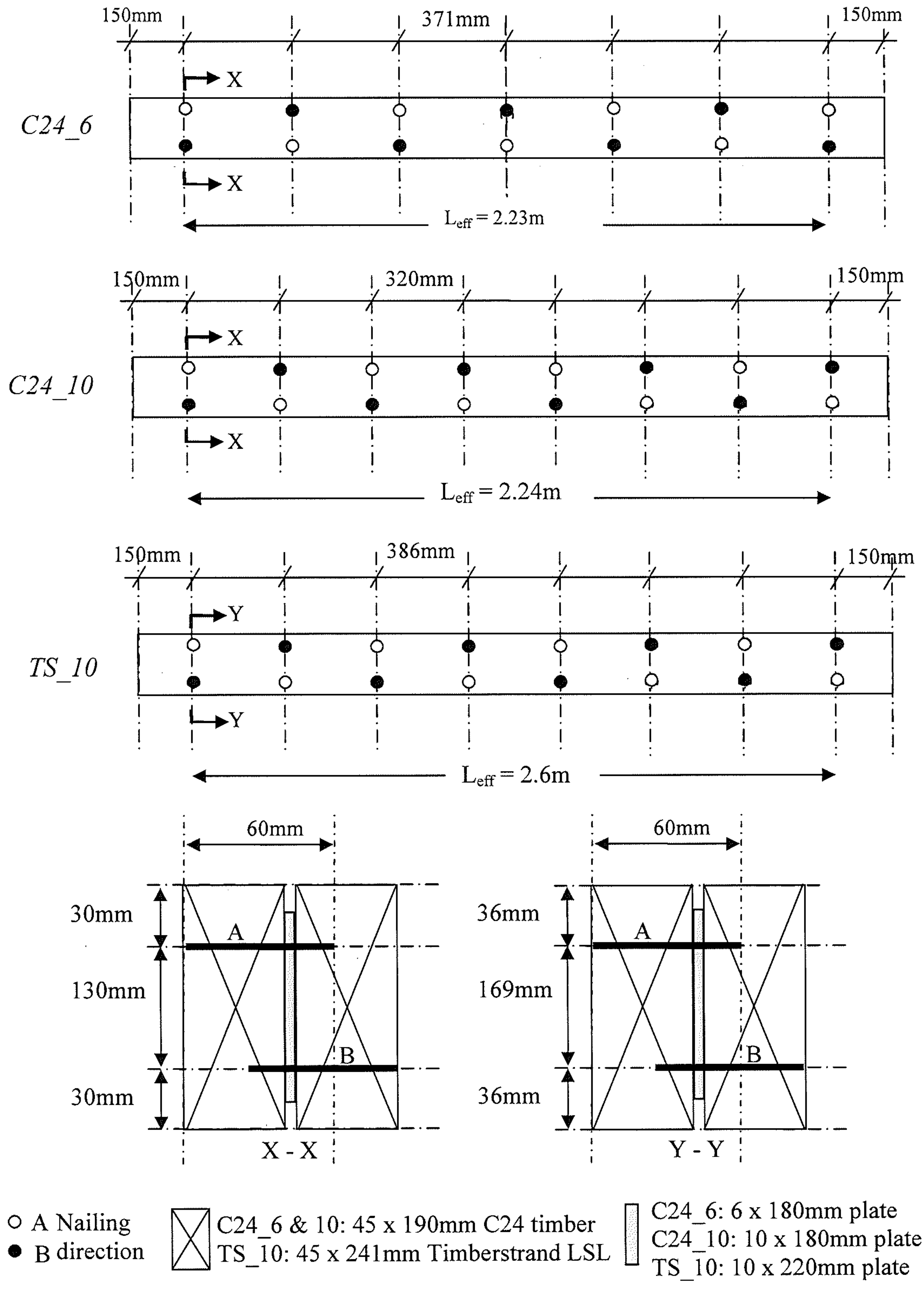
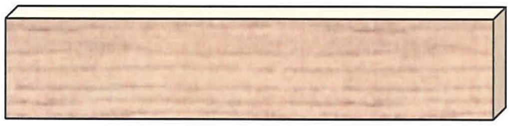
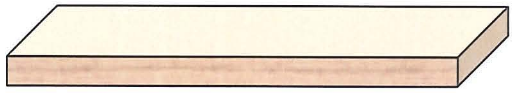


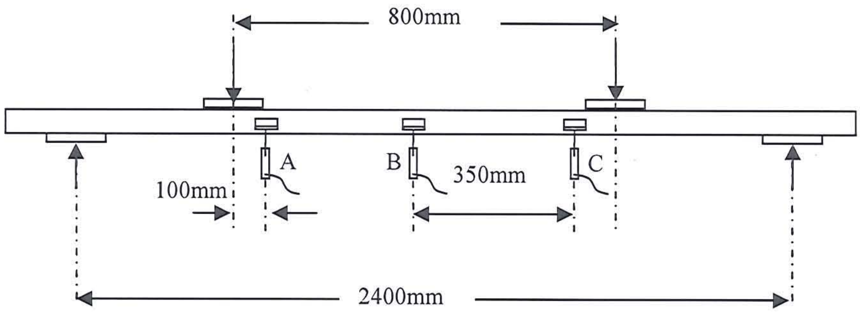
Figure 6.25 Flitch beam nailing patterns



a) Orientations A & B: edge wise

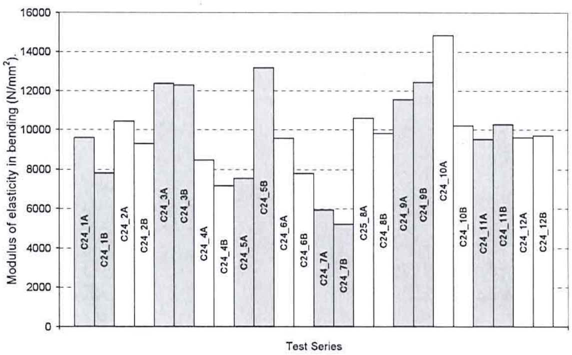


b) Orientations C & D: flat wise

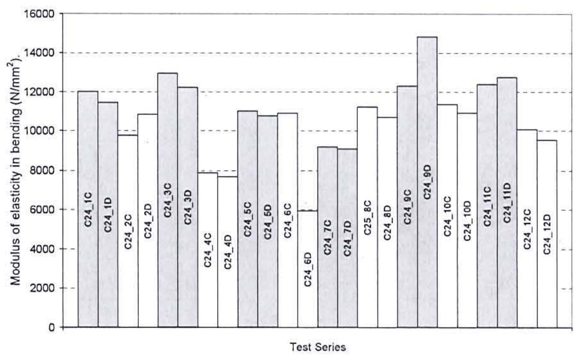


c) Test set-up

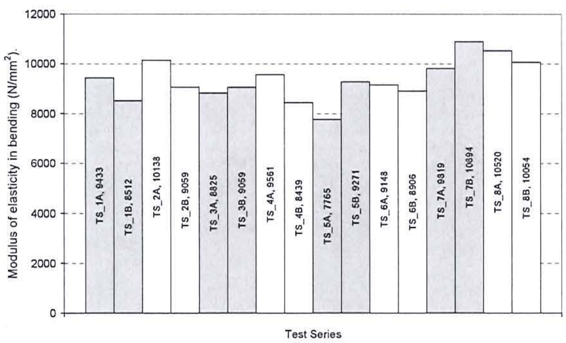
Figure 6.26 Four point bending orientations and test set-up



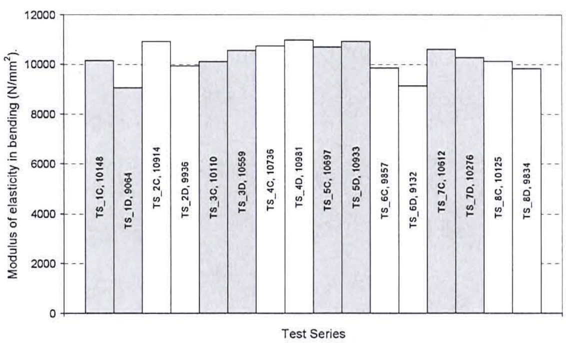
a) Modulus of elasticity of C24 timbers A & B orientations



b) Modulus of elasticity of C24 timbers C & D orientations



c) Modulus of elasticity of Timberstrand LSL timbers A & B orientations



d) Modulus of elasticity of Timberstrand LSL timbers C & D orientations

Figure 6.27 Modulus of elasticity of timber elements edgewise and flat wise

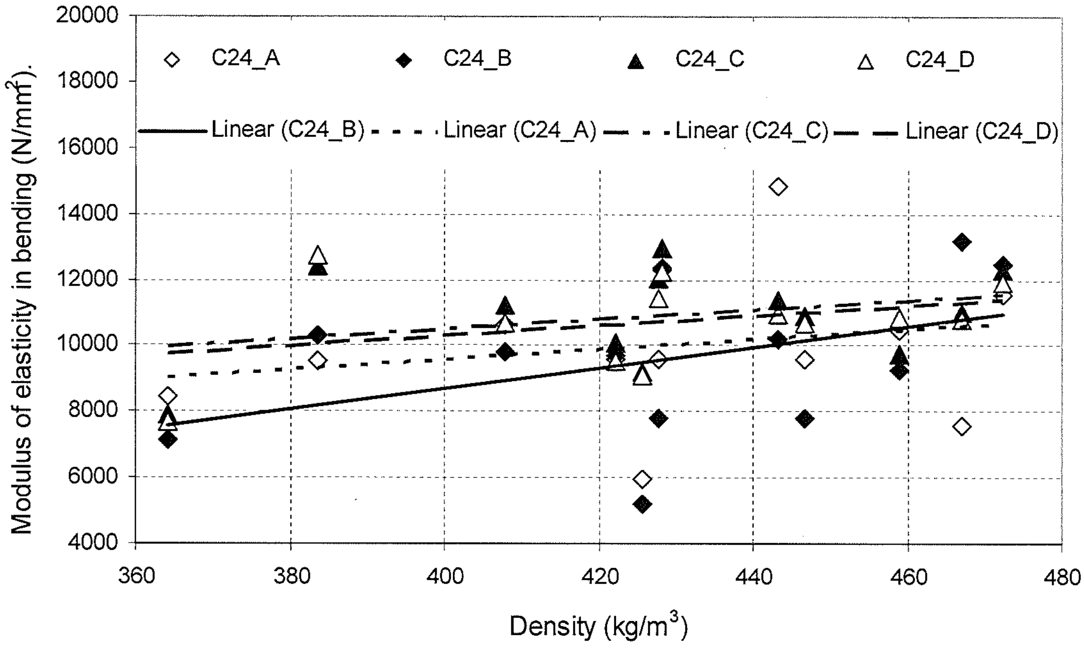


Figure 6.28 Variation in modulus of elasticity in bending of C24 elements with density about both edge wise and flat wise orientations

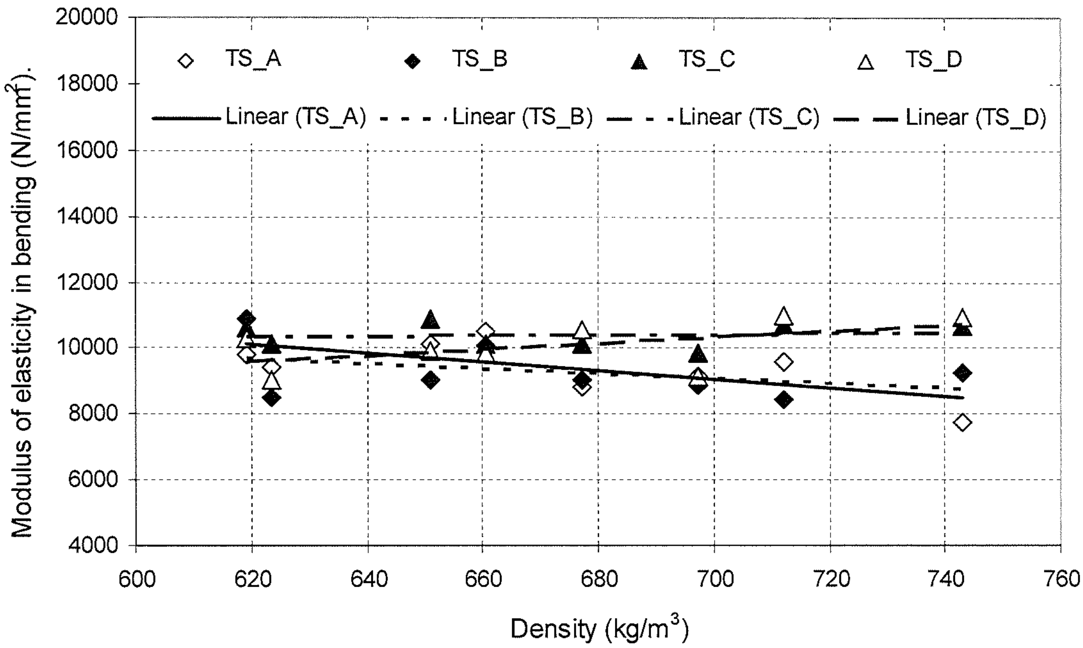


Figure 6.29 Variation in modulus of elasticity in bending of Timberstrand LSL elements with density about both edge wise and flat wise orientations

From the information contained in Figure 6.27 to 6.29 it is demonstrated that for the range of timber elements considered Timberstrand LSL in terms of stiffness is a more consistent material than C24 grade strength timber. It is also shown in Figure 6.28 that when considering C24 grade strength timber the modulus of elasticity in bending is directly proportionate to density. However, modulus of elasticity in bending of the Timberstrand LSL elements tested is demonstrated to be directly proportionate to the density when considering the flat wise direction and inversely proportionate when considering the edge wise direction. According to the literature of the manufacturer (i-Level, 2006) an increase in density should correspond with an increase in edgewise modulus of elasticity in bending as demonstrated in Table 6.7. Therefore, it is considered that the general negative trend which corresponds to a reduction in modulus of elasticity in bending with increasing density is specific to the sample range being considered and a larger range of beams could result in a positive trend line.

Table 6.7 Timberstrand LSL – Structural Use – 1.5 E & 1.7 E "S" Qualities (i-Level, 2006)

Timberstrand LSL Quality	Mean density	Mean modulus of elasticity in bending
	kg/m^3	N/mm^2
1.5E	650	10300
1.7E	690	11700

Using the information contained in Figure 6.27 the timber elements were paired and orientated on the basis of relative even stiffness, *EI* (Table 6.8). For the sets constructed using C24 strength timbers (C24_6 and C24_10) the timber elements were also paired in a manner which not only provided comparable MoE values but it also resulted in 3 beams in each set which, relative to each other, could be considered to have low, medium and high stiffness.

Considering the C24 timber elements the critical stress was determined. The critical stress is the stress induced on an element when the critical moment is applied, the critical moment being the bending moment at which instability takes place. The critical stress was therefore employed in the design calculations for the flitch elements consisting of C24 grade timbers. According to Choo (1995) the critical stress of the graded timber elements is determined by applying the following:

$$\sigma_{crit} = \frac{0.75 \cdot E \cdot b^2}{h \cdot l_{ef}} \qquad \text{Equation 6.5}$$

Where:

E is the modulus of elasticity in bending;

b is the breadth of the element;

h is the height of the element;

l_{ef} is the unrestrained length of the element.

For the Timberstrand LSL elements the bending strength was taken from the manufactures specification (i-Level, 2006).

One beam from each set was then selected and 6 strain gauges (3 on the top face and 3 on the bottom face) were applied at the middle of the beam on each flitch element so that the level of strain taking place in each element could be measured during testing (Figure 6.30).

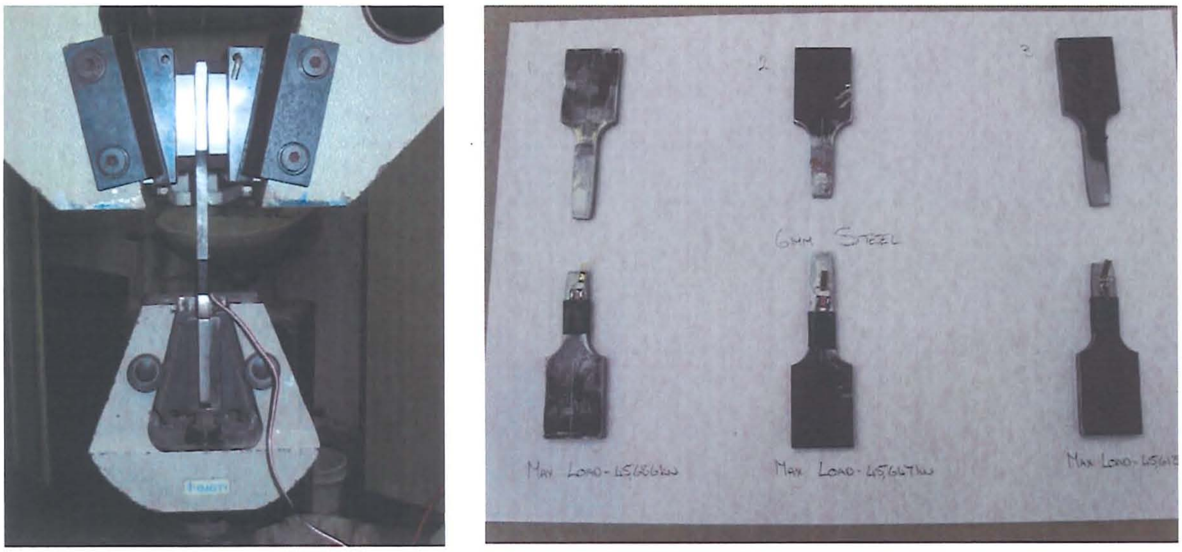
Table 6.8 Flitch beam element information

Test Set	No	Relative Stiffness, *EI*	Steel Thickness	Timber Elements				
				Type	MoE Individual	MoE Average	Shear Modulus, *G*	Critical Stress, σ_{crit}
			mm		N/mm^2	N/mm^2	N/mm^2	N/mm^2
C24_6	1	Low	6	C24	7821 7805	8697	544	27.50
C24_6	2	Medium	6	C24	9287 9803	10030	627	31.71
C24_6	3	High	6	C24	12291 11551	12165	760	38.46
C24_10	1	Low	10	C24	7172 5936	6697	419	21.16
C24_10	2	Medium	10	C24	9512 9604	9771	611	30.87
C24_10	3	High	10	C24	7560 10208	11447	715	36.17
								Bending strength, $f_{m,0,k}$
TS_10	1	N/A	10	Timberstrand LSL	8512 8439	8986	562	32.4
TS_10	2	N/A	10	Timberstrand LSL	9059 10054	9943	621	32.4
TS_10	3	N/A	10	Timberstrand LSL	8825 7765	8730	546	32.4
TS_10	4	N/A	10	Timberstrand LSL	8906 9819	9692	606	32.4

Strain gauges attached

Figure 6.30 Strain gauges at mid span

For the steel elements tensile tests were conducted (Figure 6.31) on samples from the steel plate in accordance with BS EN 10002-1:2001 and for accuracy strain gauges were used to measure the strain of the samples, this information was used to determine the MoE, yield strength and ultimate strength of the steel used, the results are shown in Table 6.9.



a) Test set-up

b) 6mm steel specimens after testing

Figure 6.31 Tensile testing of steel elements

Table 6.9 Test determined steel properties

Designation	MoE	Strength	
		Ultimate	Yield
	N/mm^2	N/mm^2	N/mm^2
6mm Steel Plate	207175	500	348
10mm Steel Plate	213148	423	275

6.5.3 Determination of Modulus of Elasticity & Bending Strength

To determine the modulus of elasticity and bending strength 4 point bending tests were carried out in accordance with BS EN 408(2003) with load and displacement measured via a data logger. Shown in Figure 6.32 is the test set-up and the corresponding load displacement graph with displacement taken as the difference between the average displacement of the 2 transducers at B minus the average displacement of the 4 transducers at A & C (bending of the beam between A & C is considered to be theoretically constant). The load displacement curve is the average of the test-set and it is to be recalled that C24_6 & 10 are tested over an effective span of 2240mm compared to TS_10 which is tested over an effective span of 2600mm.

As a result of the loading conditions the bending moment over the mid span of the beam is theoretically constant as the average displacement between the 4 transducers at A & C was calculated and the difference between this value and that of the average of the two transducers at B is used to produce the load (total applied load) against displacement curves contained in Figure 6.18. The curves contained are the average curves of the 3 beams tested in each set.

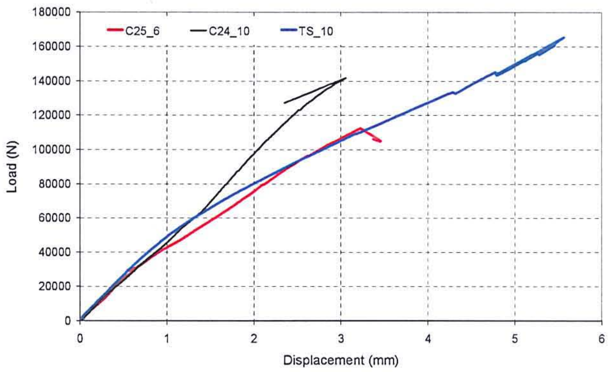
From the experimental results, which are contained in Table 6.10 and Table 6.11 the following relationships were examined (Figure 6.33, Figure 6.34, and Figure 6.35):

- Flitch beam stiffness with density of the timber elements.
- Flitch beam stiffness with MoE of the timber elements.
- Flitch beam bending capacity with density of the timber elements.
- Flitch beam bending capacity with MoE of the timber elements.
- Flitch beam bending capacity with stiffness EI of the flitch beam.

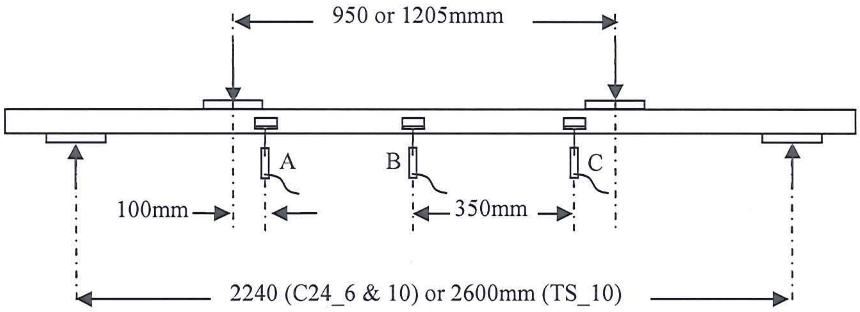
To evaluate the stiffening affect of the steel element the test determined stiffness, EI, and bending capacity of the flitch beams were also compared to calculated values using the transform section design method (see Appendix E for further information on the transform section method of design).

The comparative study used both test determined and design defined material properties. The design material properties for the graded timber elements are in accordance with BS EN 338(2003), for Timberstrand LSL the manufacturers literature has been used and the steel properties have been extracted from BS 5950: Part 1: 1990. In the case of bending capacity calculations have been carried out using both steel yield and ultimate strength values (Table 6.9).

a) Beam under test conditions



b) Load against displacement



c) Test set-up

Figure 6.32 Modulus of elasticity and bending strength test set-up

Table 6.10 Flitch beam stiffness in bending: experimental results and calculated values

Test		Experimental results	Calculations based on: Flitch element properties	Calculations based on: Design Values (BS EN 338 timber & S275 steel properties)	Ratio between experimental & design value
Set	No	Stiffness, EI (Nmm^2) $\times 10^{12}$			
C24_6	1	1.48	1.04		
	2	1.35	1.11		
	3	1.50	1.21		
	Average	1.44	1.12	1.18	1.22:1
C24_10	1	1.45	1.38		
	2	1.63	1.52		
	3	2.04	1.61		
	Average	1.70	1.50	1.59	1.07:1
TS_10				(Timberstrand LSL & S275 steel properties)	
	1	3.24	2.83		
	2	3.34	2.93		
	3	3.07	2.80		
	4	3.33	2.90		
	Average	3.23	2.86	2.94	1.09:1

Table 6.11 Flitch beam bending capacity: experimental results and calculated values

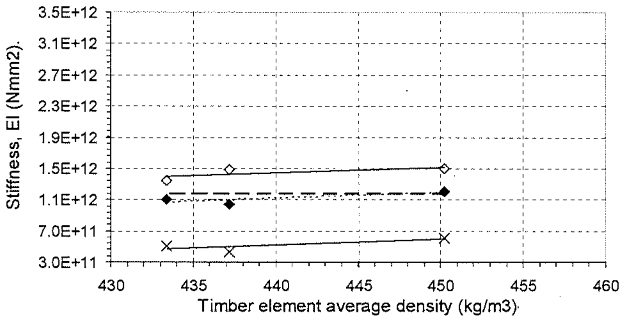
Test		Distance, *a*	Experimental bending capacity	Calculated bending capacity based on: Flitch element properties – Steel yield strength	Flitch element properties – Steel ultimate strength	Flitch – Design Values (BS EN 338 timber & S275 steel properties)	Flitch – % Diff between experimental & design value	Timber only – Design Values (BS EN 338 timber)	Timber only – Ratio between experimental & design value
Set	No	mm	kNm	kNm	kNm	kNm		kNm	
C24_6	1		31	19	28				
	2		29	21	30				
	3	545	35	23	32				
	Average		32	21	30	17	46%	13	2.46:1
C24_10	1		39	20	30				
	2		41	22	34				
	3	545	41	23	35				
	Average		40	22	33	23	43%	13	3.07:1
TS_10						(Timberstrand LSL & S275 steel properties)		(Timberstrand LSL)	
	1		53	33	51				
	2		60	34	53				
	3		56	33	51				
	4	698	63	34	52				
	Average		58	34	52	33	43%	28	2.07:1

◇ Experimental results

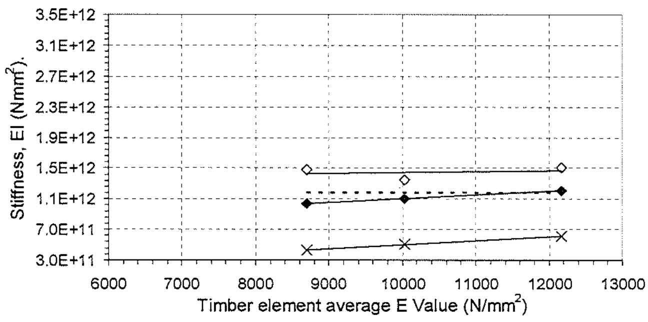
◆ Calculated results based on test determined flitch element properties

— — Calculated resutls based on design properties

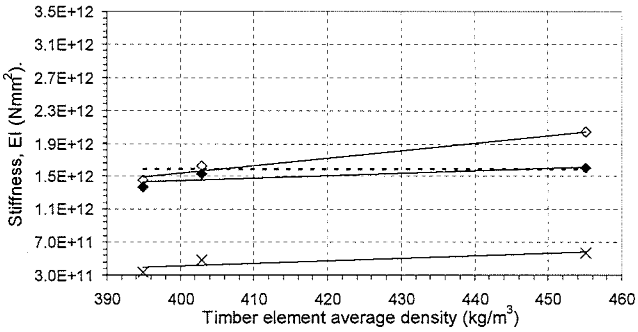
× Calculated C24 or LSL section only based on experimental properties



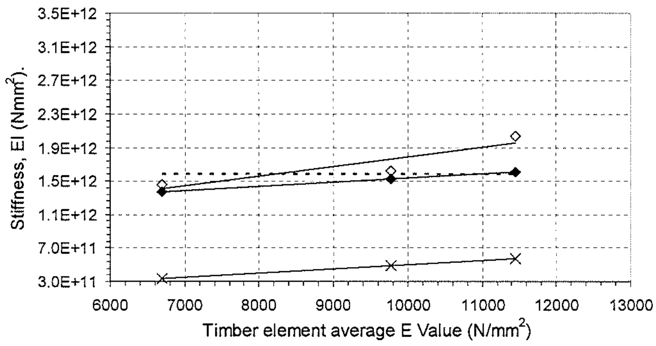
a) C24_6: Stiffness against timber density



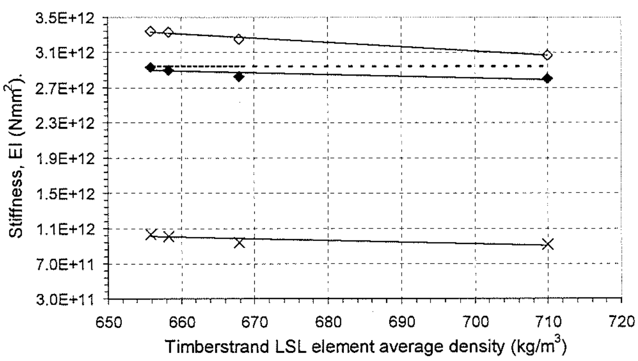
b) C24_6: Stiffness against timber MoE



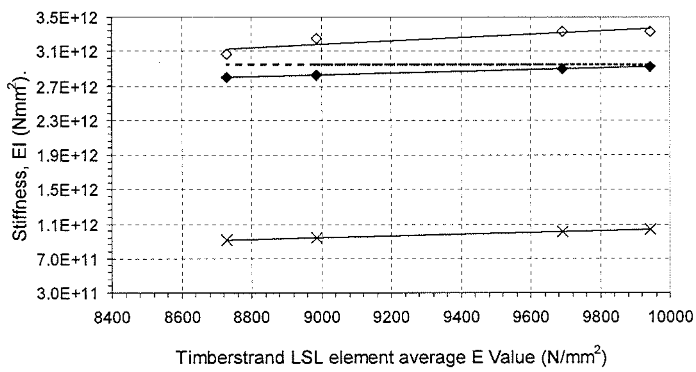
c) C24:10: Stiffness against timber density



d) C24:10: Stiffness against timber MoE

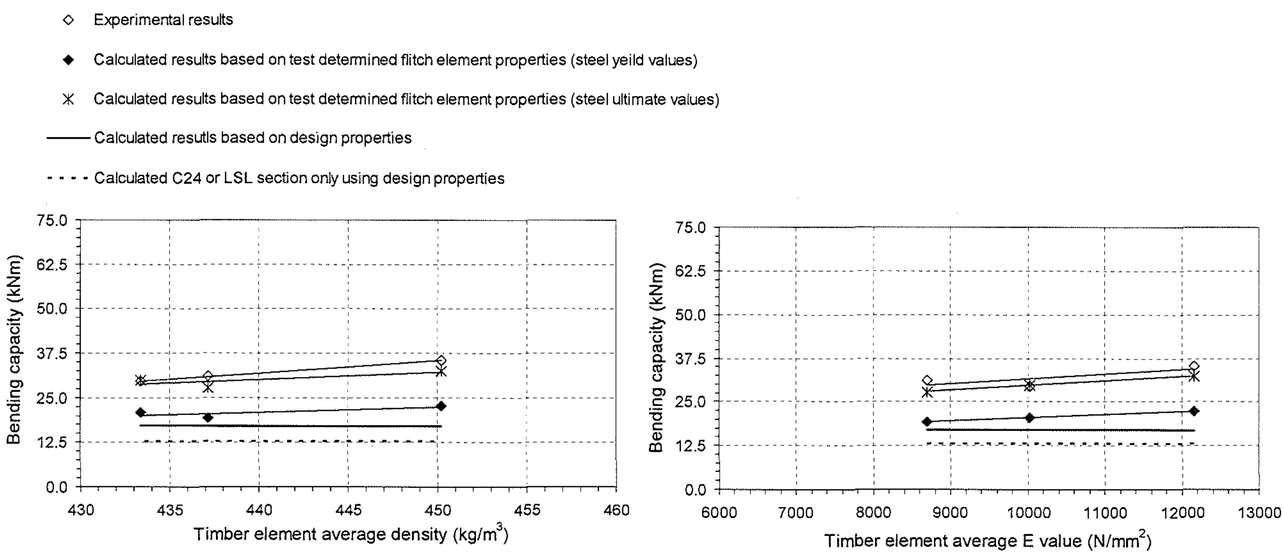


e) TS_10: Stiffness against timber density



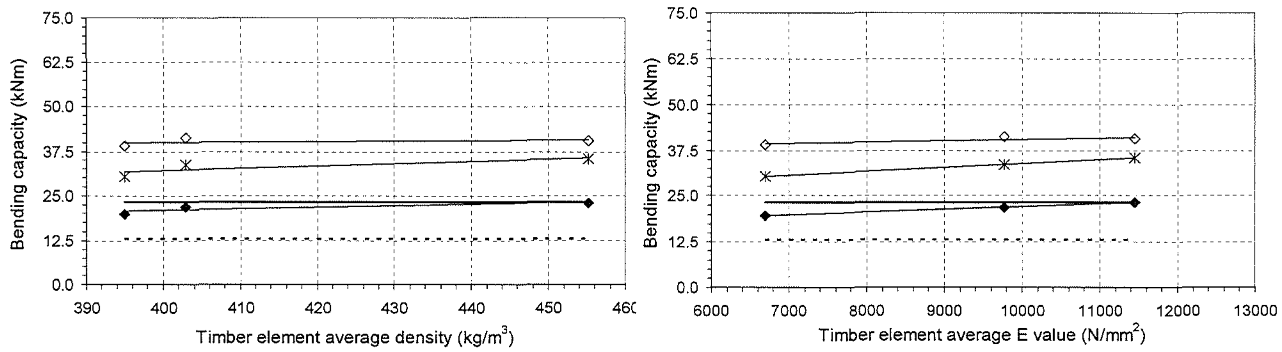
f) TS_10: Stiffness against timber MoE

Figure 6.33 Variation in flitch beam stiffness with density and MoE of timber elements



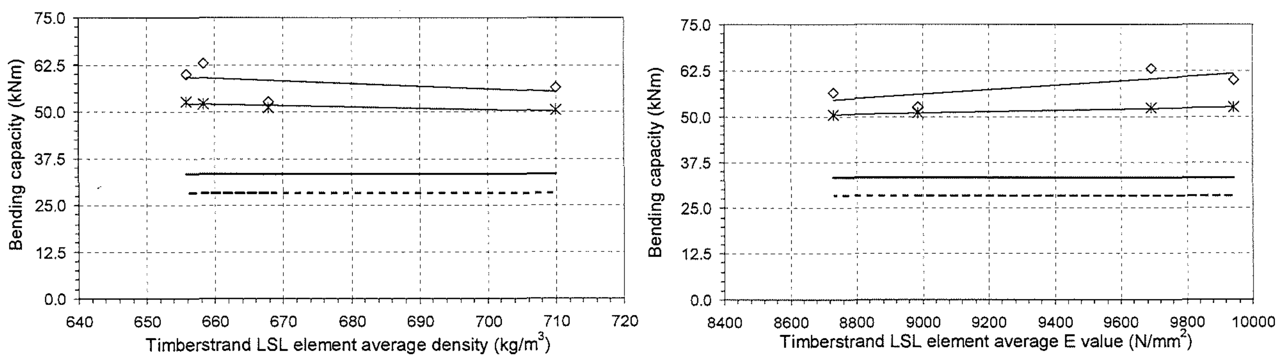
a) C24_6: Bending capacity against timber density

b) C24_6: Bending capacity against timer MoE



c) C24:10: Bending capacity against timber density

d) C24:10: Bending capacity against timer MoE



e) TS_10: Bending capacity against timber density

f) TS_10: Bending capacity against timer MoE

Figure 6.34 Variation in flitch beam bending capacity with density and MoE of timber

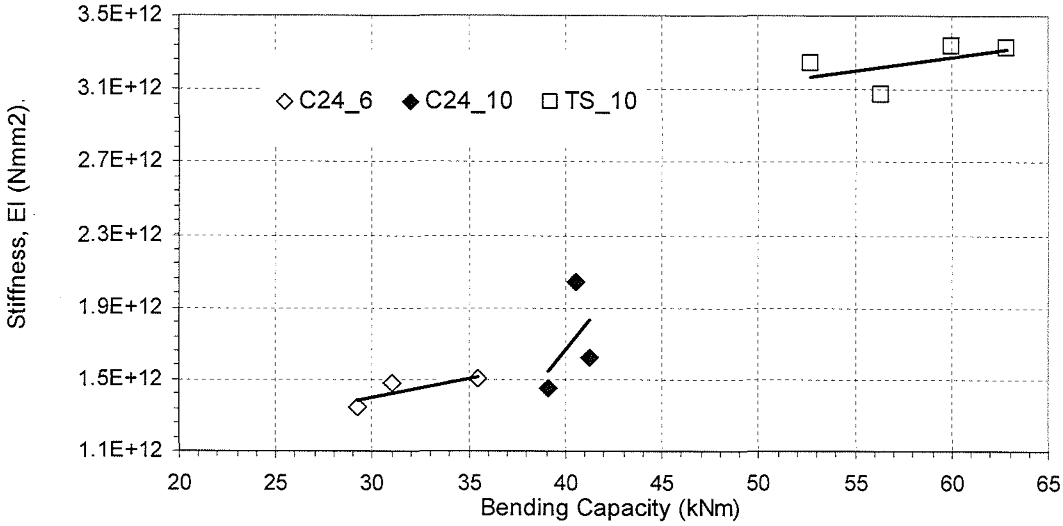


Figure 6.35 Flitch beam stiffness against bending capacity

Referring to Table 6.10 and Table 6.11 and Figure 6.33, Figure 6.34 and Figure 6.35 the following conclusions are drawn:

- The stiffness and bending capacity of flitch beams in bending constructed using C24 grade timber is directly proportional to the density and MoE of the timber elements.
- The stiffness and bending capacity of flitch beams constructed using Timberstrand LSL is indirectly proportional to the density and directly proportional to the MoE of the Timberstrand LSL elements. The reason for the inverse relationship between Timberstrand LSL density and the stiffness of a flitch beam is because the MoE of the Timberstrand LSL elements used in this study have been shown to be indirectly proportionate to density and this is demonstrated in Figure 6.33e & f and also in Figure 6.28.
- It is shown that in the majority of cases the stiffness of the tested flitch beams in bending is greater than the calculated stiffness considering steel and timber design properties. In fact there is only one instance, which is in the C24_10 flitch beam set, where the stiffness of a tested flitch beam is less than the calculated value. This is attributed to the fact that this particular underperforming flitch beam consists of timber elements which were of a particularly low average MoE (6697Nmm^{-2}) as compared with the design mean value for C24 grade timber (11000Nmm^{-2}).
- The stiffness of the test pieces in bending for all cases demonstrate improved performance compared to the stiffness values calculated based on the specific properties of the constituent elements.
- There is a minimum of 7% and a maximum of 18% improvement on design stiffness values and a minimum 51% and a maximum 68% improvement on design bending capacity values.

When considering the flitch beams constructed, and especially so when they are constructed from C24 graded timber, the level of percentage improvement will depend on the variability of the timber used.

- Flitch beam stiffness and bending capacity are directly proportionate.

6.5.4 Determination of Shear Modulus

To determine the shear modulus of the flitch beams the variable span method as prescribed in BS EN 408:2003 was used. The variable span method involves the determination of the apparent modulus of elasticity $E_{m,app}$ for each test piece over a number of spans with the same cross section at the centre. Shown in Figure 6.36 are the beam spans, load points and measurement point details of the shear modulus tests.

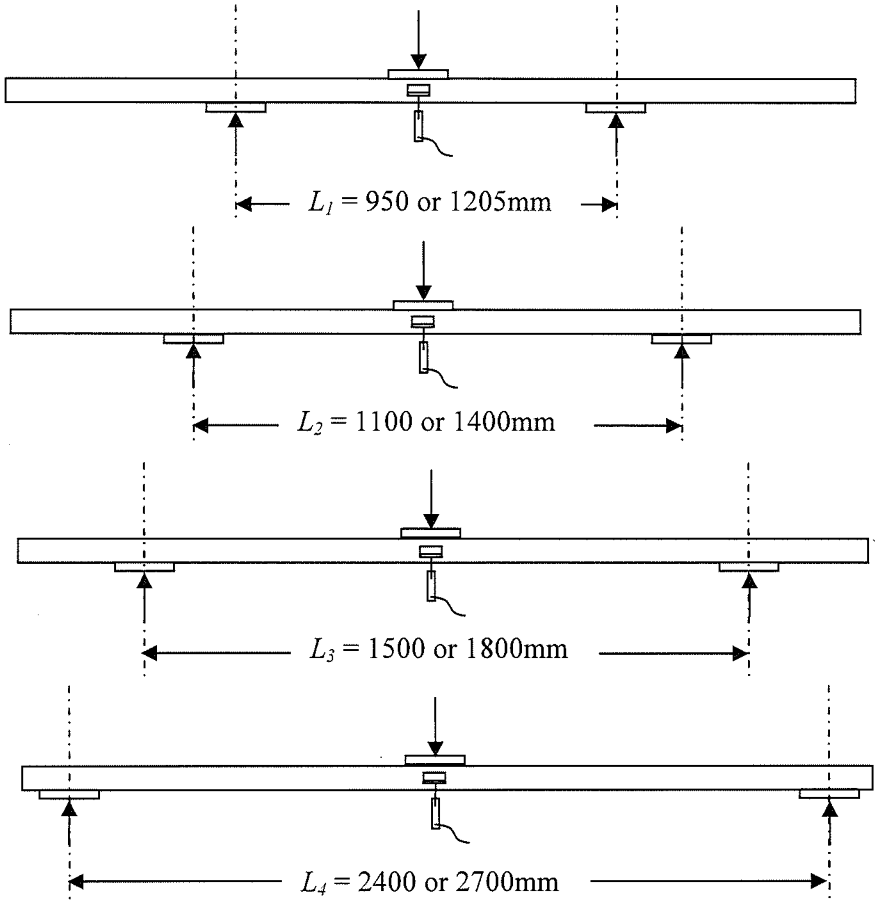


Figure 6.36 3 point bending test set-ups (smaller and larger dimensions are C24 and Timberstrand LSL flitch beams respectively).

In accordance with the processing method of BS EN 408:2003 the apparent modulus of elasticity, $E_{m,app}$, was determined for each test piece over each of the load span conditions applying the following equation:

$$E_{m,app} = \frac{l_1^{\ 3} \cdot (F_2 - F_1)}{48I \cdot (w_2 - w_1)} \qquad \text{Equation 6.6}$$

Where:

l_1 is the gauge length for the determination of modulus of elasticity as prescribed in BS EN 408:2003, in millimetres.

$F_2 - F_1$ is an increment of load on the straight line portion of the load deformation curve, in Newtons.

$w_2 - w_1$ is an increment of deformation corresponding to $F_2 - F_1$, in millimetres.

I is taken as the "transform" second moment of area (I value) of the section, converting the flitch beam into an equivalent timber section.

For each test piece, the values of $1/E_{m,app}$ were plotted against $(h/l)^2$ and the slope K_1 of the best-fit straight line through the points was determined. The shear modulus G was then calculated as follows:

$$G = k_G/K_1 \qquad \text{Equation 6.7}$$

Where k_G is the coefficient of shear modulus which according to Young and Budynas (2002) can be calculated for an I- or box section with flanges and webs of uniform thickness applying the following equation:

$$k_G = \left[1 + \frac{3(D_2^2 - D_1^2) \cdot D_1}{2D_2^3} \cdot \left(\frac{t_2}{t_1}\right)\right] \cdot \frac{4D_2^2}{10r^2} \qquad \text{Equation 6.8}$$

Where:

D_1 is the distance from the neutral axis to the nearest surface of the flange

D_2 is the distance from the neutral axis to the extreme fibre

T_1 is the thickness of the web

T_2 is the width of the flange

R is the radius of gyration with respect to the neutral axis

For a flitch beam Equation 6.8 is simplified to the following:

$$k_G = \frac{4D_2^2}{10r^2}$$ Equation 6.9

Where:

D_2 is the distance from the neutral axis to the extreme fibre

T_1 is the thickness of the steel plate

T_2 is the total width of the timber sections

R is the radius of gyration with respect to the neutral axis

The coefficient of shear modulus was therefore calculated applying Equation 6.6 the results of which are contained in Table 6.12 and for accuracy the experimentally derived E value of steel and timber sections for the given flitch beam were used to determine G.

Table 6.12 Coefficient of shear modulus

Test Set	No	Coefficient of shear modulus, k_G
C24_6	1	1.27
C24_6	2	1.26
C24_6	3	1.26
C24_10	1	1.30
C24_10	2	1.28
C24_10	3	1.27
TS_10	1	1.36
TS_10	2	1.35
TS_10	3	1.36
TS_10	4	1.35

Shown in Figure 6.37 are the $1/E_{m,app}$ against $(h/l)^2$ plots considering I transform for each of the beams tested and also shown is the average plot for each of the test sets, C24_6, C24_10 and TS_10. Using the average trend lines the G value of each beam type has been calculated in accordance with BS EN 408:2003 the results of which are contained in Table 6.13.

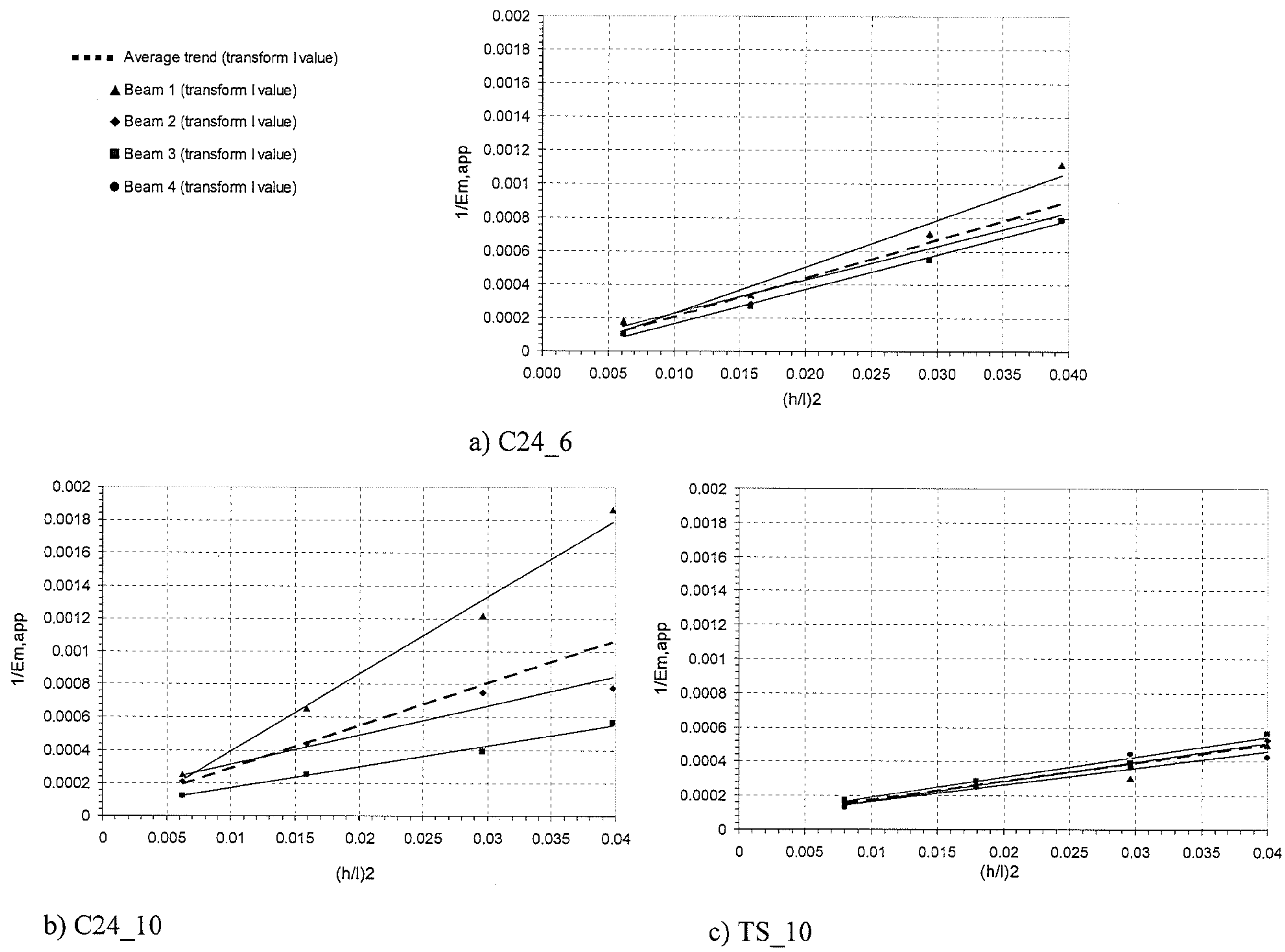


Figure 6.37 $1/E_{m,app}$ against $(h/l)^2$ plots

Table 6.13 Shear Modulus

Type	Shear modulus, G N/mm^2
C24_6	54.64
C24_10	48.52
TS_10	118.56

The standard to which the beams were tested to, BS EN 408:2003, is primarily for solid sections of structural timber and glue laminated timber. As a result, although test methods are applicable, the result processing methods may not be entirely satisfactory and for this reason a second method of resolving shear modulus, G, was used which required a degree of interpretation.

To evaluate the shear modulus, G, the following five equations were set up for each of the load/span conditions contained in Figure 6.37 and also considering the 4 point bending load span conditions illustrated in Figure 6.17:

$$\frac{L_1^{\ 3}}{48 \cdot EI} + \frac{\alpha \cdot L_1}{4G \cdot A} = \frac{1}{K_1}$$ Equation 6.10

$$\frac{L_2^{\ 3}}{48 \cdot EI} + \frac{\alpha \cdot L_2}{4G \cdot A} = \frac{1}{K_2}$$ Equation 6.11

$$\frac{L_3^{\ 3}}{48 \cdot EI} + \frac{\alpha \cdot L_3}{4G \cdot A} = \frac{1}{K_3}$$ Equation 6.12

$$\frac{L_4^{\ 3}}{48 \cdot EI} + \frac{\alpha \cdot L_4}{4G \cdot A} = \frac{1}{K_4}$$ Equation 6.13

$$\frac{L_5}{6EI} \cdot \left[\frac{3a}{4 \cdot L_5} - \left(\frac{a_5}{L_5} \right) \right] + \frac{\alpha \cdot a}{G \cdot A} = \frac{1}{K_5}$$ Equation 6.14

Where:

L_i is the span over which the beam was tested in mm.

EI is the stiffness of the flitch beam in Nmm^2.

K_i is the gradient of the load against deflection plot of the relevant test.

a is the distance between a loading position and the nearest support in a 4 point bending test.

A is the cross sectional area of the beam.

α is the shape factor calculated in accordance with (Young and Budynas, 2002).

$$\alpha = \frac{4D^2}{10r^2}$$ Equation 6.15

Where:

D is the distance from the neutral axis to the extreme fibre.

r is the radius of gyration of the section with respect to the neutral axis.

Using MathCAD the equations were combined and evaluated. The MathCAD operation evaluates by means of iteration balancing the equations until the smallest margin of error for each of the equations is returned, the output of which is the resolved EI and G value. This process was carried out for the 26 possible combinations of Equation 6.9 to Equation 6.13 and in so doing 26 solutions for *EI* and *G* were resolved, these are known as the "all inclusive" values.

On carrying out the evaluation process it was noted that certain equation combinations resulted in an *EI* output which was clearly seen to be in error, these *EI* values and corresponding *G* values were therefore removed. Values which were on inspection believed to be viable solutions to *EI* and *G* were maintained, these are known as the "selected" values.

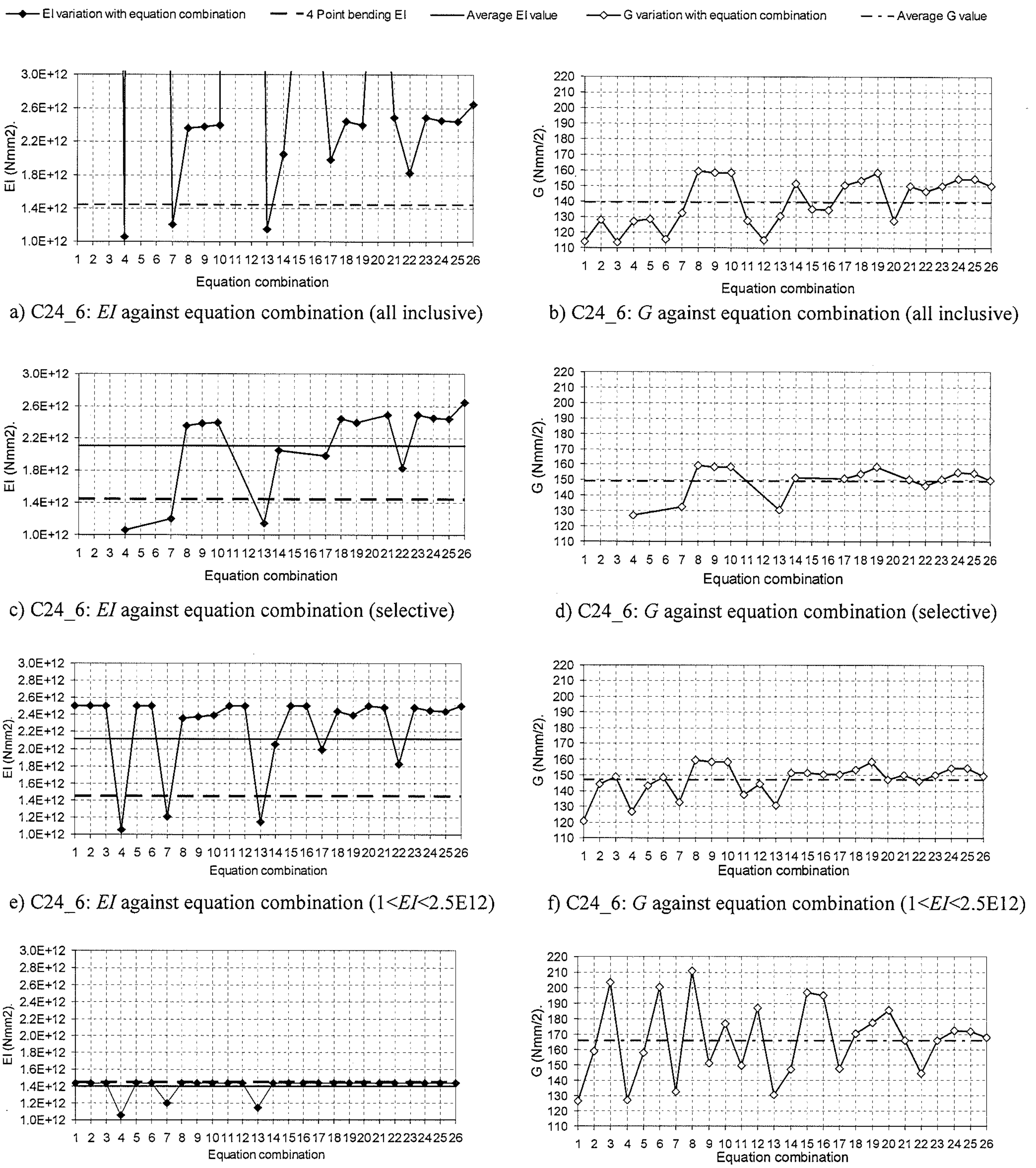
To refine the process of determining *G* defined limits of the *EI* values were placed within the iteration process. In the first instance the initial 26 equation solutions were examined and a representative limit was set:

- C24_6 *EI* limit = 2.5 x 10^{12} Nmm^2
- C24_10 *EI* limit = 2.7 x 10^{12} Nmm^2
- TS_10 *EI* limit = 4.5 x 10^{12} Nmm^2

Finally the EI value was limited in accordance with the EI value determined from the 4 point bending test (Table 6.10). The EI value from the 4 point bending test, due to the nature of testing and subsequent result processing method, should in theory not include any shear component and as a result provide the true EI value of the beam. Therefore, it is postulated that by limiting the Math CAD resolved EI solution to the 4 point bending test EI value a true reflection of G can be found. The following limits were therefore set:

- C24_6 *EI* limit = 1.44 x 10^{12} Nmm^2
- C24_10 *EI* limit = 1.70 x 10^{12} Nmm^2
- TS_10 *EI* limit = 3.22 x 10^{12} Nmm^2

Figure 6.38, Figure 6.39 and Figure 6.40 show the variation in *EI* and *G* relative to the equation combination for each of the flitch beam sets and contains a summary of the *G* value results. Tabulated results can be viewed in Appendix G.



a) C24_6: *EI* against equation combination (all inclusive)

b) C24_6: *G* against equation combination (all inclusive)

c) C24_6: *EI* against equation combination (selective)

d) C24_6: *G* against equation combination (selective)

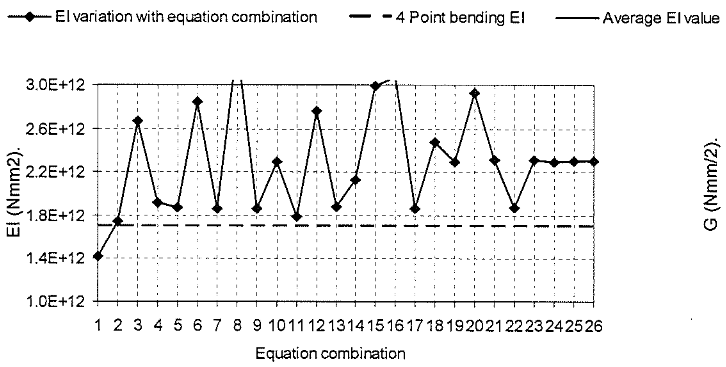
e) C24_6: *EI* against equation combination (1<*EI*<2.5E12)

f) C24_6: *G* against equation combination (1<*EI*<2.5E12)

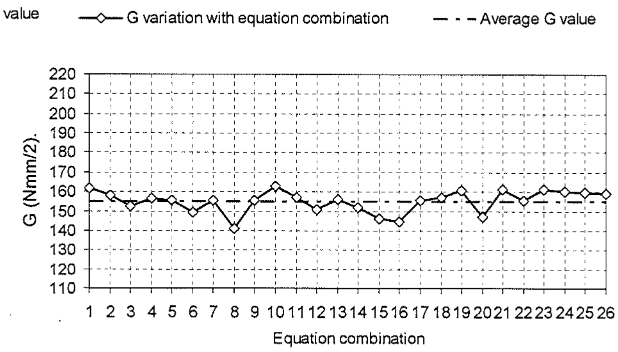
g) C24_6: *EI* against equation combination (1<*EI*<1.44E12)

h) C24_6: *G* against equation combination (1<*EI*<1.44E12)

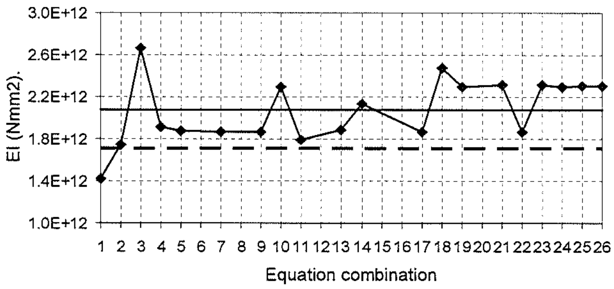
Figure 6.38 *EI* and *G* variations with equation combinations for C24_6 flitch beams



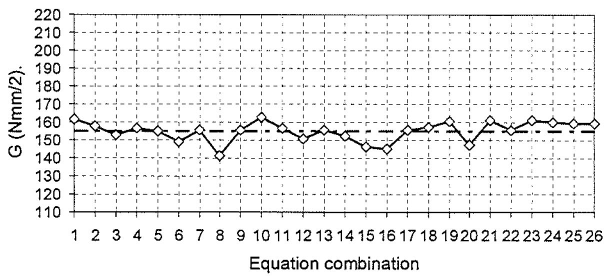
a) C24_10: *EI* against equation combination (all inclusive)



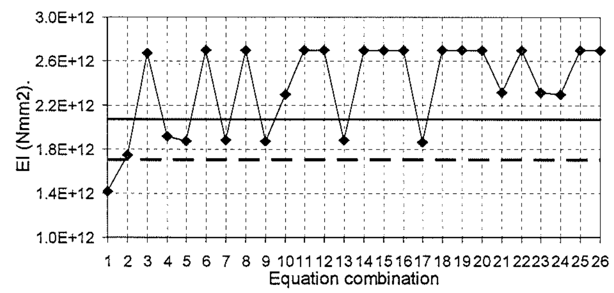
b) C24_10: *G* against equation combination (all inclusive)



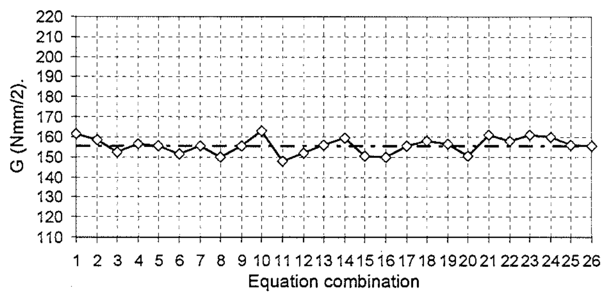
c) C24_10: *EI* against equation combination (selective)



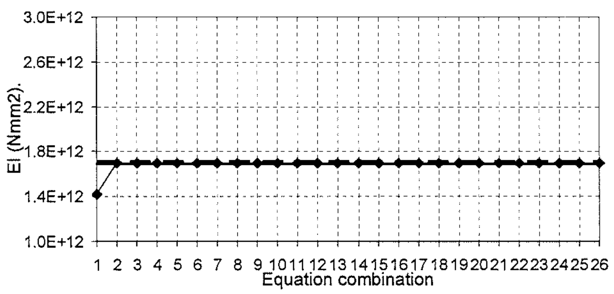
d) C24_10: *G* against equation combination (selective)



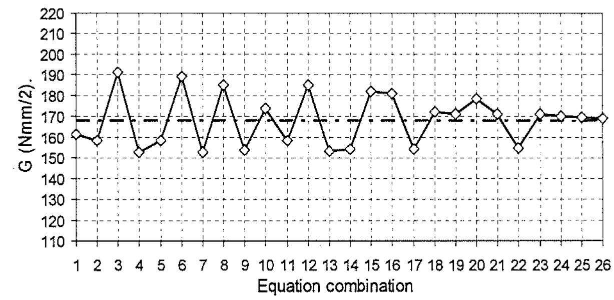
e) C24_10: *EI* against equation combination (1<*EI*<2.7E12)



f) C24_10: *G* against equation combination (1<*EI*<2.7E12)

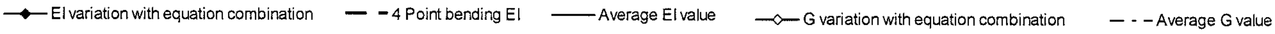


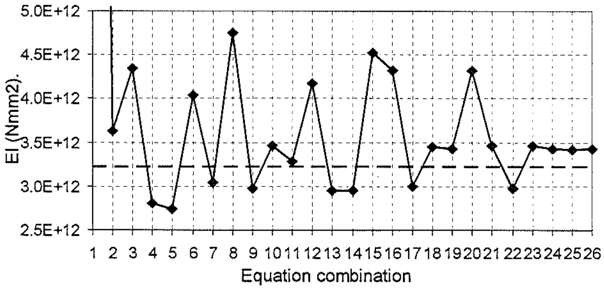
g) C24_10: *EI* against equation combination (1<*EI*<1.70E12)



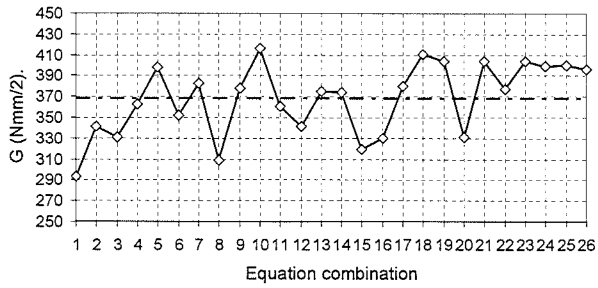
h) C24_10: *G* against equation combination (1<*EI*<1.70E12)

Figure 6.39 *EI* and *G* variations with equation combinations for C24_10 flitch beams

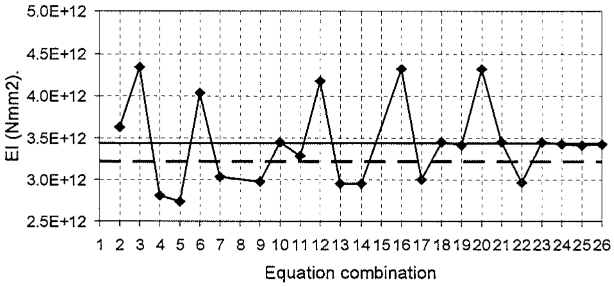




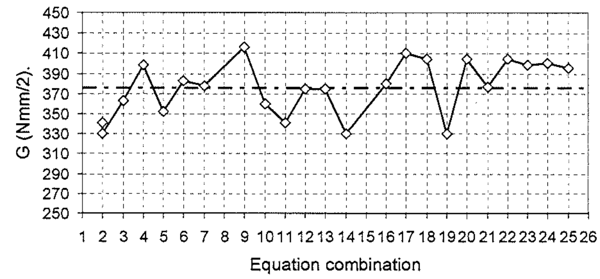
a) TS_10: *EI* against equation combination (all inclusive)



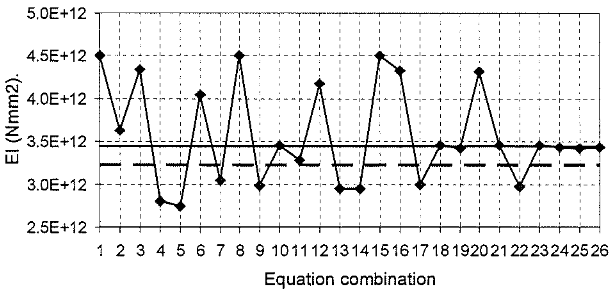
b) TS_10: *G* against equation combination (all inclusive)



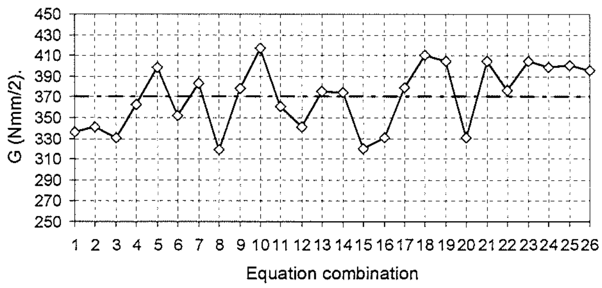
c) TS_10: *EI* against equation combination (selective)



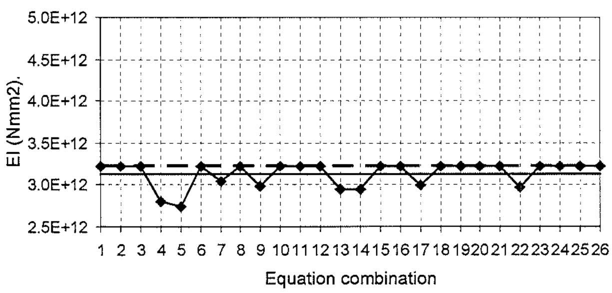
d) TS_10: *G* against equation combination (selective)



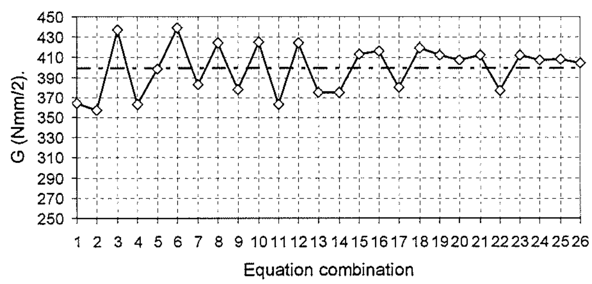
e) TS_10: *EI* against equation combination (1<*EI*<4.5E12)



f) TS_10: *G* against equation combination (1<*EI*<4.5E12)



g) TS_10: *EI* against equation combination (1<*EI*<3.22E12)



h) TS_10 *G* against equation combination (1<*EI*<3.22E12)

Figure 6.40 *EI* and *G* variations with equation combinations for TS_10 flitch beams

Table 6.14 Shear modulus G result summary

Type	Shear Modulus, G (N/mm^2)				
	BS EN 408:2003	Solving equation unknowns			
C24_6	*Transform*	*Inclusive*	*EI Selective*	*2.50E+12*	*1.44E+12*
	54.64	139.37	149.07	147.01	166.23
C24_10	*Transform*	*Inclusive*	*EI Selective*	*2.70E+12*	*1.70E+12*
	48.52	155.01	155.01	155.63	168.18
TS_10	*Transform*	*Inclusive*	*EI Selective*	*4.50E+12*	*3.22E+12*
	118.56	368.08	375.99	370.16	399.11

With due consideration to the results contained in Table 6.14, contained in Table 6.15 is the range of G values applied in the design calculations, the definitions of which are as follows:

- BS EN 408 determined is the Shear Modulus calculated for each flitch beam type by applying the method as prescribed in BS EN 408:2003 considering an equivalent I value which has been calculated in accordance with the transform section method (Appendix E) considering timber as the beam property.
- Equation Iteration is the Shear Modulus determined for each flitch beam by means of equation iteration.
- Calculated (E/16) is the Shear Modulus determined from the average timber element E value of the given flitch beam divided by 16.
- Design G_{mean} Value is the mean shear modulus according to design information (BS EN 338: 2003 and I-Level, 2006) for C24 and Timberstrand LSL respectively.

Table 6.15 Shear modulus G values

Type	Shear Modulus, G (N/mm^2)			
	BS EN 408:1995 (Transform I)	Equation Iteration	Calculated (E/16)	Design G_{mean} Value
C24_6	54.64	139.37	644	630
C24_10	49.49	155.01	582	630
TS_10	118.56	368.08	584	645

The total deflection of a beam subject to 3 or 4 point bending is as a result of two deflection components, Δ; deflection due to bending and deflection due to shear (Appendix H):

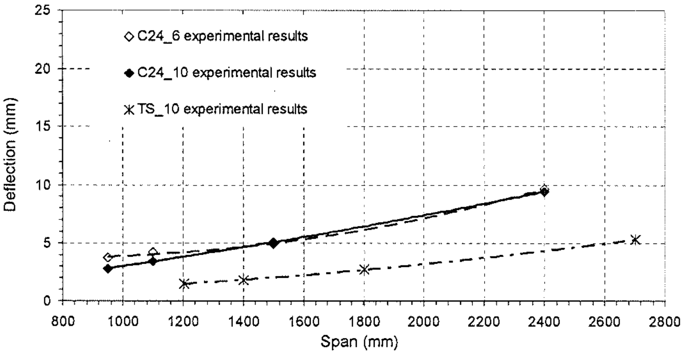
$\Delta_{total} = \Delta_{bending} + \Delta_{shear}$ ....Equation 6.16

Considering the above Figure 6.41 and Figure 6.42, which compares the 3 point and 4 point bending test results respectively, have been produced based on the following:

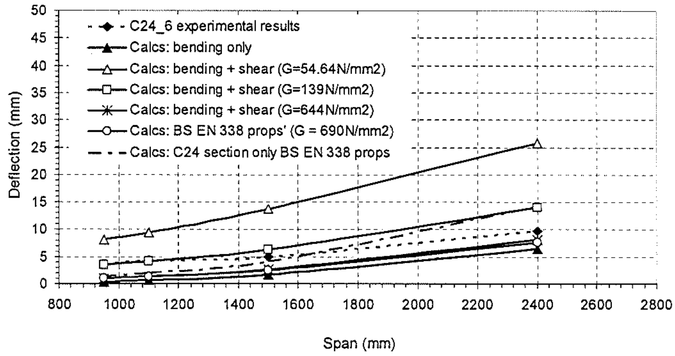
- Experimental results: are the combined bending and shear deflection results as measured from the tests conducted on the flitch beams for the given load span conditions.
- Calculated with BS EN 408:2003 determined *EI*: are the deflection results as calculated using the stiffness value, *EI*, as determined from the tests conducted on the flitch beams. This has been done for the bending component of deflection only and the combined bending and shear deflection components considering the range of shear modulus, *G*, values as given in Table 6.15.
- Calculated BS EN 338:2003 and i-Level: are the deflection results as calculated using the mean shear modulus for C24 and Timberstrand LSL as given in BS EN 338:2003 and by i-Level (2006) respectively.

Notes:

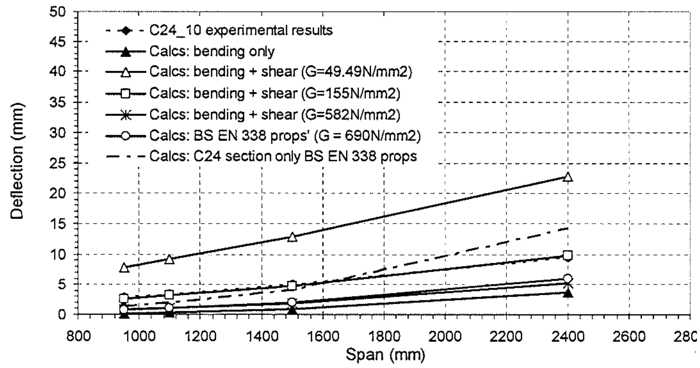
- The reason for the Timberstrand LSL section only and TS_10 flitch beams showing a higher degree of stiffness relative to the C24 sections and constructed flitch beams is as a result of increased depth of section from 190mm to 241mm deep.
- Refer to Appendix H for tabulated results.



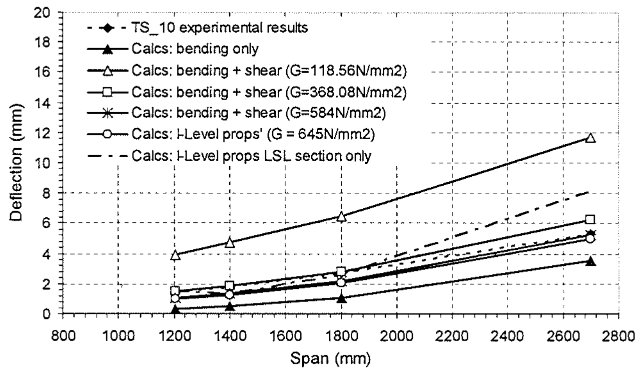
a) Experimental results



b) C24_6



c) C24_10



d) TS_10

Figure 6.41 3 point bending experimental results compared with design calculations

Referring to Figure 6.41 the following conclusions can be drawn from the 3 point bending test results and analysis:

- For C24_6 & 10 flitch beams, when span is short and shear is the dominant deflection component, the employment of a low *G* value results in good correlation between test and calculated deflection. However, as span increases the correlation between test and calculated results reduces as employing a low *G* value results in an overestimation of deflection as bending becomes the more dominant deflection component.
- For design a conservative approach should be adopted when shear is the dominant deflection component and a low *G* value, when considering flitch beams constructed from graded timber, should be employed.
- When comparing the deflection of TS_10 flitch beam test results to calculated values using the manufacturer defined material properties good correlation is demonstrated. Therefore, it is safe to use manufacturer defined properties when calculating the deflection of flitch beams constructed from Timberstrand LSL.

- It is postulated that the reason for shear deflection proving to be a more critical factor for beams constructed from C24 timber sections is as a result of nails tending to split solid section timber more readily than an engineered product. As a result of the splitting the ability of the timber element to carry longitudinal shear forces is reduced which in turn results in increased shear deflection.
- Further to the above point the method of connection is to be considered. The applied shear force will result in embedment and slip at the points of connection which will increase shear deflection.

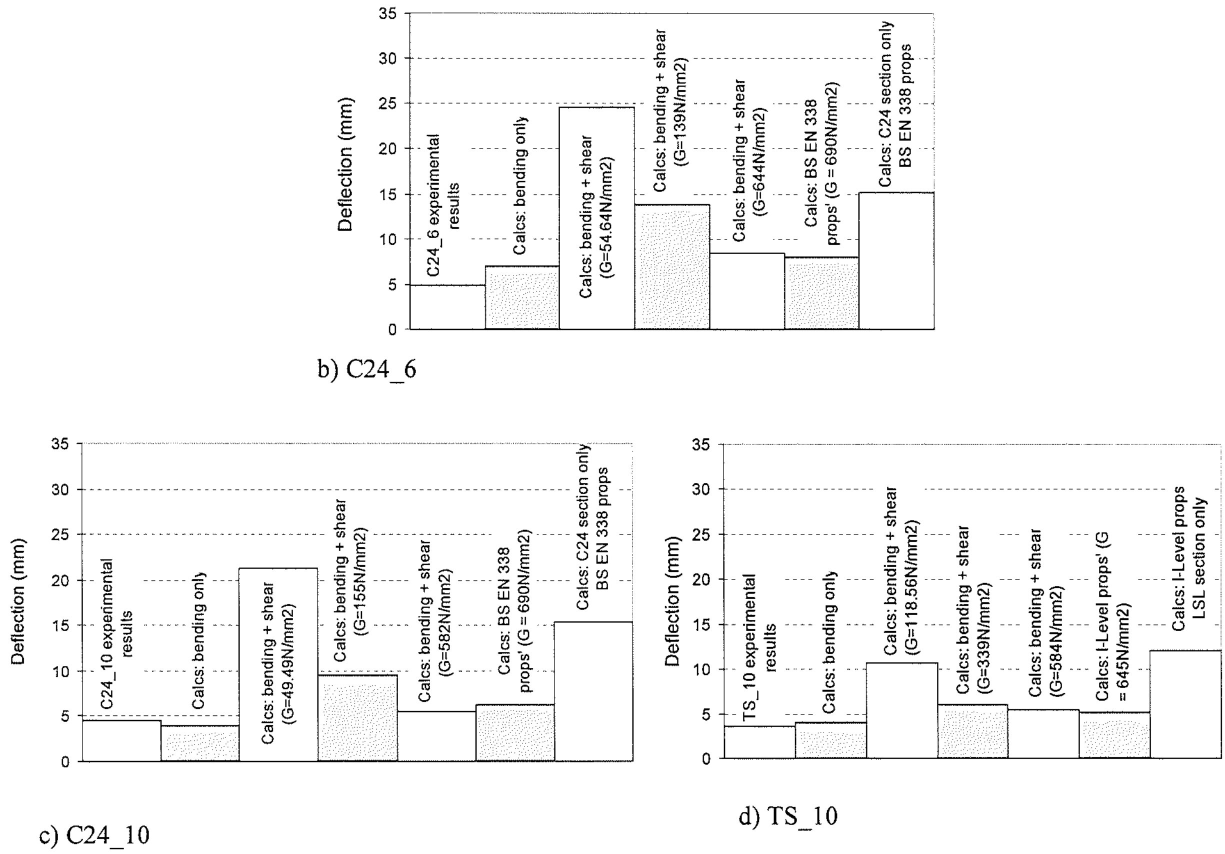


Figure 6.42 4 point bending experimental results compared with design calculations

Referring to Figure 6.42 the following conclusion is drawn from the 4 point bending test results and analysis:

- As a result of bending being the dominant deflection component the test results compare well with the calculated results, with C24_10 flitch beams being the only case where the bending only deflection result is an underestimation.

6.5.5 Stress Distribution and Beam Stiffness

During the experimental programme strain gauges were used to measure the strain of the constituent elements of selected test specimens. C24_6_3; C24_10_3 and TS_10_1 were selected, the properties of which can be viewed in Table 6.8. These flitch beams were selected as a result of the timber elements having relatively the same stiffness.

During both the 4 point and 3 point bending tests the flitch beams were bent about their neutral axis at a radius of curvature, R. As a result of the composite action even strain in each of the constituent parts of the beam, at any point above or below the neutral axis, will take place. A total of 6 strain gauges, placed at the mid-span of the flitch beams on both the top and bottom faces of all three elements, measured the strain of the elements via a data logger. From the data logged, load against strain plots were produced, the 4 timber strain gauges were used to determine the average strain of the timber elements and the 2 steel strain gauges were used to determine the average strain of the steel elements.

For comparative purposes a calculated trend was produced to demonstrate whether beams were acting as theoretically predicted. To produce the calculated trend transform section was used to determine the apportioned stresses to the elements of the beams, based on the known loading conditions and element properties:

$$\sigma_{mdt} = \frac{M}{W_{yt}} \times \frac{EI_t}{\Sigma EI} \qquad \text{Equation 6.17}$$

Where:

σ_{mdt} bending stress in timber element.

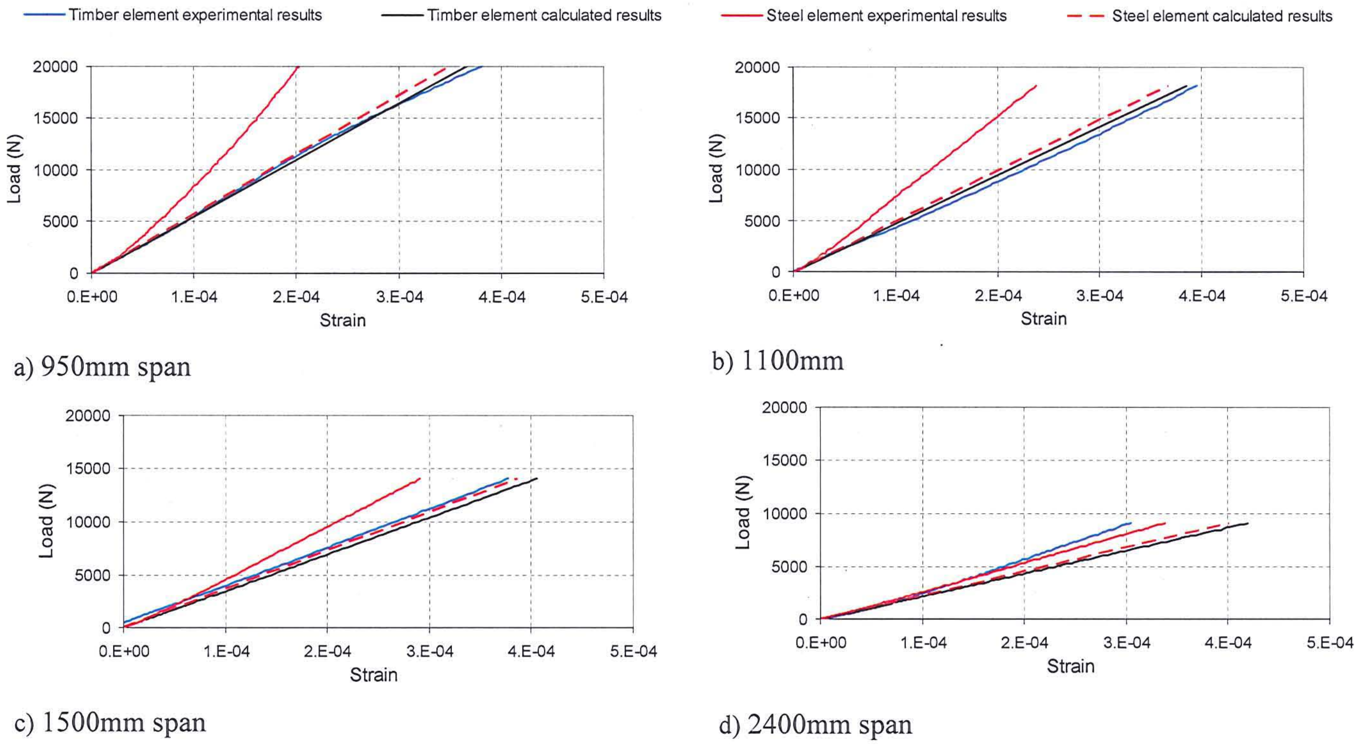
M is the resulting moment from the applied loading.

W is the section modulus.

EI_t is the combined, experimentally determined, average stiffness of the two timber elements.

ΣEI is the total stiffness of the flitch beam.

By calculating the apportioned stress to the elements and knowing that strain, ε, is equal to stress, σ, times the modulus of elasticity in bending, E; strain of the elements relative to the applied load was calculated. Used within the calculation process for accuracy are the experimentally determined material properties. Figure 6.43 to Figure 6.46 compare calculated load against strain with experimentally determined load against strain.



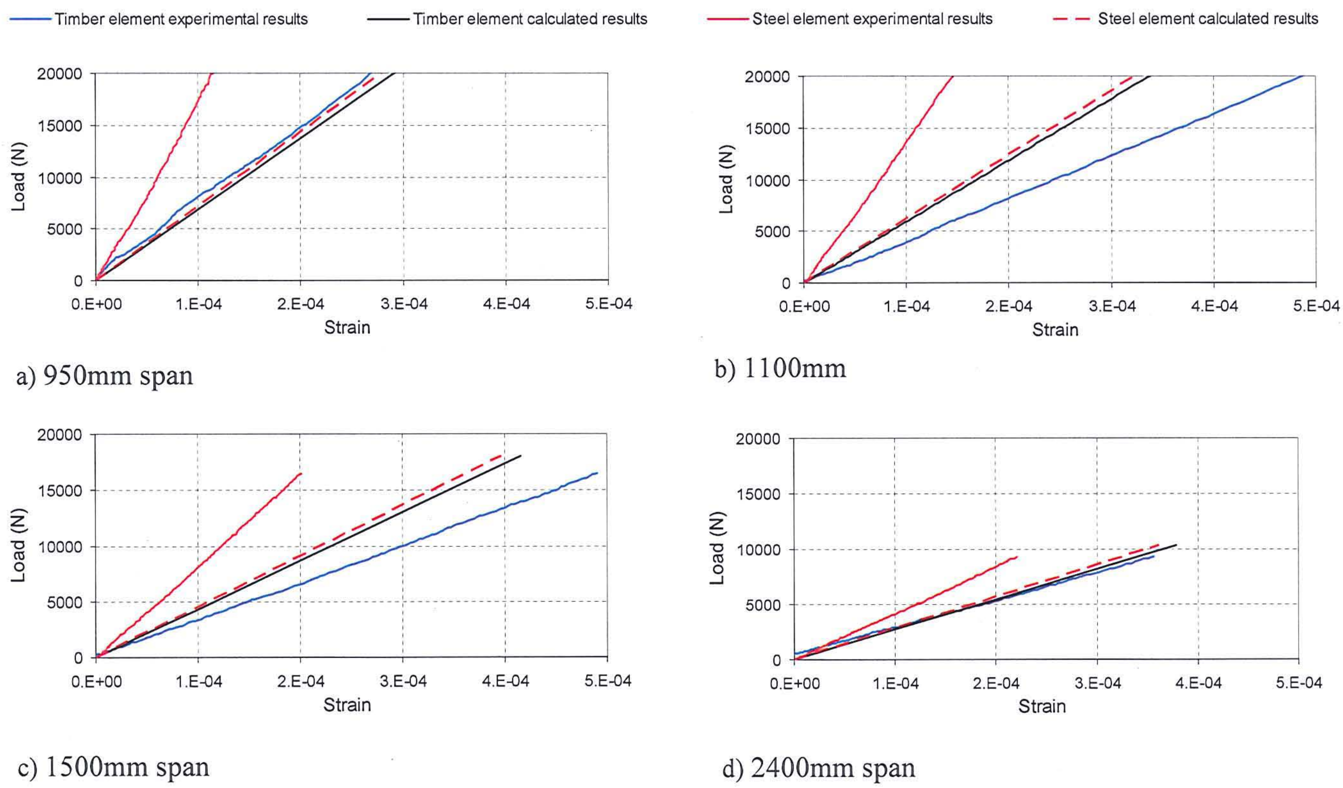
a) 950mm span

b) 1100mm

c) 1500mm span

d) 2400mm span

Figure 6.43 C24_6 load against strain in 3 point bending



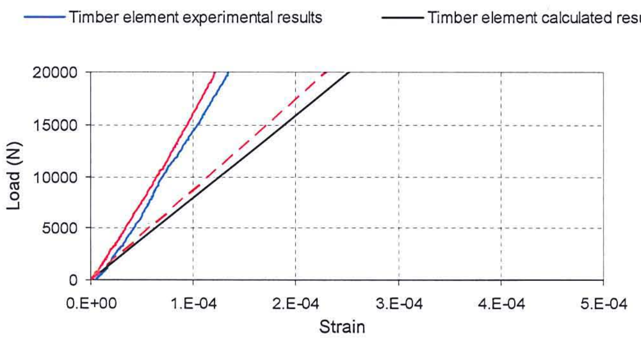
a) 950mm span

b) 1100mm

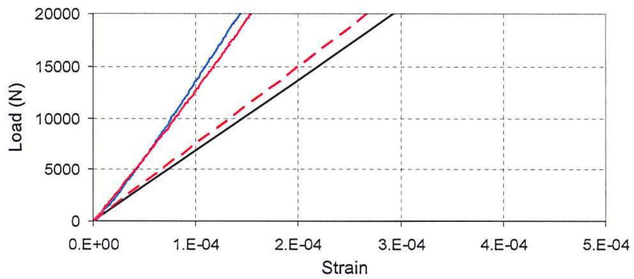
c) 1500mm span

d) 2400mm span

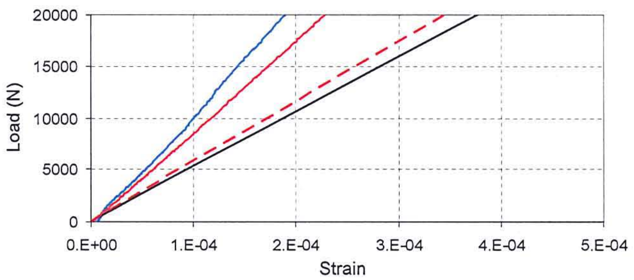
Figure 6.44 C24_10 load against strain in 3 point bending



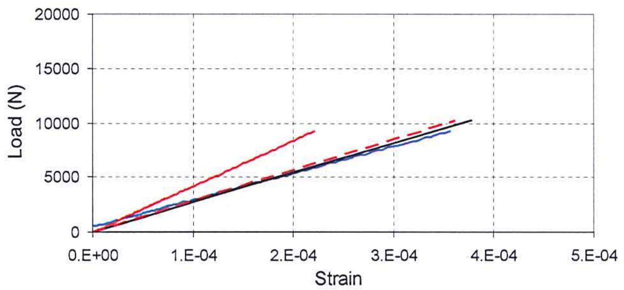
a) 950mm span



b) 1100mm

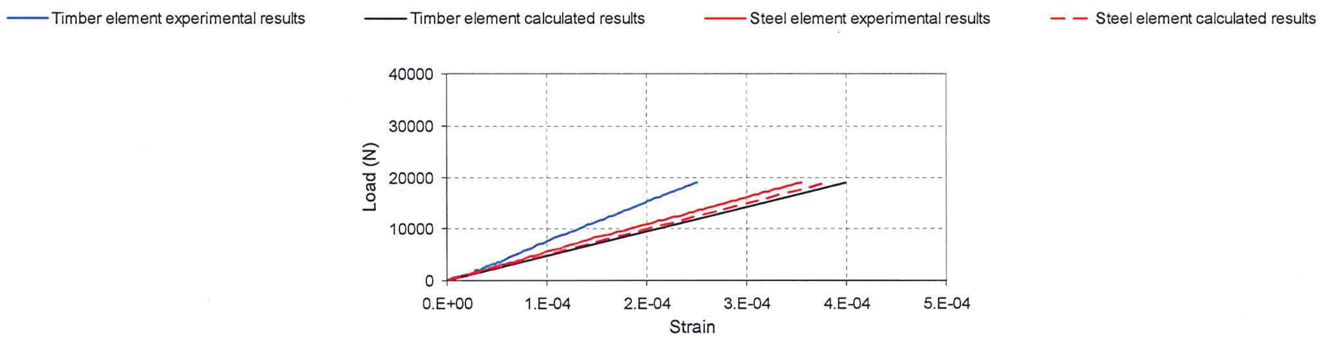


c) 1500mm span

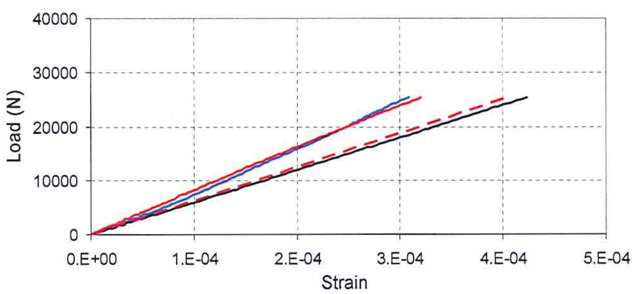


d) 2400mm span

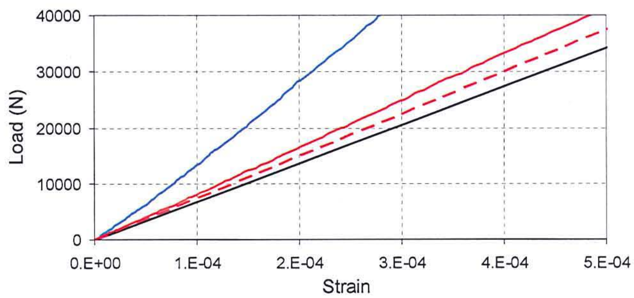
Figure 6.45 TS_10 load against strain in 3 point bending



a) C24_6



b) C24_10



c) TS_10

Figure 6.46 Load against strain in 4 point bending

The steel element of a flitch beam can be considered to be a homogenous, ductile material with a high degree of strength. Any variations in steel elements are as a result of differences in chemical composition (due to segregation of certain elements during the casting and solidification process) and in the mechanical and thermal treatment during manufacturing processes (degree of reduction during rolling, cooling rates during heat treatment etc) (Young et all, 1998). Any variations in the properties of the steel elements that do occur can therefore be considered as negligible relative to the variations of the timber elements.

The strength properties of the steel elements have been quantified experimentally therefore by considering the relationship between stress, strain and modulus of elasticity in bending and carrying out appropriate substitutions the level of stress in the timber during four and three point bending can be theoretically determined as follows:

For four point bending:

$$\sigma_t = \frac{\dfrac{P - \dfrac{2 \cdot W_s \cdot \varepsilon_s \cdot E_s}{a}}{2} \cdot a}{W_t} \qquad \text{Equation 6.18}$$

For three point bending:

$$\sigma_t = \frac{\dfrac{P - \dfrac{4 \cdot W_s \cdot \varepsilon_s \cdot E_s}{sp}}{2} \cdot sp}{W_t} \qquad \text{Equation 6.19}$$

Where:

P is the total applied load

W_s is the steel section modulus

ε_s is the measured strain of the steel

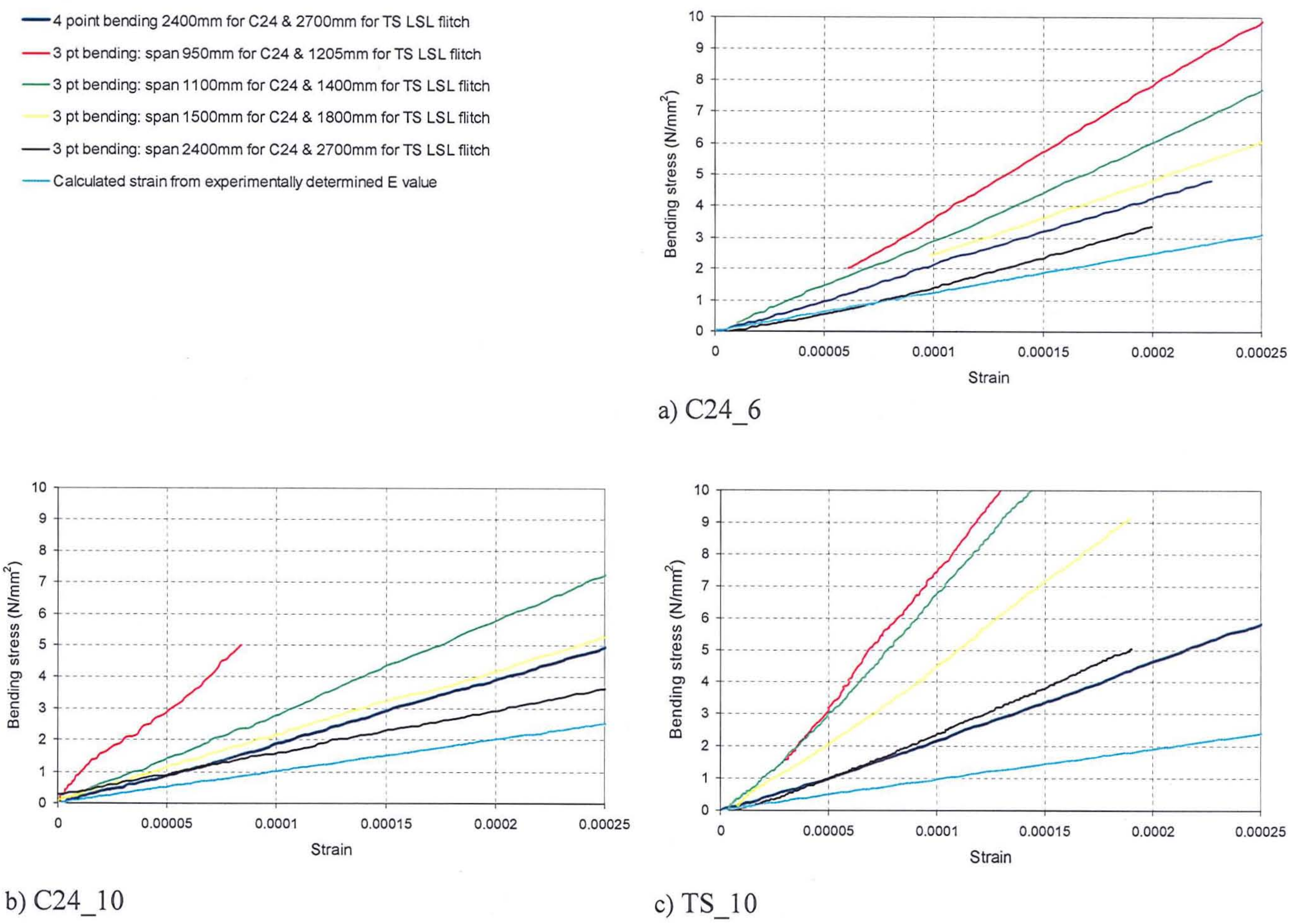
E_s is the steel modulus of elasticity.

a is the distance from the load point to the support

sp is the span

W_t is the combined timber section modulus

Using the above equations and the measured strain of the steel elements, bending stress against strain plots for the timber elements of the flitch beam were produced for the given load span conditions (Figure 6.47). Also shown for comparative purposes is the relationship between stress and calculated strain based on the experimentally determined MoE of the timber elements.



a) C24_6

b) C24_10

c) TS_10

Figure 6.47 Bending stress against strain

From the plots presented in Figure 6.43 to Figure 6.47 the following conclusions are drawn:

1. In Figure 6.43 to Figure 6.46 the actual strain of the flitch beam timber elements for all load/span combinations, with the exception of flitch beam C24_10 in three point bending over spans of 1100mm and 1500mm, is at most equal to or less than the predicted strain.
2. In Figure 6.43 to Figure 6.46 the actual strain of the flitch beam steel elements for all load/span combinations is less than the predicted strain.
3. Considering all three flitch beam types in four point bending (Figure 6.46) it is shown that the strain of the timber and steel elements is less than that predicted by calculation. Theoretically, when beams are subjected to four point bending, pure bending takes place between the load points and there is therefore a negligible shear component over this length. It can therefore be considered that the strain measured at the mid-span, during four point bending, is as a result of bending stress only and can be used for comparative purposes with calculated strain due to bending.

With reference to Figure 6.47 and from simple theory of bending the modulus of elasticity of the beams in bending, MoE, were determined. Table 6.16 contains the MoE value of the timber elements for the given flitch beam type as well as the MoE values for the composite flitch beams. The table illustrates the increase in MoE value of the beam as a result of the composite construction.

Table 6.16 Comparison of E Values

Stress v Stain Plots				
Type	Modulus of elasticity in bending		Increase in E Value	% Increase
	Average of timber components determined from testing (Table 6.8)	Composite flitch beam		
	N/mm^2			
C24_6	12408	21706	9298	75
C24_10	9984	19959	9298	93
TS_10	9497	23755	9298	98

From Table 6.16 the following conclusions are drawn:

- A shot fired dowel flitch beam formed formed 2no 45×190 C24 grade timbers sandwiching a 6×180mm steel plate results in a 75% increase in stiffness relative to the use of the C24 grade timber elements alone. However, the use of a 10mm steel plate of the same depth results in an increase in stiffness of 93%. This corresponds to an 18% increase in stiffness for 60% more steel.
- The shot fired dowel flitch beam formed from 2no 45×241 Timberstrand LSL elements sandwiching a 10×220mm steel plate has a 98% increase in stiffness although the steel plate is 21mm shorter than the timber elements, 11.5mm top and bottom (for the C24 grade flitch beams the difference in height of the steel and timber elements was 10mm, 5mm top and bottom). As a result the improved stiffness of the beam, although the depth of steel plate relative to the depth of section is less, can be considered to be because of the improved connection strength between the elements.

6.5.6 Summary

From the extended experimental programme the following conclusions are drawn:

- The standardised nailing schedule is sufficient and is a balance between optimisation (minimum design requirement resulting in improved manufacturing time) and standardisation which allow the process to be improved through repeatability but can on occasion result in an over specification.

- The strength and stiffness of flitch beams in bending constructed using C24 grade timber is directly proportionate to the density and MoE of the timber elements.
- The strength and stiffness of flitch beams in bending constructed using Timberstrand LSL is inversely proportionate to the density and directly proportionate to the MoE of the Timberstrand LSL elements as a result of the inverse relationship between density and MoE value of the Timberstrand LSL sections used in this study.
- The stiffness of flitch beams in bending, constructed using a shot fired dowel connection, is greater than calculated predictions. The level of improvement is dependent on the variability of the timber component and as a result the level of percentage improvement is more predictable for an engineered timber component due to its higher degree of consistency.
- When solid section timber is being used for the construction of flitch beams using shot fired dowels a conservative approach to design should be taken when shear deflection governs and deflection is the limiting design criteria. The shot firing of dowels tends to split the timber and in turn reduces the capacity of the beam to carry longitudinal shear forces resulting in increased deflection due to shear. However, this problem is reduced significantly when using a product such as Timberstrand LSL due to its composite construction which considerably reduces splitting.
- The applied shear force on the beam will result in embedment and slip of the fixings at the points of connection which results in increased shear deflection. The use of Timberstrand LSL will reduce the level of embedment and slip as a result of providing improved connection strength.
- The use of Timberstrand LSL for the production of flitch beams when using a shot fired dowel connection is recommended. Testing has demonstrated that a more robust beam is constructed.

6.6 Conclusions & Recommendations

The following conclusions have been drawn and recommendations made as a result of the research work conducted on shot fired dowel flitch beams:

- The use of a shot fired dowel flitch beam results in the same material cost as a traditional bolted flitch beam but does result in the introduction of a lean concept. The study conducted has resulted in the determination of generic nailing schedule which simplifies the design process and due to the improved speed of the operation the manufacturing process has also improved.

- According to Bainbridge et al (2001) when carrying out the design calculations for a shot fired dowel flitch connection in accordance with EC5 the mean penetration depth of the nail should be taken to determine embedment. However, from the experimental work conducted in this section it has been demonstrated that the full embedment depth of the nail can be used.
- The axial withdrawal capacity of a fixing improves the lateral load carrying capacity of a connection. When considering a shot fired dowel connection a cold weld forms between the shot fired dowel and the steel element which results in the headside pull through of the fixing corresponding to the axial withdrawal capacity.
- The study of nailing patterns demonstrated that increasing the number of nails reduces the stiffness and the strength of flitch beams due to splitting. As a result the minimum nailing specification based on the ultimate shear carrying capacity of the beams was used in the extended study. It is recommended that further work on nailing patterns is carried out to clarify the initial findings and also to quantify the effect that nailing has on shear modulus.
- In relation to shear modulus the test programme shows that deflection due to shear can be onerous and in certain circumstances govern, particularly when solid timber sections are used in shot fired dowel flitch beam construction. The splitting of the timber due to the shot fired dowels results in additional deflection due to the longitudinal shear forces within the beam. As a result extra precautions should be taken in design when using solid timber sections. It is therefore recommended that engineered products such as Timberstrand LSL are used due to their inherent properties resulting in a dissipation of splitting energies and consequent reduction in splitting.
- In terms of stiffness it has been demonstrated that Timberstrand LSL flitch beams have an improved level of stiffness relative to the size and thickness of steel section used compared to flitch beams constructed using C24 sections. Further analysis of this for a range of comparable beams with different steel plate thicknesses and dimensions could be carried out to determine which flitch beam configurations correspond to the most added value.
- In design long term creep affects need to be taken into account. According to Larsen and Mettem (2001) 18% more cumulative final creep should be allowed for in the design of shot fired dowel flitch beams. However, further work on duration of load should be carried out to ensure the incorporation of such an onerous factor is required.

Published work

1. **Hairstans, R., Dodyk, R. and Kermani, A.** (2006) *"Timberstrand LSL Nailed flitch beams."* Proceedings of the 9th World Conference on Timber Engineering, 6-10 August 2006, Oregon USA

2. **Hairstans, R. and Kermani, A.** (2004). *"Performance of shot-fired flitched timber beams."* COST Action E29 - International Symposium on Advanced Timber and Timber-Composite Elements for Buildings, 27-29 October 2004, Florence, Italy.

3. **Hairstans, R., and Kermani, A**. (2005). *"Shot fired dowel flitch beams."* The Second Scottish Conference for Postgraduate Researchers of the Built & Natural Environment (PRoBE), Glasgow Caledonian University, 16-17 November 2005, Glasgow, pp441-449, ISBN: 1-903661-82-X

CHAPTER 7

CRANE ERECT OF TIMBER PLATFORM FRAME CONSTRUCTION

7.1 Introduction

As mentioned in Chapter 2 a crane erect method of timber platform frame construction has evolved which incorporates the off-site fabrication of the wall and floor components and the on-site preparatory construction of the roofing system at ground level. This chapter details the development of a Best Practice Procedure (BPP) for the evolved Modern Method of Construction (MMC) with a view to eliminating client scepticism and improving efficiency. To achieve this, the following three key drivers were set:

1. Health & Safety
2. Speed of Erection
3. Cost

Based on the three key drivers a feasibility study has been carried out which compares three alternative methods of timber platform frame construction (at height construction with tele-handler, at height construction with crane and crane erect construction).

Of the three key drivers Health & Safety was deemed as being the most critical on the basis that effective planning for Health & Safety is essential if projects are to be delivered on time, without cost overrun, and without experiencing accidents or damaging the Health & Safety of site personnel (CIOB, 2003). Considering this the operation of the crane erect construction method posing the major Health & Safety risk was identified and this was the lifting into position of the roofing system constructed at ground level.

Although preparatory construction of the roof system at ground level reduces the time spent working at height, one of the single biggest causes of casualties in the construction industry, there is the risk of the roof system failing during lifting operations. To reduce the associated risk of failure of the roof system when being lifted into position an analytical model was developed to analyse the behaviour of a roof system under lifting conditions. However, due to the nature of the support conditions when analysing a roof system under lifting conditions (one point of support would result in a mechanism forming) extensive laboratory testing was required to verify the developed model.

Using the verified model a range of lifting methods were analysed and two methods of lifting roof systems were derived which can be used depending on the nature of the system. Further to this guidance notes and product mass information is provided to ensure that lifting operations can be carried out safely and efficiently.

7.2 Feasibility Study

7.2.1 General

With the foreseen expansion in the timber platform frame market there will be increased pressure on contractors to deliver construction projects on time and within budget. Traditional methods of timber platform frame construction are labour intensive, time consuming and relatively high risk with the major risk associated with working at height.

The Health & Safety Executive (2003) has also recognised the associated risk of working at height by reporting that over the past five years there have been 437 fatalities on construction sites in the UK of which 225 were as a result of a fall from height, equating to almost one person every week on average. When considering the traditional methods of timber platform frame domestic dwelling construction working at height is required to install both the floor and roof systems. There are two main types of fall arrest methods which can be employed when considering traditional methods of construction (Gillan et al, 2003):

1. Active fall arrest: *"a system that requires actual physical activity by an individual to ensure that the system operates correctly, e.g. harness and lanyard clipped to an anchor point. Normally, active systems would protect only the individual wearing/using the equipment."*
2. Passive fall arrest: *"described as a system, once installed, that requires no active measures by the users, or those who are likely to rely on the system in the event of a fall. Passive systems can protect numerous individuals at any given time."*

Active based systems for domestic dwelling construction are viewed as being impractical and the most common methods of providing safety for operations at height in traditional methods of construction are passive. The passive system employed is required to provide safety both at the 'Leading Edge' (the opposite side to that being worked on); and the open 'Working Edge' (the side where work is being carried out i.e. flooring is being installed). To achieve this, the methods available to the builder are safety nets and fall arrest mats, *"air"* or *"bean"* bags (Figure 7.1).

a) Air mats (Marwood Group Ltd, 2007) b) Bean bags (Response Safety Netting, 2007) c) Safety nets (Response Safety Netting, 2007)

Figure 7.1 Passive fall arrest systems

According to Gillan et al (2003) fall arrest mats have an advantage over safety net systems when considering the domestic dwelling market. Safety nets have limitations when used during low level construction as they require strict management of the space below the net to ensure a clear net deflection height is maintained. However, there are still problems associated with fall arrest mats including installation time, cost (approx £5/m^2 per week), storage and maintenance.

The major components of a timber frame dwelling can be pre-assembled off-site and craned into position on-site providing a working platform. Taking the construction of these components to a factory environment alleviates the problem of the current construction industry skills shortages, provides a safer working environment and is also proven to have a higher level of best practice production time (Gibb and Isack, 2003).

At height construction of the roofing system poses a major risk in the timber platform frame house construction process which requires to be engineered out (HSE, 1999). The preparatory construction of the roofing systems at ground level to be craned into position results in a large reduction in time spent working at height.

The construction method developed which envelopes the procedures reliant on the use of a crane is known as 'Crane Erect'. This method of erection results in limited man handling and can, with good planning, eliminate the need for fall protection and further reduce erection time.

For the crane erect method of construction to be carried out safely and to an optimum there are pre-requisites which must be covered. Studies have shown that there is a change in risk of accidents from committing to off-site fabrication and on-site preparatory work. The return of accidents switches from minor consequence and high risk to major consequence and low risk (Gibb and Isack, 2003). It is imperative therefore that good construction, design and management procedures are implemented (CDM Regulations, Clause 11, 2007).

At the design stage the risk of failure is required to be engineered out:

- Any system component being craned into position should be designed for this purpose and the weight of the component should be supplied to the on-site staff.
- Lifting points are required to be designed, and if required manufactured into, the products to be lifted.

This has implications at both the design and manufacturing level resulting in increased work load, however the increased time spent carrying out these tasks is seen as minor in comparison to the on-site advantages.

The success of crane erect is reliant on good project planning. The delivery sequence of components is altered to allow for the construction of the roof system at ground level prior to other construction events. Just in time principles are necessary to limit the need for long term storage. The stacking arrangement of the floor cassettes and wall panels is required, as much as practically possible, to be in an order which reflects the construction sequence, this limits the need for temporary storage and quickens operations. The crane requires adequate space to be made available in close proximity to the plot being developed and a designated area for the temporary storage of the pre-constructed roof system within its lifting range.

Project planning has to ensure that other trades will not be disturbed or indeed any risk to others is created from the congestion of activities in a confined area. For this reason the crane erect method lends itself to larger scale projects and those on green and brown field sites. On small scale, congested sites the planning of activities is more difficult to allow for crane erect however in most circumstances not impossible.

Good infrastructure for ease of access and egress, unrestricted visibility, a predetermined temporary storage area for the constructed roof system and no over head hazards are further pre-requisites. In normal circumstance the pre-requisites are raised at the pre-start meeting of the construction project.

To make this process simple in the future partnering is beneficial, the client and the erector have a mutual understanding and can tailor their planning to work in tandem resulting in operational efficiency.

The catalyst for safe and efficient construction is training. The major training requirement is in the safe working practice of carrying out lifting procedures and for this reason all those involved in the erection process are approved Slingers and Signallers and those who deal with the planning of the erection process are Appointed Persons, in line with the Health & Safety Executive guidelines. Further to this there is a requirement to produce a lifting plan and method statement for every frame. However, standardisation of the procedures has allowed a generic method statement to be produced and as a way of receiving endorsement from the on-site staff it has been done with their input. One of the main advantages of involving the on-site staff in carrying out construction planning to improve the erection process is benefiting from their knowledge and expertise, a finding also reported on by Hare et al (2005).

7.2.2 Comparison of Erection Methods

To provide evidence of the benefits of the crane erect method of construction a feasibility study was conducted. Three main areas were investigated; Health & Safety, Speed of Erection and Cost.

The Health & Safety statistics available were not directly related to crane erect therefore to alleviate client scepticism a study was carried out of the different methods of erection and their associated risks to provide circumstantial evidence. The study conducted used weighted risk assessments of the different methods of timber frame construction to determine which one had the lowest associated risk. The risk assessments were completed by people at all levels of the erection process, including site managers, contract managers, erectors and Health & Safety officers (the averaged out-put of these risk assessments is contained in Appendix I). The out come of this study demonstrated that if crane erect construction was considered as the datum then at height construction with a tele-handler had 63% more associated risk and at height construction with a crane had 46% more associated risk.

Time and cost are very much interlinked and there is a trade-off between the two variables. *"Construction planning involves the selection of proper methods, crew sizes, equipment, and technologies, to perform the tasks of a construction project. In general, there is a trade-off between the time and cost to complete a task; the less expensive the resources, the larger duration they take to complete an activity"* (Hegazy, 1999). Using a project planning tool, Microsoft Project, the three different methods of timber platform frame construction which are mapped out in Figure 7.2 were compared for time saving benefits, in particular these three methods were looked at because they are compatible with the off-site fabrication of system components, although, it is to be noted that method 1 requires the on-site construction of the flooring system at height. Each method of construction was broken into tasks and each of the tasks allocated resource and time requirements. From the study crane erect was proven to produce a time saving of 53% if planning and resource allocation were optimised.

The time performance of crane erect is dependent on best practice procedures being implemented. Allocation of resources and in particular that of the crane due to the hired cost, is important if the method of crane erect is to be cost effective. Good planning and training are therefore prerequisites for operational success.

The added costs due to increased crane hire time and training requirements will be counter-balanced by the increased market value of products and erection procedures due to efficiency in time and safety. Shown in Table 7.1 is the cost analysis of a typical house type which demonstrates that as a result of reduced labour and safety equipment requirements a 25% cost saving is made (for a further financial break down of the methods see Appendix J).

Table 7.1 Construction method costing (Based on 2004 figures)

	At height with tele-handler	*At height with crane*	*Crane erect method*
Labour	£1,435.50	£1,287.00	£792.00
Plant	£120.00	£273.00	£546.00
Safety equipment	£216.95	£216.95	£0.00
Total cost	£1,772.45	£1,896.95	£1,338.00
Cost/m^2	£22.42	£23.99	£16.92

Further to the cost savings made improved market perception and client satisfaction will result in additional work load leading to increased turnover and profitability. It is noted that the reduction of associated risk will in time reduce the incidence of accidents which will subsequently reduce insurance premiums. Also of note is the recent development of Open Learning Training and Accreditation Scheme for Timber Frame Erectors developed by the UK Timber Frame Association (UK TFA, 2007) which leads to a full UKTFA/City & Guilds Accreditation which covers the crane erect method of construction and is recognised by all the leading manufacturers in the industry.

Safety is a decisive factor behind the implementation of the crane erect procedure as it results in eliminating the majority of the risks associated with timber frame construction through safer working practice. Time and cost savings have been proven to be possible through the implementation of good planning and the information available can be used to alleviate client scepticism.

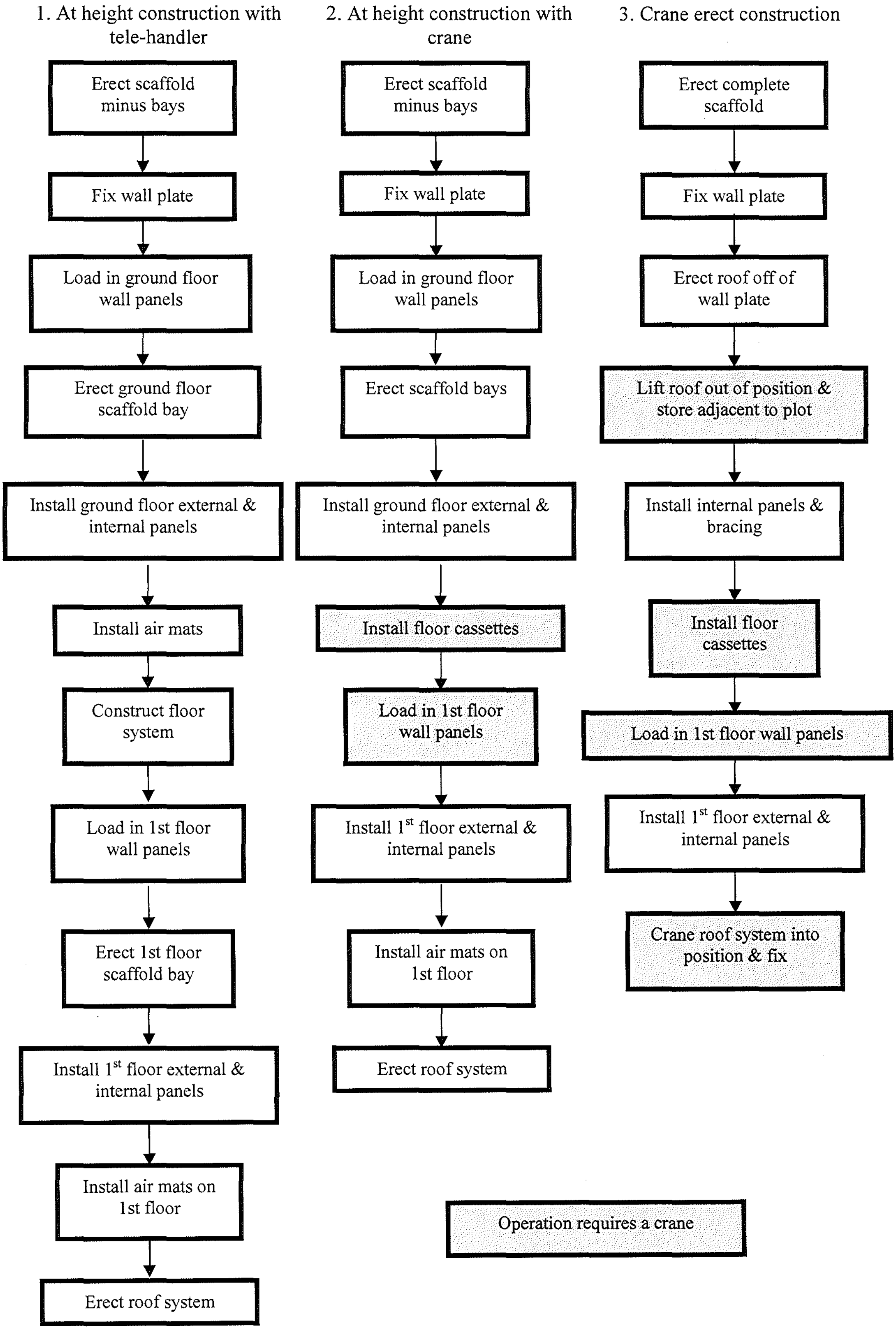


Figure 7.2 Timber frame erection methods supported by off-site fabrication

A weighted design matrix, using Time, Cost and Safety as the three variables, was used to judge the methods of construction (Table 7.2):

1. At Height with Tele-handler.
2. At Height with Crane
3. Crane Erect

Table 7.2 Quantification of variables

Variable	Rating	Reason
Time	2	The housing market is one which is determined by supply and demand. The present market climate shows that there is a high demand for housing and therefore contractors want to complete projects as quickly as reasonably possible creating larger turnover and subsequent profits. The time of construction is therefore more important than the actual cost due to the market climate, but a balance has to be struck.
Cost	1	Cost would be more important if there was a reverse in market trends where costs would have to be kept to a minimum so large profits could be made on small scale projects. Therefore, the present requirement of large scale developers is to have a level of expenditure which will see houses constructed to supply the demand but meet profit targets.
Safety	3	Safety is paramount. As with all construction practices there is always an element of risk involved but the more this risk can be engineered out, without being overly detrimental to the timescale and cost of projects, the better.

Using the information associated with each method of timber frame construction a weighted comparison matrix was produced to compare each method relative to one another (Table 7.3).

Table 7.3 Weighted matrix comparing timber frame construction methods

Construction Method	Factor			Total
	Time	Cost	Safety	
1. At Height with Tele-handler	1	2	1	7
2. At Height with Crane	2	1	2	11
3. Crane Erect	3	3	3	27

From the information contained above and that of Table 7.3 the crane erect method (Figure 7.3) is shown to be the most efficient method of timber platform frame construction.

a) Roof constructed on ground floor slab

b) Roof lifted out of position

c) Ground floor panels erected

d) Cassette flooring installed

e) 1st floor panels installed

f) Roof system craned into position

Figure 7.3 Crane erect method of timber frame construction

7.2.3 Summary

The current climate in the UK housing market is one where demand outstrips supply. Therefore, the trade off between time and cost is in the favour of time. In relation to timber platform frame the crane erect method of construction, which requires the off-site construction of wall panels and flooring systems and the on-site preparatory construction of the roof system to be craned into position, is at both a cost and time advantage relative to other methods such as at height construction using a tele-handler or crane. However, there is a change in associated risk when considering the crane erect method from minor consequence and high risk to major consequence and low risk. To ensure that the crane erect method is indeed low risk the accident which would result in major consequence, failure of the roof system during lifting requires to be engineered out and a best practice procedure endorsed.

7.3 Modelling & testing of lifting procedures

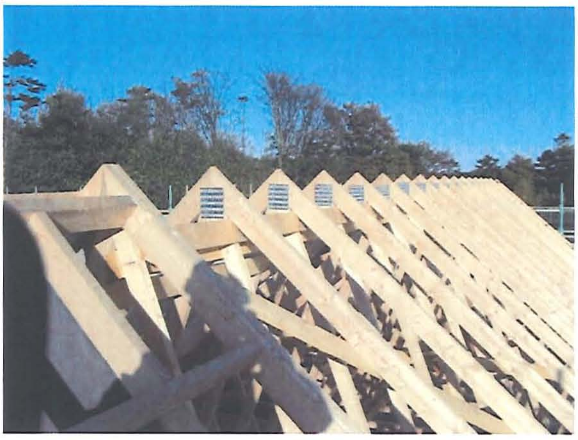
To understand how a roof system is behaving under lifting conditions so that a safe method of lifting could be implemented a computer model of a roof system was developed using structural analysis software. The limited support conditions of the computer model required it to be verified. Therefore, laboratory testing of an equivalent roof system was conducted to measure how the roof system reacted considering a range of lifting conditions. The measured reaction was compared to the out-put of the computer model to determine how realistic the model was and whether it could be used to derive a best practice lifting procedure.

7.3.1 Roof Truss Information & Modelling Consideration

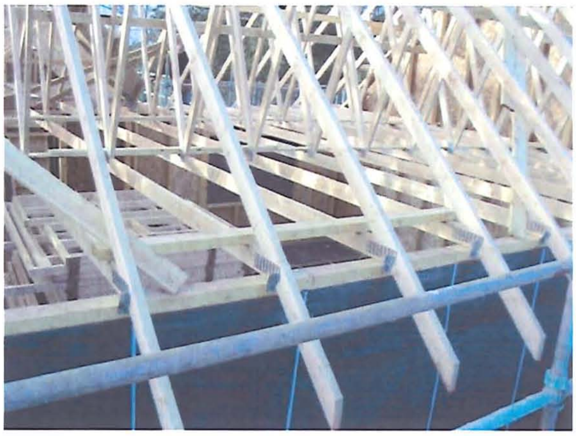
Timber trussed rafter roofs were first introduced to the UK in the 1960s and are now the most common type of roof system used when considering domestic dwelling construction (Bainbridge et al, 1998). Timber trussed rafters are structural frames which are individually designed to support roofs and ceilings; principally at a spacing of 600mm. Spans of up to about 22m can be manufactured with longer spans achievable by splicing two or more sections together on-site. Manufactured from strength graded timbers the elements of a truss are joined together with punched metal plate fasteners. The trusses are then formed into a roof system by means of being braced and connected to a headbinder using truss clips. In Figure 7.4 examples of connection detailing and a full truss system are given.

Connection between the trusses and the headbinder is important as it transfers applied loading on the roof system to the timber frame and this connection is normally formed by truss clips. Bracing of the system forms two basic functions:

- Stability bracing holds the trusses firmly in place and keeps them straight so that they can resist all the loads applied (with the exception of wind).
- Wind bracing, often required in addition to stability bracing so wind forces on the roof and walls can be withstood.



a) Apex of a roof truss system showing nail plates



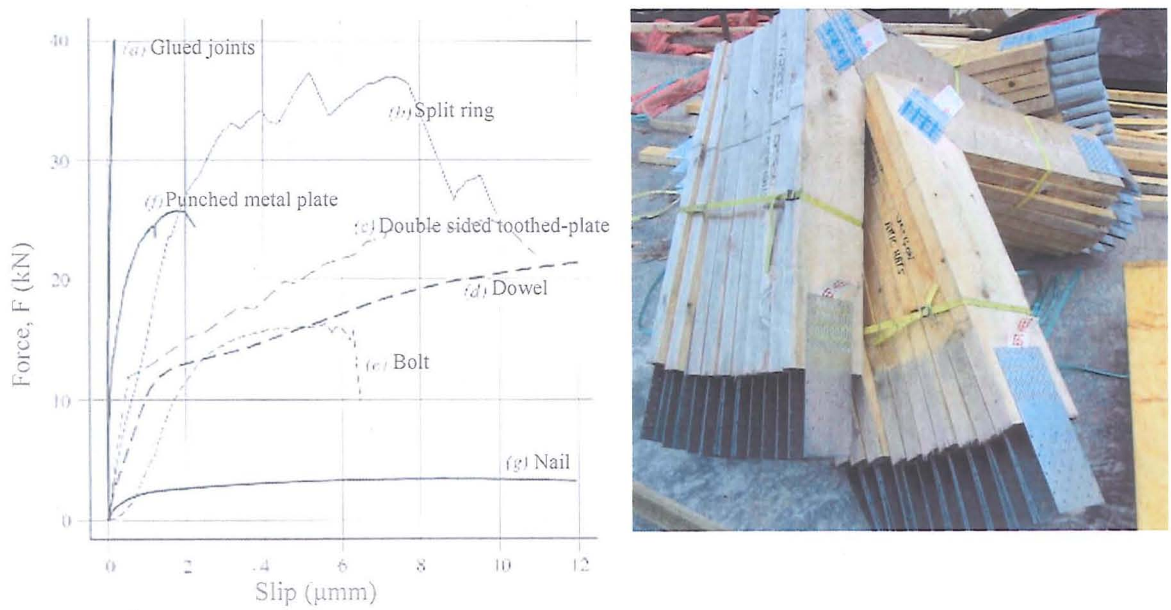
b) Wall head detail showing trusses attached to the headbinder via truss clips

c) Truss rafter roof system

Figure 7.4 Truss rafter roof system and examples of connection details

In terms of design although a truss system forms a complex 3 dimensional structure commercial programs split the system into 'simple' 2 dimensional static models in which the forces are added directly to the truss model and the reactions are transferred from one 2 dimensional model to another (Nielsen, 2003). The members of the truss are normally modelled as linearly elastic beam elements with the moment distribution between these elements dependent on the level of rigidity of the connection assigned.

The distribution of load within a structural system is affected by the rotational stiffness of the joints taken into account, with accurate modelling resulting in more economical design. According to Kanerva et al (2004) it is usual in timber design to assume the rotational stiffness of a joint to be either infinitely rigid or zero. The punched metal plates used in the fabrication of timber trussed rafters, described by Whale (1995) as *"fasteners made of metal plate having integral projections punched out in one direction and bent perpendicular to the base of the plate"* offer a degree of rigidity. Another factor which influences the transfer of forces and final deformation of the system is local displacement of the connections, known as joint slip. Relative to other timber connection methods punched metal plates perform relatively well in terms of joint slip and demonstrate a small plastic deformation capacity.



a) Experimental load slip curves for joints in tension parallel to the grain (Racher, 1995)

b) Example of truss nail plates

Figure 7.5 Experimental load slip curves of connections and examples of nail plates

Rotational stiffness and joint slip are required to be considered in combination for accurate modelling of a nail plate connection. To do this the developed model has to quantify the magnitude of forces in the plate, nail group and contact zone (nail area in contact with the timber element).

Foschi (1977) introduced a model that was capable of estimating the stiffness and sectional forces in each nail group and plate. The nail and plate elements are developed with non-linear load-slip relationships. Nail plate and contact elements are used to join the beam elements together. This model, regarded currently as the most advanced, has been used in research activities but as yet not in commercial truss programmes (Nielsen, 2003).

7.3.2 Analytical Modelling and Laboratory Testing

LUSAS structural analysis software was used to create a 3 dimensional, duo pitch roof truss system consisting of fink trusses representative of what is commonly used in industry. Figure 7.6 provides the dimensions and material specification of the individual trusses of the system which were braced in accordance with BS 5268-3:1998 details of which are given in Figure 7.7.

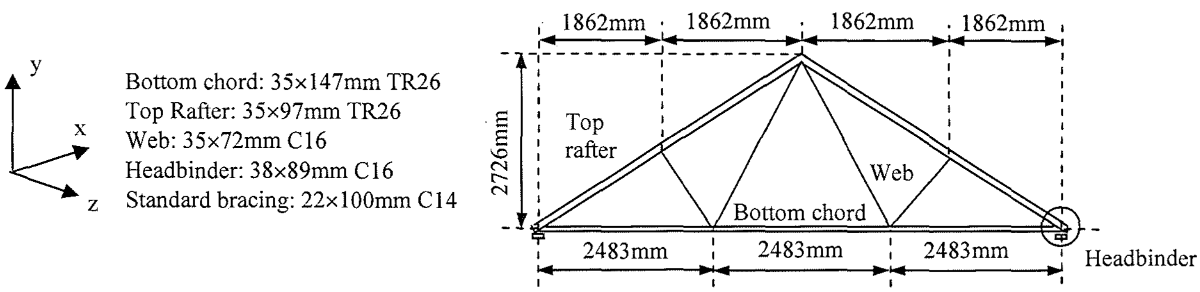
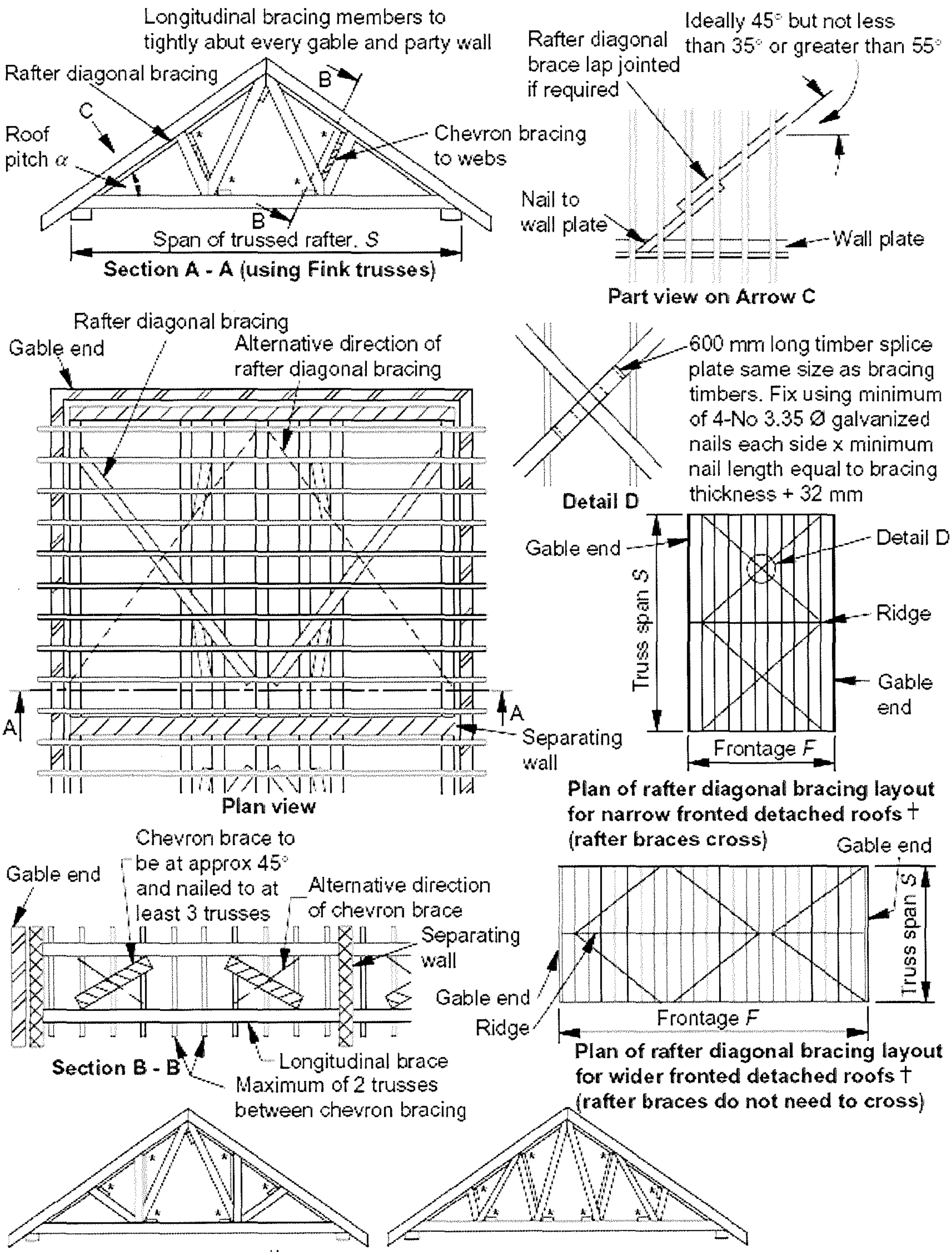


Figure 7.6 Truss dimensions, material and support conditions of analytical model

Initially a single plane frame truss was modelled considering the truss to be formed of linear elastic beam elements set out along system lines (lines coinciding with the centre line of the truss members) (Kessel, 1995) with the bottom chords and rafters of the system taken to be continuous.

The final 3 dimensional model was to be simple to allow numerous lifting procedures and, potentially systems variations, to be analysed. The purpose of the model analysis conducted is to ensure that forces within the original system, designed to withstand in service applied actions, are within design tolerance. The developed model was not to be used to optimise the performance of the truss system or rationalise the use of material. As a result the connections of the system were pinned and load slip was not taken into account. In terms of force distribution this is a conservative approach and although it is recognised that additional deformation of the system will take place due to joint slip robustness of the system is the primary concern not serviceability.

To verify the plane frame model it was subjected to a range of loading conditions and the axial forces of selected members and support reactions were compared to long hand calculations to ensure the model was providing expected results. The individual verified truss was then used to build up a system of trusses which was braced accordingly giving due consideration to the eccentricity of the bracing elements. The full system was then subjected to loading conditions and the axial forces of selected members and support reactions (the end nodes of the bottom chord of each truss were simply supported) were then checked relative to long hand calculations to verify the final system.



NOTE 1 Chevron bracing shown is not required on internal members of trusses with spans of 8 m or less.

NOTE 2 denotes longitudinal bracing not required when the criteria described in A.11(2) are met.

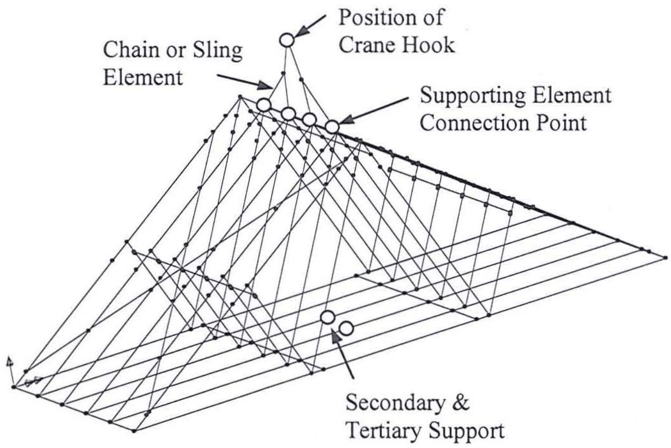
† NOTE 3 For the purposes of this annex, a building may be defined as narrow fronted when the frontage $F < \dfrac{S}{1.4 \cos \alpha}$

Figure 7.7 Standard bracing for rafter and web members of duopitch trussed rafters (BS 5268-3:1998 Figure A.1)

During a lifting operation the support conditions of a system are limited to one point the 'crane hook'. Modelling of such support conditions is not feasible, the computer model would fail due to lack of restraint resulting in a mechanism forming, and as a result the model had to be given extra restraint for successful analysis. Two extra restraint points were placed at the mid-span of the bottom chord of the middle two trusses of the system (Figure 7.8), restricting translational movement of the system in the X and Z directions. How representative this model was of the actual system was unknown and for this reason laboratory testing of the modelled system was undertaken to provide results for verification.

Due to laboratory restrictions the size of the roof truss system which could be tested was limited to a run of six trusses. Therefore, the initial computer model was also limited to a run of six trusses so that for verification purposes a direct comparison could be made.

a) Laboratory set-up



b) Analytical model

Figure 7.8 Laboratory set-up and analytical model showing demonstrating support conditions

The roof system was erected according to the following procedure:

1. The headbinder of the system was set-out and the truss clips were secured in position.
2. The first truss in the run was erected, checked to be straight and upright and temporarily braced.
3. In sequence the remaining five trusses were erected and, once checked to be straight and upright, connected to the longitudinal bracing.
4. A final check of the system was carried out to ensure that all trusses were straight and level then all other bracing in accordance with BS 5268-3:1998 was applied and any temporary bracing was removed.

The testing of the system under lifting conditions is not without complications. The measurements taken from the system during the lifting procedure had to be practical to measure, have a good degree of accuracy to qualify the computer model and be suitable to allow comparison.

The most practical measurements which could be accurately taken during lifting conditions were those of system deflection. On lifting the system is suspended in space and susceptible to sway therefore deflection measurements of the system components were made relative to a fixed point on the system itself.

Two measurement axes on the system were set-up, Axis A and Axis B (Figure 7.9), to measure the deflection of the system and 9 lifting configurations were tested as well as the application of an eccentric load. The lifting configurations tested were selected such that they were representative of methods which could be practically employed on-site and would provide a range of responses so that a comprehensive analysis of the system could be carried out.

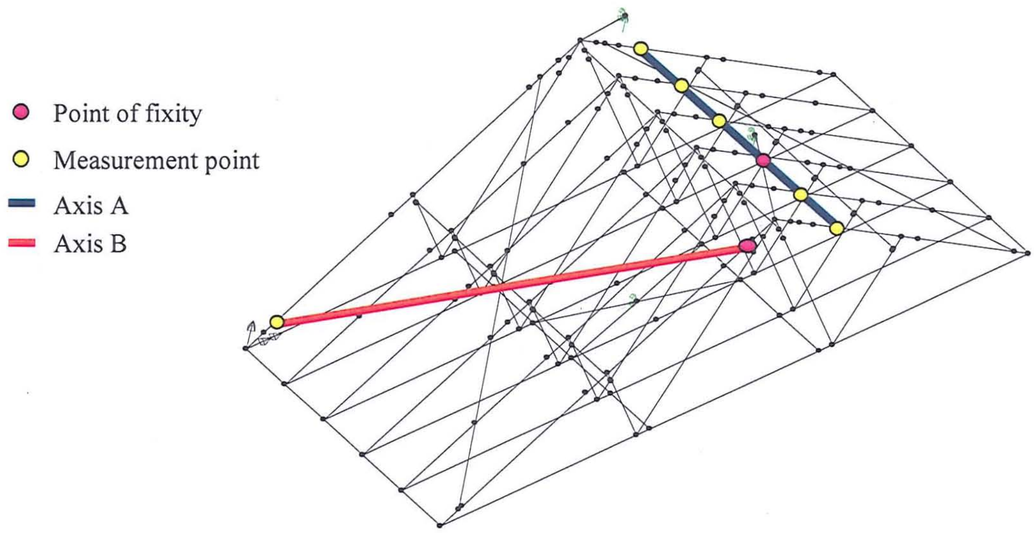


Figure 7.9 Measurement axis on system

In the first range of tests conducted the roof system was lifted from designated points on the system which included Two-point apex, Four-point apex and Four-point rafter mid-span lifts, as illustrated in Figure 7.10 (1a to 4). During these lifting conditions additional strain is placed on the system as a result of the applied out of plane forces at the node points due to the configuration of the lifting equipment. Therefore, to improve the accuracy of the model the lifting equipment was brought to a position whereby it was strained to a point just prior to applying forces to the nodes and the position and angle of the lifting equipment was measured.

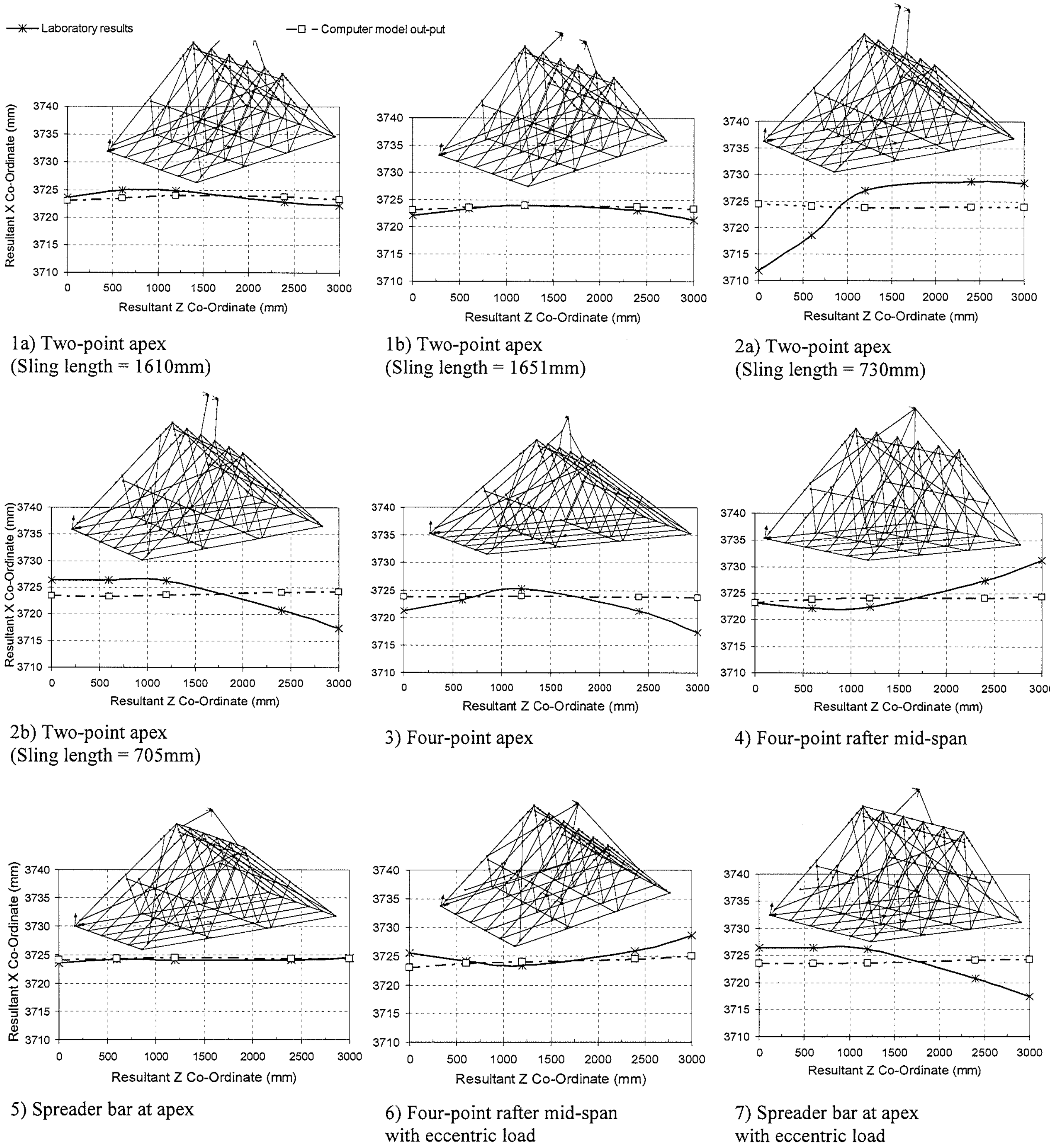


Figure 7.10 Analysis and test results for apex node point displacements (Axis A) compared.

During the lifting action a degree of slippage of the lifting equipment took place until the system was suspended. However, the computer model carries out a static analysis of the system and can not account for movement of the lifting equipment due to slippage and as a result discrepancies between the results were expected. In an attempt to alleviate slippage in the system during each test the following procedure was carried out:

1. A pre-test suspending the system in the lifting condition to be examined was conducted.
2. The system was taken back to the position prior to when the lifting equipment was strained without out of plane force being applied to the system.
3. A datum measurement was taken and the positioning of the lifting equipment was measured.
4. The system was lifted and suspended.
5. A test displacement measurement was taken.

Following on from lifting from designated points on the system, lifting using a spreader bar was carried out as shown in Figure 7.10 (5). Although it was expected that a reduced level of slippage in the system would take place due to the even spread of an applied load acting in the vertical plane the same test procedures were implemented primarily to ensure quality of data but also for consistency.

Finally to simulate possible asymmetric weight distribution in a truss system an eccentric load was applied to the system under lifting conditions. Again laboratory restrictions limited the level of eccentric load which could be applied. The application of a high magnitude load at too large an eccentricity would result in the system swaying and coming into contact with the laboratory floor. Considering the above an eccentric load (6kg mass) was applied to the second truss in from the end truss at the dimensions shown in Figure 7.11.

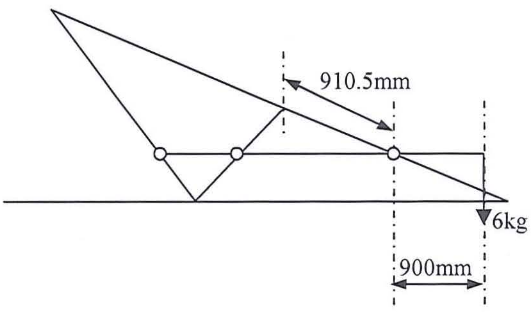


Figure 7.11 Eccentric load (attached to the second truss in from the left hand side).

For each lifting procedure tested the resultant local deflections of the measurement points relative to the fixed point were calculated and converted to global deflections. The lifting procedures were analysed using the computer model and from the processed data the resultant global deflections of the measurement points relative to the fixed points were determined. In Figure 7.10 the resultant X and Z deflections of the apex node point displacements (Axis A) from both the laboratory tests and computer model are plotted relative to each other to allow comparison. The Axis B measurement method was restricted to measuring displacement in the global X direction. In Figure 7.12 the correlation between the resulting deflection measured from the laboratory testing and the computer model for each lifting procedure is illustrated.

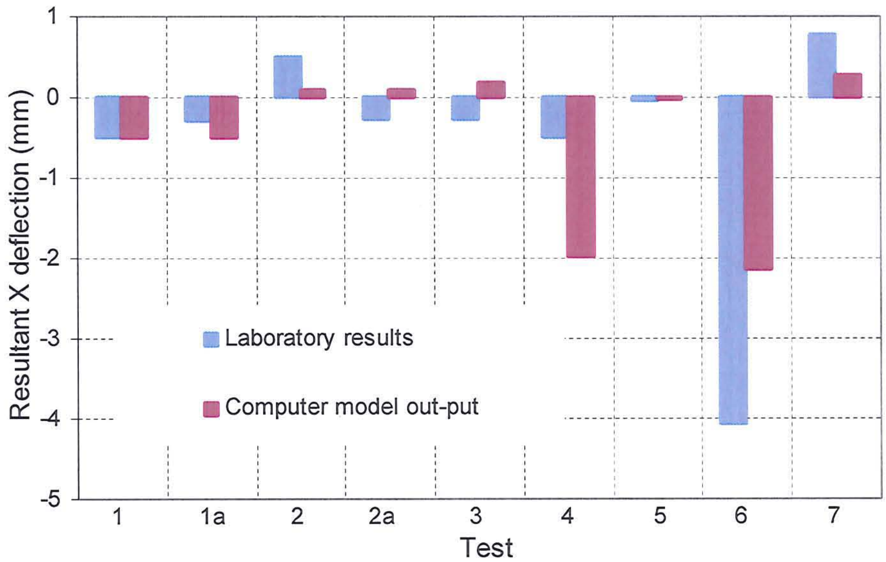


Figure 7.12 Local X displacement of measurement point along Axis B

Considering the results contained in both Figure 7.10 and Figure 7.12 a relatively high degree of correlation is demonstrated between the laboratory results and the simulated model out-put. The level of correlation is however considered to be relative to the lifting procedure.

As discussed the lifting procedure tested has a bearing on the sensitivity of the results. Therefore, to further prove that the laboratory test results correlate with the analytical model output a rating system was set up to compare the conclusive statements taken from each test depending on the level of correlation exhibited (Overall Conclusion) with the expected accuracy of results due to the nature of the testing procedure (Accuracy Factor).

In Table 7.4 the expected accuracy of each test is rated between 0.25 (Depleted) and 1.0 (V. Good) depending upon what the perceived accuracy of the test would be based on circumstantial evidence from when the tests were conducted. For example the use of a spreader element is expected to give a high degree of accuracy as it resulted in an even distribution of load being applied to the system when carrying out the laboratory test, this corresponds to how the load is applied in the computer model and as a result is allocated a rating of 1.0 (V. Good). However, lifting from the mid-point of four rafters was difficult to configure in the laboratory and resulted in uneven strain being applied to the system which was exaggerated by lifting equipment slippage, this could not be reflected in the model due to the analysis method being static and as a result is rated 0.25 (Depleted).

Considering the results presented in Figure 7.10 and Figure 7.12 the level of correlation between the laboratory and computer model results are rated between -3 (Error) and +3 (Excellent). For example Test 1b tends to show good correlation although the computer model tends to be stiffer and is therefore allocated a rating of 2 (Good).

The multiple of the two factors provides the Total = "Overall Conclusion" × "Accuracy Factor", the purpose of which is to average out the results to show whether the test results provide evidence to support the analytical model as being representative of the truss system being lifted. In Table 7.5 result scenarios are presented to which the total from Table 7.4 can be compared.

Table 7.4 Weighted comparison.

Test	Overall Conclusion[1]		Accuracy Factor[2]		Total
1a Two Point Apex	Good	2	Good	0.75	1.5
1b Two Point Apex	Good	2	Good	0.75	1.5
2a Two Point Apex	Sceptical	-1	Good	0.75	-0.75
2b Two Point Apex	Sceptical	-1	Good	0.75	-0.75
3 4 Point Apex	Favourable	1	Average	0.5	0.5
4 Four Point Rafter Mid-Point	Sceptical	-1	Depleted	0.25	-0.25
5 Spreader Bar at Apex	Excellent	3	V. Good	1.0	3
6 Four Point Rafter Mid-Point with Eccentric Load	Favourable	1	Depleted	0.25	0.25
7 Spreader Bar at Apex with Eccentric Load	Favourable	1	Depleted	0.25	0.25
				Total	5.25

1. Overall Conclusion ratings are determined taking into consideration Figure 7.10 & Figure 7.12.
2. Accuracy Factor is a prediction of how accurate the testing results will be depending on the nature of testing.

Overall Conclusions Ratings	Accuracy Factor
Error = -3 Poor = -2 Sceptical = -1 Favourable = 1 Good = 2 Excellent = 3	Depleted = 0.25 Average = 0.5 Good = 0.75 V. Good = 1.0

Table 7.5 Result scenarios.

Conclusion rating	Accuracy factor	Number of tests	Output
Error	Very good	9	-27
Error	Depleted	9	-6.75
Excellent	Depleted	9	6.75
Excellent	Very good	9	27

The end result of the Weighted Comparison (Table 7.4) is a value of 5.25 giving further evidence that the laboratory testing provided results of 'excellent' conclusion rating but of 'depleted' accuracy (Table 7.5) due to the nature of testing and this was true of the testing scenario. The computer model is a relatively accurate way of modelling lifting conditions considering the influencing factors on laboratory test results although the model does tends to have a higher degree of stiffness. As a consequence of load carrying capacity being the limiting criteria when considering lifting procedures increased stiffness is a conservative approach as it will reduce load sharing. Load sharing within truss systems has been reported by Wolfe and McCarthy (1989) who from conducting full scale roof truss assembly tests demonstrated that 35-66% of the applied load was distributed to the unloaded trusses when one truss was loaded individually in truss roof assemblies. Due to the requirement that it is essential to ensure the system is safe during lifting conditions as the risk of an accident is to be reduced to a negligible degree as the consequences would be major this conservative approach is endorsed. However, if the model was to be improved such that semi-rigid behaviour and slip at the joints is taken into account, without the need of using the Foschi model, a simplified method explored by Zhong et al (1998) could be adapted. The modelling method developed by Zhong et al (1998) employs spring elements with no physical dimensions to represent the semi-rigid behaviour at the ends of truss members connected by metal plates, with the metal plates modelled by "rigid" links.

7.3.3 Summary

The support conditions of lifting are limited to one point and this can not be modelled directly as it would result in a mechanism forming. However, providing additional translational restraint in the X and Z directions at the mid-span of the bottom chord of the middle two trusses has been demonstrated to be an effective modelling solution.

Pinning the connections and as a result not taking into consideration the level of rigidity or the slip of the connections has reduced the load sharing capacity of the system and also increased its stiffness. Although this is the case the model is still deemed suitable as the important design consideration when deriving a lifting procedure is system robustness not serviceability and as a result this is a suitably conservative approach.

As a result of the model being static it does not account for movement of the lifting equipment during lifting operations which can tend to apply increased out of plane actions on individual trusses especially when lifting at an angle from designated points. Lifting at an angle from designated points on truss rafter systems is therefore considered to be inappropriate as poor configuration of the lifting equipment results in the overstressing of system elements and would require additional bracing.

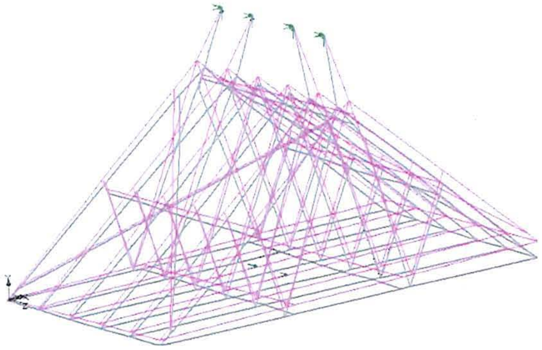
Considering that the purpose of the model is to derive a best practice lifting procedure and is not going to be used to optimise the in-service performance of the truss system or rationalise the use of material it is appropriate. It is not overly complicated which will allow numerous lifting procedures to be examined and potentially system variations to be made. However, the model could be improved if necessary using spring elements and "rigid" links to represent the semi-rigid behaviour at the ends of truss members.

7.3.4 Best Practice Lifting Procedure

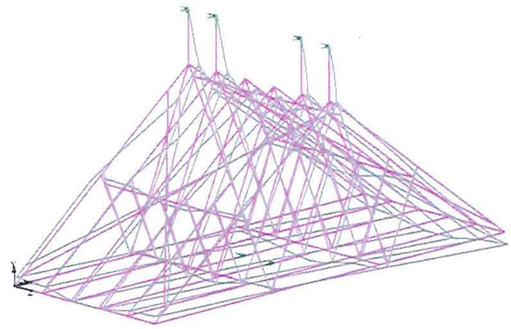
To develop a Best Practice Lifting Procedure a range of factors are required to be considered such as practicality, capital investment, logistics and Health & Safety. Of the range of criteria to be considered safety is the most important. The main Health & Safety issue relating to the lifting of a roof system is that of ensuring structural integrity. Failure of the roofing system under lifting conditions could result in an accident of major consequence. Therefore, to ensure structural integrity the verified structural model has been used to analyse a roofing system under a variety of lifting conditions. Shown in Figure 7.13 are examples of roof lift options considered and the deflected shapes of the system under the lifting conditions. Wind loading was not considered because lifting operations are deemed to be too hazardous during adverse weather conditions.

When developing the best practice procedure for lifting the following points were taken into consideration:

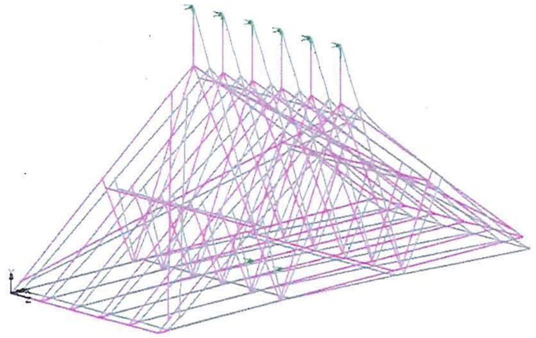
1. Method to be generic: The lifting procedure developed should be capable of being used on systems of varying configurations and bracing specifications.
2. Even load spread: Spreading the load evenly over the system will result in the load being transferred to the support point or 'crane hook' in a manner which will not overstress any particular elements of the system.
3. Redundancy: The devised method should, when considering large scale complicated systems, have a degree of redundancy so that if a lifting point fails the system will not fall.



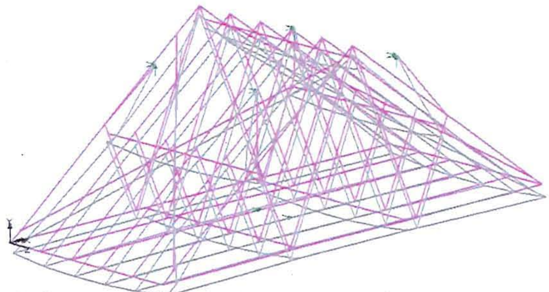
a) four point angled apex lift



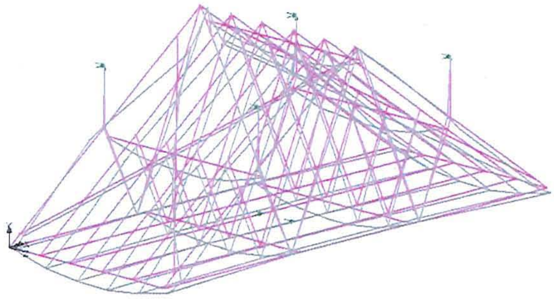
b) four point vertical apex lift



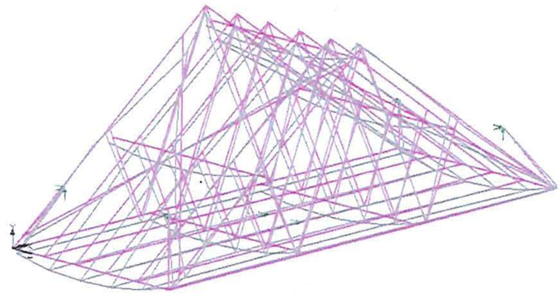
c) six point vertical apex lift



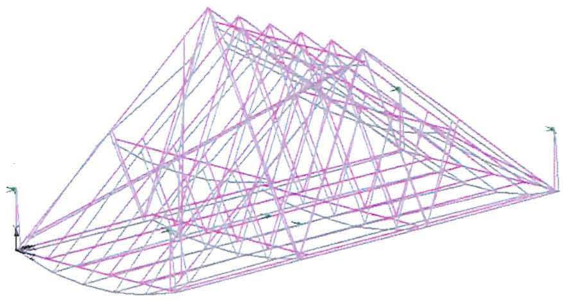
d) four point angled mid-span lift



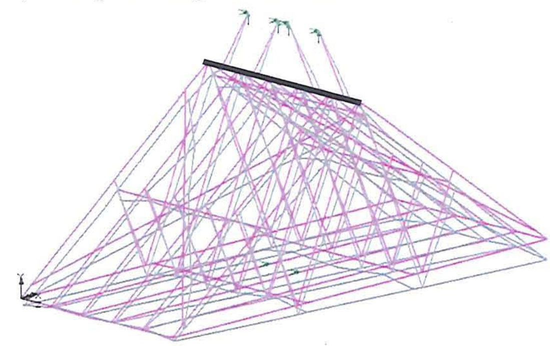
e) four point vertical rafter mid-span lift



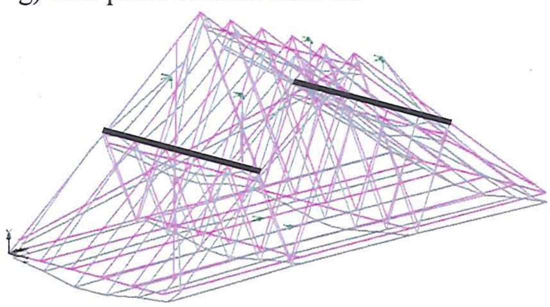
f) four point angled base lift



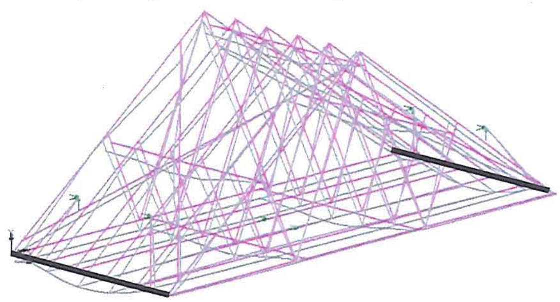
g) four point vertical base lift



h) 1 x spreader bar at apex



i) 2 x spreader bar at rafter mid-span lift



j) 2 x spreader bar at base lift

Figure 7.13 Sample lifting analysis and resulting deflected shapes (1:30 Scale)
(The chain/sling angle is set at a maximum of 60 degrees from the horizontal (LEEA, 1998)

Considering the above points different lifting methods were analysed and evaluated based on the following criteria:

- Even distribution of stresses between system elements.
- Even support reactions.
- Minimum system deflection.

7.3.5 Apex point lifting for complex systems

From the lifting analysis conducted it was concluded that apex point lifting is the optimum solution in terms of safety and practicality for complicated roof systems. Lifting directly from node points on the system with the lifting equipment set such that lifting forces act in the vertical plane eliminates out of plane deflection of the trusses and as a result additional stiffening and strengthening of the system is not required. The number of lifting points required can be optimised for practicality and apex point lifting would allow the lifting of complicated systems, examples of which are shown in Figure 7.14, because the configuration of lifting equipment would be a simple procedure.

Figure 7.14 Examples of varying roof types

To develop the Best Practice Lifting Procedure, which could be applied to all roof systems inclusive of complicated ones, a model representative of the largest run of trusses to be lifted was created. This model, which consisted of a run of 17 trusses and two end gable panels, was then used to optimise the number and positioning of lifting points.

The specification of diagonal and chevron bracing is dependent on individual circumstance and as a result cannot be relied upon in all cases for stability. For this reason only longitudinal bracing was included in the model.

The roof system section modelled may form part of a larger system consisting of extra sections such as hipped ends or T-sections. Differential movement of the system section is avoided by providing adequate support to the whole of the system i.e. each section of the system would be individually supported by means of apex point lifting. This would alleviate any problems associated with the position of the centre of gravity of a complicated system causing excessive sway and also ensure that elements of the system are not overstressed due to eccentric loading.

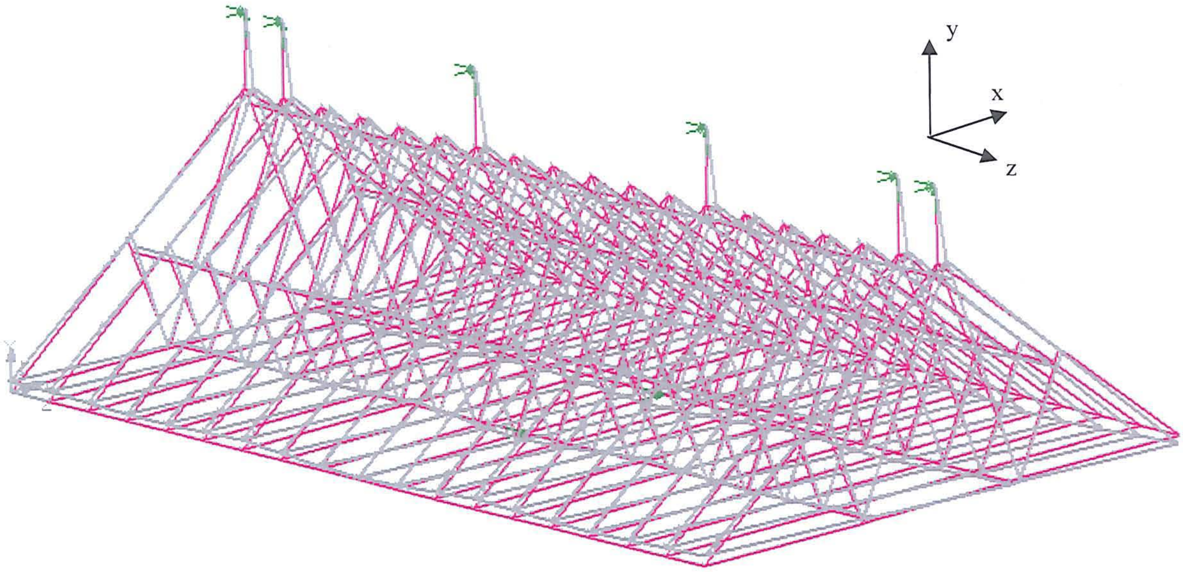
To reduce the number of required lifting points to an optimum level structural analysis of the modelled system was carried out and the components were checked to be within the design limitations of EC5 which are given in Table 7.6. Shown in Figure 7.15 and Figure 7.16 are examples of the system being analysed. It is demonstrated in Figure 7.15 and Figure 7.16 that the transfer of axial and bending forces through the system are within the limits set and there is a degree of redundancy should failure of a lifting point occur. Final design checks were then made on the optimised lifting procedure to ensure that the load carrying capacities of the system connections were not breached and finally guidance for different roof systems was provided. Figure 7.17 provides an illustrative guide to the optimum positioning of lifting points.

Table 7.6 Element design parameters to EC5

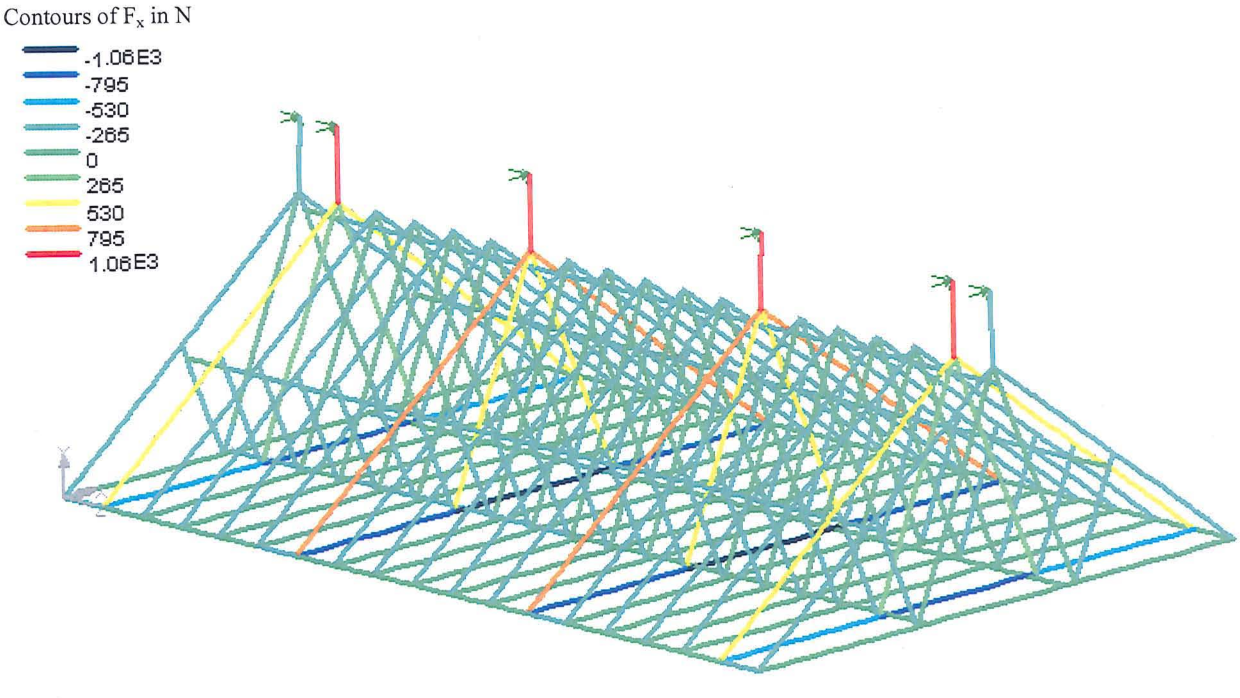
Element Information					Maximum allowable					
					Bending Moment		Tension	Buckling		Shear
Type	Grade	Breadth	Depth	Area	yy	zz	xx	zz	yy	
		mm	mm	mm^2	Nmm		N	N	N	N
Bottom chord	TR26	35	147	5145	660E+03	2770E+03	69655	52677	9578	10361
Top rafter	TR26	35	97	3395	436E+03	1210E+03	45963	34760	6320	6837
Bracing material	C14	22	100	2200	96E+03	434E+03	14892	43665	19360	2690
Web	C16	35	72	2520	199E+03	409E+03	21323	9480	3625	3262
Headbinder	C16	38	89	2875	290E+03	679E+03	20582	41510	30812	3722
Spreader element*	C24	45	190	8550	3.37E+06	2.63E+06	85702	15193	15193	15374

Notes:

- *C24 spreader element is used in the "upgraded longitudinal bracing method" (Section 7.4.2) and the beam is at a 35 degree angle to the horizontal.
- Instantaneous load conditions have been considered for services classes 1 & 2 therefore $k_{mod} = 1.1$.
- A material factor of $\gamma_M = 1.3$ has been applied.
- Material properties are in accordance with BS EN 338
- Compression modification factors k_{cz} and k_{cy} have been interpolated from Figure 3.5 of IstructE (2007).



a) Deflected shape (1:30 scale)

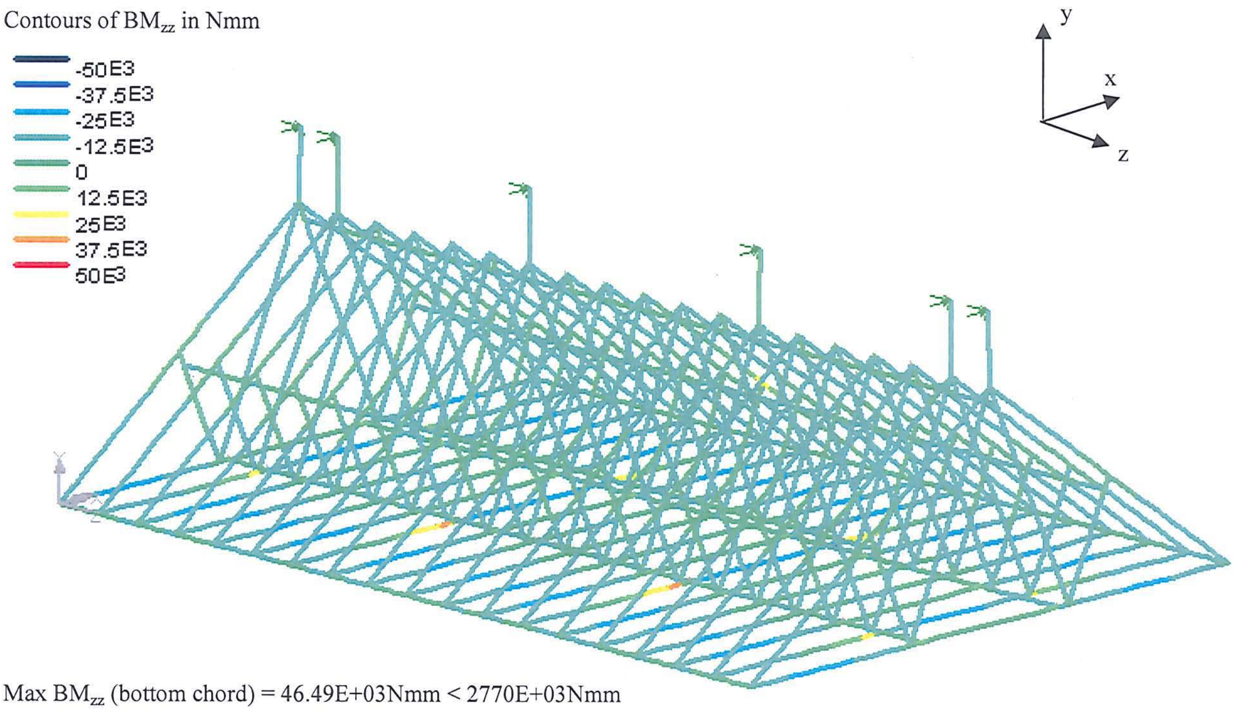


Max tension (rafter) = 1060N < 45693N
Max compression (bottom chord) = 1057N < 9578N

b) Axial force in elements

Figure 7.15 Deflection and element axial force in large scale system

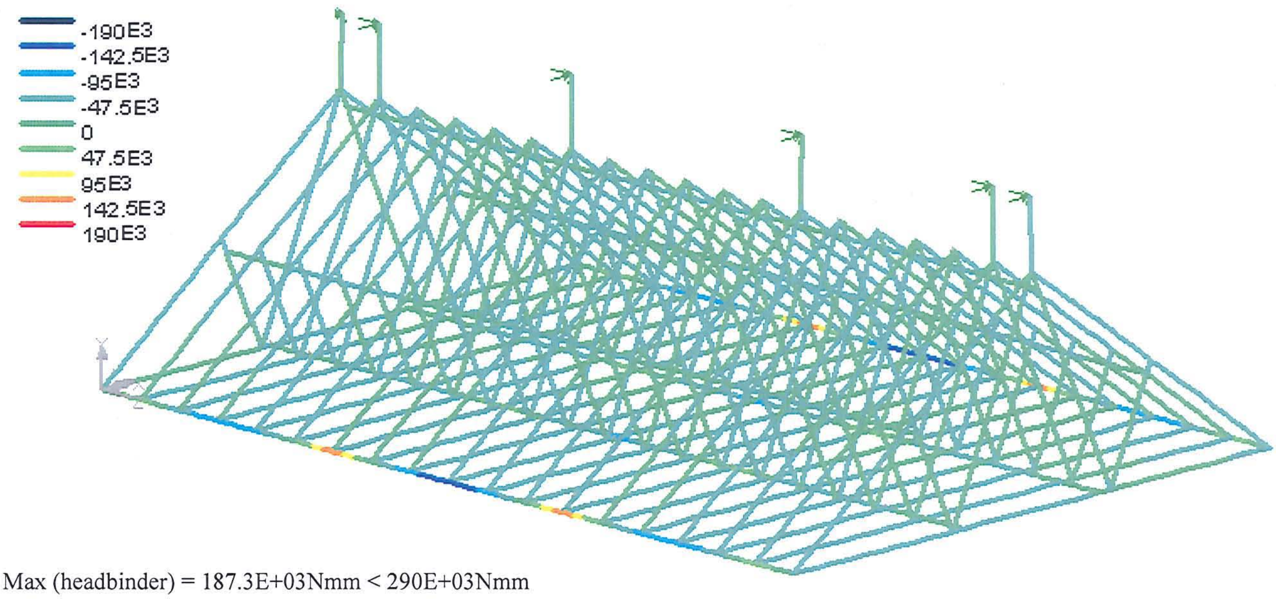
Contours of BM_{zz} in Nmm



Max BM_{zz} (bottom chord) = 46.49E+03Nmm < 2770E+03Nmm

c) Bending moment about z – z of elements (BM_{zz})

Contours of BM_{yy} in Nmm



Max (headbinder) = 187.3E+03Nmm < 290E+03Nmm

d) Bending moment about y – y of elements (BM_{yy})

Figure 7.16 Bending moments in large scale system

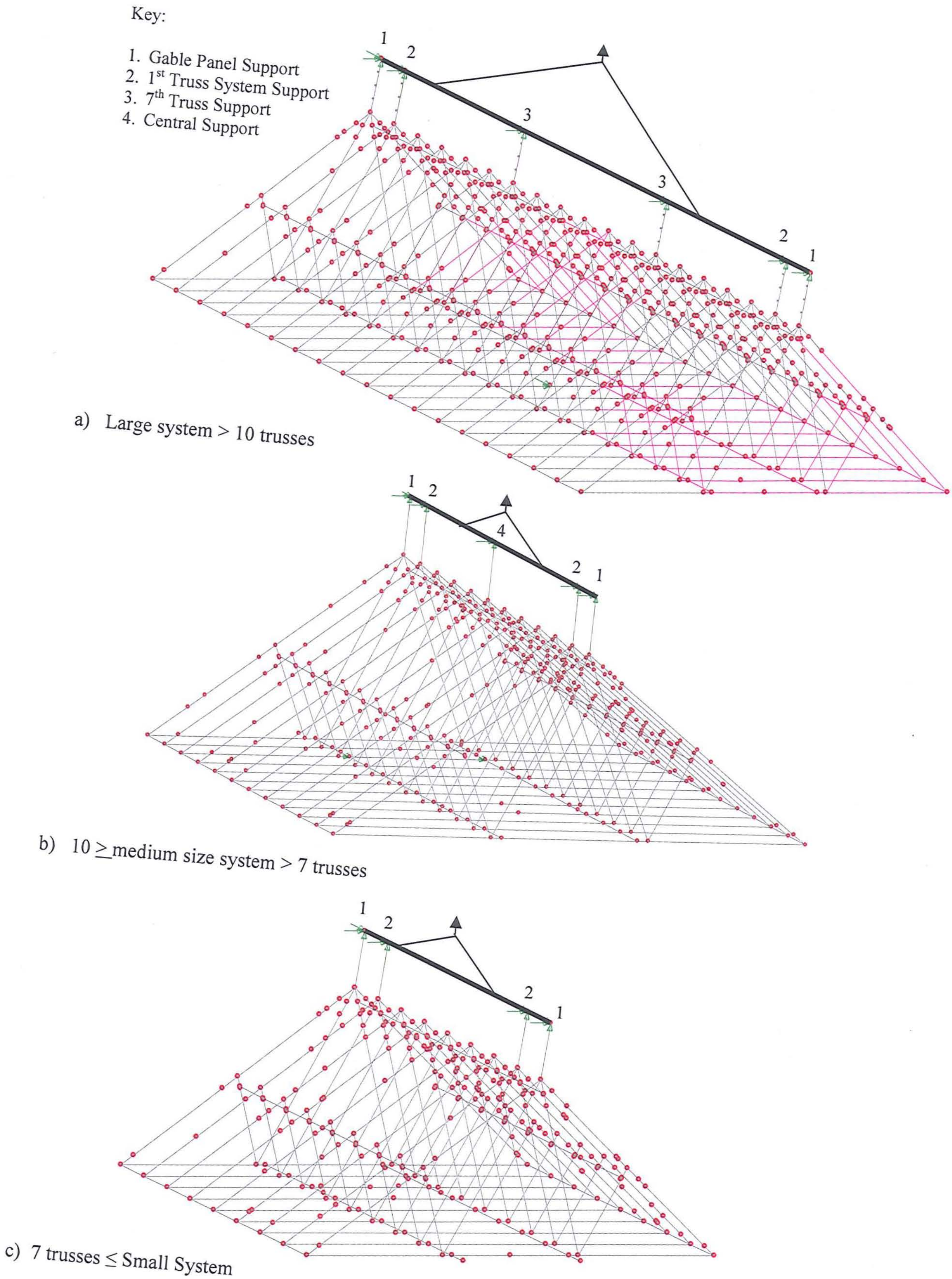


Figure 7.17 Optimal lifting points

7.3.6 Upgraded longitudinal bracing: standard roof systems

When considering a standard roof system, which consists of a run of fink trusses with a pitch angle not more than 35 degrees, the method derived for complicated roof systems is overly rigorous. A lifting beam would require a relatively high level of capital investment and would need to conform to European legislation. According to the Lifting Equipment Engineers Association (1998) the detailed requirement of the current legislation and the new regulations vary but collectively, in the context of lifting equipment, they require:

1. The equipment is safe and suitable for its purpose.
2. The personnel who use the equipment are suitably trained.
3. The equipment is maintained in a safe condition.
4. Records of conformity, test, examination etc are kept (a lifting beam would require to be tested once every 12 months).

As a result further research was conducted to find a "*simplified method*" of roof lifting which could be applied to standard roof systems which did not require special equipment. From the sample lifting analysis shown in Figure 7.13 lifting method "i) 2 × spreader bar at rafter mid-span lift" was considered. However, instead of the use of a lifting beam the concept of upgrading the longitudinal bracing elements from 22×100mm C14 to 45×190mm C24 timbers was investigated. The upgraded size was selected on the basis that it is the largest available size of standard lintel material which is not at a cost premium. To upgrade bracing for a run of 12 trusses would result in an additional cost of £10 per house (cost of 45×190 C24 taken as £2.10 per m run).

Upgraded bracing would form part of the roof system and would therefore not be classified as lifting equipment and as such would not have to conform to lifting equipment legislation. The upgraded bracing would function as bracing once the roof is in service and would improve the structural integrity of the system as it is an over-specification. In accordance with BS 5268:1998 – Part 3 Annex A.1 "*all bracing members are of minimum width 89mm and minimum depth 22mm*" and the following points from the code are noted due to their level of importance:

- *"All bracing members are nailed to every trussed rafter they cross with two 3.35mm diameter galvanized wire nails with a minimum length equal to the bracing thickness plus 32mm".* Therefore, the minimum nail length to be used is 77mm.
- *"Where bracing members are provided in two pieces, they are lap jointed over at least two trussed rafters and nailed as described above."*

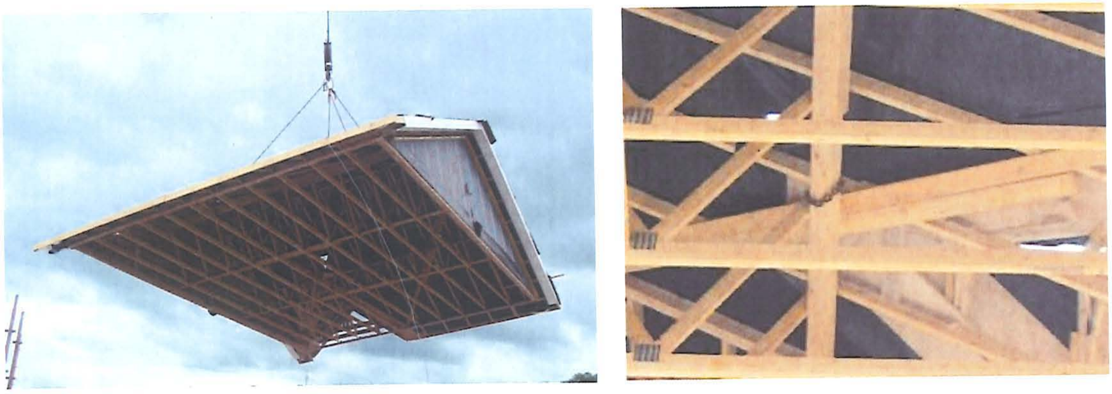
The provision of bracing above a minimum level derived from existing best practice for trussed rafter roofs can significantly enhance the stiffness of a roof for a comparatively small investment in materials and effort, a finding reported by Bainbridge et al (1998).

To ensure that the method of upgrading the longitudinal bracing (Figure 7.18) was structurally robust it was analysed using the qualified model (Figure 7.20). The analysis conducted demonstrated that the members of the truss system were capable of withstanding the applied stresses in the system under lifting conditions. Therefore, a safe lift will take place if the following points are adhered to:

- A maximum run of 12 trusses can be lifted using the simplified method if four lift points are assigned.
- The lifting points are designated so that even load distribution takes place. Setting the lift points one quarter of the way in from each end ensures that the maximum bending stress to be withstood by the lifting beam is minimised.
- The angle of the slings or chains should not be less than 60 degrees from the horizontal.

For systems where the run of trusses is greater than 12 the following recommendations are made:

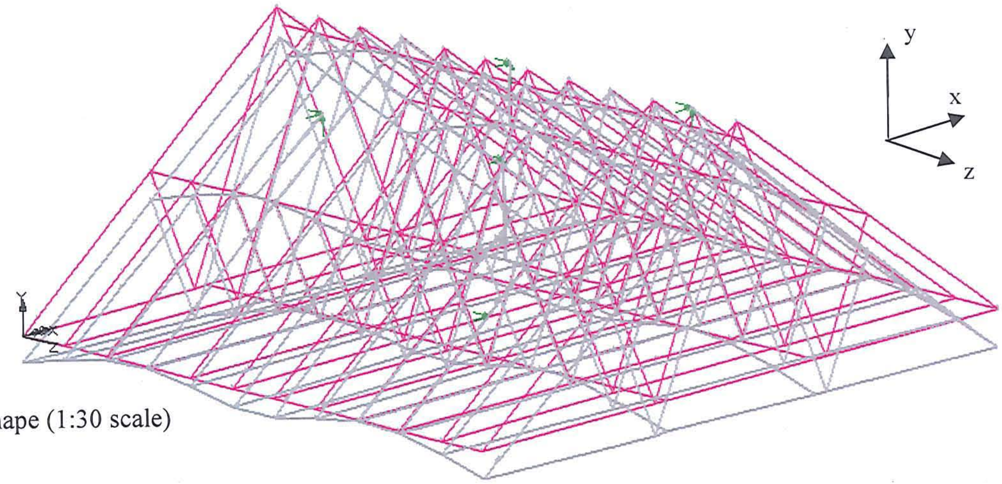
- More lift points could be specified as long as it is possible to configure the lifting slings or chains so that their angle from the horizontal is not less than 60 degrees, even strain takes place upon lifting and the number of trusses between lift points is not greater than 6 as shown in Figure 7.20.
- Higher strength timber or larger section timber could be specified. One option is the use of an engineered product such as laminated strand lumber (LSL) which has a higher bending strength capacity of 37.6N/mm^2 compared to 24N/mm^2 for C24. However, specification of an engineered product such as LSL would cost approximately 60% more and as a result the commercial viability is doubtful when considering a large number of houses.



a) 12 truss system being lifted

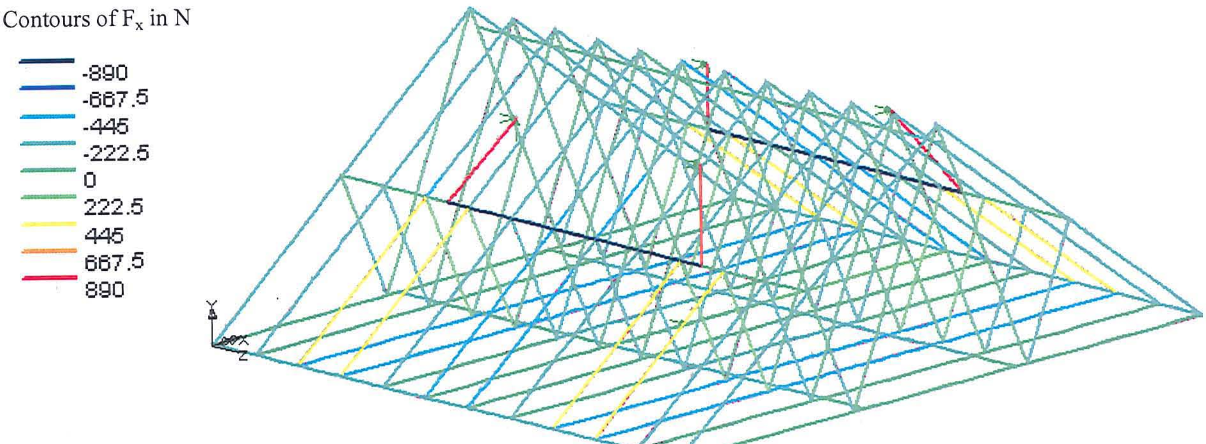
b) Reinforced bracing

Figure 7.18 Simplified method being applied



a) Deflected shape (1:30 scale)

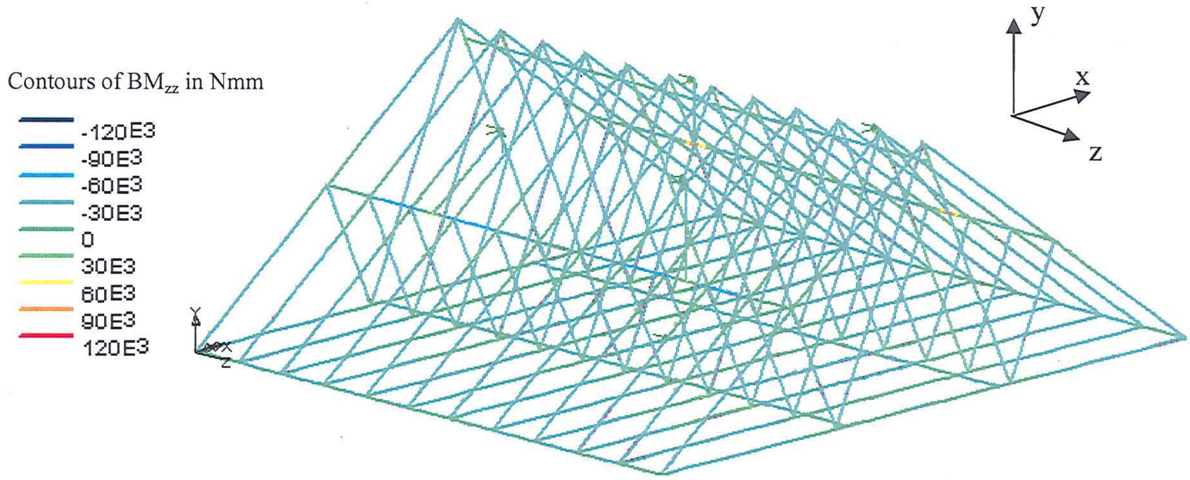
Contours of F_x in N



Max tension (rafter) = 528N < 85702N
Max compression (longitudinal bracing) = 897N < 15193N

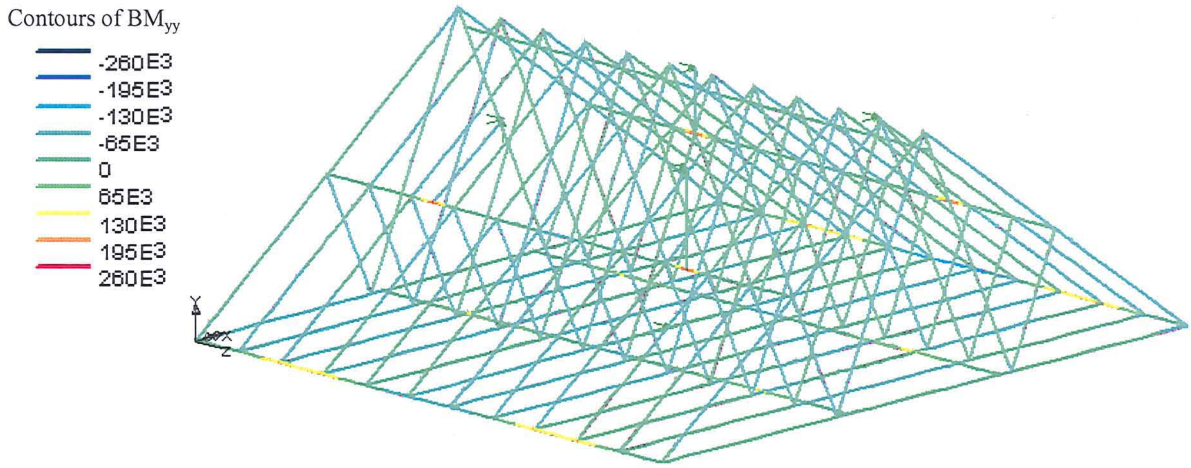
b) Axial force in element

Figure 7.19 Deflection and element axial force when lifting from upgraded longitudinal bracing



Max BM_{zz} (longitudinal bracing) = 121E+03Nmm < 337E+03Nmm

c) Bending moment about z – z of elements (BM_{zz})



Max BM_{yy} (longitudinal bracing) = 267E+03Nmm > 263E+03Nmm (1.5% failure accepted as neglible)

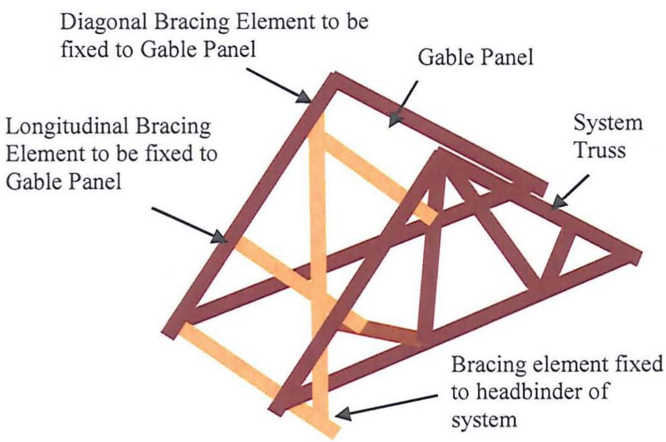
d) Bending moment about y – y of elements (BM_{yy})

Figure 7.20 Bending moments when lifting from upgraded longitudinal bracing

For lifting operations to be applied safely on-site the following recommendations are made:

1. Method statements and risk assessments are produced and provided to on-site staff prior to the execution of the work.
2. It is a Health & Safety requirement that the weight of anything which is to be lifted is known and supplied to site.
3. From the estimated weight of the roof the size and positioning of the crane is determined.

4. It is checked that any operations to be carried-out are in-line with site regulations.
5. Lifting operations are not to be carried out during adverse weather conditions as attempting to control the system during the lifting procedure would be hazardous.
6. Quality assurance procedures on-site should ensure that all system elements are in good conditions prior to construction.
7. Gable panels are tied into the system by the attachment of the bracing elements to them and the diagonal bracing elements are also to be fixed to the headbinder of the system (Figure 7.21).
8. All fixings are secured in accordance with the fixing specification.
9. The truss system is lifted and fixed into position:
    a. Lifting will be carried out in accordance with the specified lifting method and the chains/slings as provided will have been designed, checked and verified for safe working loads.
    b. The chains/slings are to be fixed to the pre-specified lifting points which have been designated in accordance with the specified method.
    c. Strain is placed on the chains/slings evenly such that the lift is even and optimum load spread is achieved.
    d. Gable panels are supported during lifting to eradicate the risk of failure in the headbinder.
    e. Chains/slings are applied with care to restrict movement on the lift and also limit the risk of damage to bracing elements.



a) Bracing detail

b) On-site application

Figure 7.21 Bracing details & on-site application

The crane erect method of construction requires floor cassettes, packs of panels as well as the roof system to be lifted. It is a health and safety requirement that the weight of any product being lifted is known and supplied to the crane operator. The weight of products is also required to be known by the appointed person who determines the lifting plan and positioning of the crane on-site. The size and positioning of the crane depends on the weight of products to be lifted. Therefore, knowing the weight of the products to be lifted is important for both Health & Safety and project planning reasons.

The three main products which are lifted into position during the crane erect method of construction are currently:

1. Wall panels packs.
2. Floor cassettes.
3. Roof systems.

To provide the information required a study was conducted to determine the average mass of the products being lifted and form a readily available and user friendly method for carrying out relatively accurate estimations. In communication with the people who require the information it was decided upon that a series of charts would be the most efficient method of providing the information from the study conducted examples of which are contained in Figure 7.22 to Figure 7.24.

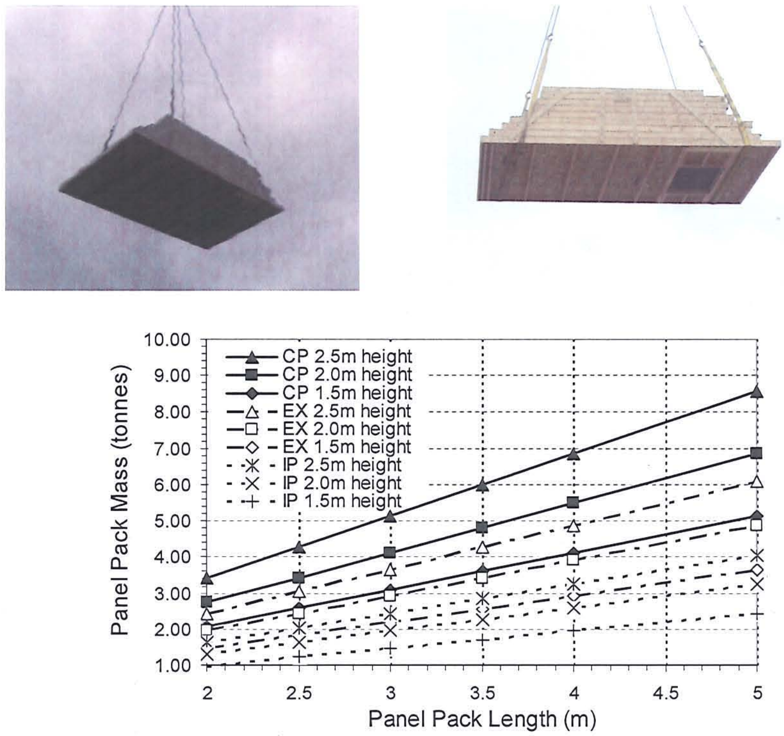


Figure 7.22 Panel pack mass plot and examples of panel pack lifting operations (CP – Closed panel; EX – External panel; IP – Internal partition)

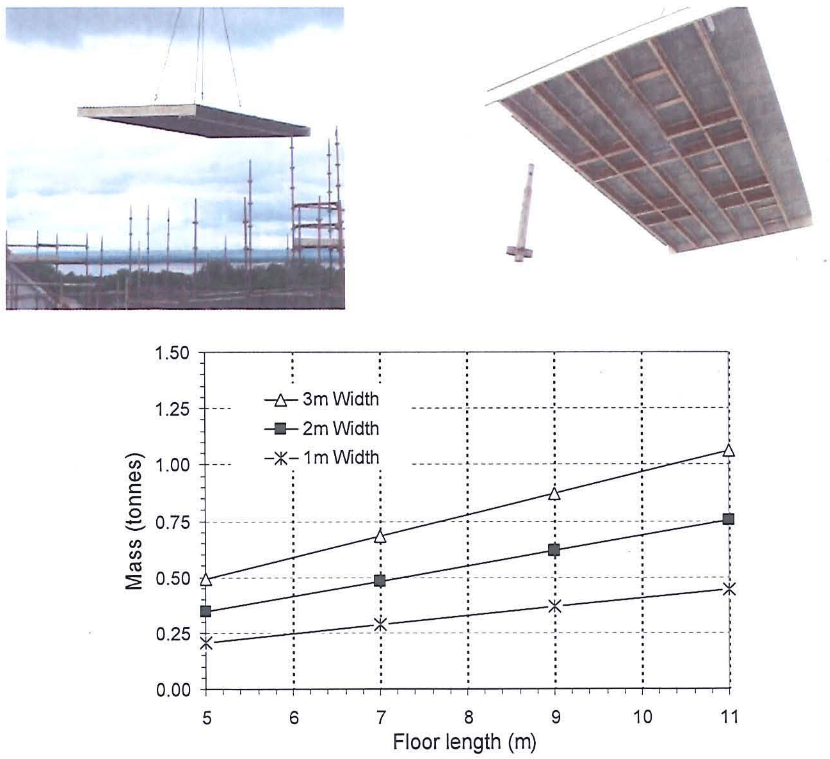


Figure 7.23 Cassette floor mass plot and examples of cassette floor lifting operations

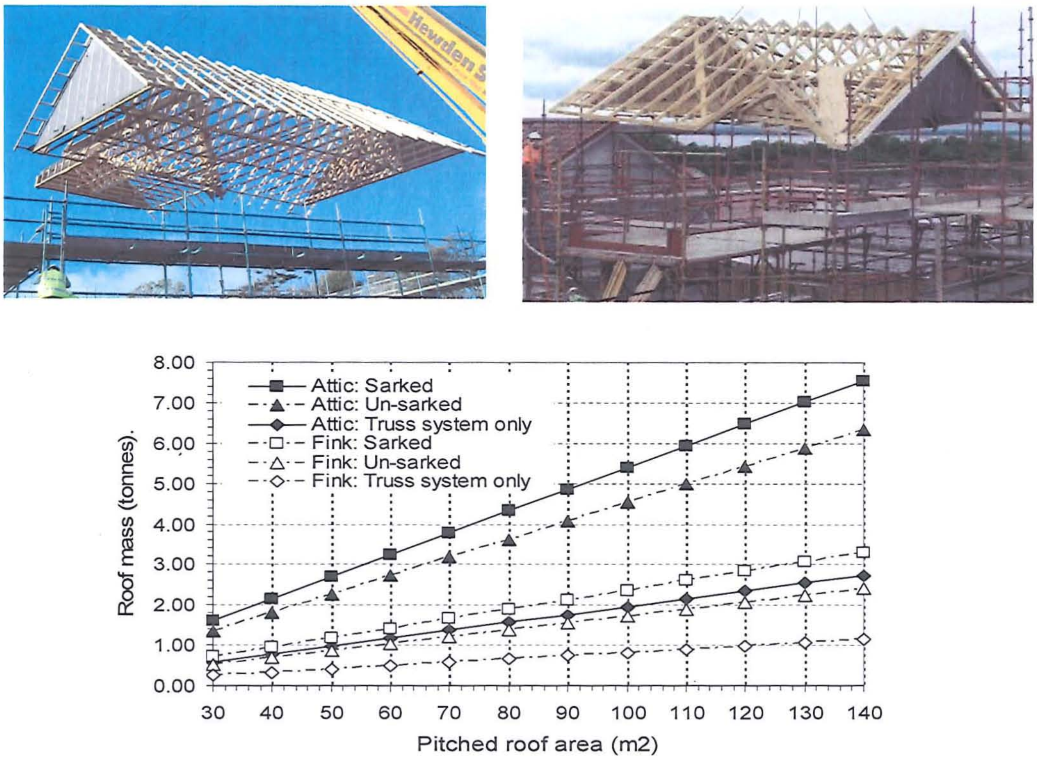


Figure 7.24 Roof mass plot and examples of roof lifting operations
(Truss system only is not including felt and battens.)

7.3.7 Summary

Using the developed model a large range of lifting procedures were investigated and evaluated based on even distribution of stresses between system elements, even support reactions and minimum system deflection. From the analysis conducted two best practice lifting procedures, apex point lifting from a spreader bar for complicated systems and mid-span lifting from upgraded longitudinal bracing for non-complicated systems, were derived. Further to this a study was conducted to quantify the mass of the products to be lifted into position during the crane erect process as a result of Health & Safety and project planning requirements.

7.4 Conclusions

The Crane Erect method of timber platform frame construction utilises off-site fabrication and on-site preparatory work to optimise the process. With the implementation of good project planning and improved on-site practices the crane erect construction method is a quicker, more cost effective and safer practice.

Of the lifting operations to be conducted roof lifting was highlighted as being the most hazardous. Developed in this chapter are two methods of lifting roofs which, depending on the system to be lifted, can be appropriately applied:

1. Apex point lift using a lifting beam: this method of lifting would allow complex roof configurations to be lifted without additional strengthening but would require capital investment, additional logistics planning and new quality procedures to be implemented.
2. Mid-span lift from upgraded longitudinal bracing: If a sufficient number of lifting points can be configured so that even strain takes place and adequate support is provided, then this method of lifting can be applied to the majority of standard house types constructed by major house builders.

The development of the Best Practice Lifting Procedures have engineered out the major safety issue of working at height and reduced the risk of system failure during lifting to a negligible amount. Further to this information required by on-site staff, so that safe working practices can be implemented has been provided in an easy to understand format facilitating adoption of the devised methods.

The devised methods are now used to erect the majority of units manufactured and supplied by Oregon Timber Frame Ltd which is approximately 1600 units per annum (2006 figures).

Published work

1. **Hairstans, R., Kermani, A. and Lawson R.** (2004). *"Crane erection of timber-trussed rafter roofs"*. Proceedings of the Institution of Civil Engineers. ICE Journal of Engineering Sustainability, No. 157, Issue ES2, pp 89-98.
2. **Hairstans, R., Kermani, A. and Lawson R.** (2003). *"Optimising timber frame housing construction"*. The First Scottish Conference for Postgraduate Researchers of the Built & Natural Environment (PRoBE), Glasgow Caledonian University, 18-19 November 2003, Glasgow, pp 261-269, ISBN: 1-903661-50-1.
3. **Hairstans, R.** (2004) *"Optimising timber frame construction"*, Proceedings of the Young Researchers Conference. Institution of Structural Engineers, London.

CHAPTER 8

CONCLUSIONS & RECOMMENDATIONS FOR FUTURE WORK

8.1 Introduction

The requirement to meet the demand for housing in the UK, the sustainability agenda, the skills shortage and the implementation of revised regulations and codes of practice fuelled the need for the research work conducted and documented in this thesis. Initiated under a government funded research programme, known as a Knowledge Transfer Partnership (KTP), a mutually benefiting alliance between Oregon Timber Frame Ltd and Napier University has been formed. The research work conducted through the collaborative partnership has resulted in the successful implementation of applied research improving the competitive position and awareness of the timber platform frame industry. Receiving a national award for excellence in 2005 from the Department of Trade and Industry the research work has resulted in a number of peer reviewed published work providing simplified design and specification techniques, implemented lean technology and assisted in the harmonisation process between current British Standards and new European Codes of Practice.

Further to the direct outputs, the project has also positively affected Oregon Timber Frame Ltd indirectly. The commercial decision making process of the business has been enhanced through increased levels of technical support and the availability of valuable information and resources through the knowledge based partner. The design procurement process is now more robust and the information streams between internal departments, clients and suppliers have been refined and improved.

Detailed in this section are the key findings of the research activities conducted, information on the level of implementation and future work requirements.

8.2 Conclusions

The main conclusions drawn from the research project are segregated relative to each sub-project for clarity.

8.2.1 Development of Timber Platform Frame Construction

Timber platform frame has evolved as a system from its early inception in North America in the 19th century and has adapted to become an off-site Modern Method of Construction (MMC). Specific to the UK market there are new challenges arising which require to be met:

1. Regarded as a sustainable material if sourced correctly timber is environmentally sound. The challenge for the timber platform frame industry is to improve the whole life cycle cost of the finished product which is the building envelope. This requirement is both consumer driven as environmental issues become more and more prevalent, and Government driven due to the endorsement of European legislation. To achieve the objective of improved building performance and environmental efficiency the use of new and existing products will have to be optimised and new technologies endorsed.
2. The design procurement process requires to become more efficient to ensure that timber platform frame is a robust, safe and serviceable system capable of adapting to the specification of new materials whilst also changing to meet the needs of new codes of practice, building regulations and certification procedures.
3. To compensate for the industry skills shortage off-site activities require to be increased and where appropriate automation used. To improve the efficiency of off-site activities, and also to allow for greater variations in the end product without impinging upon quality or cost, 'lean' techniques require to be endorsed. As the levels of off-site activities increase there will be more onus on quality assurance and improved system interfacing.
4. On-site procedures will have to adapt to suit the changes of the industry. As more work is carried out off-site the associated risks of a project will change. To reduce the level of associated risk global education is required such that an appreciation of the product, how it can be used and what it can achieve are understood at all levels. To eliminate client scepticism and reduce the associated risks of the off-site MMC used strong partnering arrangements between the manufacturer and the developer are imperative.

8.2.2 System Stability Analysis of Timber Platform Frame

As a result of carrying out a "Whole House Engineering" exercise a number of issues relating to the overall stability of timber platform frame systems were highlighted and the following conclusions were drawn:

5. Stiffness proportionality of a system is important if the optimum transfer of the applied wind actions to the foundations is to be achieved.
6. Connections are often the weakest links in a system and when considering racking resistance it is the connections between the component parts which can prove to be critical in determining the level of resistance a system has to applied wind actions.
7. The allowable level of racking resistance of a wall diaphragm is subject to the allowable level of shear force transfer between the wall footer and sole plate and subsequently the sole plate and substrate. When the make-up of the wall panel is specified so that a high level of racking resistance is to be attained increased levels of connectivity may be required to ensure that the level of force transfer required can take place. Increased levels of connectivity will require an improved level of fixing specification (either by number or type) between both the wall footer and sole plate and the sole plate and substrate. In connection detailing ductile behaviour is advantageous so that the on-set of failure is recognised and can be remedied prior to becoming critical.
8. The nailed connection between the wall footer and sole plate will tend to fail in a ductile manner relative to the connection between the wall footer and substrate as a result of this the connection between the sole plate and substrate should be over specified relative to the connection between the wall footer and sole plate. Using a derived function the level of fixity between the wall footer and sole plate and the sole plate and substrate has been optimised with due consideration to practicality and robustness. Further to this recommendations are also made to improve the cost effectiveness of the specification.
9. It is advisable to specify the connection between the sole plate and the substrate based on test results. However, the characteristic lateral load carrying capacity of timber to concrete connections when considering dowel type fixings can with good engineering judgement be safely determined using EC5 design methods.
10. The racking resistance of a wall panel is directly related to the level of holding down and it is normal for holding down to be provided by holding down straps although there may be a level of redundancy due to the self weight of the system. Reducing the holding requirement as a result of system redundancy is not recommended as the weight of the system is also used to increase the level of racking resistance of a wall diaphragm. It is not good practice to assign a level of holding down to shear fixings if they are being considered to provide lateral

resistance. However, if appropriately specified shear fixings could provide both shear and holding down resistance negating the requirement of holding down straps.

8.2.3 Development of a Stability Model

A stability model (based on the current British Standard codes of practice due to the non-availability of the UK National Annex for the superseding European Code) was developed covering a range of parameters corresponding to the normal limits of domestic dwellings constructed for volume house builders. The derived model was used to evaluate the influence of the variables which affect the level of percentage opening, often required architecturally in the front and back of houses, and subsequently the corresponding financial implications of the requirements. The following are the main conclusions of the study:

11. The developed model demonstrates a relatively high degree of accuracy when considering the level of variables involved and can be used with confidence for the initial design and financial analysis of systems within the boundaries set.
12. To achieve the often onerous level of opening required in the front and back of houses as a result of architectural layout there are two main options available which would help to alleviate the resulting cost implications. Foremost, the option of preference is to reduce the level of opening so that the required racking resistance is attainable through additional nailing rather than secondary sheathing. Secondly, the aspect ratio of the system can be optimised so that an increased level of opening is achieved without reducing the living area of the house itself.

8.2.4 Wall Diaphragms

A range of wall diaphragm options were considered in this section of the work. Initially the concept of optimising current wall detailing options was carried out with a view to meeting European Legislative requirements. Secondly, the use of Structural Insulated Panels (SIPs) was considered as an alternative wall construction solution due to its insulation properties. From the studies conducted several conclusions were drawn:

13. If the use of current timber sizes for framing material (38×89mm C16 strength grade timber) is to be continued then there are options available which employ a thermal laminate to improve the wall detail U-value. The options available, relative to each other, have comparable Life Cycle Assessment ratings and can achieve a target U-value of 0.27 W/m^{2}K. The advantage of maintaining 38×89mm framing members is that it would not require existing timber frame designs to be altered. However, it is more cost effective to increase the framing material size

to either 115mm or 140mm and use a glass wool with improved thermal conductivity. In so doing a U-value of 0.28 W/m^2K can be achieved and either other aspects of the whole building can be considered to achieve envelope compliance or a sheathing material such as bitumen impregnated fibre board, which has higher thermal resistance, can be used to attain the target U-value of 0.27 W/m^2K.

14. Although more expensive SIPs are a viable alternative to traditional stud wall diaphragms if the environmental whole life cycle cost of the house is considered to be of high importance as a result of SIPs providing additional insulation. In terms of structural performance they are capable of withstanding (to the levels required for domestic dwelling construction), vertical loading (direct compression), transverse wind loading (combined bending and compression) and in-plane lateral forces (racking loads).
15. From the comparative study conducted, in which both EC5 and British Standard racking resistance results for a comparable timber frame stud wall were compared to the experimentally determined racking resistance of a SIP wall, it was demonstrated that both design methods were capable of providing a conservative estimate of the racking resistance of a SIP wall. However, what the study also highlighted was the short comings of EC5 method B (the current adopted method of the UK for determining racking resistance to EC5) whereby openings formed in a wall can result in overly conservative design values depending on their size and position.

8.2.5 Shot Fired Dowel Flitch Beams

The primary function of a lean technology is to improve product quality by means of employing a holistic management philosophy whereby design and production of a product are improved by implementing more efficient practice requiring the understanding of the needs of the employees and customer. Research work conducted on shot fired dowel flitch beams has resulted in the implementation of a lean technology and the main conclusions from the research work carried out are as follows:

16. A feasibility study provides evidence that there are occasions where flitch beams are a legitimate beam solution due to restrictions within a timber platform frame system resulting in the requirement for enhanced beam stiffness due to onerous load span requirements. In such circumstance flitch beams offer a beam solution, which relative to other available alternatives, can be cost effective and practical.
17. The traditional method of flitch beam fabrication is a time consuming process due to the use of a bolted connection. The use of a shot fired dowel connection is a more efficient method of fabrication requiring a nominal level of staff training and dissemination of information

18. In design the specification of connections can be determine applying the design methods of EC5 and through conducting a study of the range of beams to be fabricated a generic nailing pattern can be implemented. To achieve this experimental work was conducted on connection strength which demonstrated that, for the range of samples considered, it is safe to take the full embedment depth of the nail and the withdrawal strength of the nail to equal head side pull through when carrying out design calculations.
19. Relative to design determined values, using code or manufacturer prescribed material properties in calculation, for the range of shot fired dowel flitch beams tested, it has been demonstrate that the fabrication method employed results in a beam which is on average 11% stiffer and 59% stronger in bending with the level of percentage improvement dependent on the variability of the timber elements.
20. Although shot fired dowel flitch beams are stiffer in bending their ability to resist deflection due to shear is reduced especially when considering flitch beams constructed from solid section timber. The application of shot fired dowels results in splitting of the timber elements which reduces their ability to transmit longitudinal shear forces and this subsequently results in increased levels of deflection. Therefore, a level of caution is required in design when high shear forces are to be carried. The use of engineered timber products such as Timberstrand LSL is recommended as a result of their ability to dissipate splitting forces during nail application.

8.2.6 Crane Erect Method of Timber Platform Frame Construction

The Crane Erect method of timber platform frame construction is demonstrated to be a quicker, more cost efficient and safer practice if good project planning is implemented. To develop a best practice procedure for roof lifting laboratory experiments and computer modelling techniques were used. As a result of the research work carried out the following conclusions have been drawn:

21. By committing to the Crane Erect method of timber platform frame construction there is a major change in associated risk to low incidence but high impact. To ensure that the risk of system failure during lifting is reduced to a negligible amount lifting points are required to be engineered in to all components to be lifted and the weight of the component should be supplied to the on-site staff.
22. Although when under lifting conditions a roof system only has limited points of support it is possible to produce a relatively accurate computer model such that the response of the system to varying lifting configurations can be analysed.
23. The model used to analyse different lifting procedures assumed the connections of the system to be pinned and did not account for joint slip. The connections of a truss rafter are formed

using punched metal plates which are semi-rigid resulting in a degree of moment transfer and, although marginal relative to other methods of timber connection, are subject to a degree of slip. Although this is the case the developed model was accepted as the important design consideration when deriving a lifting procedure is system robustness not serviceability.

24. Two lifting options are recommended for the safe lifting of roof systems, apex point lifting using a lifting beam and mid-span lifting from upgraded longitudinal bracing. For complicated roof systems apex point lifting is advised. However, considering the majority of standard house types constructed for volume house builders upgrading of the longitudinal bracing is sufficient if the lifting equipment can be correctly configured.

8.3 Future Work

As a result of the work contained in this thesis there has been a degree of implementation and with the research partnership between Oregon Timber Frame Ltd and Napier University on-going there is scope for future work. Again information on what has been implemented from the project and what the future work will be is segregated to each sub-project for clarity.

8.3.1 System Stability Analysis

Although the work carried out in this project area has been used to make recommendations to improve the stability of timber frame systems and also to allow the safe specification for connecting timber shear walls to the foundations further work is required in the following areas:

- The range of fixings tested could be extended further and also be inclusive of other forms of anchorage as higher strength racking panels may require the transfer of shear and overturning forces of a much larger magnitude.
- The range of timber products and substrates could be extended. In particular the use of engineered products such as Laminated Strand Lumber (LSL) and Parallel Strand Lumber (PSL) are to be considered.
- Methods of improving the pull-out and pull-through resistance of the fixings tested could be investigated. It is considered at this stage that the method of fixity could be evolved to provide both shear and holding down resistance negating the requirement for holding down straps in timber platform frame construction improving the build process.
- In addition to the study conducted into the shear & holding down capacity of the fixings; further information relating to cost, practicality and detailing could be included in the research. In particular foundation tolerance effects should be quantified and the effects of moisture ingress as a result of capillary action could also be investigated.

- The information from additional studies could be used in combination with the existing body of work to combine and optimise the use of shear fixings, racking/holding down plates and timber frame wall panel materials and levels of fixity in the form of a hybrid racking panel product capable of high levels of shear resistance.

8.3.2 Development of a Stability Model

Future work on the developed stability model and how it will be used are as follows:

- Due to the non-availability at the time of writing of the UK National Annex for the new European Code of Practice the model has been derived in accordance with British Standards. As a result future work will be required to re-model in accordance with the European Code.
- Further work on the model could result in it being used in conjunction with the enterprise resource planning (ERP) system being implemented by Oregon Timber Frame Ltd. It could be used to facilitate the estimating process as a means of determining what contracts are more cost effective in terms of architectural layout.
- The model could also be used to demonstrate to house builders what can be appropriately achieved in terms of building layout from a structural perspective with a view to striking an improved balance with architectural requirements.
- Currently the model is restricted to buildings up to 3 storeys and two types of roof construction. The parametric study could be extended further to cover more building and roof types so that it has a wider scope for application.

8.3.3 Wall Diaphragms

The investigation into the implications of new European environmental legislation and SIPs as an alternative form of wall construction has clarified future legislative requirements and quantified perceived industry threats. The information from this study could be used and progressed in the following ways:

- The current model for estimating the U-value of wall details could be developed further so that alternative cladding materials can be considered and financial and environmental costs are more intrinsic to its output.
- Currently the majority of panel product supplied by the timber platform frame industry is open panel; the future strategy of the industry is to evolve this product to a closed panel system capable of meeting 2010+ building regulations. The research work on wall detailing could form the basis of this research program with a view to quantifying building performance (thermal, sound and fire) through testing. This project could be combined with information

from the structural racking performance project to produce a certified panel product with optimal building and structural performance which is cost effective and capable of being manufactured without high levels of capital investment.

8.3.4 Shot Fired Dowel Flitch Beams

Shot fired dowel flitch beams have been incorporated in timber platform frame systems on the basis of the information from this sub-project. Information has been provided to the exterior consulting engineers to allow specification and nailing schedules have been provided to the off-site business sector to facilitate production. However, the product and level of available information could be improved further by committing to the following work:

- The research work conducted demonstrated that the application of nails at particular points can result in reduced beam strength. As a result further nailing patterns could be investigated such as increased end nailing and mid-span central nailing to determine whether improved performance can be achieved.
- Timberstrand LSL flitch beams have an improved level of stiffness relative to the size and thickness of steel section used compared to flitch beams constructed using C24 sections. Further analysis of this for a range of comparable beams with different steel plate thicknesses and dimensions could be carried out to determine which flitch beam configurations correspond to the most added value.
- The positioning of steel plate should be considered. The influence of steel along the neutral axis of a beam is negligible compared to the steel close to the top and bottom chord. Bending moment at the mid-span of a beam, when considering uniformly distributed loads will be greater. Therefore, it would be advantageous to test flitch beams with different steel configurations to determine whether the use of steel in flitch beams could by optimised whilst still being practical to fabricate.
- The majority of design circumstances in timber platform frame construction require flitch beams to carry uniformly distributed loads and as a result shear deflection is not normally the critical deflection component, although, additional levels of caution are required as a result of the research findings. Further investigation is deemed necessary as a means of improving the shear modulus of shot fired dowel flitch beams by improving the nailing specification.
- In accordance with Larsen and Mettem (2001) 18% more cumulative long term creep is to be allowed for in design. Further investigation into duration of load affects is to be carried out to quantify whether such an onerous design allowance is required especially when considering the use of engineered products for the production of shot fired dowel flitch beams.

8.3.5 Crane Erect Method of Timber Platform Frame Construction

The implementation of the Crane Erect method of timber platform frame construction has been successfully implemented and endorsed by major house builders. The research work conducted ensures the roof lifting methods are safe and provides the information required to allow the accurate estimation of all products to be lifted. The following future work would further enhance the research work already conducted:

- Continual monitoring of the roofing systems to be erected is required to ensure that the derived methods are suitable for all new roofing systems to be lifted. If required re-modelling of new systems may be necessary to prove that the recommended lifting procedures can be safely used.
- If new products are implemented into the timber platform frame system then the site information on component mass will have to be reviewed to incorporate the mass of new products.

REFERENCES

1. **Aicher, S., Klock, W., Dill-Langer, G. and Radovic, B.** (2003) *"Nails and nail plates as shear connectors for timber-concrete composite constructions*", Otto-Graf-Journal. Volume 14, pp 189-210.
2. **Alam, P and Ansell, M** (2003) *"Failure analysis and finite element modelling of timber-steel-timber composites by shot-fired bainitically-hardened nails and subjected to short-term lateral load",* ESWM-Stockholm.
3. **Alam, P.** (2004) *"The Reinforcement of Timber for Structural Applications and Repair*", PhD Thesis, University of Bath.
4. **Al-Mashari, M.,** (2002). *"Enterprise resource planning (ERP) systems: A research agenda*", Industrial Management and Data Systems 102 (3), 165–170.
5. **Alsmarker, T.** (1995), *"Diaphragms and Shear Walls"*, Timber Engineering Step 1, Lecture 4, ISBN 90 5645 001 8.
6. **American Plywood Association (APA).** (1983). *"Design and Fabrication of Plywood Sandwich Panels"*, Supplement No.4, APA, Tacoma, WA.
7. **American Plywood Association (APA).** (2007) www.apawood.com
8. **Anderson, B.** (2006) *"Conventions for U-value Calculations"*. British Research Establishment (BRE) Report BR 443, Garston, BRE Press, 2006.
9. **Anderson, J. and Howard, N** (2000) *"The Green Guide to Housing Specification"*, BRE Press, ISBN 1860813763.
10. **Andreasson, A.** (2000) *"Three dimensional interaction in stabilisation of multi-storey timber frame building systems"*, Licentiate Thesis, Division of Structural Engineering, Lund Institute of Technology, Lund University.
11. **Bainbridge, R. J. and Mettem, C. J.** (2002) *"Limit States Design to EC5 – Feedback From Designers in the UK"*, Journal of Institute of Wood Science, Vol 16 No 1.
12. **Bainbridge, R. J., Larsen, P., Mettem, C. J., Alam, P. and Ansell, M. P.** (2001) *"Timber-Steel Shot Fired Nail Connections at Ultimate Limit State"*, CIB-W18 Meeting 34, Venice, Italy.
13. **Bainbridge, R. J., Mettem, C. J., Gordon, J. A., Reffold, A. and Studer, T.** (1998) *"Stability bracing for non-domestic timber trussed rafter roofs"* The Structural Engineer, Volume 76/No 17, pp 334 – 338.
14. **Barker 33 Cross Industry Group** (2006) *"Modern Methods of Construction (MMC0 for the provision of housing – Technical report covering the barriers to the greater use of modern methods of construction and the mechanisms to overcome them"*. http://www.scri.salford.ac.uk/MMCFinalreport.pdf

15. **Barker, K.** (2004) *"Review of Housing Supply - Securing our Future Housing Needs: Final Report Recommendations"*, Her Majesty's Stationery Office, ISBN 1 84532 010 7.

16. **Bergstrom, M and Stehn, L** (2005) *"Matching industrialised timber frame housing needs and enterprise resource planning: A change process"*, International Journal of Production Economics, pp 172 – 184.

17. **Blass, H. J.** (1995) *"Multiple Fastener Joints"*, Lecture C15, Timber Engineering Step 1 Basis of Design, Material Properties, Structural Components and Joints, ISBN 90 5645 001 8.

18. **British Board of Agrément (BBA).** (2002), *"Kingspan Tek Haus Building Systems"*, Agrément Certificate No 02/S029, P O Box No 195, Bucknalls Lane, Garston, Watford, Herts WD25 9BA

19. **British Standard Institution (BSI).** (1999) BS EN 1380:1999: *"Timber structures – Test methods – Load bearing nailed joints"*, ISBN 0 580 32889 9.

20. **British Standard Institution (BSI).** (1999) BS EN 1382:1999: *"Timber structures – Test methods – Withdrawal capacity of timber fasteners"*, ISBN 0 580 32888 0.

21. **British Standard Institution (BSI).** BS EN 409:1993: *"Timber structures – Test methods – Determination of yield moment of dowel type fasteners – nails"*. ISBN 0 580 22317 5.

22. **British Standards Institution (BSI).** (1993) BS EN 409:1993: *"Timber structures – Test methods – Determination of yield moment of dowel type fasteners – nails"*, ISBN 0 580 22317 5.

23. **British Standards Institution (BSI).** (1992) BS EN 1382:1999: *"Timber structures. Test methods. Withdrawal capacity of timber fasteners"*. ISBN 0 580 32888 0.

24. **British Standards Institution (BSI).** (1996). BS 5268: Section 6.1: 1996 *"Structural Use of Timber, Part 6. Code of Practice for Timber Frame Walls, Section 6.1 Dwellings Not Exceeding Four Storeys"*, ISBN 0 580 24969 7.

25. **British Standards Institution (BSI).** (1996). BS EN 594: *"Timber Structures – Test Methods – Racking strength and stiffness of timber frame wall panels"*, ISBN 0 580 25724 X.

26. **British Standards Institution (BSI).** (1997) BS 8110-:1997: *"Structural use of concrete. Code of practice for design and construction"*, ISBN 0 580 26208 1.

27. **British Standards Institution (BSI).** (1998) BS 5268-3:1998 *"Structural use of timber - Part 3: Code of practice for truss rafter roofs"*. ISBN 0 580 29516 8.

28. **British Standards Institution (BSI).** (2001) BS 5950-1:2000: *"Structural use of steelwork in building. Code of practice for design. Rolled and welded sections"*, ISBN 0 580 33239 X.

29. **British Standards Institution (BSI).** (2001) BS 5950-1:2000: *"Structural use of steelwork in building. Code of practice for design. Rolled and welded sections"*, ISBN 0 580 33239 X.

30. **British Standards Institution (BSI).** (2001) BS EN 10002-1:2001: *"Metallic materials – Tensile testing – Part 1: Method of test at ambient temperature"*, ISBN 0 580 38459 4.

31. **British Standards Institution (BSI).** (2002) BS 5268-2:2002: *"Structural Use of Timber – Part 2: Code of practice for permissible stress design, materials and workmanship"*, ISBN 0 580 33314 0.

32. **British Standards Institution (BSI).** (2002) BS EN 1990:2002: *"Eurocode – Basis of structural design",* ISBN 0 580 40186 3.

33. **British Standards Institution (BSI).** (2002) BS EN 1991-1-4:2005 *"Eurocode 1: Actions on Structures - Part 1-4: General actions – Wind"*. ISBN 0 580 45959 4.

34. **British Standards Institution (BSI).** (2002). BS 6399-1:1997: *"Loadings for buildings, Part 1: Code of practice for dead and imposed loads"*, ISBN 0 580 26239 1.

35. **British Standards Institution (BSI).** (2002). BS 6399-1:1997: *"Loadings for buildings, Part 3: Code of practice for imposed roof loads"*, ISBN 0 580 16577.

36. **British Standards Institution (BSI).** (2003) BS EN 1991-1-1:2003: *"Eurocode 1. Actions on structures. General actions. Snow loads"*. ISBN 0 580 42307 7.

37. **British Standards Institution (BSI).** (2003) BS EN 338:2003: *"Structural timber. Strength classes"*, ISBN 0 580 41845 6.

38. **British Standards Institution (BSI).** (2003) BS EN 338:2003: *"Structural timber. Strength classes"*, ISBN 0 580 41845 6.

39. **British Standards Institution (BSI).** (2006) BS EN 1995-1-1:2004: *"Eurocode 5: Design of timber structure – Part 1.1: General – Common rules and rules for buildings"*, ISBN 0 580 45147 X.

40. **British Standards Institution (BSI).** (2006) BS EN 408:2003 *"Timber Structures – Structural Timber and Glued Laminated Timber – Determination of Some Physical and Mechanical Properties"*, ISBN 0 580 45171 2.

41. **Brunskill, R. W.** (1994) *"Timber Building in Britain"*, Butler and Tanner Ltd, ISBN 0-304-36665-X.

42. **Byfield, M. and Nethercot, D.** (2001) *"Eurocodes – Failing to standardise safety"*, Proceedings of Institution of Civil Engineers, Civil Engineering 144 Nov 2001, pp 186–188 Paper 12633.

43. **Camichael, E. N.** (1984) *"Timber Engineering – Practical Designers Guide"*, E & F. N Spoon, ISBN 0 419 12700 3.

44. **Cathcart, C. M.** (June, 1998) *"Technology: SIPs, Not Studs"*, Architecture, 148 – 152. Canada (http://www.kisscathcart.com/articles.html).

45. **Ceccotti, A.** (1995) *"Timber-concrete composite structures"*, Timber Engineering Step 2, Lecture 4, ISBN 90 5645 001 8.

46. **Chilton, J.C.** (1995) *"History of Timber Structures"*, Timber Engineering Step 2, Lecture 4, ISBN 90 5645 001 8.

47. **Choo, B. S.** (1995) *"Timber Engineering Step 1 Basis of design"*, material properties, structural components and joints, Lecture B3 Bending, ISBN 90-5645-001-8.

48. **CINTRAFOR** (2001) *"Center for International Trade in Forest Products"*, University of Washington, College of Forest Resources, Box 352100, Seattle, WA 98125-2100.

49. **Chartered Insitute of Building (CIOB)** (2003) *"Code of Practice for Project Management for Construction and Development"*. Ascot, Englemere Limited.

50. **Cohen, D. H.** (1994) *"A history of marketing of British Columbian softwood lumber"*, Forestry Chron. 70(5):578-584.

51. **Construction Task Force** (1998) *"Rethinking Construction, Department of Trade and Industry"*, Crown Copyright, URN 03/951.

52. **Cook, N. J.** (1998a) *"Comparing CP3 and BS 6399: Part 2 – ground roughness"*, The Structural Engineer, Volume 76/No 4.

53. **Cook, N. J.** (1998b) *"Comparing CP3 and BS 6399: Part 2 –'division by parts'"*, The Structural Engineer, Volume 76/No 6

54. **Cook, N. J.** (1998c) *"Comparing CP3 and BS 6399: Part 2 – UK wind climate and design dynamic pressure"*, The Structural Engineer, Volume 76/No 13.

55. **Crowley, A.,** (1998). *"Construction as a manufacturing process: Lessons from the automotive industry"*, Computers and Structures 67 (5), 389–400.

56. **Davis, T. J and Claisse, P. A.** (2000) *"Resin-Injected Dowel Joints in Glulam and Structural Timber Composites"*, Construction and Building Materials, v 15, n 4, June, 2001, p 157-167.

57. **Department for Environment Food and Rural Affairs (DEFRA).** (2005) *"The Government's Standard Assessment Procedure for Energy Rating of Dwellings"*, BRE, Garston, Watford WD25 9XX.

58. **Department of Trade and Industry (DTI).** (2005), *"Knowledge Transfer Partnerships Annual Report 2004/2005"*, http://www.ktponline.org.uk.

59. **Desai, S. (2003)** *"Load Test on Flitch Beams: Lessons From History"*, The Structural Engineer, Vol 81 no4 pp 20 – 21.

60. **Desch, H. E and Dinwoodie, J. M.** (1996), *"Timber Its Structural Properties and Utilisations"*, 7th Edition, MacMillan, ISBN 0 333 25752 9

61. **Dias, A and Cruz, H.** (2004) *"Experimental shear-friction tests on dowel type fastener timber-concrete joints"*, Proceedings of the World Conference on Timber Engineering 2004, pp 305 – 308, ISBN 0356-9403.

62. **Dinwoodie, J. M.** (2000) *"Timber Its Nature and Behaviour"*, Van Nostrand Reinhold Company, ISBN 0 419 23580 9.

63. **Doran, S. M.** (2006) *"Timber frame dwellings - Conservation of fuel and power: ADL1A guidelines"*, Special Digest SD2:2006, BRE Press, Garston, Watford, WD25 9XX, ISBN 1 86081 923 0.

64. **Department of Trdde and Industry (DTI)** (2004) *"Global Watch Mission Report – Modern methods of construction in Germany – playing the off-site rule"*, Department of Trade and Industry, URN 04/1430.

65. **Eaton, R. A. and Hale, M. D. C.** (1993) *"Wood: Decay, pests and protection"*, Chapman and Hall.

66. **Elmhurst Energy Rating Systems Limited** (2007), *"Elmhurst SAP Energy Rating Software"*, Elmhurst Farm, Bow Lane, Withybrook, Nr. Coventry CV7 9LQ, e-mail: enquiries@elmhurstenergy.co.uk.

67. **Enjily, V. and Griffiths, R. D.** (1996) *"The Racking Resistance of Large Wall Panels"*, Proceedings of the Institute of Wood Engineering Conference., Volume 2, Omnipress, Madison, Wisconsin. 321 – 328.

68. **European Committee for Standardization (CEN).** (2004) Eurocode 5 Design of timber structures Part 1-1: General – Common rules and rules for buildings, London.

69. **European Parliament** (2002) *"Directive 2002/91/EC of the European Parliament and of the Council of 16 December 2002 on the energy performance of buildings"*, OJ L1, 4.1.2003, p. 65-70.

70. **Foschi, R. O.** (1977) *"Analysis of wood diaphragms and trusses. Part II: Truss-plate connector"*. Can. J. Eng., 4, 353-362.

71. **Gibb, A. G. F and Isack, F.** (2003) *"Re-engineering Through Pre-Assembly: Client Expectations and Drivers"*, Building Research and Information, Issue 31(2), pp 146 – 160, ISSN 0961 3218.

72. **Gilfillan, J. R.** (2001) *"Enhancement of the Structural Performance of Glue Laminated, Homegrown Sitka Spruce Using Carbonfibre-Reinforced Polymer"*, Structural Engineer, Vol 79 No 8, ISSN 0039-2553.

73. **Gillan, G. Cameron, I. and Duff, R. A.** (2003) *"A technical evaluation of fall prevention and protection methods utilised for efficient and effective height safety management"*, Proceedings of The 1st Scottish Conference for Postgraduate Research of the Built and Natural Environment, Glasgow Caledonian University, UK. pp 479 – 492.

74. **Glos, P.** (1995) *"Strength Grading, Timber Engineering Step 1"*, Lecture 4, ISBN 90 5645 001 8.

75. **Goodier, C. and Gibb, A** (2005) *"The value of the UK market for off-site, BuildOff-site: promoting construction Off-site"*, www.buildOff-site.co.uk.

76. **Grimsdale, P.** (1995) *"Timber Frame Construction, Timber in Construction"*, Butler & Tanner Ltd, ISBN 0 7134 5053 3.

77. **Hare, B., Maloney, B., Cameron, I. and Duff, A. R.** (2005) *"Improving consultation and worker engagement in the construction industry"*, Proceedings of the Second Conference for

Postgraduate Research of the Built and Natural Environment, Glasgow Caledonian University, UK. pp 151 – 160.

78. **Harris, R.** (2004) *"21st Century Timber Engineering – The Age of Enlightenment for Timber Design Part 1: Environmental Credentials"*, The Structural Engineer, December 7th, pp 23 – 28.

79. **Harris, R.** (2005) *"21st Century Timber Engineering – The Age of Enlightenment for Timber Design Part 2: Environmental Credentials"*, The Structural Engineer, January 18th, pp 52 – 57.

80. **Hegazy, T.** (1999) *"Optimisation of Construction Time-Cost Trade-Off Analysis Using Genetic Algorithms"*. Proc. Institution of Canadian Civil Engineers, Vol 26, pp 685-697.

81. **Heikkilä, J. and Suikkari, R.** (2001) *"Log Structures in Finnish Architecture – Continuing the Tradition"*, University of Oulu, Department of Architecture, P.O. Box 4100, FIN 90014 Oulu, Finland.

82. **Hilson, B. O.** (1995) *"Joints with dowel-type fasteners – Theory"*, Timber Engineering Step 1, Centrum Hout, ISBN 90 5645 001 8.

83. **Hoffmeyer** (1995) *"Wood as a Building Material"*, Timber Engineering Step 1, Lecture A4, ISBN 90 5645 001 8.

84. **Hunt, D.R.** (1996) *"The genera of temperate broadleaved tree"*, *Broadleaves* 2:4-5 IUCN.1994. *IUCN Red List Categories*. IUCN, Gland, Switzerland

85. **Hunton Fibre Ltd** (1994) *"Hunton Bitroc and Hunton Bitvent"*, British Board of Agrement, Agrement Certificate No 02/3966.

86. **I-Level** (2006) *"Timberstrand® LSL – Structural Use – 1.5 E & 1.7 E "S" Qualities"*, Characteristic Mechanical Properties (values to be used with prEN 1995-1-1).

87. **The Institution of Structural Engineers (IstructE).** (2007) Draft *"Manual for the design of timber building structures to Eurocode 5"*, Published by The Institution of Structural Engineers, London, SW1X 8BH.

88. **JAWIC** (2001) *"Wood supply and demand information service"*, December 2001, Tokyo, Japan. 16pp.

89. **Johansen, K.W.** (1949) *"Theory of timber connections"*. International Association of Bridge and Structural Engineering. Publication No. 9:249-262. Bern.

90. **Johansson, C. J.** (2003) *"Grading of Timber with Respect to Mechanical Properties"*, Timber Engineering, WILEY ISBN 0 470 84469 8.

91. **Johnson, A. C. and Dolan, J. D.** (1996). *"Performance of long shear walls with openings"*, Proceedings of the Institute of Wood Engineering Conference., Volume 2, Omnipress, Madison, Wisconsin. 337-344.

92. **Kanerva, P., Peltola, S. and Vesa, J.** (2004) *"Design method of rotational stiffness of mechanical joints on the design of timber structures"*, Proceedings of the World Conference on Timber Engineering, Lahti, Finland, Volume 2, pp 265 – 271.

93. **Kermani, A.** (2005) *"Performance of Structural Insulated Panels: Combined Bending and Axial Compression"*. Institution of Civil Engineers, Structures and Buildings, Vol. 159. Issue SB1. pp 13–19, (http://www.ice.org.uk/services/services_journals.asp).

94. **Kessel, M. K.** (1995) *"Trusses"*, Timber Engineering Step 1, Lecture B12, ISBN 90 5645 001 8

95. **Prion, H. G. L and Lam, F.** (2003) *"Engineered Wood Products for Structural Purposes"*, Timber Engineering, Wiley, ISBN 0 470 84469 8.

96. **Larsen, H. J.** (1995) *"Limit State Design and Safety Format"*, Timber Engineering Step 1 Basis of Design, Material Properties, Structural Components and Joints, Salland De Lange, Deventer, ISBN 90 5645 001 8.

97. **Larsen, P. and Mettem, C. J.** (2001) *"New Age Flitch – Case Study of Innovation Into Practice"*, TRADA Technology, UK.

98. **Lawson R. M., Ogden R. G., Pedreschi R., Grubb P. J. and Popo-Ola S. O.** (2005) *"Developments in pre-fabricated systems in light steel and modular construction"*, The Journal of the Institution of Structural Engineers.

99. **Lee, A. D.** (1997) *"Innovations in Manufactured Housing: Structural Insulated Panels"*, Pacific Northwest National Laboratory George James, U.S. Department of Energy.

100. **Lifting Equipment Engineers Association (LEEA).** (1998) *"Lifting Equipment a Users Pocket Guide"*. Lifting Equipment Engineers Association, Bishop's Stortford, Herts.

101. **Martensson, A.** (2003), *"Short and Long-Term Deformations of Timber Structures"*, Timber Engineering, Wiley, ISBN 0 470 84469 8.

102. **Marwood Group Ltd** (2007) www.marwoodgroup.co.uk

103. **Mettem C. J., Gordon J. A. and Bedding B.** (1996) *"Structural Timber Composites"*, TRADA Technology, ISBN 1 00510 01 4.

104. **Milner, M.** (2003) *"A Briefing Guide to the Use of Structural Insulate Panels (SIPs)"*, TRADA Technology Publication, UK.

105. **Mungwa, M. S., Jullien, J., Foudjet, A. and Hentges, G.** (1999) *"Experimental study of a composite wood-concrete beam with the INSA-Hilti new flexible shear connector"*, Construction and Building Materials, pp371-382.

106. **National Audit Office** (2005) *"Using Modern Method of Construction to Build Homes Quickly and Efficiently"*, NAO Information Centre, DG Ref 5704rf.

107. **Nielsen, J.** (2005) *"Timber Engineering – General Introduction"*, Timber Engineering, Wiley, ISBN 0 470 84469 8.

108. **Office of the Deputy Prime Minister (ODPM)** (2006) *"Approved Document L1A: Conservation of fuel and power (New dwellings)"*, ISBN 978 1 85946 217 1.

109. **Pan, W., Gibb, A and Dainty, A.** (2005) *"Modern Methods of Construction in Housebuilding - Perspectives and Practices of Leading UK Housebuilders"*, Loughborough University, ISBN 0 86017 915 X Publication X117.

110. **Parliamentary Office of Science and Technology (POST).** (2003) *"Modern methods of house building"*. POST Note Number 209

111. **Patton-Mallory, M., Wolfe, R, W., Soltis, L. A., and Gutkowski, R. M.** (1985). *"Light-frame Shear Wall Length and Opening Effects"*, Journal of Structural Engineering., ASCE, 111(10), 2227–2239.

112. **Persaud, R and Digby, S.** (2006) *"Design and testing of a composite timber and concrete floor system"*, The Structural Engineer, 21st February 2006 pp 22-30.

113. **Prion, H. G. L. and Lam, F.** (2003) *"Shear Walls and Diaphragms"*, Timber Engineering, Wiley, ISBN 0 470 84469 8.

114. **Pryce, W.** (2005) *"Architecture in Wood a World History"*, Thames & Hudson Ltd, ISBN 0 500 34213 X.

115. **Racher, P.** (1995) *"Mechanical Timber Joints – General"*, Step Lecture C1, Timber Engineering Step 1 Basis of Design, Material Properties, Structural Components and Joints, ISBN 90 5645 001 8.

116. **Response Safety Netting** (2007) www.safetynettingcompany.co.uk

117. **Robertson, R. A. and Griffiths, D. R.** (1981) *"Factors Affecting the racking resistance of timber framed panels"*, Journal of the Institution of Structural Engineers, Volume 59B, Number 4, pp 49 – 63.

118. **Saint-Gobain Isover** (2007) www.isover.co.uk.

119. **Scottish Buildings Standards Agency (SBSA)** (2007) *"The Scottish Building Standards Agency: Domestic Technical Handbook 2007"*, Denholm House, Almondvale Business Park, Livingston, EH54 6GA.

120. **SERA** (2007) www.sera.org, SERA Labour Environmental Group, London Bridge, London.

121. **Smith, I., Landis, E., Gong, M.** (2003) *"Fracture and Fatigue in Wood"*, WILEY ISBN 0 471 48708 2.

122. **Sousa, P.** (1995) *"European Standardisation"*, Timber Engineering Step 1 Basis of Design, Material Properties, Structural Components and Joints, Salland De Lange, Deventer, ISBN 90 5645 001 8.

123. **Steer, P. J.** (1995), *"Timber in Construction"*, Timber Engineering Step 1 Basis of Design, Material Properties, Structural Components and Joints, Salland De Lange, Deventer, ISBN 90 5645 001 8.

124. **Stehn L.** (2002) *"Environmental Labelling of Timber-Framed Dwellings and their Building Components"*, Building Research & Information Vol 30 No 4 pp 248 – 254.

125. **Stehn, L.** (2005) *"Industrialised Construction of Timber Frame Buildings"*, Presentation received at Woodtech – Advanced Timber Engineering, Lund, Sweden.

126. **Stern, E. G. and Kumar, V. K.** (1973) *"Flitch Beams"*, Forest Product Journal, Vol 23 Part 5 pp 40 – 47.

127. **Structural Engineers Register (SER)** (2004) *"Scheme for Certification of Design (Building Structures) – in accordance with the requirements of the Building (Scotland) Act 2003, Building (Procedure)(Scotland) Regulation 2004"*, Structural Engineers Register Ltd , London, SW1X 8BH.

128. **Structural Insulated Panel Association (SIPA)** (2007) P.O. Box 1699 Gig Harbor, WA 98335, America. www.sips.org.

129. **Tan, K. C., Kannan, V.R., Handfield, R.B. and Ghosh, S.,** (1999). *"Supply chain management: An empirical study of its impact on performance"*, International Journal of Operations & Production Management 19 (10), 1034–105.

130. **Tarn, J.M., Yen, D.C., Beaumont, M.,** (2002). *"Exploring the rationales for ERP and SCM integration"*, Industrial Management & Data Systems 102 (1), 26–34.

131. **The Construction (Design and Management) Regulations (CDM).** (2007) Clause 11.

132. **The Health & Safety Executive (HSE).** (1999). *"Health & Safety in Roof Work"*. Her Majesty's Stationery Office. ISBN 0 7176 1425 5.

133. **The Health & Safety Executive (HSE).** Press Release E0:03 (14 March 2003), Designers to Demonstrate Risk Reduction of Falls from Height in Construction.

134. **The Institution of Structural Engineers (IstructE).** (2007) *"Manual for the design of timber building structures to Eurocode 5"*, Published by The Institution of Structural Engineers, London, SW1X 8BH.

135. **Thelandersson, S.** (2003), *"Timber Engineering – General Introduction"*, Timber Engineering, Wiley, ISBN 0 470 84469 8.

136. **Thermalite** (2005) *"Thin Joint Masonry – Aircrete Building Blocks"*, Marley Building Materials Limited.

137. **Timber Research and Development Agency (TRADA)**. (1994), *"Eurocode Design Guidance"*, ISBN 0 901348 95 3.

138. **Timber Research and Development Agency (TRADA).** (2001) *"Timber Frame Construction"*, ISBN 1900510 32 4.

139. **Timber Research and Development Agency (TRADA).** (2005) *"Softwood sizes"*, Wood Information Sheet, Section 2/3 Sheet 37, TRADA Technology Ltd

140. **Toratti and Kevarinmaki** (2006) *"Development of wood-concrete composite floors"*, VTT Building and Transport, Finland.

141. **Tracy, J. M.** (2000). *"SIPs Overcoming the Elements"*, Forest Products Journal, Vol 50 No 3. 12-18.

142. **Turnbull, D. B., Milner, M. W. and Mettem, C. J.** (1998) *"Better construction through the potential of timber I-beams"*, The Structural Engineer Volume 77/No 15 3 August 1999.

143. **UK Land Directory** (2007) www.uklanddirectory.co.uk, UK Land Directory Ltd, Alma Road, St Albans, Herts.

144. **UK Timber Frame Association (UKTFA).** (2005) *"Timber Frame Facts and Figures"*, www.timber-frame.org, UKTFA, The e-Centre, Cooperage Way Business Village, Alloa

145. **Ward, T.** (2001) *"Assessing the effects of thermal bridging at junctions and around openings"*, BRE Information Paper IP 17/01, ISBN 1 86081 506 b5.

146. **Waters, J. R.** (2003) *"Energy Conservation in Buildings. A Guide to Part L of the Building Regulations"*, Blackwell Publishing. London.

147. **Wolfe, R. W. and McCarthy** (1989) *"Structural performance of light frame roof assemblies-I. Truss assemblies designed for high variability and wood failures"*. Res. Paper FPL-RP-492, U.S. Department of Agriculture, Forest Products Laboratory, Madison, Wis.

148. **Young, J. F., Mindess, S., Gray, R. J. and Bentur, A.** (1998) *"The Science and Technology of Civil Engineering Materials"*, Prentice Hall, ISBN 0 13 659749 1.

149. **Young, W. C. and Budynas, R. G.** (2002) *"Roark's Formulas for Stress and Strain"*, 7th Edition, ISBN 0-07-07542-X.

150. **Zonhg, Li., Gupta, R. and Thomas, H. M.** (1998) *"Practical approach to modelling of wood truss assemblies"*, Practice periodical on structural design and construction, pp119 – 124.

APPENDIX A Method for Determining the Centre of Rotation and Applied Shear Forces to Timber Frame Walls in an Asymmetric System

Where several walls parallel to the wind direction resist the wind load on a timber platform frame building it is normally assumed that they share the load in proportion to their strength, on the assumptions that the strength of a wall is proportional to its stiffness and that the horizontal diaphragms create a stiff structure.

Hence $$F_{v,d,i} = \frac{F_{v,d} R_{d,i}}{\sum R_{d,i}}$$

where $F_{v,d,i}$ = design load on racking wall i

$F_{v,d}$ = total racking load

$R_{d,i}$ = design racking resistance of wall i

For most timber frame buildings the above assumptions are adequate. However, if the shear walls on one side of a building are significantly less strong and stiff than those on the other side then the share of the load which they carry may be greater than the load calculated as above.

One example of this is a built-in garage where the opening provides little shear resistance in the front wall unless special measures are taken (Case 1, Figure 3.4) and another is an end terrace house with a full gable wall on one side and a plasterboard party wall on the other (Cases 2 & 3, Figure 3.4).

In such cases it is assumed that the building acts like a rigid box which resists both the shear force of the wind load and a torsional moment (Prion and Lam, 2003). This torsional moment is equal to the wind load multiplied by the distance between the geometrical centre of the building and the building's centre of rotation (CR) measured perpendicular to the wind direction.

For building plans on an x-y grid with an origin (0, 0) in one corner, the distance of the CR from the origin for wind perpendicular to the x-axis (Figure A.1) is calculated from the formula:

$$\bar{x} = \frac{\sum R_{d,i} x_i}{\sum R_{d,i}}$$

where $R_{d,i}$ = design resistance of racking wall i which is parallel to the wind direction

$\bar{x}$ = distance of CR from origin, measured along x-axis

x_i = distance of wall i from origin, measured along x-axis therefore $$R_1(\bar{x}-x_1)+R_2(\bar{x}-x_2)=R_3(x_3-\bar{x})$$

hence $$\bar{x}=\frac{R_1x_1+R_2x_2+R_3x_3}{R_1+R_2+R_3}$$

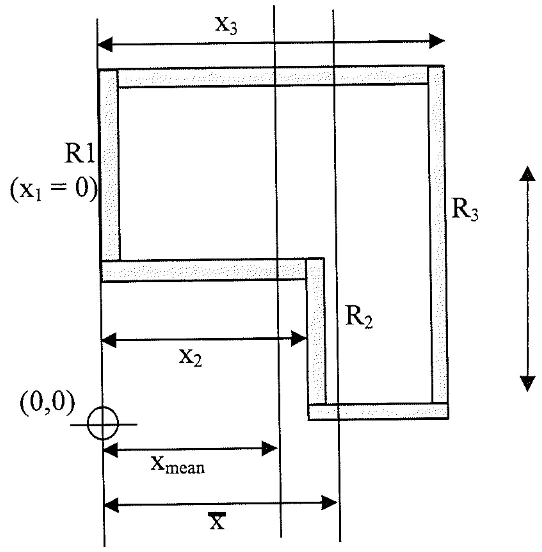


Figure A.1 Centre of rotation for wind perpendicular to the x-axis

The resulting torsional moment, $F_{v,d}(\bar{x}-x_{mean})$ is resisted by all the walls, with each wall contributing to the total moment in proportion to its (stiffness) × (lateral displacement) × (perpendicular distance to the centre of rotation), i.e.

$$F_{v,d}(\bar{x}-x_{mean})=k_x\sum R_{d,i}z_i^{\;2}$$

where $F_{v,d}$ = design racking load on building (sum of wind force on windward and leeward walls)

x_{mean} = distance of geometrical centre of building from the origin, along x-axis

k_x = a constant calculated from the above equation

z_i = perpendicular distance of any racking wall i from CR, i.e. $(\bar{x}-x_i)$ or $(\bar{y}-y_i)$ as appropriate.

The additional load which each wall perpendicular to the x-axis takes to resist the torsional moment is then

$$F_{tor,d,i} = k_xR_{d,i}x_i$$

The total load carried by each wall perpendicular to the x-axis is then

$$F_{\mathrm{d,i}} = F_{\mathrm{v,d,i}} + F_{\mathrm{tor,d,i}}$$

And it is checked that

$$F_{\mathrm{d,i}} \leq R_{\mathrm{d,i}}$$

APPENDIX B Optimisation of Shear Fixing Specification

Table B.1 Optimised annual shear fixing costs

Nail Spacing (90× 3.1mm)	MSC		BTB		KF		KMN	EXPN			
	36070	36082	4C70	4C82	75× 80	75× 100	72	8× 70	8× 90	6× 60	6× 100
mm	Cost per annum (£/annum)										
50	86400	86400	92571	98743	75130	115200	249600	96565	82944	93600	81000
100	45474	45078	46286	49371	38400	57600	132141	48282	42318	46800	40500
150	30857	30494	30857	33717	25412	38400	89856	32188	28405	31200	27000
200	23351	23040	23564	25135	19200	29206	68073	24141	21159	23400	20250
250	18383	18514	18783	20035	15292	23299	54790	19313	16997	18720	16200
300	15429	15475	15614	16859	12800	19379	45845	16094	14203	15600	13500
350	13292	13292	13500	14400	10937	16723	38731	13912	12198	13371	11571
400	11676	11649	11782	12567	9600	14603	34036	12160	10634	11823	10125
450	10286	10368	10452	11239	8512	12960	30357	10800	9468	10497	9063
500	9290	9341	9391	10091	7680	11715	27395	9714	8533	9439	8151
600	7784	7795	7855	8429	6400	9735	22922	8087	7101	7855	6785

Note:

- Level of shear fixity has been optimised base on Equation 3.5 (rounded to the nearest 10mm).
- 2007 unit cost information.
- Based on 1800 units per annum approximately 8400m (48m per unit) of sole plate

APPENDIX C Basic Racking Resistances for a Range of Materials and Combinations of Materials

Table C.1 BS 5268-2: part 6.1: 1996 Table 2

Primary board material	Fixing	Racking resistance kN/m	Additional contribution of secondary board on timber frame wall: Category 2 or 3 materials kN/m	Additional contribution of secondary board on timber frame wall: Category 1 material kN/m
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 0SB (type F2)	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 perimeter, 150 mm internal	0.90	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, for the outmost layer should be at least 60 mm long	0.90	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	0.90	0.45	1.06

NOTE 1 Timber members in wall panels should be not less than 38 mm × 72 mm rectangular section with linings fixed to the narrower face, with ends cut square and assembled in accordance with the relevant clauses of section 6.

NOTE 2 Timber members of rectangular section less than 38 mm × 72 mm, but not less than 38 mm × 63 mm, should be taken into account for internal walls (excluding separating walls), but in such cases all values for basic racking resistance given in this table should be reduced by 15 %.

NOTE 3 Studs should be spaced at centres not exceeding 610 mm.

NOTE 4 Board edges should be backed by, and nailed to timber framing at all edges except in the case of the underlayers in separating wall construction where it is normal to fix boards horizontally, in which case the intermediate horizontal joint may be unsupported.

NOTE 5 Studs should be of species and stress grade satisfying strength class C16 or better (as defined in BS 5268-2).

NOTE 6 The additional contribution from a secondary layer of category 1, 2 or 3 materials should only be included once in the determination of basic racking resistance, no matter how many additional layers may be fixed to the wall panel.

NOTE 7 The values given in Table 2 together with the modification factors in 4.8 and 4.9 assume that the wall under consideration is adequately fixed to ensure resistance to sliding and overturning.

NOTE 8 Where a secondary board is fixed on the same side of a wall as the primary sheathing then the nail lengths given in the table should be increased to take account of the additional thickness.

APPENDIX D Material Cost of Timber Frame Wall Diaphragm Details

Table D.1 38×89mm stud wall detail cost breakdown

Wall element	Standard Detail		Detail 1		Detail 2		Detail 3	
	Specification	Cost £	Specification	Cost £	Specification	Cost £	Specification	Cost £
Sheathing	9mm OSB/3	8.12	9mm OSB/3	8.12	9mm OSB/3	8.12	9mm OSB/3	8.12
Vapour Barrier	Breather membrane	4.61	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73
Fire Stop	50x38mm Timber	0.72	Barrier Sock	2.66	Barrier Sock	2.66	Barrier Sock	2.66
Insulation	Fibre glass wool ($\lambda = 0.038$ W/mK)	7.78	Fibre glass wool ($\lambda = 0.038$)	7.78	Fibre glass wool ($\lambda = 0.038$)	7.78	Rigid polyurethane ($\lambda = 0.025$)	62.93
			35mm Thick polyurethane thermal laminate ($\lambda = 0.025$ W/mK)	42.39	35mm Thick polyurethane thermal laminate ($\lambda = 0.025$ W/mK)	42.39		
Timber	38x89 C16	20.16	38x89 C16	20.16	38 x 89 C16	20.16	38 x 89 C16	20.16
					25x38mm Battens	4.90	25x38mm Battens	4.90
Plasterboard	Vapour check plasterboard	9.00	Vapour check plasterboard	9.00	Vapour check plasterboard	9.00	Normal plasterboard	6.58
	Total	41.38	Total	90.85	Total	95.74	Total	108.50
	Cost/m run	17.24	Cost/m run	37.85	Cost/m run	39.89	Cost/m run	45.21

Note: Based on a 2.4×2.4m wall at 2007 prices.

Table D.2 38×115mm stud wall detail cost breakdown

Wall element	Detail 4 Specification	Detail 4 Cost £	Detail 5 Specification	Detail 5 Cost £	Detail 6 Specification	Detail 6 Cost £
Sheathing	9mm OSB/3	8.12	22mm Bitumen fibre board	15.56	9mm OSB/3	8.12
	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73
Vapour Barrier	Vapour barrier	1.73	Vapour barrier	1.73	Vapour barrier	1.73
Fire Stop	Barrier Sock	2.66	Barrier Sock	2.66	Barrier Sock	2.66
Insulation	Fibre glass wool ($\lambda = 0.035$ W/mK)	17.28	Fibre glass wool ($\lambda = 0.035$)	17.28	Fibre glass wool ($\lambda = 0.032$)	26.50
Timber	38 x 115 C16	25.92	38 x 115 C16	25.92	38 x 115 C16	25.92
	25x38mm Battens	4.90	25x38mm Battens	4.90	25x38mm Battens	4.90
Vapour barrier	Reflective paper	1.73	Reflective paper	1.73	Reflective paper	1.73
Plasterboard	Normal plasterboard	6.58	Normal plasterboard	6.58	Normal plasterboard	6.58
	Total	70.34	Total	77.78	Total	79.56
	Cost/m run	29.31	Cost/m run	32.41	Cost/m run	33.15

Note: Based on a 2.4×2.4m run wall at 2007 prices.

Table D.3 38×140mm stud wall detail cost breakdown

Wall element	Detail 4		Detail 5		Detail 6	
	Specification	Cost £	Specification	Cost £	Specification	Cost £
Sheathing	9mm OSB/3	8.12	22mm Bitumen fibre board	15.56	9mm OSB/3	8.12
Vapour Barrier	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73	Low emissivity breather membrane	9.73
Fire Stop	Barrier Sock	2.66	Barrier Sock	2.66	Barrier Sock	2.66
Insulation	Fibre glass wool ($\lambda = 0.035$ W/mK)	17.46	Fibre glass wool ($\lambda = 0.035$)	17.46	Fibre glass wool ($\lambda = 0.032$)	26.56
Timber	38 x 140 C16	31.68	38 x 140 C16	31.68	38 x 140 C16	31.68
Plasterboard	Vapour check plasterboard	9.00	Vapour check plasterboard	9.00	Vapour check plasterboard	9.00
	Total	78.65	Total	86.09	Total	87.76
	Cost/m run	32.77	Cost/m run	35.87	Cost/m run	36.57

Note: Based on a 2.4×2.4m run wall at 2007 prices.

APPENDIX E The Transform Section Method of Design

The transform section method considers the whole section as one equivalent beam (Figure E.1) which consists of a single material. The equivalent beam will be of a cross-sectional area which is in proportion to the stiffness of the adopted material, if the stiffer of the two materials which constitute the beam is used then the cross-sectional area of the beam will be reduced.

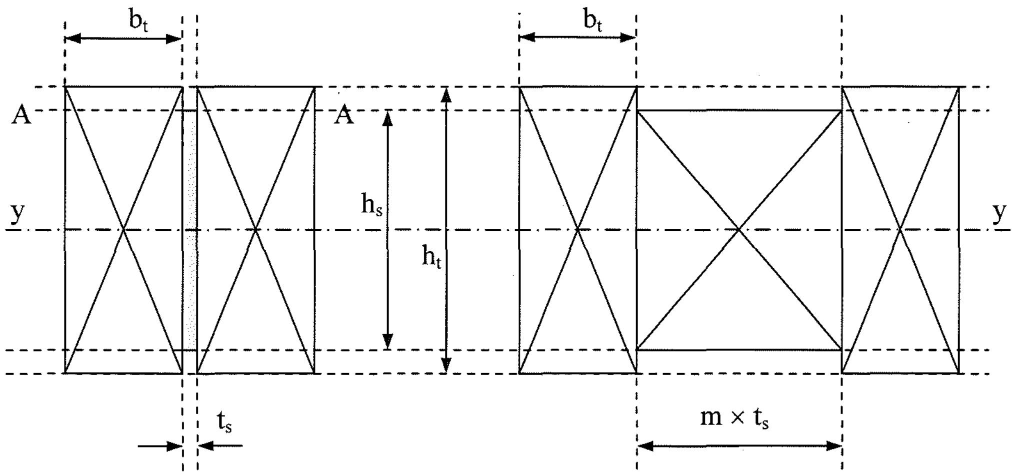


Figure E.1 Transform Section

Transform section is based on the theory of bending:

$$\frac{M}{I} = \frac{\sigma}{y} = \frac{E}{R}$$

Where:

M is the bending moment

I is the second moment of area

σ is the bending stress

y is the distance from the neutral axis

E is the modulus of elasticity (MoE) of the material in bending

R is the radius of curvature

If the beam shown in Figure A.1 were to be deflected about its neutral axis, y – y, at a radius of *R*, and full transmission of stress were taking place the strain in the timber at A – A would be equal to the strain in the steel at A – A and because it is known that:

$$E = \frac{Stress}{Strain}$$

The modular ratio, *m,* is derived as:

$$m = \frac{E_{steel}}{E_{timber}}$$

Because the strain in both beam elements is taken to be equal:

Stress in steel $= m \times$ *stress in timber.*

To consider the beam as an equivalent timber element the area of steel is increased width wise by modular ratio, *m*.

APPENDIX F Stiffness EI & Shear Modulus G Evaluation Tables

Table F.1 Stiffness *EI* & shear modulus *G* evaluation of C24_6 flitch beams

Equation Combination		Stiffness, EI Evaluation: All Inclusive	Selected	*EI* Capped 2.50E+12	*EI* Capped 1.44E+12	Shear Modulus, G Evaluation: Inclusive	*EI* Selective	*EI* Capped 2.50E+12	*EI* Capped 1.44E+12
		Nmm^2				Nmm^{-2}			
1	1,2	-1.94E+23		2.50E+12	1.44E+12	113.77		120.80	126.56
2	1,3	3.85E+23		2.50E+12	1.44E+12	127.78		144.04	158.93
3	1,4	1.96E+13		2.50E+12	1.44E+12	113.29		149.29	203.34
4	1,5	1.05E+12	1.05E+12	1.05E+12	1.05E+12	126.85	126.85	126.85	126.85
5	2,3	5.56E+23		2.50E+12	1.44E+12	128.60		143.28	158.19
6	2,4	1.32E+13		2.50E+12	1.44E+12	115.51		148.65	200.28
7	2,5	1.20E+12	1.20E+12	1.20E+12	1.20E+12	132.67	132.67	132.67	132.67
8	3,4	2.36E+12	2.36E+12	2.36E+12	1.44E+12	159.46	159.46	159.46	211.10
9	3,5	2.38E+12	2.38E+12	2.38E+12	1.44E+12	158.37	158.37	158.37	150.94
10	4,5	2.40E+12	2.40E+12	2.40E+12	1.44E+12	158.36	158.36	158.36	177.21
11	1,2,3	1.33E+20		2.50E+12	1.44E+12	127.59		137.36	149.41
12	1,2,4	1.52E+13		2.50E+12	1.44E+12	114.84		144.44	186.87
13	1,2,5	1.15E+12	1.15E+12	1.15E+12	1.15E+12	130.48	130.48	130.48	130.48
14	1,3,5	2.05E+12	2.05E+12	2.05E+12	1.44E+12	151.33	151.33	151.33	147.15
15	1,3,4	4.09E+12		2.50E+12	1.44E+12	134.84		151.31	196.87
16	2,3,4	4.08E+12		2.50E+12	1.44E+12	134.68		150.75	194.90
17	2,3,5	1.98E+12	1.98E+12	1.98E+12	1.44E+12	150.58	150.58	150.58	147.30
18	2,4,5	2.44E+12	2.44E+12	2.44E+12	1.44E+12	153.54	153.54	153.54	170.67
19	3,4,5	2.40E+12	2.40E+12	2.40E+12	1.44E+12	158.44	158.44	158.44	177.68
20	1,2,3,4	5.55E+12		2.50E+12	1.44E+12	127.59		147.18	185.27
21	1,2,4,5	2.49E+12	2.49E+12	2.49E+12	1.44E+12	150.00	150.00	150.00	166.09
22	1,2,3,5	1.83E+12	1.83E+12	1.83E+12	1.44E+12	146.34	146.34	146.34	144.49
23	1,2,4,5	2.49E+12	2.49E+12	2.49E+12	1.44E+12	150.00	150.00	150.00	166.09
24	1,3,4,5	2.45E+12	2.45E+12	2.45E+12	1.44E+12	154.63	154.63	154.63	172.63
25	2,3,4,5	2.44E+12	2.44E+12	2.44E+12	1.44E+12	154.26	154.26	154.26	172.09
26	1,2,3,4,5	2.65E+12	2.65E+12	2.50E+12	1.44E+12	149.77	149.77	149.77	168.04
Average		2.87E+22	2.11E+12	2.25E+12	1.40E+12	139.37	149.07	147.01	166.23

Table F.2 Stiffness *EI* & shear modulus *G* evaluation of C24_10 flitch beams

Equation Combination		Stiffness, EI Evaluation				Shear Modulus, G Evaluation			
		All Inclusive	Selected	EI Capped		Inclusive	EI Selective	EI Capped	
				2.70E+12	1.70E+12			2.70E+12	1.70E+12
		Nmm^2				Nmm^{-2}			
1	1,2	1.42E+12	1.42E+12	1.42E+12	1.42E+12	161.53	161.53	161.53	161.53
2	1,3	1.75E+12	1.75E+12	1.75E+12	1.70E+12	157.86	157.86	158.28	158.28
3	1,4	2.67E+12	2.67E+12	2.67E+12	1.70E+12	152.60	152.60	152.60	191.37
4	1,5	1.92E+12	1.92E+12	1.92E+12	1.70E+12	156.41	156.41	156.37	152.65
5	2,3	1.87E+12	1.87E+12	1.87E+12	1.70E+12	155.25	155.25	155.25	158.24
6	2,4	2.84E+12		2.70E+12	1.70E+12	149.05	149.05	151.54	189.35
7	2,5	1.86E+12	1.86E+12	1.88E+12	1.70E+12	155.29	155.29	155.66	152.72
8	3,4	3.35E+12		2.70E+12	1.70E+12	141.10	141.10	149.97	185.11
9	3,5	1.87E+12	1.87E+12	1.87E+12	1.70E+12	155.34	155.34	155.49	153.89
10	4,5	2.30E+12	2.30E+12	2.30E+12	1.70E+12	162.84	162.84	163.19	173.89
11	1,2,3	1.79E+12	1.79E+12	2.70E+12	1.70E+12	156.76	156.76	147.85	158.28
12	1,2,4	2.76E+12		2.70E+12	1.70E+12	150.66	150.66	151.65	185.18
13	1,2,5	1.88E+12	1.88E+12	1.88E+12	1.70E+12	155.74	155.74	155.74	153.39
14	1,3,5	2.13E+12	2.13E+12	2.70E+12	1.70E+12	152.00	152.00	159.37	154.38
15	1,3,4	2.99E+12		2.70E+12	1.70E+12	146.02	146.02	150.22	181.98
16	2,3,4	3.07E+12		2.70E+12	1.70E+12	144.86	144.86	149.93	180.89
17	2,3,5	1.87E+12	1.87E+12	1.87E+12	1.70E+12	155.39	155.39	155.36	154.36
18	2,4,5	2.47E+12	2.47E+12	2.70E+12	1.70E+12	156.97	156.97	158.03	172.00
19	3,4,5	2.29E+12	2.29E+12	2.70E+12	1.70E+12	160.57	160.57	156.42	171.04
20	1,2,3,4	2.93E+12		2.70E+12	1.70E+12	147.08	147.08	150.15	178.62
21	1,2,4,5	2.32E+12	2.32E+12	2.32E+12	1.70E+12	161.18	161.18	161.18	170.94
22	1,2,3,5	1.87E+12	1.87E+12	2.70E+12	1.70E+12	155.51	155.51	158.00	154.74
23	1,2,4,5	2.32E+12	2.32E+12	2.32E+12	1.70E+12	161.18	161.18	161.18	170.94
24	1,3,4,5	2.30E+12	2.30E+12	2.30E+12	1.70E+12	160.06	160.06	160.06	170.14
25	2,3,4,5	2.30E+12	2.30E+12	2.70E+12	1.70E+12	159.68	159.68	155.81	169.75
26	1,2,3,4,5	2.31E+12	2.31E+12	2.70E+12	1.70E+12	159.27	159.27	155.60	169.00
Average		2.29E+12	2.07E+12	2.36E+12	1.69E+12	155.01	155.01	155.63	168.18

Table F.3 Stiffness *EI* & shear modulus *G* evaluation of TS_10 flitch beams

Equation Combination		Stiffness, EI Evaluation				Shear Modulus, G Evaluation			
		All Inclusive	Selected	EI Capped		Inclusive	EI Selective	EI Capped	
				5.50E+12	3.22E+12			4.50E+12	3.22E+12
		Nmm^2				Nmm^{-2}			
1	1,2	1.98E+13		4.50E+12	3.22E+12	293.72		336.78	364.27
2	1,3	3.63E+12	3.63E+12	3.63E+12	3.22E+12	341.32	341.32	341.32	357.33
3	1,4	4.34E+12	4.34E+12	4.34E+12	3.22E+12	330.60	330.60	330.60	437.03
4	1,5	2.80E+12	2.80E+12	2.80E+12	2.80E+12	362.56	362.56	362.56	362.56
5	2,3	2.74E+12	2.74E+12	2.74E+12	2.74E+12	398.51	398.51	398.51	398.51
6	2,4	4.04E+12	4.04E+12	4.04E+12	3.22E+12	351.74	351.74	351.74	438.86
7	2,5	3.04E+12	3.04E+12	3.04E+12	3.04E+12	382.98	382.98	382.98	382.98
8	3,4	4.75E+12		4.50E+12	3.22E+12	309.09		319.19	424.38
9	3,5	2.98E+12	2.98E+12	2.98E+12	2.98E+12	377.89	377.89	377.89	377.89
10	4,5	3.46E+12	3.46E+12	3.46E+12	3.22E+12	416.52	416.52	416.52	425.52
11	1,2,3	3.28E+12	3.28E+12	3.28E+12	3.22E+12	360.23	360.23	360.23	363.05
12	1,2,4	4.18E+12	4.18E+12	4.18E+12	3.22E+12	341.41	341.41	341.41	424.29
13	1,2,5	2.95E+12	2.95E+12	2.95E+12	2.95E+12	374.86	374.86	374.86	374.86
14	1,3,5	2.95E+12	2.95E+12	2.95E+12	2.95E+12	374.40	374.40	374.40	374.40
15	1,3,4	4.52E+12		4.50E+12	3.22E+12	319.51		320.31	413.76
16	2,3,4	4.32E+12	4.32E+12	4.32E+12	3.22E+12	330.27	330.27	330.27	415.88
17	2,3,5	3.00E+12	3.00E+12	3.00E+12	3.00E+12	379.45	379.45	379.45	379.45
18	2,4,5	3.45E+12	3.45E+12	3.45E+12	3.22E+12	410.28	410.28	410.28	419.09
19	3,4,5	3.42E+12	3.42E+12	3.42E+12	3.22E+12	404.12	404.12	404.12	412.66
20	1,2,3,4	4.32E+12	4.32E+12	4.32E+12	3.22E+12	330.48	330.48	330.48	407.91
21	1,2,4,5	3.46E+12	3.46E+12	3.46E+12	3.22E+12	404.17	404.17	404.17	412.72
22	1,2,3,5	2.97E+12	2.97E+12	2.97E+12	2.97E+12	376.51	376.51	376.51	376.51
23	1,2,4,5	3.46E+12	3.46E+12	3.46E+12	3.22E+12	404.17	404.17	404.17	412.72
24	1,3,4,5	3.43E+12	3.43E+12	3.43E+12	3.22E+12	398.98	398.98	398.98	407.33
25	2,3,4,5	3.42E+12	3.42E+12	3.42E+12	3.22E+12	400.36	400.36	400.36	408.75
26	1,2,3,4,5	3.43E+12	3.43E+12	3.43E+12	3.22E+12	396.00	396.00	396.00	404.23
Average		4.16E+12	3.44E+12	3.56E+12	3.13E+12	368.08	375.99	370.16	399.11

APPENDIX G Methods of Determining Bending and Shear Deflection Components

For four point bending:

$$\Delta_{bending} = \frac{P \cdot L^3}{6EI}\left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3\right]$$

$$\Delta_{shear} = \frac{P \cdot \alpha \cdot a}{G \cdot A}$$

For three point bending:

$$\Delta_{bending} = \frac{P \cdot L^3}{48EI}$$

$$\Delta_{shear} = \frac{P \cdot \alpha \cdot L}{4G \cdot A}$$

Where:

Δ_{total} is the total deflection

$\Delta_{bending}$ is the deflection component due to bending

Δ_{shear} is the deflection component due to shear

L is the span over which the beam was tested

EI is the stiffness of the flitch beam

a is the distance between a loading position and the nearest support in a 4 point bending test

A is the cross sectional area of the beam

α is the shape factor calculated in accordance with (Young, W. C. and Budynas, 2002).

APPENDIX H Calculated and Measured Deflection for Varying Lad/Span Conditions

Table H.1 C24_6 Flitch beam calculated and measured deflection for varying load/span conditions

Type	Load	Span	Distance, *a*	Experi-mental results	Calculations based on:							
					Flitch beam						C24 Section Only	
					BS EN 408 determined *EI*				BS EN 338 Properties		BS EN 338 Properties	
					Bending only	& Including shear using *G* (N/mm^2) =			Bending only	Including shear	Bending only	Including shear
						54.64	139	644				
	N	mm	mm	Deflection, mm								
C24_6	25000	2230	545	4.86	6.97	24.53	13.86	8.46	6.62	8.01	13.78	15.18
	25000	950	475	3.70	0.40	8.05	3.40	1.05	0.38	0.99	0.79	1.40
	25000	1100	550	4.22	0.62	9.48	4.09	1.37	0.59	1.29	1.23	1.93
	25000	1500	750	4.88	1.57	13.66	6.31	2.60	1.49	2.45	3.11	4.07
	25000	2400	1200	9.66	6.44	25.77	14.02	8.08	6.11	7.64	12.72	14.27

Notes: Four point bending test
Distance, *a* is the distance from the support to the nearest load point

Table H.2 C24_10 Flitch beam calculated and measured deflection for varying load/span conditions

Type	Load	Span	Distance, *a*	Exper-imental results	Calculations based on:							
					Flitch beam						C24 Section Only	
					BS EN 408 determined *EI*				BS EN 338 Properties		BS EN 338 Properties	
					Bending only	& Including shear using *G* (N/mm^2) =			Bending only	Including shear	Bending only	Including shear
						49.49	155	582				
	N	mm	mm	Deflection, mm								
C24_10	25000	2240	545	4.48	3.96		9.50	5.44	4.96	6.21	13.91	15.31
	25000	950	475	2.83	0.22		2.64	0.87	0.28	0.82	0.79	1.40
	25000	1100	550	3.40	0.35		3.14	1.09	0.44	1.06	1.23	1.93
	25000	1500	750	5.08	0.88		4.70	1.90	1.11	1.96	3.11	4.07
	25000	2400	1200	9.43	3.62		9.72	5.25	4.54	5.91	12.72	14.27

Notes: Four point bending test
Distance, *a* is the distance from the support to the nearest load point

Table H.3 TS_10 Flitch beam calculated and measured deflection for varying load/span conditions

Type	Load	Span	Distance, *a*	Experi-Mental results	Calculations based on:							
					Flitch beam						LSL section Only	
					BS EN 408 determined *EI*				I-Level Properties		Trusjoist Properties	
					Bending only	& Including shear using *G* (N/mm^2) =			Bending only	Including shear	Bending only	Including shear
						118.56	399	584				
	N	mm	mm	Deflection, mm								
TS_10	25000	2700	545	3.51	4.08	10.65	6.03	5.42	3.99	5.19	10.86	12.04
	25000	1204	602	1.52	0.32	3.94	1.39	1.05	0.31	0.98	0.84	1.49
	25000	1400	700	1.78	0.50	4.71	1.75	1.35	0.49	1.26	0.64	1.40
	25000	1800	900	2.72	1.06	6.48	2.67	2.16	1.03	2.03	1.63	2.60
	25000	2700	1350	5.29	3.56	11.70	5.98	5.22	3.48	4.98	6.66	8.12

Notes: Four point bending test
Distance, *a* is the distance from the support to the nearest load point

APPENDIX I Assessing the Risk of Crane Erect Construction Relative to Other Available Methods of Timber Platform Frame Construction

Table I.1 Weighted risk assessment (values are average from the full survey conducted)

Safety Issue	At Height With Tele-handler			At Height With Crane			Crane Erect		
	Associated risk	Relative ranking	Factored	Associate risk	Relative ranking	Factored	Associate risk	Relative ranking	Factored
Lack of knowledge of good safety techniques.	2	3	6	2	2	4	3	1	3
Incorrect method of construction and misuse of equipment.	1	1	1	1	2	2	1	3	3
Unsafe manual handling, lifting loading, moving, stacking and storing.	3	3	9	2	2	4	1	1	1
Overloading of working places, scaffold, false work, hoists, ropes, etc	3	3	9	3	2	6	1	1	1
Removal of guards from scaffolds and working platforms.	2	3	6	2	2	4	1	1	1
Failure to use protective safety equipment.	2	1	2	2	1	2	2	1	2
Unauthorized use of tools, machinery or equipment.	2	1	2	2	1	2	2	1	2
Ignoring established rules, safe procedures or working methods.	3	1	3	3	1	3	3	1	3
Throwing or accidentally dropping things from height.	3	2	6	3	2	6	1	1	1
Failing to adapt and adhere to safe systems of work and procedures.	3	2	6	3	2	6	1	3	3
Illegal methods of access/egress to workplace.	3	2	6	3	2	6	1	1	1
Unauthorised interference with and misuse of plant and machinery	1	1	1	1	2	2	2	3	6
Failure to observe statutory requirements.	2	3	6	2	2	4	2	1	2
Congestion on-site from equipment and material storage.	1	1	1	3	2	6	3	3	9
Adverse wind conditions leading to unsafe working environment.	3	1	3	3	1	3	3	1	3
Total			67			60			41

Associated risk: The actual likelihood of the safety issue causing an accident considering the construction method being used: 1 - Low Risk; 2 - Medium Risk: 3 - High Risk

Relative ranking: The relative ranking of the associated safety issue taking place in the construction method being considered relative to the other construction methods:
1 - Least likely to occur in this construction method; 2 - Medium likelihood of occurrence in this construction method; 3 - Most likely to occur in this construction method

Note:

- Where two methods of construction have the same risk of the safety issue arising they are weighted the same and the third method is compared to them and vice-versa and the weighting is applied.
- If all three methods are the same in comparison then a weighting of 1 is applied.

Factored: The factored number is the multiple of Risk and Comparison, summing this will allow the comparison of the construction methods in terms of safety by ranking them.

APPENDIX J Financial Break Down of Construction Methods

Table J.1 At Height with tele-handler

Resource Cost					
Resource	Supplier	Unit	Cost/Unit	Quantity	Total Cost
3 Joiners	Sub-Contractor	Hours	£11.00	130.5	£1,435.50
Scaffolder	Client	N/A	N/A	N/A	N/A
Telehandler	Crane Hire Company		£15.00	8	£120.00
Additional Costs					
Resource	Supplier	Unit	Cost/Unit	Quantity	Total Cost
Air Mats	Hire company	Days		3	£216.95
Notes: • Based on 2004 figures. • Where the client is the supplier they incur the cost • Crane minimum hire period is 8 hours and travel time to and from site will be charged as "working time" and limited to 1 hour each way and included in the minimum hire period.				Total	£1,772.45
				Cost/m^2	£22.42

Table J.2 At height with crane

Resource Cost					
Resource	Supplier	Unit	Cost/Unit	Quantity	Total Cost
3 Joiners	Sub-Contractor	Hours	£11.00	117.00	£1,287.00
Scaffolder	Client	-	-	-	-
Telehandler	Crane Hire Company		£15.00	8	120.00
Crane + Operator*	Crane Hire Company		£273.00	1	£273.00
Additional Costs					
Resource	Supplier	Unit	Cost/Unit		Total Cost
Air Mats	Hire company	Days		3	£216.95
Notes: • Based on 2004 figures. • Where the client is the supplier they incur the cost. • Crane minimum hire period is 8 hours and travel time to and from site will be charged as "working time" and limited to 1 hour each way and included in the minimum hire period.				Total	£1,896.95
				Cost/m^2	£23.99

Table J.3 Crane erect

Resource Costs					
Resource	Supplier	Unit	Cost/Unit	Quantity	Total Cost
Crane Erect					
3 Joiners	Sub-Contractor	Hours	£11.00	72	£792.00
Scaffolder	Client	-	-	-	-
Crane + Operator	Crane Hire Company	Days	£273.00	2	£546.00
Additional Costs					
Specific Costs					
Resource	Supplier	Unit	Cost/Unit	Quantity	Total Cost
Ground Prep	Contractor	-	-	-	-
Notes: • Based on 2004 figures. • Where the client is the supplier they incur the cost. • Crane minimum hire period is 8 hours and travel time to and from site will be charged as "working time" and limited to 1 hour each way and included in the minimum hire period				Total	£1,338.00
				Cost/m^2	£16.92