THE ASSESSMENT AND APPLICATIONS OF A NEW CONNECTOR TYPE FOR USE IN TIMBER STRUCTURAL SYSTEMS

By

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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.

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Synopsis

Helically shaped fasteners, as structural ties, were first developed in 1984. Their innovative helical design proved to be very efficient and structurally viable in numerous structural applications in masonry and stone construction. Over the years, their uses widened to include amongst others crack stitching, warm roof batten fixing and creating masonry lintels. Following the understanding that helically shaped fasteners could have considerable potential providing highly efficient jointing systems and offer a number of advantages in structural applications for connecting timber to timber as well as timber to masonry/concrete a research programme was developed.

By conducting a review on the state of the art of timber jointing, the numerous methods for structural timber connections and the range of parameters that can influence the resistance of such joints were highlighted. Such a review allowed the development of an extensive experimental programme design to characterise helically shaped fasteners as structural timber connectors.

The mechanical properties of helically shaped fasteners were first investigated and compared to common timber connectors. In accordance with the relevant European and British standards, the investigation showed that helically shaped fasteners exhibited a very ductile behaviour compared to other common fasteners. However the design equations of Eurocode 5, which were developed for common timber fasteners, did not accurately predict the characteristic values of helically shaped fasteners. Consequently, specific design equations were developed for predicting the characteristic helically shaped fasteners' yield moment and embedment strength.

The innovative helical shape of helically shaped fasteners was designed to increase the bonding between the fastener and the substrates to connect. Hence, the axial resistance of helically shaped fasteners in timber was extensively investigated. The results showed that the helical shape of the fasteners gives them high axial resistance in timber. The investigation showed that numerous parameters affected the withdrawal resistance of helically shaped fasteners, and that they could be combined in semi empirical models to predict the resistance and behaviour of helically shaped when axially loaded in timber.

The investigation was also focused on the lateral shear resistance of timber connections with helically shaped fasteners loaded in single and double shear. The results showed that the connections exhibited very ductile behaviour while reaching similar resistance to common timber connectors. As a result semi empirical models were developed to predict the lateral shear resistance and behaviour of timber connections with helically shaped fasteners.

In addition to timber connections, the research also examined the use of helically shaped fasteners in timber to concrete connections for use as sole-plate fixing and timber-concrete composite flooring systems.

The research showed that the helically shaped fasteners have considerable potential for use in a wide range of timber connection systems as they provide a unique solution combining strength, flexibility, durability and holding power. The study also developed an in-depth understanding of the factors that influence their strength and stiffness properties. A series of semi-empirical models were developed to predict the performance characteristics of helically shaped fasteners, in withdrawal and lateral shear, which provide powerful analysis-design tools for architects and engineers as they predict the connection behaviour, up to failure loads, with good accuracy.

Acknowledgements

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To Vanessa and Maxim.

Contents

Publications and Awards

1. **G. Coste & A. Kermani**, Axially loaded helically shaped connectors in timber, Second Scottish Conference for Postgraduate Researchers of the Built & Natural Environment (PRoBE), Glasgow, UK, 16-17 November 2005.

Award: *The International Council for Research and Innovation in Building and Construction (CIB) supported runner-up prize for best paper.*

2. **G. Coste & A. Kermani**, Development of a semi empirical model for a new helically shaped metal fastener axially loaded in timber, Poster presentation at the Institution of Structural Engineers' Young Researchers Conference, 15th March 2006, London

3. **G. Coste & A. Kermani**, Performances of Helically shaped connectors in timber structural systems, 9th World Conference on Timber Engineering (WCTE), Portland, Oregon, USA, 6-10 August 2006.

4. **G. Coste, A. Kermani & A. Porteous**, Performance of helically shaped metal fasteners in timber, Proceedings of the Institution of Civil Engineers, Construction Materials, May 2006, Vol: 159, No: 2, 53-60.

Award: *Institution of Civil Engineers' Howard Medal 2007*

Notations

Greek Notations:

α: angle of fastener's axis to the timber fibres (degrees)

β: ratio between the timber members embedment strengths

γ_m: partial safety factor for the material property

δ: displacement at load P (mm)

δ_{max}: displacement at load P_{max} (mm)

δ_y: displacement at load P_y (mm)

θ: yield moment angle (degrees)

ρ_k: characteristic timber density (kg/m^3)

ρ: mean timber density (kg/m^3)

Latin Notations:

d: fastener nominal diameter (mm)

d_t: Helically shaped fasteners thread diameter (mm)

d_r: Helically shaped fasteners root diameter (mm)

D: measured timber density (kg/m^3)

e_i: gap between the timber member and substrate (mm)

f_{ax}: withdrawal strength in timber (N/mm^2)

$f_{ax,\alpha,k}$: characteristic withdrawal strength at an angle α to the grain (N/mm^2)

$F_{ax,Rk}$: characteristic axial withdrawal capacity of the fastener (N)

f_h: embedment strength (N/mm^2)

$f_{h,i,k}$: characteristic embedment strength in timber member i (N/mm^2)

$F_{tc,int\text{-}free}$: load carrying capacity of the connection with interlayer (N)

f_u: fastener tensile strength (N/mm^2)

$f_{u,k}$: characteristic fastener tensile strength (N/mm^2)

$F_{v,ef,Rk}$: multiple fasteners connection load (N)

$F_{v,Rk}$: characteristic load-carrying capacity per shear plane per fastener (N)

k_{ef} : effective number of fasteners factor

k_{mod}: modification factor taking into account the combined effect of moisture content and the duration of load

l_{ef}: pointside penetration length of the threaded part minus one screw diameter (mm)

l_p: fastener penetration length in timber (mm)

L_S: spacing between lines of fasteners in a joint (mm)

mc: timber moisture content (%)

m_ω: timber sample initial mass (g)

m_0: timber sample oven-dried mass (g)

M_y: dowel type fastener yield moment (N.mm)

$M_{y,k}$: characteristic dowel type fastener yield moment (N.mm)

n_{ef} : effective number of screws

N_L: number of lines of fasteners in a joint

N_R: number of rows of fasteners in a joint

P: lateral shear load (N)

p_e: fastener cross sectional perimeter (mm)

p_h : pilot hole diameter, mm

P_{max}: lateral shear load per shear plane (N)

P_y: lateral shear yield load (N)

r_d: ratio of pilot hole diameter to fastener root diameter

R_D: ductility ratio ($\delta_{\mathrm{max}}/\delta_{\mathrm{y}}$)

R_S: spacing between rows of fasteners in a joint (mm)

R_k: characteristic value of material property or strength

S_d: design action effect

t_i: timber thickness or fastener penetration depth (mm)

t_{pen}: fastener pointside penetration (mm)

W: withdrawal load in timber (N)

W_{pen}: withdrawal load per unit length in timber (N/mm)

Chapter 1 Introduction

1.1 General

The advantages of timber for use as a primary structural material are numerous. It is available throughout the world, and with proper sustainable management it has a potential unlimited availability. Its environmental impact compared to other construction materials is greatly reduced, the production of timber products consume only about 50% of the energy required to produce concrete and only 1% of the energy to produce steel. Timber has a low weight to strength ratio which is advantageous for transport, erection and production, which also permits a simplification of the foundations of timber structures. These advantages coupled with growing customer demands for environmentally better products, more aesthetically pleasing structures, along with stricter environmental and building regulations have all contributed to a reawakening of the uses of timber as a primary structural material.

Throughout the world the predominant use of timber is in timber-framed housing; where in North America and Scandinavia 90% of the houses are timber-framed houses. In the UK the market for timber-framed construction is steadily growing and now represents 20% of the total of new-built houses (Hairstans, 2007). This method of construction is proving popular as in addition to complying with the environmental and ecological requirements from public opinion and governments, it presents many advantages compared to other structural materials allowing for architectural and design flexibility, fast site erection, low weight construction and in turn reducing the cost of supporting structures.

In addition to the main use of structural timber in timber frame housing, timber is being used in more and more challenging structures through innovative design, such as gridshells and compression-net structures, and with the development of timber composite materials, such as glulam and Laminated Veneer Lumber (LVL) opportunities for more innovative structural systems are increasing. However in order for timber to be structurally comparable to other building materials, the load carrying capacity of its elements and more importantly of the jointing methods need to be improved and optimised.

In timber structures, joints have always been the most critical components, as they govern their overall strength, stiffness, serviceability and durability. It is often said that a timber structure is primarily an assembly of joints separated by members. In addition the need for stronger and larger structures using timber based materials coupled with the limited availability or the increased cost of large sections of solid timber have necessitated the need to improve the fastening mechanisms and techniques, in order to achieve effective transfer of loads between timber members. This in return has led to the development of new fasteners and connector types, such as threaded fasteners, spilt rings, or nail plates.

Helically shaped connectors were developed after a need for efficient, economical and non-disruptive wall ties was identified. The innovative product created to fulfil this need, a unique helical stainless steel wall tie, has since, over two decades, formed the basis of a range of special purpose ties, fixings, masonry repairs and reinforcement for buildings, bridges and other masonry buildings. While the main uses of Helically shaped fasteners were in masonry buildings, preliminary experimental research showed that the threaded fasteners could be used for timber connections.

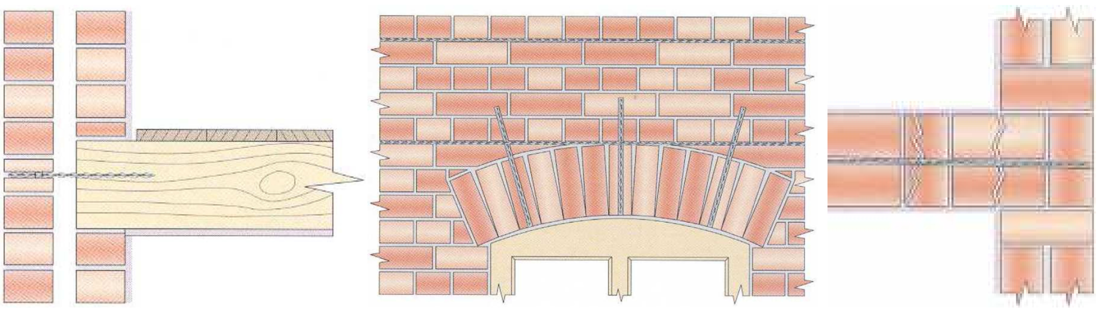


Figure 1.1: Examples of current uses of helically shaped fasteners.

1.2 Helically shaped fasteners

1.2.1 General

After identifying the need for efficient, economical and non-disruptive remedial wall ties and masonry repair helically shaped fasteners were developed (Keitley, 2003). It created a new type of tie made of austenitic stainless steel, with its own helical design, Figure 1.2. Since its creation, the tie has evolved and been developed to a range of stainless ties, fixings and masonry reinforcement products, with a series of repair techniques that provide concealed, stress free solutions to the problems of masonry failure.

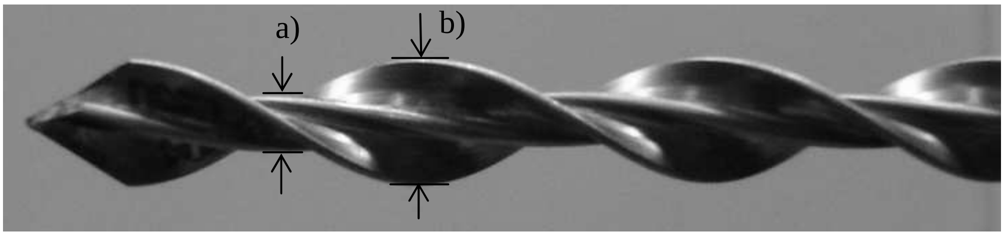


Figure 1.2: Helically shaped connector, a) root diameter, b) shank diameter.

Helically shaped fasteners are made of stainless steel, the manufacturing process consists of three different stages:

- The raw material is extruded through dies to form the cross section of the fasteners in a continuous manner, the formed product is stocked in rolls,
- The length of material with the formed cross section is then spun in tension in length of 7 or 14m which creates the helixes of the ties, in addition the reference of the ties being manufactured is printed on the product,
- The formed length of section is then cut to length, and a point is cut at one end.

1.2.2 Current uses

Following the gathered experience in more than 20 years, the company has created fasteners of different length, diameter and steel class or material composition. This range of fasteners is used in numerous applications for connecting concrete based materials, bricks and timber, and for repairs or remedial ties in masonry structures.

The main, and original, application of helically shaped fasteners is for masonry repair. Over the years, masonry structures and primarily houses of Victorian type can develop various structural damages, and the different manufacturers have developed different techniques for repairs, as illustrated in Figure 1.3.

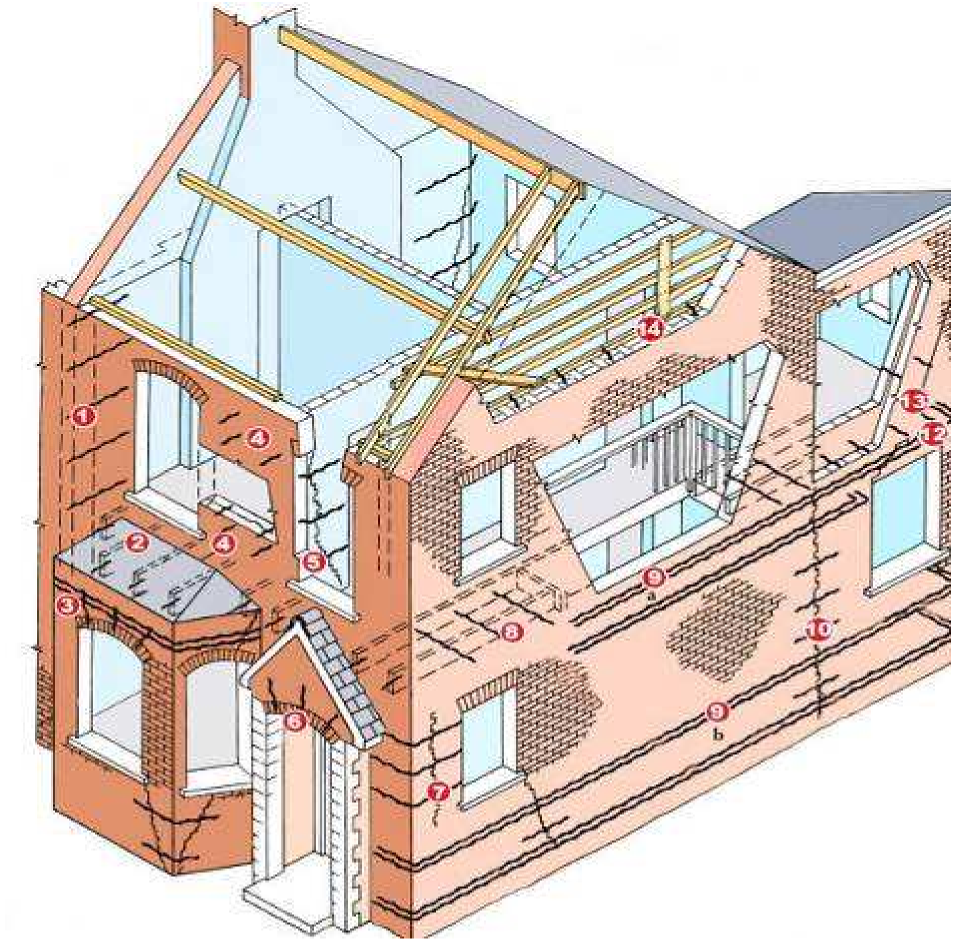


Figure 1.3: Typical masonry damages and repair techniques (from www.Helically shaped.co.uk)

In a typical masonry building as shown in Figure 1.2, helically shaped fasteners can be used for:

a) **Repair**: flat or arched masonry lintels (6, 12), separated masonry (4), bay windows (3), cracks in corners and openings (7);
b) **Reconnect**: party or internal walls with external walls (1, 5), and ceiling joists (14);
c) **Stabilise**: bowed walls into joists end or sides (2, 8);
d) **Create**: masonry beams (9) and movement joints (10);
e) **Replace**: cavity wall ties (13);

These repair techniques have nowadays been extended and applied in numerous other applications. For example in repairs of masonry arch bridges, the techniques employed have allowed to considerably minimise the disruption to road or rail services, preserving the existing structure and avoiding the need for expensive rebuilding.

In addition several products have been developed for widening the range of applications for connection in various materials, and notably timber. Helically shaped fasteners are now also used for:

- Timber frame wall ties for cavities up to 125mm,
- Warm roof batten fixings, where the need for eliminating cold bridges is high,
- Timber or Medium Density Fibreboard (MDF) connections to bricks, blocks and concrete, in numerous applications essentially non structural such as fixing window frames, fences and cupboards.

Helically shaped fasteners can be obtained from various manufacturers. A review of the different fasteners from the different providers showed that the products available have similar geometry, material properties and characteristics.

The fasteners used in this research were provided by Helifix Ltd. Four fastener references made of austenitic stainless steel grade 304 were used: StarTie 10, StarTie 8 , InsKew and TimTie with nominal thread diameters of 10mm, 8mm, 6mm and 4.5mm respectively. In addition to stainless steel fixings, helically shaped Ltd developed a range of grouts, resins and tools in order to complete the applications and uses of the steel ties. In numerous applications the combination of helically shaped steel ties and helically shaped resins are recommended for best results of the repairs. In view of the advantages of such products and techniques, Helifix Ltd developed internationally, and is now present in North America, Australia, and mainland Europe.

1.3 Experimental research and objectives

Helically shaped fasteners are widely used in masonry and stone buildings however the behaviour and performances of such fasteners in timer to timber and timber to concrete connections have never been investigated. This research programme was developed on the back of realising that the helically shaped fasteners could have considerable potential providing highly efficient jointing systems and offer a number of advantages in structural applications for connecting timber to timber as well as timber to masonry/concrete. Therefore an experimental research programme was composed with the aim utilise the helically shaped fasteners' advantages and mechanical properties for use in a variety of structural timber connection systems, and to examine the viability of the use of helically shaped connectors in comparison to other available connector systems.

The objectives of the research were therefore:

- To undertake a review of the existing research on timber connections with dowel type fasteners, including load displacement behaviour subjected to lateral loading or direct withdrawal, load carrying capacities, and design methods
- To develop an appropriate experimental program for the investigation of Helically shaped connectors as timber fasteners in a variety of existing timber connection systems, in comparison to conventional timber fasteners;
- To determine the mechanical properties of Helically shaped fasteners, and examine how they compare to conventional fasteners;
- To develop numerical models for the simulation of the load displacement behaviour for predicting the structural behaviour and performance of connections with Helically shaped fasteners;
- To compare the experimental results with the design rules for dowel type connections of Eurocode 5;
- To develop design procedures for the use of Helically shaped fasteners in variety of timber connections;
- To examine new possible uses for helically shaped fasteners in timber structural systems.

In chapter 2, the various parameters that can influence the resistance and behaviour of helically shaped fasteners in timber are detailed. To achieve these objectives, and following the extensive review, a schematic diagram of the experimental programme aimed at investigating the viability of new type of fasteners in timber structural systems was designed, Figure 1.4. The diagram below shows the various steps involved in the experimental programme. It is to be noted that the diagram shows all the steps necessary for a complete analysis of the fastener in timber structural systems. This study focused on the behaviour of timber connexion systems with helically shaped fasteners under static loading. The various stages not investigated in this study are framed in red.

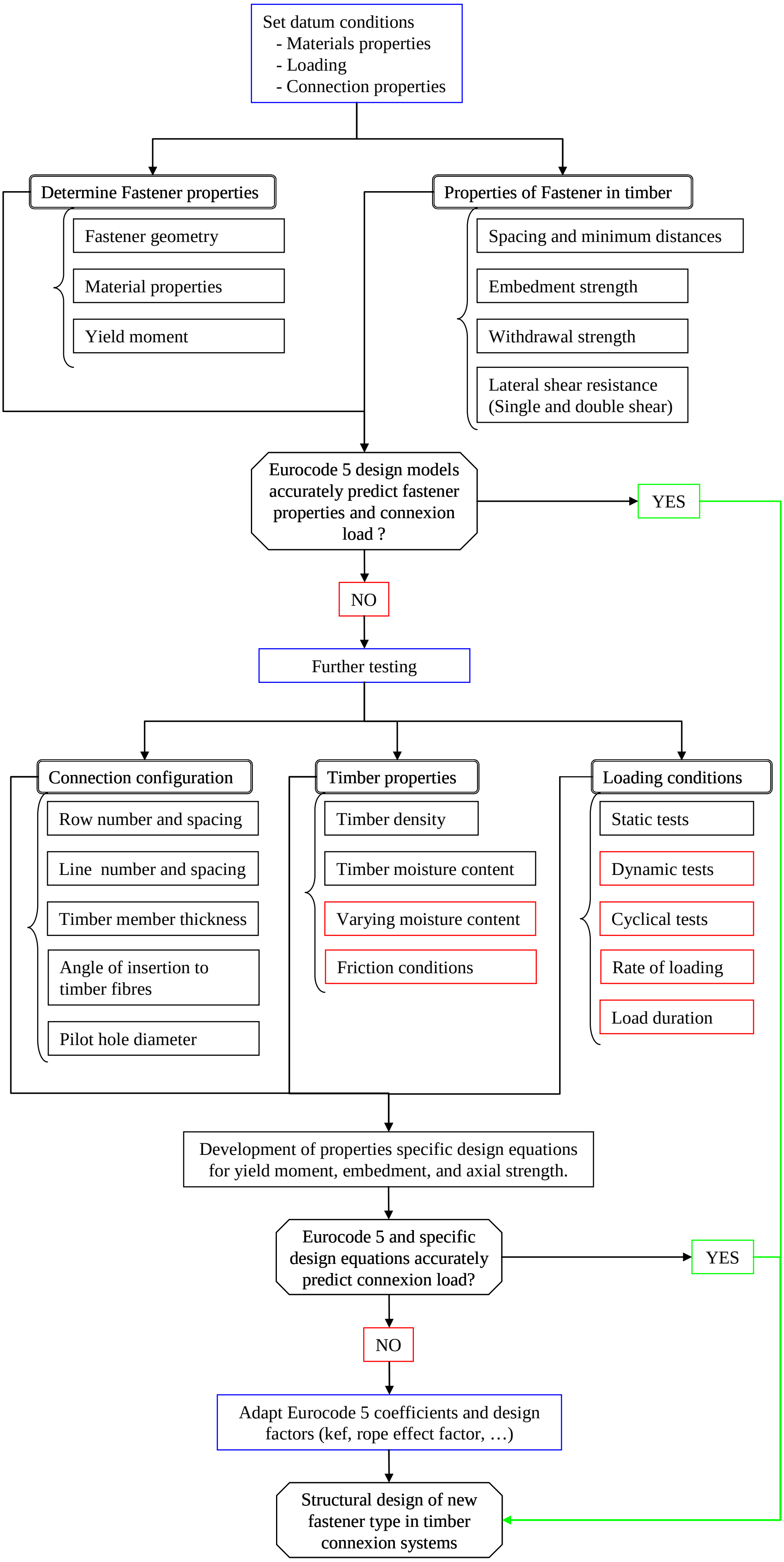


Figure 1.4: Schematic diagram of the experimental investigation

Figure 1.5, shows the organisation of the thesis relative to the programme detailed in Figure 1.4. In addition the methodology used for each stage is shown, whether it refers to standard European and UK test methods or to the analysis of results of normalised tests.

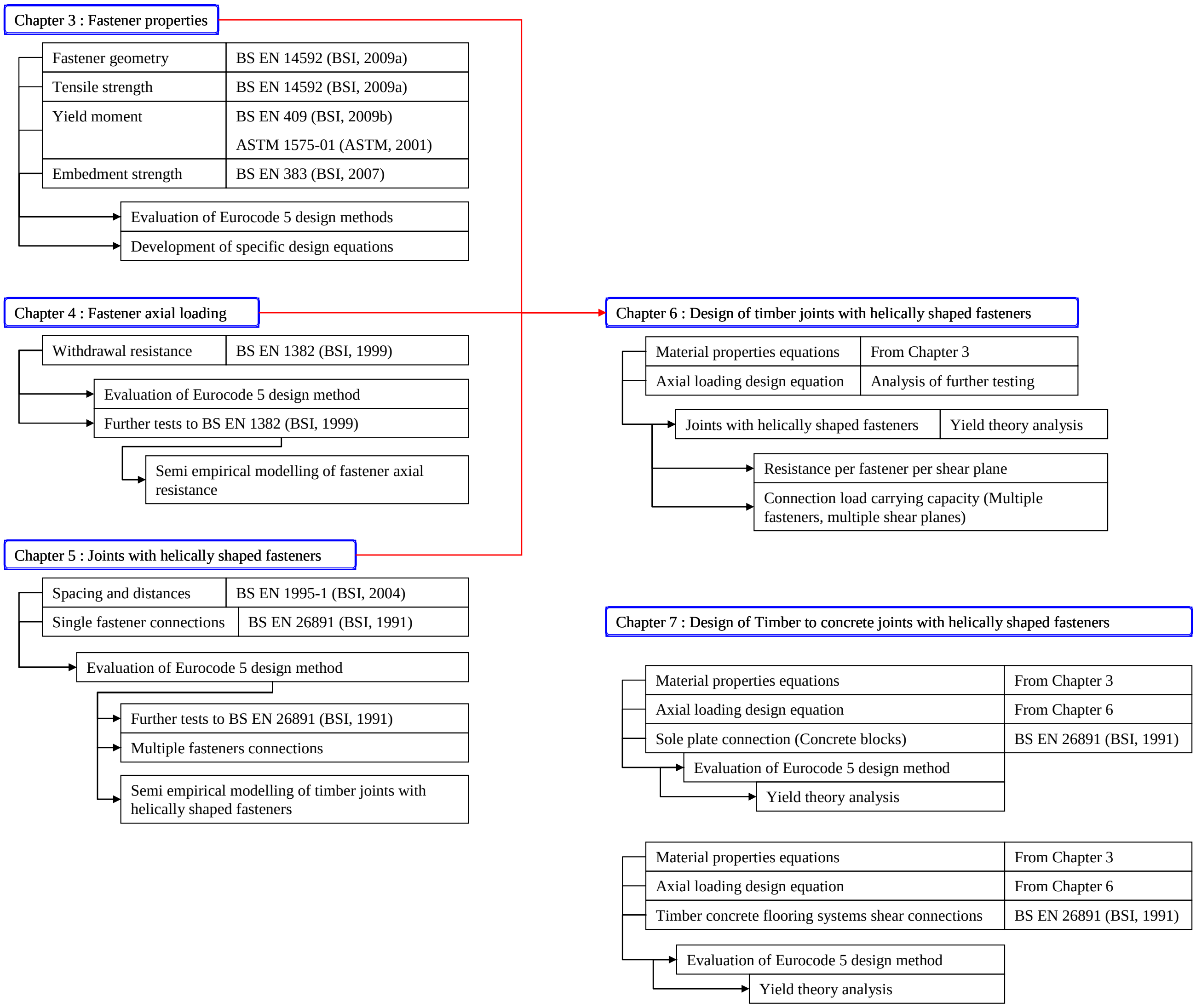


Figure 1.5: Organisation of the experimental work and determination methods

1.4 Outline of thesis

This thesis is divided into eight chapters. Chapter 2 reviews the existing state of the art for timber fasteners and connections, and Chapter 8 draws the conclusions from the study detailed in the chapters 3 to 7. A brief description of the chapters is given below.

Chapter 2 – Literature Review

The literature review first describes the different means for connecting timber, from dowel type fasteners to nail plates and bearing connectors. Then, as helically shaped fasteners fall in this category, connections with dowel type fasteners are reviewed. The parameters that influence such connections are examined – timber and fasteners properties, joint configuration and loading. Then the review was focused on the withdrawal strength of fasteners and timber, and the parameters that may affect axially loaded fasteners in timber. Finally timber to timber connections with dowel type fasteners are reviewed.

Chapter 3 – Properties of Helically shaped fasteners

The mechanical properties of timber fasteners influence the behaviour and resistance of connections, and are used for design of timber structures. In this chapter helically shaped fasteners were investigated along with commonly used timber screws and nails to determine their mechanical properties. The fasteners tensile strength, yield moment and embedment strength were evaluated and compared to the design rules from Eurocode 5.

Chapter 4 – Axially loaded helically shaped fasteners in timber

In this chapter the withdrawal behaviour and capacity of helically shaped fasteners was investigated. First, tests were carried out for evaluating helically shaped withdrawal performance~~s~~ compared to common timber fasteners. Then an extensive experimental programme was performed in order to investigate the parameters that may influence the behaviour and resistance of helically shaped fasteners when subjected to axial loads in timber. From the experimental results a semi empirical model was developed for simulating the load displacement behaviour and capacity of helically shaped fasteners in withdrawal.

Chapter 5 – Laterally loaded connections with helically shaped fasteners

Timber to timber connections with helically shaped loaded in single and double shear are investigated in this chapter. First, single fastener joints are examined with helically shaped and common timber fasteners for comparison purposes. Subsequently; single and double shear timber to design method for timber connections with helically shaped fasteners are considered, and the connection configuration parameters that may influence such connections are investigated. An extensive experimental programme was carried out to explore these parameters from which semi empirical models were developed for simulating the load displacement behaviour and capacity of connections with helically shaped fasteners.

Chapter 6 – Design methods for timber joints with helically shaped fasteners

In this chapter the applicability of the available design method for timber connections is examined for helically shaped connections. First the Eurocode 5 dowel type connection design method is detailed. Then, the results of the experimental programmes from the chapter 3 to 5 are used in order to investigate the applicability of the design method to timber connections with helically shaped fasteners. Alternative design equations and design rules for helically shaped fasteners are proposed for connection design where Eurocode 5 method is not applicable.

Chapter 7 – Helically shaped as shear connectors in timber concrete composite systems

As well as timber to timber connections helically shaped fasteners were investigated as connectors for timber to concrete connections. In the first part of this chapter a review of timber to concrete composite systems was undertaken. The various techniques, applications and design methods are described. Then two of the main timber to concrete applications were investigated: sole plate connections and timber-concrete floors shear connections. The experimental programmes are detailed and the results in comparison to common type connectors were studied. Finally the design method for timber to concrete shear connector are examined.

Chapter 8 – Conclusion and future work

The conclusions of the experimental and investigation work undertaken in the chapters 3 to 7 are drawn in this chapter. Also proposals for future work are presented.

Chapter 2 Literature review

2.1 Introduction

Two main forms of joints can be found in timber structures: mechanically fastened and glued joints. The use of glued joints dates from thousands of years with examples dating from Egyptian times. Glued joints can be divided into two categories: pure wood-to-wood joints and hybrid joints where the adhesive part is used as reinforcement of a mechanical connector. Compared to mechanical joints, glued joints offer more rigidity, higher load carrying capacity for similar joint area and usually the possibility of automation. On the other hand, glued joints have the disadvantages of requiring high level of skills for manufacturing therefore preventing on-site manufacturing, and they generally exhibit complex and brittle behaviour (Thelanderson, 2003).

Mechanical connections are constructed by using a metal connector between the timber members to be joined, which transmit lateral shear or withdrawal loads. Two main categories of mechanical connectors exist: dowel type and bearing connectors (American Society of Civil Engineers, 1996).

Due to the fundamental differences between the two jointing methods introduced above, and the nature of this research, a literature review was conducted focusing on mechanically fastened joints. The different types of mechanical fasteners used in structural timber systems, their applications, structural behaviour and performance are described. First, details of the different connectors types used in timber structural systems are given, with emphasis on dowel type connectors. Then a review of the knowledge on the connection behaviour, the parameters that influence connections with dowel type connectors has been carried out. Finally the different methods currently used for modelling timber joints are described.

2.2 Mechanical timber connectors

Mechanical connectors can be divided into two categories depending on the types of forces they can transmit. The first, and the most commonly used types of mechanical fasteners are, dowel type fasteners; which include nails, screws, lag or coach screws, staples, bolts and dowels. They can transmit lateral shear loads due to the fastener bending resistance and the wood bearing capacity, and axial loads parallel to their axis through friction or bearing, or a combination of the two. Bearing type connectors only transmit lateral loads through increased bearing in the connected members. Bearing connectors include shear plates, split rings, and toothed plates (STEP 1, 1995).

2.2.1 Dowel type fasteners

Dowel type fasteners are the most common types of connectors in timber connections. Archaeological evidence has shown that their use dates back form the days of ancient civilisations. However it is with the industrial revolution in the 19th century and the possibility of mass production that dowel type fasteners, and nails in particular, became the most used mean of connection between timber members (Porteous, 2003). Nowadays, dowel type fasteners are engineered products, designed for transmitting lateral shear and axial loads. Examples of dowel type fasteners are shown in Figure 2.1.

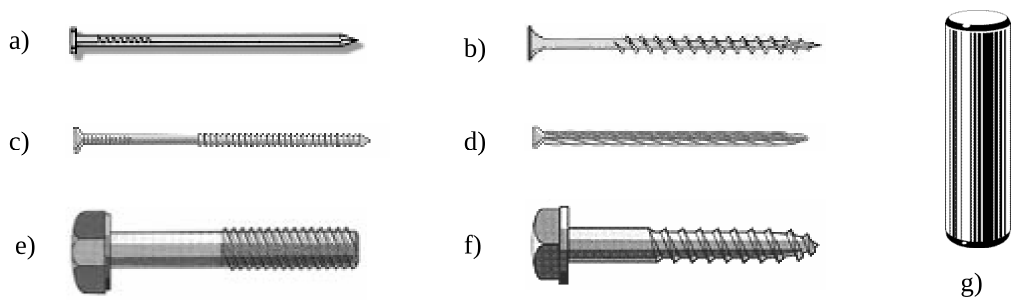


Figure 2.1: Examples of dowel type fasteners. a) round wire nail b) wood screw c) annularly threaded nail d) helically threaded nail e) bolt f) lag screw g) dowel.

Due to the complexity and wide range of timber structural applications different types of dowel type fasteners were developed to answer specific problems; thus they can be classed into three groups:

- Nails,
- Screws,
- Bolts and dowels.

A nail is defined by three main characteristics: the shank, which offers the most possibilities for variation, the head, which provides a strike area for insertion into the timber and a bearing area, and usually a point, which purpose is to facilitate driving into the wood. Nails can be manufactured with variations in material, shape, deformations, qualities, finishes, treatments and coatings to answer specific applications. In 1979 a limited survey listed approximately 2900 types of nails in the national standards of 16 countries (American Society of Civil Engineers, 1996).

Round wire nails are the most basic type of nails, they are manufactured from steel rods drawn through dyes to form the required diameter, cut to length, then one end of the dowel obtained is compressed along its axis to form the head, and the other end is pinched to form the point, eventually mechanical deformations are rolled into the shank of the nails. Treatments, coatings and finishes are manufacturing processes applied after forming of the nail. Deformed shank nails, or threaded nails, can have annular or helical threads rolled into the shank. Annularly threaded nails have multiple rings rolled into the shank perpendicularly to their axis, resulting in a smaller root diameter than the original wire diameter. Helically threaded nails have multiple helixes rolled into the shank, resulting in a deformation but without reduction of the cross sectional area (Wills et al., 1996). Because threaded nails are generally deformed after pointing and heading, part of the nail shank remains plain.

The common materials used for manufacturing nails are a low carbon steel ($c \leq 0.15\%$), medium low carbon steel ($0.15\% \leq c \leq 0.23\%$) or stiff stock steel which is a bright non-hardened medium-low or medium-high carbon steel ($0.23\% \leq c \leq 0.44\%$). For specific applications nails can be manufactured from stainless steel, aluminium alloys, brass, copper or even bronze (Elhbeck, 1979).

Screws are helically threaded fasteners where the angle between the thread and the fastener axis is steep; therefore a greater force of insertion is required. Two categories of screws exist: woodscrews and lag or bolt screws. The main characteristic of screws is that they can be removed or reinserted without significant loss of holding power in shear or withdrawal applications. They can also be used to fasten brittle materials (American Society of Civil Engineers**,** 1996). Wood screws are commonly used in connections to transmit lateral or withdrawal forces, or a combination of the two. Compared to nails they provide a more positive connection in withdrawal.

Woodscrews are manufactured with continuous single or double helical threads rolled on about two thirds of the shank. As opposed to helically threaded nails the root diameter of screws usually measures about two thirds of the shank diameter. As they are used in a variety of applications they measure between 6 and 100 mm in length, and 1.5

to 10 mm diameter. As nails they are manufactured with a head and point, while serving the same purposes as for nails, the head of woodscrews is also provided with a slot or recess which allows for insertion with a screw-driver or power tool. This recess also allows for the screws to be removed or retightened.

Lag screws, also called bolt or coach screws, are larger and stouter than woodscrews. The head of lag screws is usually square or hexagonal with no slot; they are designed to be inserted into predrilled members using a wrench or power tool. Lag screws are usually used instead of bolts or dowels when high withdrawal resistance is required or where the presence of a washer and nut is objectionable for aesthetics reasons, or where fastening a bolt would prove a difficult or impossible operation.

Dowels and bolts are slender cylindrical fasteners, with mainly smooth shanks, manufactured from steel rods. As opposed to bolts, dowels do not contain an integral head but can be threaded at both ends to receive a nut. Bolts have a square or hexagonal head and are threaded at the other end to receive a nut. These allow the bolt to be tightened so the members fit closely, and can be retightened in case of dimensional variations of the timber members. Dowels have to be inserted into predrilled holes of a diameter no larger than the dowel diameter, while for bolts the pilot hole can have a diameter up to 1mm larger than the bolt diameter (STEP 1, 1995). Both types of fasteners are used in joints transmitting high lateral forces, mostly on glulam or heavy timber construction.

Early in the 1900’s in-plane connections of timber members for truss systems could be achieved by nailing steel or plywood gusset plates to members to be connected. However in the 1950’s preformed metal nail plates were developed and permitted enhancing the level of pre fabrication and industrialisation of truss manufacture, Figure 2.2. Metal punched nail plates have teeth stamped out by a die so they are perpendicular to the plane. Usually made of light gauge galvanised steel between 0.9 to 2.5 mm thick, they can cover an area of 30 cm^2 up to 1 m^2. Nail plates are nowadays widely used for connecting two or more members of the same thickness in the same plane. Two plates are used per joint, on either side of the members to be connected, Figure 2.2. The strength of a punched nail plate is determined by the nails pattern, shape and length, but also importantly on the angle between the joint line and the main direction of the plate.

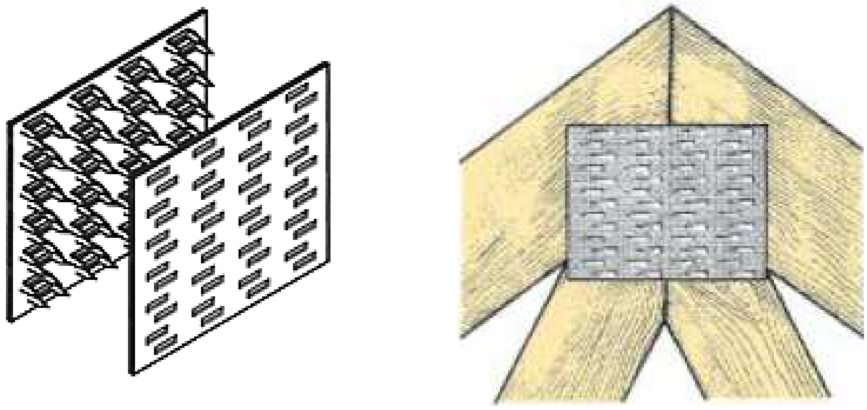


Figure 2.2: Punched nail plate, Truss connection with nail plate (from mii.com/unitedkingdom).

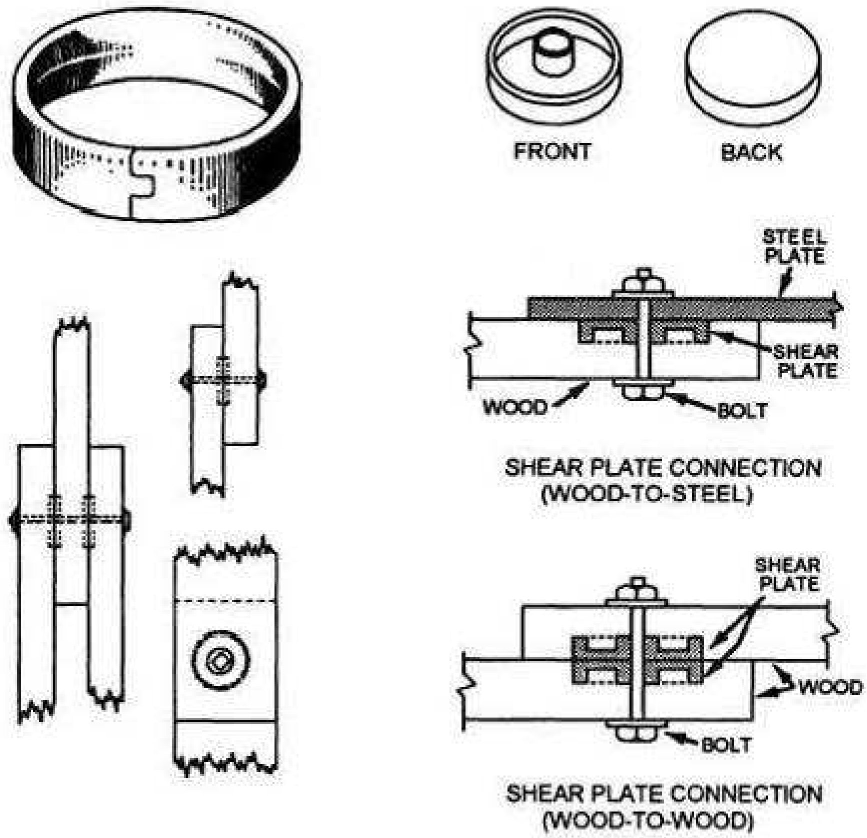
2.2.2 Bearing connectors

Bearing type connectors are capable of transmitting the highest lateral shear loads per unit of all the mechanical connector types available for timber construction. Bearing type connectors were developed and have been used for more than a hundred years, with the first patented in 1889 in the U.S. (American Society of Civil Engineers, 1996). They were created to increase the bearing, shear areas in timber joints by utilizing rings and shear plates. They are used in timber to timber and steel to timber connections in combination with bolts.

Three main types of bearing connectors have been developed and are still widely used in heavy timber construction: split rings, shear plates and toothed plates.

Split rings and shear plates are always circular as they are placed into grooves predrilled by circular cutters, their diameter vary between 60 and 260 mm. The manufacturing of a joint is similar for both types of connectors. First a bolt hole and groove are pre-cut in the timber members – this operation requires accuracy for the grooves to match on the opposite sides of the timber members to be connected, and involves specialist equipment – then the connectors are placed into the cuts, followed by the timber members, finally the bolts are inserted and tightened to form the joint, Figure 2.3.

Split rings are used in timber to timber joints, and are the most efficient connectors for these types of joints. They were developed as flat rings; however the shape evolved to now be double levelled which eases the installation and provides a tighter fit, with the split in the cross section allowing the ring to expand during insertion into the groove. In a split ring connection lateral shear loads are transferred from one timber member to the ring through embedding stresses, then through its shear resistance to the second timber member. The purpose of the bolts in split ring joints is to hold the timber to be connected together, and its resistance is usually ignored in design (STEP 1, 1995).



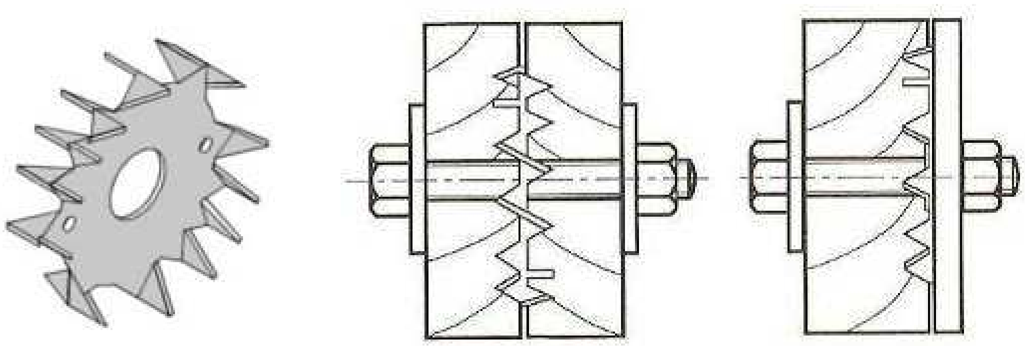
a) b)

Figure 2.3: Bearing connectors: a) Split ring and connections, b) Shear plate and connections. (from www.tpub.com)

Shear plates can be used in steel to timber, and timber to timber connections when a pair is used back to back; they are placed into a groove totally embedded into the wood. The load transfer in connections using shear plates uses the same principles as split ring joints, with the only difference being that the load is transferred between the members through the shear resistance of the bolt. Split rings and shear plates are usually made of low carbon steel or malleable iron, however for use in corrosive environment they can be manufactured from stainless steel or fibreglass (American Society of Civil Engineers, 1996).

Toothed plate connectors are available in various shapes, however the most common are circular with a diameter measuring 38 to 165mm. Double-sided and single-sided toothed plates exist for timber to timber and steel to timber connections respectively, Figure 2.4. They do not require a pre-cut groove to be used as they are pressed into the timber members to be connected; however it is therefore recommended that they are only used in timbers with a characteristic density of 500 kg/m^3 or less (STEP 1, 1995).

As the teeth of the plates need to be pressed into the timber, hydraulic presses or high strength bolts need to be used for manufacturing the joints due to the high forces required to embed the plates in the wood. The load is transferred from the timber to the plate through embedment resistance of the teeth, and further through the plate into the other timber member with double sided plates. In single sided plates the load is transferred from the timber through embedding of the teeth of the plate, then the bolt is loaded which in turns loads a second single sided toothed plates in timber to timber connections, or a steel member in timber to steel connections.



a) b) c)

Figure 2.4: a) double sided toothed plate, b) timber to timber joint with double sided toothed plate, c) timber to steel joint with single sided toothed plate (STEP 1, 1995).

2.3 Timber connections with dowel type connectors

As section 2.2 demonstrates the range of mechanical connectors available for use in structural timber is wide, with each type of connector having different properties and advantages as a solution to a connection problem. In this section a review has been undertaken on the load carrying capacity and behaviour of timber connections with dowel type fasteners. The main factors influencing their behaviour and resistance are reviewed, and their effects detailed.

2.3.1 On timber joints

Historically, experimental research on timber joints was conducted on similar joint arrangement. The samples usually consisted of a timber member sandwiched between two other members that were of timber, timber based material or steel, with connectors penetrating the members and acting in single or double shear under lateral loading. Typically, from testing of connections samples with dowel type fasteners the following observations are made from the deformation between the middle and sides members:

1) The fastener bends due to the relative displacement of the timber members,
2) The timber close to the shear plane under the fastener is crushed,
3) The fastener is being pulled out of the timber,

From these observations it is possible to identify the three main parameters affecting the load carrying capacity of timber connections with dowel type fasteners:

- *Fastener yield moment*; the resistance of the fastener to bending,
- *Embedment capacity*; the crushing resistance between the fastener and timber,
- *Withdrawal capacity*; the pull out capacity of the fastener from the timber.

These three parameters, and therefore the load displacement behaviour and ultimate resistance of a timber connection with dowel type fasteners, are influenced by a number of variables which can be grouped into three categories, shown in Table 2.1 (Goh, 1997):

- Material properties and dimensions of the fastener and timber,
- Joint configuration,
- Loading conditions.

Table 2.1: *Factors influencing timber connections.*

Material and dimensions		**Joint configuration**	**Load conditions**
Fastener	**Timber member**		
Type	Density	Number of fasteners	Type of loading
Length	Moisture content	Number of shear planes	Static
Size	Swelling / Shrinkage	Member thickness	Dynamic
Shape	Relaxation	Predrilling	Cyclical
Surface	Friction	End and edge distances	Rate of loading
Mechanical Properties	Mechanical Properties	Fastener spacing	Load duration
Tensile strength	Compressive strength	Angle between fastener axis and grain orienation	Time between assembly and loading
Flexural properties	Embedding strength		
Ductility	Modulus of elasticity	Depth of penetration	
Buckling	Foundation modulus		
Stiffness	Creep modulus		

These factors have been studied for a number of years, and have different degrees of influence on timber joints. The main factors and their effect on timber connections are detailed in the following sections.

2.3.2 Effect of timber properties

2.3.2.1 Timber density

Timber density is the most important physical property of timber (STEP1, 1995) almost all mechanical properties of timber, including strength, stiffness, are positively correlated to it, as are the strength and stiffness or timber joints. The density of timber varies greatly between and within species, from 160 kg/m^3 to 1040 kg/m^3. However for most structural timbers it falls between 300 and 500 kg/m^3 in Europe and 720 kg/m^3 in America, (Forest Product Laboratory, 1999)

Timber density is determined by the amount of wood substance and the amount of water present, which varies depending on the environmental conditions. Therefore for comparison purposes between species the specific gravity is used as reference. The traditional definition of the specific gravity is the ratio of the density of wood to the density of a reference material, usually water. However to further reduce the uncertainty over the moisture present in the timber it is common practice in engineering to use the specific gravity based on oven-dry mass and volume at green, oven-dry or 12% moisture content. The use of the specific gravity instead of the timber density allows taking the effect of the moisture content on its own.

Porteous (2003) noted that research on the behaviour and performance of timber joints, using either empirical or elastic analyses approaches found that in both methods the strength and stiffness of timber joints are a linear function of the wood density. When using the yield theory it was found that the strength and stiffness of timber joints are function of the embedment strength, this is examined later in this chapter.

2.3.2.2 Moisture content

Wood is a hydroscopic material, meaning that it is constantly exchanging moisture with its surroundings, and its moisture content will always vary towards equilibrium with the environment. The rate of change is slow and will correspond only to weekly or monthly changes in the humidity (Thelandersson & Larsen, 2003). The moisture content (*mc*, in %) of timber is the ratio of the mass of removable water to the dry mass of wood:

$$mc = \frac{m_{\omega} - m_0}{m_0} \cdot 100 \qquad \textbf{…(2.1)}$$

Where m_{ω} is the initial mass, and m_0 the oven- dry mass.

The moisture content of a living tree can be as high as 200%, however in most covered structures the moisture content of the timber members generally varies between 7% and 14%. Moisture in timber is held in two ways: in the cell cavity (free water) and within the cell wall (bound water) (Breyer et al., 2003). During drying the first water removed is the free water. When it is removed completely, and the bound water is still present the timber reaches the stage named fibre saturation point; for most species this stage correspond to a moisture content of 25% to 35%, a convenient average is 28% (STEP1, 1995). The fibre saturation point is of particular significance since below this point changes in volume and structural properties happen. With varying moisture content below saturation point, and the relatively slow rate of variation which results in a non linear moisture distribution in the timber, internal stresses are induced due to the constrained swelling and shrinkage of the wood. These stresses are negligible in the longitudinal direction where the strength of timber is high; however they can cause some cracking in the direction perpendicular to the grain where the strength is relatively low. Surface cracking of structural timber elements in the direction perpendicular to the grain is common due to varying moisture content (Thelandersson & Larsen, 2003).

Generally an increase of moisture content has an adverse effect on the mechanical properties of timber. The effects on the mechanical and physical properties of timber are not linear over the full range, and dependant on each specific property, where a variation of 1% of the moisture content induces a change ranging between 0.5% and 5% (STEP1, 1995). However for practical reasons it is generally assumed that between 8% and 20% the relationship between moisture content and timber properties is linear.

Mack (1966) studied nailed joints with green and dry timber which resulted in a reduction factor of 1.39 in joint reduced load strength with green wood. He also found that between nailed joints made of dry wood and timber at 12% moisture content there was no significant difference. Morris in 1970 expanded on the work, and found different reduction factors from Mack, but when the moisture content was increased to 18% the factor agreed with Mack. These results therefore suggested that moisture content influences greatly the performance of joints. Morris further investigated the moisture content effect and accommodated its influence in a semi empirical model (Goh, 1997). Further research on the moisture content found that it influences the effect of the duration of the load, where at high moisture content the effect of the duration of the load is more evident (Fridley et al., 1992). This effect has been included in Eurocode 5 in the modification factors for service classes and load duration factors (Porteous, 2003).

2.3.2.3 Grain orientation

The grain of timber is the vertical alignment of the fibre cells as the tree grows. Due to this arrangement the timber properties vary in three orthogonal ways, the longitudinal direction parallel to the fibres, the radial and tangential directions perpendicular to the growth rings, Figure 2.5.

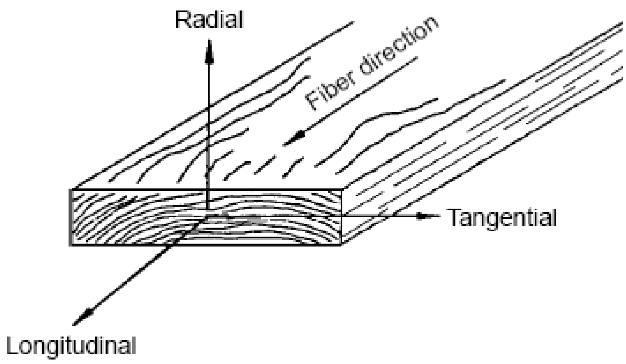


Figure 2.5: The three principal axes of wood (Forest Product Laboratory, 1999)

The difference between the two perpendicular directions compared to the longitudinal direction is small, and therefore in engineering two axes are used: parallel and perpendicular to the grain. Due to the nature and cellular arrangement in trees, the properties relative to the grain direction exhibit no symmetry, therefore anisotropy in tension and compression is unavoidable.

Work by Hankinson (1921) confirmed that the relationship of timber strength properties to the grain orientation can be described by the following equation:

$$n = \frac{p \cdot q}{p \cdot \sin^2 \varphi + q \cdot \cos^2 \varphi} \qquad \textbf{…(2.2)}$$

Where, n is the unit strength at angle φ, p is the unit strength parallel to the grain, and q is the unit strength perpendicular to the grain.

On investigating the effect of grain orientation on the mechanical properties of joints with dowel type fasteners, Smith and whale (1985) found that it had little influence, under lateral loading, on the stiffness and strength characteristics of the joints with nails and bolts. However for joints with shear plates and split rings it had a significant influence. Further research on the influence of the grain orientation on the embedding strength of fasteners concluded that grain direction influence was dependent on the fastener diameter (Wilkinson, 1991; Whale et al., 1988). Small diameter dowels were not influenced by the loading direction to the grain, whereas the larger diameter dowels embedment strength was dependent on the loading direction.

These findings are accepted by the engineering community, and have been included in the European design standard for timber. For nails the embedment strength is independent of the grain direction, for bolts the embedment strength is dependent on the grain direction and its influence is described by a Hankinson type formula (Thelandersson & Larsen, 2003).

2.3.3 Effect of fasteners properties

2.3.3.1 Effect of material properties

As mentioned in section 2.2 fasteners, and nails in general can be manufactured, treated, or have coating added on their shank. Coatings on the shank of nails aim to improve the fastener surface to decrease the driving resistance or increase the withdrawal resistance, or a combination of both (American Society of Civil Engineers, 1996). Cement coating is a common coating in America, it does not contain as its name suggest cement, but compositions of resins depending on the manufacturer. Cement coating generally increase the withdrawal performance of nails by increasing friction, its impact is immediate in softwood, but almost negligible in hardwood where the coating is removed during driving of the fastener. Other coatings include thermo-plastics and thermo-setting polymers, usually referred to as plastic coatings. When used in mechanical guns for insertion they improve the drivability of the fasteners in the timber as the plastic coating melts during insertion, and then increase the withdrawal performance as the plastic hardened in the timber, creating a strong bond between the wood fibres and fastener. However the effectiveness of each coating depends on the bond created between the steel fastener and resin as well as bond between wood fibres and resin. Consequently improvement of the mechanical properties of the nails due to the presence of a coating varies greatly between manufacturers and the compositions of the resins, (Forest Product Laboratory, 1999).

In addition to coatings the shank of nails which can be plain, toothed, formed or deformed are available for structural purposes in order to increase the withdrawal resistance, and in some cases also improve the lateral shear strength of joints. The aim of such surface modification of nails is to increase the contact surface between the wood and nail without increasing the nail weight. Deformed nail, as opposed to formed nail, are nails that have the shank modified or improved after forming the nail, and therefore deformed nails have a clear part on their shank not modified. At constant moisture content, nails with improved shank have an increased withdrawal resistance of about 40% compared to equivalent diameter round wire nails. However the improvement is even greater when moisture conditions are varying, with the withdrawal resistance of improved nails four times that of equivalent round nails.

2.3.3.2 Fastener yield moment

The yield moment of the fasteners is a critical parameter in the behaviour of joints. This parameter was first taken into account by Johansen in 1949 when he included in the yield equations the plastic bending capacity of the fasteners due to the relative displacement of the timber members of the connections.

When Eurocode 5 connection design method was drafted it followed Johansen method and assumed that the materials behaved in a rigid plastic manner, with the plastic moment capacity of the fasteners taken into account. To characterise the moment capacity of fasteners BS EN 409:2009 (BSI, 2009b) was adopted, it provides a method of testing fasteners using a four points bending set up. The yield moment of the fasteners of a diameter inferior or equal to 8mm is determined for an angle of 45°, at which the whole cross section of the fasteners is assumed to be fully plasticized (Blass, 2001). However at angles less than 45°, only the outer areas of the cross section of the fastener are under plastic deformation, and therefore the true moment capacity of the fastener is between the elastic and plastic bending strength. This fact is particularly important in connections with “large” diameter fasteners, as various studies showed that while testing in accordance to BS EN 26891 (BSI, 1991) to a maximum relative displacement between timber members of 15mm, the angle measured on the fasteners is never of 45°, but can be as low as 5 to 10°.

In a paper presented in the Working Commission 18 on timber structures of the International Council for Research and Innovation in Building and Construction (CIB-W18) in 1998 Jorissen & Blass reported that the bending moment increases with the bending angle, and as a result the failure modes and load carrying capacities are in turn affected by this angle. They calculated that for a bending angle of 5° the bending capacity of an M12 bolt is only 60% of the plastic capacity, and the load carrying capacity of a connection is therefore reduced by 13% compared to the calculated capacity with full plastic bending capacity. Following these studies an equation was developed for determining the bending capacity of bolts and dowels that was then extended to all fastener diameters and included in Eurocode 5 (BSI, 2004):

$$M_{y,Rk} = 0.3 \cdot f_{u,k} \cdot d^{2.6} \qquad \textbf{...(2.3)}$$

Equation 2.4 was developed for "small" diameter fasteners and was included in the Eurocode 5 draft in 1994:

$$M_{y,Rk} = 0.133 \cdot f_{u,k} \cdot d^{3} \qquad \textbf{...(2.4)}$$

However a recent study (Jorissen & Leijten, 2005) investigated the applicability of Equation 2.4, developed for "large" diameter fasteners, for "small" diameter fasteners.
The research concluded that for "small" diameter fasteners Equation 2.3 results in more accurate results, and also that Equation 2.4 did not result in unsafe yield moment capacity for "small" diameters fasteners. It also suggested that only one equation should be included in Eurocode 5 in order "to keep it as simple as possible", consequently Equation 2.3 is the only referenced equation in Eurocode 5 (BSI, 2004).

2.3.4 Embedment strength

When developing the yield equations for determining the load carrying capacity of joints, Johansen assumed that the timber was a rigid plastic material, and therefore that crushing of the timber was constant with varying dowel diameters, or other material properties. The first studies that investigated the embedding strength of dowel were published by Siimes et al. (1954) and Noren **(**1968), in which the embedment strength was a function of the timber density, moisture content and dowel diameter and shape (Porteous, 2003). However the results of the different studies could not be related as the methods, procedures and interpretations were not uniform or standardised. To answer to this lack of knowledge an extensive research program was developed at the Timber Research and Development Association in the U.K. using the previous research as a basis to develop a test method, procedure and test apparatus to characterise the embedment strength of timber fasteners (Rodd et al, 1987). To achieve this, the researchers used a set up that can be regarded as a three member connection in which a thin timber is sandwiched between two thick steel plates ensuring that failure is achieved by crushing of the timber. Rodd et al. (1987) considered that the thickness of the timber sample was critical in determining the embedment strength and therefore recommended that it should be limited to twice the fastener diameter. It was also recommended in the test protocol that a gap should be available between the steel and timber to avoid any friction forces, and that the fastener should be inserted in the timber following site practice (predrilling, insertion tool …), Figure 2.6. This set up was then adopted by the European committee developing the harmonised design codes and test methods for use across the countries of the European community in BS EN 383:2007 (BSI, 2007).

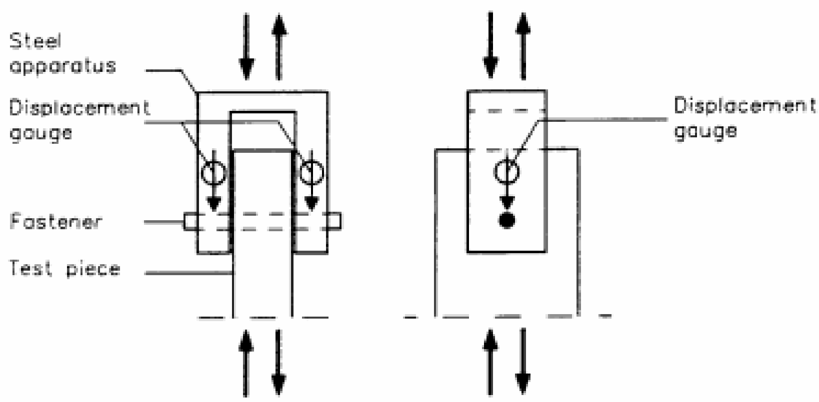


Figure 2.6: European embedment test setup (BSI, 2007).

In parallel, similar research on the embedding strength was being conducted in America by Wilkinson (1991). The set up developed for measuring the embedding strength differed from the European method by placing the fastener on top of the timber sample and applying the load uniformly on the fastener, Figure 2.7. This approach presents the advantage to avoid any bending of the fastener during testing. However it does not enable an accurate measurement of the embedment stiffness, as the possible shortening of the timber sample and the slack in the testing machine cannot be evaluated and separated from the relative displacement of the fastener to the timber samples. (Pope & Hilson, 1995).

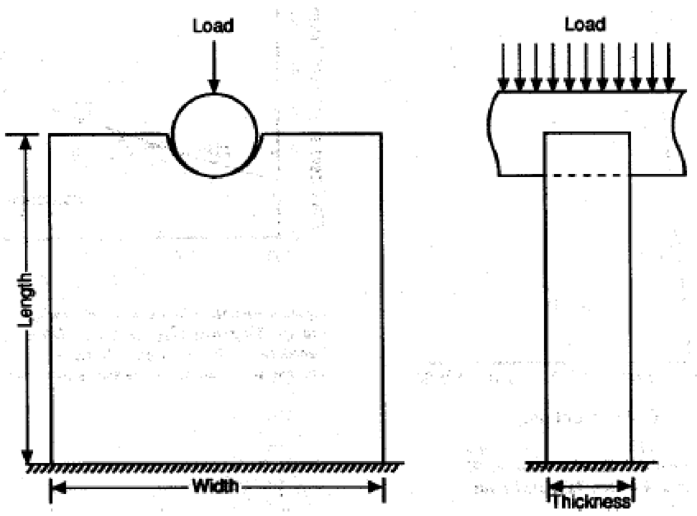


Figure 2.7: American embedment test setup (Wilkinson, 1991)

The most comprehensive research in Europe on the embedment strength of fasteners in timber was carried out by Smith & Whale (1988) using the test set up shown in Figure 2.6. The study concluded that the embedment strength of a fastener was positively correlated to the timber density, and a design equation was derived and applied to the design of timber joints. The conclusions of this research were adopted and are now part of Eurocode 5.

Since the adoption of the tests methods in Europe and America, studies focused on the parameters that influence the embedding strength of fasteners (Ehlbeck & Werner, 1992; Mohammad & Smith, 1997; Foschi et al., 2000; Hwang & Komatsu, 2002). From numerous studies it is clear that with moisture content increasing the bearing strength is decreasing. This fact was observed to be independent of timber species and fasteners diameter (Rammer & Winistofer 2001). In addition these studies showed that when reaching high moisture content, the embedment strength stays constant with further increasing the moisture content in the timber; this point was 25.3% from Rammer & Winistorfer (2001), 22.5% from Koponen (1991), 21% from Wilkinson (1971), and 23% from Green & Kretschmann (1994). This point at which the bearing strength keeps constant is close to the timber saturation point, however no clear conclusion has been made on the relationship between the two parameters. Also, from their extensive research Whale et al. (1987) concluded that predrilling influences greatly the bearing strength of fasteners, due to the contact area between the timber and fastener. This was also translated to Eurocode 5 with a design equation for determining the embedment strength of fastener with and without predrilling.

2.3.5 Effect of load conditions

Numerous researches showed that the type of loading applied to a timber structure have a great influence on its behaviour, strength and stiffness (Girhamman & Andersson, 1988). However there is limited amount of results on the behaviour of timber connections to the rate of loading and duration of loading. In a preliminary study Rosowsky et al. (1999) concluded that the duration of load effects for a timber connection varied from those of timber, and were to be investigated independently. In the same study it was also concluded, however with reservations due to the limitation of the experimental programme, that the rate of loading in static tests did not influence nailed connections subject to lateral or withdrawal load.

2.4 Withdrawal strength of fasteners

2.4.1 General

Fasteners in timber structures are subject to withdrawal loads when the load is applied parallel to their longitudinal axis, and when used in lateral shear connections, where at relatively large displacement the load acts parallel to the nail (Forest Product Laboratory, 1965). Common round wire nails resist these loads by the friction forces between the shank of the fastener and the wood fibres (Rammer et al., 2001). However various studies showed that these frictional forces are greatest just after driving, before any relaxation in the wood happens, and that the withdrawal resistance reduces with time. A study in 1938 showed that the withdrawal resistance of round wire nails decreased by 57% after 105 days. This observation was one of the main influences for developing nails with deformed shank. Threaded nails, and screws, also resist axial loads by friction of the wood fibres and the nail shank, but more importantly by lodging wood fibres between the threads. Threaded nails are withdrawn from the timber when the fibres locked into the threads are broken, therefore wood relaxation or shrinkage has little influence on their withdrawal resistance. Threaded nails offer about 40% increase in withdrawal resistance compared to common nails when inserted into timber that stays at constant moisture content, (Forest Product Laboratory, 1999).

The behaviour and performance of fasteners axially loaded were investigated and showed that the withdrawal capacity of fasteners is influenced mainly by the timber density and moisture content, fastener diameter, shank surface and condition, depth of penetration and grain orientation.

Research on the influence of moisture content showed that when a fastener is inserted into green timber and pulled before any seasoning occurs it has the same withdrawal resistance to nails inserted into seasoned wood and pulled soon after driving. However the withdrawal capacity of smooth nails can be decreased by as much as 75% when inserted into green timber that is allowed to season or into seasoned timber that is subject to varying moisture conditions (Senfit & Suddarth, 1971). The influence of moisture content on the axial capacity of threaded nails has not been widely published, however the Wood Handbook (Forest Product Laboratory, 1999) states that at varying moisture condition their withdrawal strength is about 4 times that of common nails. In 2004 Rammer & Zelinka (2004) reviewed the research on the withdrawal strength and behaviour of nails axially loaded in end grain timber. They concluded that the ratio of end to side grain withdrawal strength varied with the timber species from 0.5 to 0.8, but was independent of wood density. For threaded nails this ratio is somewhat lower due to the greater strength in side grain. It was also noted that for smooth nails the ratio between the side and end grain withdrawal strength reduces when the time between sample fabrication and testing increases.

To increase the withdrawal resistance of common nails different techniques, other than shank deformation, have been developed over the years. Cement and plastic coatings are the two most used techniques that increase the withdrawal of round nails. Their effectiveness is influenced by a number of parameters such as the quality of the bond between the coating and nail shank or wood fibres, the capacity of the coating to resist driving into the timber, and the chemical interaction between the coating and the timber treatment (Forest Product Laboratory, 1999).

Stern et al. (1994) investigated the load slip behaviour of smooth round and threaded nails. The results showed that the ultimate axial resistance of smooth nails is reached when the friction is overcome, also at constant rate of loading as the nail is pulled out of the timber friction is regained until it is exceeded again and again, Figure 2.8. As the nail is pulled out of the timber the friction between the timber and nail is decreasing.

Figure 2.8 also shows the load displacement behaviour of annularly and helically threaded nails. Annular shank nails have greater withdrawal resistance, and helical nails are the most ductile fasteners. It was also noted that the three types of nails have similar stiffness.

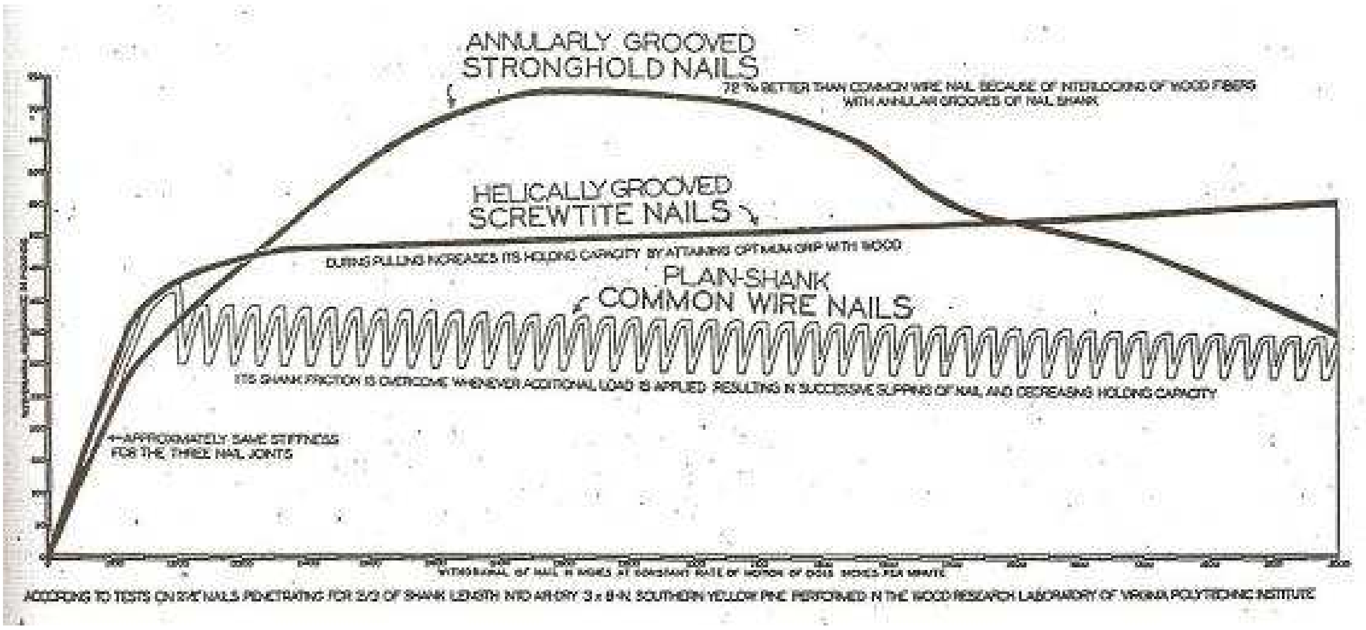


Figure 2.8: Withdrawal load displacement behaviour of common and threaded nails – Nail displacement (in Inches) vs withdrawal resistance (in pounds) (Dolan, 1995)

2.4.2 Evaluation of withdrawal strength

As round nails only resist withdrawal loads by the frictional forces between the timber and nail shank, and these forces decrease in time and with varying moisture conditions Eurocode 5 does not allow the use of such nails to be subjected to permanent or long term axial loads when inserted perpendicular to the grain. Eurocode 5 also recommends that when inserted parallel to the timber fibres no nails, smooth or threaded, should be considered to be capable to transmit axial loads.

The withdrawal strength of fasteners should be determined by tests according to BS EN 1382 (BSI, 1999), as Eurocode 5 (BSI, 2004) only provides design equations for smooth nails and screws. In this standard the withdrawal strength is defined as the load per unit nail diameter times the penetration length. The withdrawal capacity of fasteners is then calculated in Eurocode 5 by multiplying the withdrawal strength by the nail diameter and the penetration length.

In North America various research were conducted for characterising the withdrawal strength of nails. Following extensive testing, empirical equations were developed and are used in calculating design withdrawal values. The equations used are of the following form:

$$W_{pen} = adg^{b} \qquad \textbf{...(2.5)}$$

Where W_{pen} is the load per unit penetration (N/mm), d the nail diameter (mm), g the timber specific gravity, a and b constants to fit test data.

2.5 Laterally loaded timber joints

Due to the large number of parameters influencing the behaviour and load carrying capacity of joints with dowel type fasteners, and therefore the possible combinations in a joint, researchers have largely focused on characterising and predicting the ultimate, yield strength of connections or their load displacement behaviour.

2.5.1 The yield theory

The yield theory was developed by Johansen in 1949, it consists in applying the plastic theory to timber joint behaviour. To predict the ultimate strength of nailed joints Johansen assumed that both the nails and timber were ideal-plastic materials, Figure 2.9. With these assumptions he simplified the analysis without significantly impacting on the final result (STEP1, 1995).

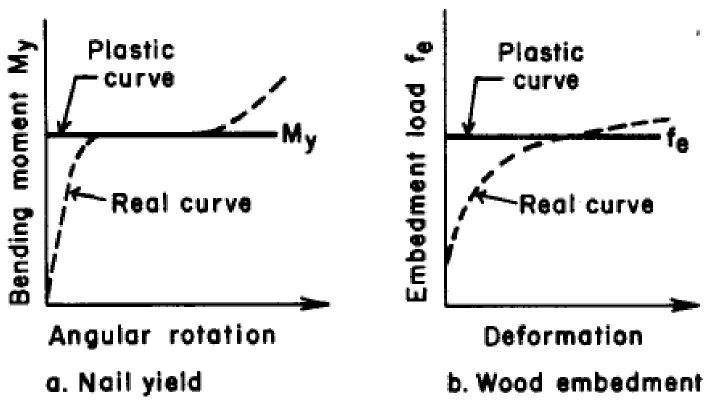


Figure 2.9: Idealisation of nail and wood properties (Aune & Patton-Mallory, 1986(a))

In addition to making assumptions on the material properties, the model developed by Johansen (1949), assumes that the connection does not fail due to insufficient spacing between fasteners or end distances before reaching yielding point, and ignores the friction that may exist between timber members. With these assumptions Johansen derived equations predicting the ultimate strength of single and double shear joints due to either a bearing failure of the timber member or the simultaneous development of a bearing failure and plastic hinge formation in the fastener. Each of the equations derived relate to a particular mode of failure of the joint, Figure 2.10.

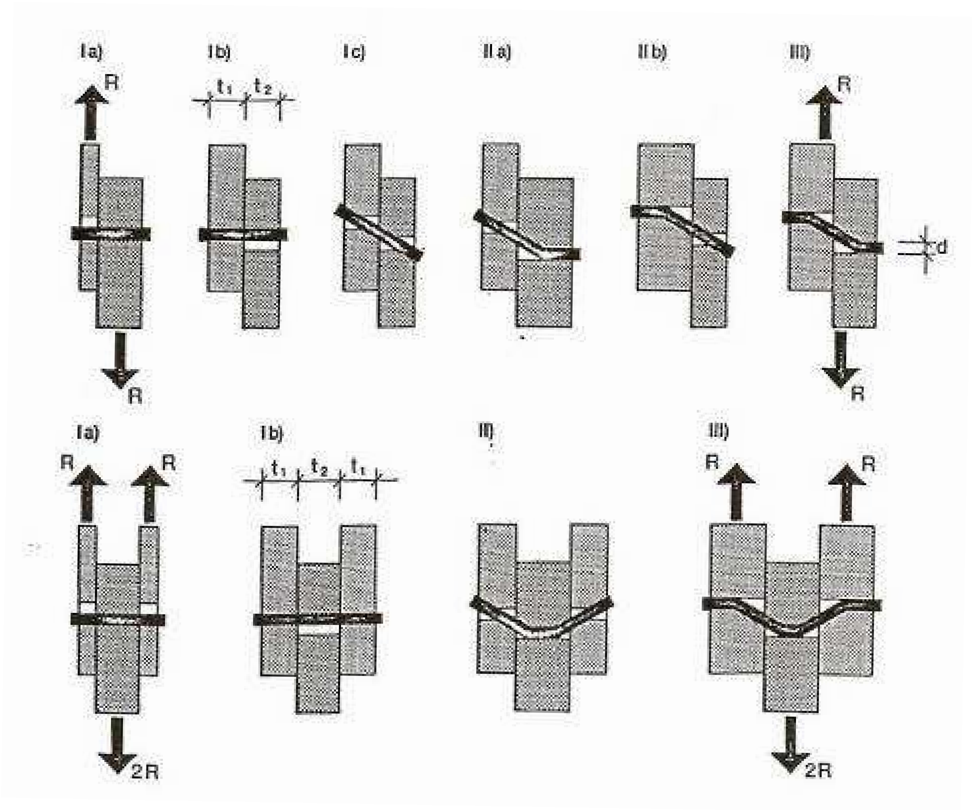


Figure 2.10: Modes of failure for single and double shear connections (Ehlbeck & Larsen, 1993)

Using Johansen's work as basis various researchers (Mack,1966; Larsen, 1979; Aune & Patton-Mallory, 1986; Smith & Whale, 1987) validated the theory with experiments. With time enhancements were provided following extensive research work, the yield theory is now used to accurately predict the yield load of single and double shear joints with dowel type fasteners including the effects of different embedding strength, joint geometry and joints with steel side plates and gaps from layers of insulation or local reinforcement using timber based materials (Aune & Patton-Mallory, 1986(b)).

In 1988 the drafting panel for Eurocode 5 adopted the yield theory as the basis for design of joints (Hilson & Whale, 1990). The current version of EC5 provides equations for determining the characteristic strength of joints with timber members and with steel side plates. In the design equations the rope effect is also included as previous versions did provide too conservative values, however limiting factors were introduced to avoid relying on the withdrawal of the fasteners when designing connections.

The current design equations for timber to timber connections for dowel type fasteners are as follow:

- For single shear connection:

$$F_{\mathrm{v,Rk}} = \min \begin{cases} f_{\mathrm{h,1,k}} t_1 d & \text{1a)} \\ f_{\mathrm{h,2,k}} t_2 d & \text{1b)} \\ \dfrac{f_{\mathrm{h,1,k}} t_1 d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{1c)} \\ 1{,}05\dfrac{f_{\mathrm{h,1,k}} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,1,k}}\, d\;\, t_1^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{2a)} \\ 1{,}05\dfrac{f_{\mathrm{h,1,k}} t_2 d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,1,k}}\, d\;\, t_2^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{2b)} \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{\mathrm{y,Rk}} f_{\mathrm{h,1,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{3)} \end{cases}$$

- For double shear connection:

$$F_{\mathrm{v,Rk}} = \min \begin{cases} f_{\mathrm{h,1,k}} t_1 d & \text{1a)} \\ 0{,}5 f_{\mathrm{h,2,k}} t_2 d & \text{1b)} \\ 1{,}05\dfrac{f_{\mathrm{h,1,k}} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{\mathrm{y,Rk}}}{f_{\mathrm{h,1,k}}\, d\;\, t_1^2}} - \beta\right] + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{2)} \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{\mathrm{y,Rk}} f_{\mathrm{h,1,k}} d} + \dfrac{F_{\mathrm{ax,Rk}}}{4} & \text{3)} \end{cases}$$

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

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

d is the fastener nominal diameter;

t_i is the timber thickness or fastener penetration depth;

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

β is the ratio between the timber members embedment strength;

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

2.5.2 Load displacement models

While the yield theory was chosen as the method of design in the European and Canadian standards, a major disadvantage of the method is that there is no indication of the load displacement behaviour of the connection. Therefore different methods were developed either simultaneously or with the aid of advancing technologies for modelling the load displacement behaviour of connections with dowel type fasteners (Erki, 1990).

2.5.2.1 Empirical models

The first empirical model developed to relate the strength of a nailed connection to the slip of the connection was by Ivanov in 1949, in the form of a second order equation. However it was only in the 1960's that practical models were developed, the first published by Mack in 1966, (Porteous, 2003).

Mack developed an empirical model for determining the load displacement relationship of a short term laterally loaded nail by assuming that the different factors affecting the joint behaviour did not interact, and that the relationship between the joint load and displacement was a function of the product of each of these factors. The experiments performed during the research showed that the variables chosen by Mack did not significantly interact, and therefore his approach was valid. As various variables were investigated Mack used the concept of reduced load to derive a displacement function for all the different joint configurations tested. The reduced load was defined as the ratio between the load at slip δ to the load at the upper limit displacement which was arbitrary set at 2.54mm. The displacement function used was of the following form:

$$f(\delta) = (A\delta + B)(1 - e^{-C\delta})^D \qquad \textbf{…(2.6)}$$

This method has since been used in various studies to examine the effects of different variables on the load displacement behaviour of joints with dowel type fasteners. Recently Porteous and Kermani (2005) used a similar method to that used by Mack and expressed the relation between the load and displacement for a connection with fully overlapping nails as follows:

$$P_x = f(\delta), k \qquad \textbf{…(2.7)}$$

Where P_x is the load at displacement x, $f(\delta)$ the displacement function and k the product of the functions of the variables that influence the joint behaviour.

Other empirical models were developed and most notably by McLain in 1976 in the form of:

$$P = A \cdot \log(1 + B\delta) \qquad \textbf{…(2.8)}$$

Where P is the load at displacement δ, and A and B are curve fitting constants. This model was later enhanced by SaRibeiro and SaRibeiro (1991) by incorporating the effect of moisture content, timber specific gravity, gusset thickness and nail diameter in the constant A and B (Kermani & Goh, 1999).

2.5.2.2 Elastic theory approach

This approach was first used by Kuenzi (1955) where the fastener was represented as a beam on elastic foundations, Figure 2.11 (Foschi &Bonac, 1977).

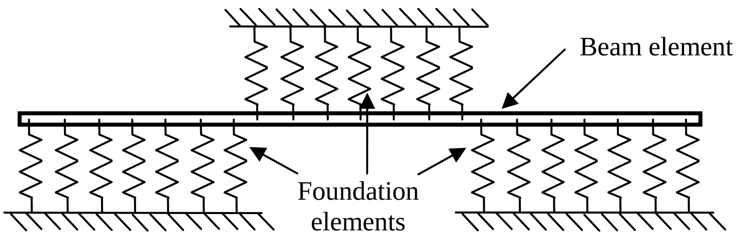


Figure 2.11: Elastic theory representation of a double shear connection.

The deflection curve of a beam on elastic foundation is given by:

$$\frac{EId^4 y}{dx^4} = -ky \qquad \textbf{…(2.9)}$$

Kuenzi used the differential equation with E the elastic modulus of the nail, I the moment of inertia of the nail, and k the foundation modulus of the timber (Porteous, 2003). The solutions derived for representing the curve combined trigonometric and hyperbolic functions, and allowed the calculation of pressure, moment, shear and deflection at any point of single or double shear joints. These equations were experimentally validated by Stluka (Wilkinson, 1971(b)). Improvements were brought to the solutions developed by Kuenzi by various researchers (Noren 1968, Wilkinson 1971(b)).

However the use of the elastic theory to characterise the load displacement relationship of joints with dowel type fasteners suffers two major drawbacks (Porteous, 2003):

- The theory is only valid for the elastic range of the load displacement curve of joints which is widely believed to be to a slip of 0.3mm,
- Numerous researchers argue that the load displacement behaviour of joints is non-linear from the beginning of loading.

2.5.2.3 Finite element models

Finite elements models are nowadays widely used (Chen et al., 2003), as complex and time consuming operations are processed using commercially available software. This method allows researchers to address both the elastic and plastic behaviour of the materials in the connection, and also allows customising the model to the properties of the elements (Porteous, 2003). However to accurately predict the load displacement behaviour of the connections, the different input parameters need to be accurately characterised which often require extensive supplementary experimental work (Goh, 1997).

Research on timber connections with dowel type fasteners by finite element method considered two distinctive approaches:

- The fastener is modelled using springs, either in 2D or 3D, where the springs stiffness represent the fastener behaviour in the connection (yield moment, embedding and withdrawal strengths),
- Full 3D modelling of the connection, where the elements of the model are given the material properties.

2.6 Summary

The literature review focused on dowel type fasteners, and shows that timber connections can be achieved using various methods, and that each is influenced by numerous parameters. These parameters can be classified in three main groups: material and dimension properties, joint configuration and loading conditions.

Timber properties are some of the most important factors influencing timber connections. The orthotropic nature of timber, its density and the variability of moisture content in timber due to environmental exchanges need to be taken into account when designing or predicting the load capacity of joints. The fastener's properties affect the joint capacity and behaviour. But maybe even more, it is the interactions between the wood and fasteners that need to be characterised for each type of fastener.

It is also clear from the literature review that extensive work has been carried out to model the behaviour of the parameters influencing connections and the load displacement relationship or strength of the connections.

From the findings of the literature review, an experimental research programme was developed to study the behaviour of helically shaped fasteners as timber connectors. All modelling methods require that experimental work is to be carried out in order to obtain accurate results for predicting and characterising the load displacement behaviour or strength of joints.

Chapter 3 Properties of Helically shaped fasteners

3.1 General

The fasteners properties are factors that influence the load carrying capacity and behaviour of timber joints. These properties include the geometrical dimensions and material of the fasteners, but also the yield moment and embedding strength. Over the years researchers have developed test methods and procedures in order to characterise these properties, enabling the development of design equations. The aim of this chapter is to examine the material properties of helically shaped fasteners in accordance with the relevant British and European standards, and verifying the applicability of the design rules.

To gain a better understanding of the behaviour of Helically shaped fasteners used in timber structural systems; a comparative experimental programme was also carried out on conventional timber fasteners such as nails (plain and profiled) and a range of screw types. This was important as most equations and relationships detailed in previous research and Eurocode 5 (BSI, 2004) were developed for common timber fasteners. Hence, tests results on nails and woodscrews would also provide an indication of the performance of helically shaped fasteners, also providing the validation of the results. The fixings considered are shown in Figure 3.1. In Table 3.1 the dimensional details of each of the fasteners used in this study are detailed.

a) b) c) d) e) f) g) h) i) j) k) l) m)

Figure 3.1: a), b), c), & d) Helically shaped StarTie 10, StarTie 8, InSkew & TimTie; e) & f) round wire nails; g) & h) annularly threaded nails; i) helically threaded nail; j) & k) Ulti-Mate woodscrews; l) & m) BZP woodscrews.

In an effort to harmonise and standardise the types and characteristics of fasteners used in structural timber available within the European Community, a new European Standard has been drafted by the European Committee for standardization (CEN) outlining the requirements for fasteners for use in timber structural applications. The new standard, prEN 14592 (CEN, 2007) specifies the requirements for materials, geometry, strength, stiffness and corrosion resistance of the fasteners. It also aims to provide information on dimensional and mechanical properties and strength values to be used in conjunction with the design method outlined in Eurocode 5 for all types of fasteners. A nominal diameter needs to be given for all fasteners according to prEN 14592, with which calculations are made for determining the mechanical properties. For screws the effective diameter as defined in Eurocode 5 is also required.

Table 3.1: *Fasteners dimensions*

Fastener	Length		Diameters as measured			Diameters values		
	Nominal	**Measured**	**Wire**	**Thread**	**Root**	**Nominal**	**Thread**	**Root**
	mm	**mm**	**mm**	**mm**	**mm**	**mm**	**mm**	**mm**
StarTie 10	N/A	N/A	N/A	9.80	4.26	10.00	10.00	4.25
StarTie 8	N/A	N/A	N/A	7.83	3.77	8.00	8.00	3.75
InSkew	N/A	N/A	N/A	5.85	3.34	6.00	6.00	3.35
TimTie	N/A	N/A	N/A	4.44	3.03	4.50	4.50	3.00
RWN 4.50	100.00	102.10	4.48	N/A	N/A	4.50	N/A	N/A
RWN 6.00	150.00	151.60	6.01	N/A	N/A	6.00	N/A	N/A
HTN 3.10	90.00	87.72	3.08	3.21	2.99	3.10	3.20	3.00
ATN 3.75	75.00	76.02	3.76	4.28	3.47	3.75	4.20	3.50
ATN 5.00	100.00	100.02	4.94	5.59	4.61	5.00	5.60	4.60
UMW 5	80.00	79.84	3.75	4.91	3.46	4.90	4.90	3.50
UMW 6	100.00	99.22	4.48	5.89	4.18	5.90	5.90	4.20
BZPNo 10	76.20	75.34	3.67	4.94	3.18	4.90	4.90	3.20
BZPNo 12	88.90	87.70	4.21	5.51	3.86	5.50	5.50	3.90

Note: 5 fasteners were randomly selected and measured to obtain average values

3.2 Tensile strength

3.2.1 General

The tensile strength of a fastener is used in the calculations for determining the load carrying capacity of a joint. It is a parameter that influences the bending resistance of the fasteners and therefore the yield moment. It is a requirement of Eurocode 5 that all dowel type fasteners have a minimum tensile strength of 600 N/mm^2. This requirement is particularly important for fasteners produced from wire. The tensile strength of the fasteners was determined to ensure that the condition of Eurocode 5 is fulfilled for all the fasteners used in this research.

The fasteners were inserted between the “jaws” of the testing machine and tested in direct tension, with the speed of the travelling head of the machine set at 2 mm/min, Figure 3.2. Five specimens were tested for each fastener, and the maximum load attained during testing was recorded.

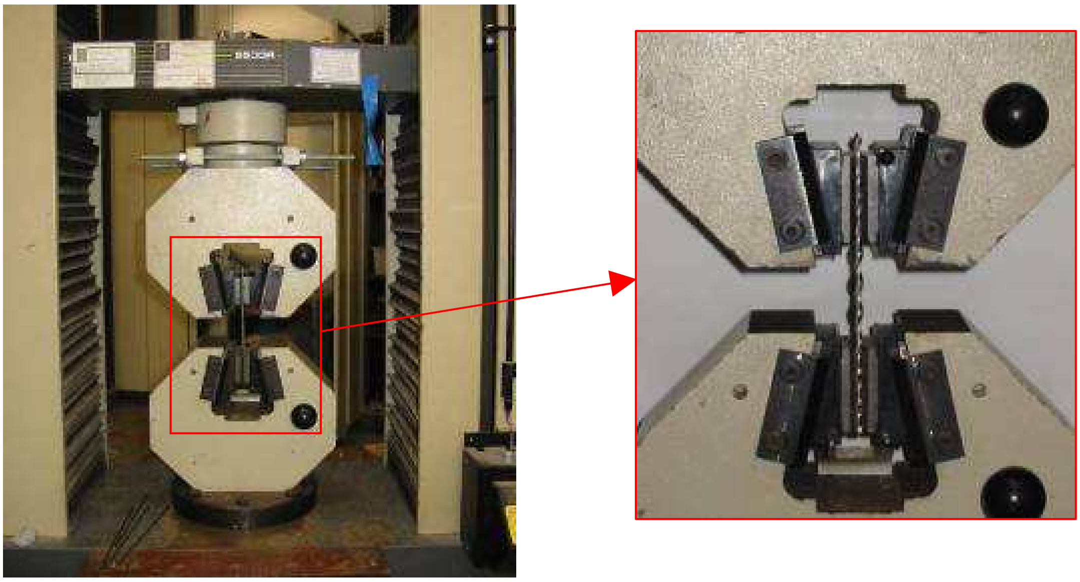


Figure 3.2: Tensile test set up

3.2.2 Tests results

The maximum loads recorded during the tests were divided by the cross section of the fasteners to obtain their tensile strength; the results are shown in Table 3.2.

The cross section of the helically shaped fasteners was determined by a simple procedure which consisted in weighing fasteners in air and in water, then dividing the volume obtained by the total length of fastener. The fasteners had the points cut in order to have a constant section along the measured lengths for determining their cross sectional area. This procedure was repeated by using a bundle of 5, 10 and 20 fasteners for each of the four sizes of helically shaped connectors; the average cross section per size was then calculated and taken as the value to use in the calculations. For all other profiled fasteners, the root diameter was used for calculating their cross sectional area.

Table 3.2: Tensile tests results

Fastener	Maximum Tensile load	Cross section	Tensile strength	Characteristic tensile strength
	N	mm^2	N/mm^2	N/mm^2
StarTie 10	16584	16.26	1020.11	867.10
StarTie 8	11567	10.25	1128.16	958.93
InSkew	9238	7.54	1224.71	1041.00
TimTie	7035	6.22	1130.48	960.91
RWN 4.50	11611	15.90	730.07	620.56
RWN 6.00	17205	28.27	608.51	517.23
HTN 3.10	6493	7.55	860.23	731.19
ATN 3.75	7804	9.62	811.14	689.47
ATN 5.00	11268	16.62	677.94	576.25
UMW 5	11584	9.62	1204.03	1023.43
UMW 6	16616	13.85	1199.36	1019.46
BZPNo 10	10604	8.04	1318.58	1120.79
BZPNo 12	14937	11.95	1250.38	1062.82

The tensile tests results show that all fasteners, except round nails of 6mm diameter, fulfil the Eurocode requirement. Round wire nails have the lowest tensile strength. Most profiled fasteners have greater tensile strength, which is often necessary as the manufacturing process require higher quality material to be used.

3.3 Yield moment of fasteners

The yield moment of a fastener is one of the main parameters that are used for determining the resistance of a joint. It represents the fastener capacity to resist the loads transmitted between a timber member to the next. The yield moment of a fastener is influenced by the fastener material, dimensions and shape.

In Eurocode 5 (BSI, 2004) the yield moment of dowel type fasteners is either derived from design equations that were originally developed for round nails with diameters up to 8.0 mm or following the test method described in BS EN 409:2009 – Timber structures – Determination of the yield moment of dowel type fasteners – nails (BSI, 2009b). The principle of the test method described in BS EN 409:2009 (BSI, 2009b) involves the loading of the fastener in such a manner that "*the loading points do not move along the nail and the loads remain normal to the axis of the fastener during the test*." In order to achieve the loading configuration described, it is given as an annex in the standard a drawing of a possible apparatus capable of achieving the desired loading conditions. However this apparatus required a level of manufacturing that was too important to be justified in this study and therefore this equipment or a similar respecting the test principle was not available. In this circumstance, it was decided to test the fasteners using a set up approaching the European Standard method, but also to determine their yield moment according to the American standard published by The American Society for Testing Materials. The results of both sets of tests are compared to the design equations given in Eurocode 5.

3.3.1 Tests set up and procedures

The first test performed on the fasteners to determine their yield moment was a four-point bending moment of the fasteners, using the set-up dimensions and loading rate described in BS EN 409:2009 (BSI, 2009b). This set up did not comply with the principle of the test as the load and bearing points stayed vertical during the test, Figure 3.3.

The dimensions recommended in BS EN 409 are as follow:

- Distance, l_1, between load and support point: $l_1 \geq 2d$,
- Distance, l_2, between the two load points: $d \leq l_2 \leq 3d$.

Due to the range of fasteners to be tested two set-ups and rate of loading were used, the details are shown in Table 3.3.

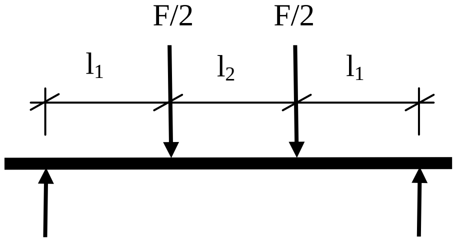


Figure 3.3: Four-point fastener bending test set up

In BS EN 409:2009 (BSI, 2009b) it is recommended that the load should be applied in such a way that maximum load is reached in 10 ± 5 seconds. For ductile fasteners the maximum load is defined as the load at which the fastener has deformed through an angle of 45º. In order to keep some consistency between fasteners tested the rate of loading was kept constant within each test set, the details are given in Table 3.3.

Table 3.3: *Four points nail bending tests details.*

Fasteners	Dimension l_1	Dimension l_2	Rate of loading
ATN 3.75	12 mm	12 mm	72 mm/min
HTN 3.00			
BZP No10			
UMW 5			
TimTie			
ATN 5	17 mm	17 mm	102 mm/min
BZP No12			
UMW 6			
RWN 4.50			
RWN 6.0			
StarTie 10			
StarTie 8			
InSkew			

The yield moment, M_y, of the fasteners is then calculated as follows:

$$M_y = \frac{F_{max}}{2} \times l_1 \qquad \textbf{…(3.1)}$$

Where $F_{max} = \min \begin{cases} \text{Maximum load sustained during testing.} \\ \text{Load at which the fastener has deformed through an angle } \alpha. \end{cases}$

In BS EN 409 (BSI, 2009b), the fastener is thought to have developed a plastic hinge, when deformed at an angle $\theta = 45^{o}$. However in BS EN 14592:2008 (BSI, 2009a) the values of θ, to be considered in the case where no ultimate load has been recorded during testing and for limiting the bent angle for screws, vary depending on the fastener type. The bending angle allowed is as follow:

- For nails, $\theta = 45^{o}$
- For screws, $\theta = (45/d^{0.7})^{o}$ (where d is the nominal diameter)

The angle, θ, is defined as the angle measured between the two parts of the fasteners between the loading and bearing points. Jorissen & Blass (1998) showed that the fastener deformation depends on its slenderness, and therefore the deformation angle is not measured similarly for fasteners with low or high slenderness, Figure 3.4. It can be considered that configuration (b) is configuration (c) with L = 0.

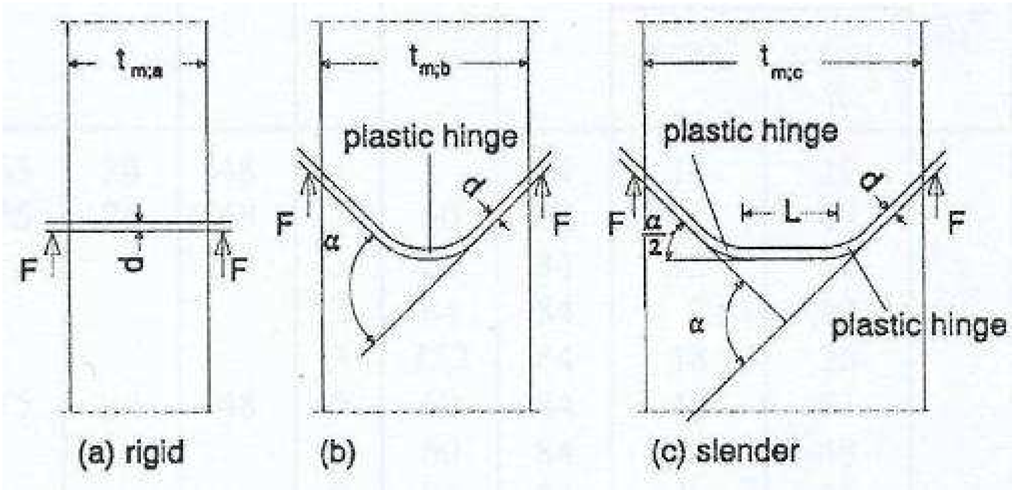


Figure 3.4: Deformation of dowel type fasteners depending on fastener slenderness. (Jorissen & Blass, 1998)

As the tests performed are straightforward four-point bending tests, the fasteners are deformed as shown in configuration (c) in Figure 3.4. The angle to consider when assessing if a plastic hinge as formed in the fastener is (θ /2).

In Table 3.4 the displacement limits calculated for achieving the different values of the angle α according to BS EN 14592:2008 (BSI, 2009a) for each fastener are shown.

Table 3.4: *Displacement corresponding to a bending angle θ according to BS EN 14592:2008 (BSI, 2009a)*

Fastener	Nominal diameter	Angle θ	Displacement
	mm	º	mm
StarTie 10	10.00	9	1.33
StarTie 8	8.00	10	1.48
InSkew	6.00	13	1.93
TimTie	4.50	16	1.68
RWN 4.50	4.50	45	7.04
RWN 6.00	6.00	45	7.04
HTN 3.10	3.10	45	4.97
ATN 3.75	3.75	45	4.97
ATN 5.00	5.00	45	7.04
UMW 5	4.90	15	1.58
UMW 6	5.90	13	1.93
BZPNo 10	4.90	15	1.58
BZPNo 12	5.50	14	2.08

The second test performed on the fasteners was a three point bending test developed by the American Society for Testing Materials (ASTM) aiming at determining "*the bending yield moment of nails, used in engineered connection applications.*" (ASTM, 2001). In this standard the test set up is also dependant on the fastener dimensions, as the loading span, bearing and load point radius vary with the fastener diameter. Following the standard strictly for the range of fastener used in this study, two loading spans should be used. However as only one of the fasteners required a different set-up it was decided to test all the fasteners using the loading span shown in Figure 3.5.

It is also recommended in the American standard that the bearing and loading points should have the same diameter as the fastener being tested. However due to the variety of fasteners used in this study it was decided that only one size of bearing would be used as it was assumed that the influence on the bearing point diameter was not significant. This assumption was comforted in a research paper by Showalter and Pollock (1994), where reviewing yield moment tests on series of small diameter nails noted that "*there was no significant radius effect for bearing and load points for the nails diameter range 2.87 to 4.83mm*". The load and bearing points used in this study were 5.00 mm in diameter.

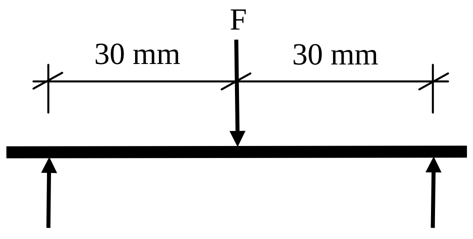


Figure 3.5: Three points fastener bending test set up.

The yield moment of fasteners is determined from each test as follows:

1 – From the load displacement relationships obtained, a straight line is fitted to the initial linear portion of the curve, Figure 3.6,

2 – The line is then offset by a distance equal to 5% of the nail diameter,

3 – The yield moment of the fastener is determined using the load at which the straight line and the load displacement curve intersect.

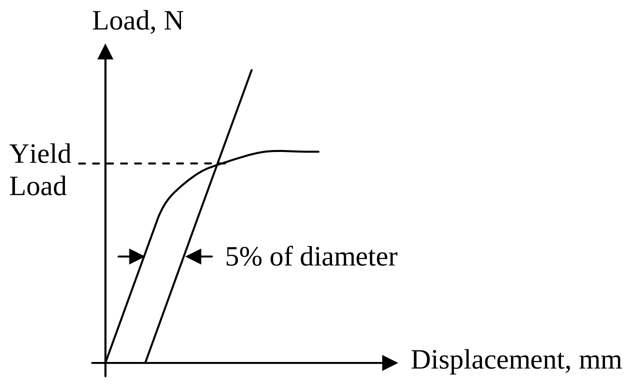


Figure 3.6: Typical load displacement curve and 5% offset yield moment load.

3.3.2 Yield moment results

The tests showed that the fasteners behave either in a brittle or a ductile manner. Therefore a limit for the bending angle allowed for determining the yield moment is necessary. Typical brittle and ductile load-displacement relationships from the four points tests are shown in Figure 3.7; typical load-displacement relationships with 5% offset load used for determining the yield load are shown in Figure 3.8.

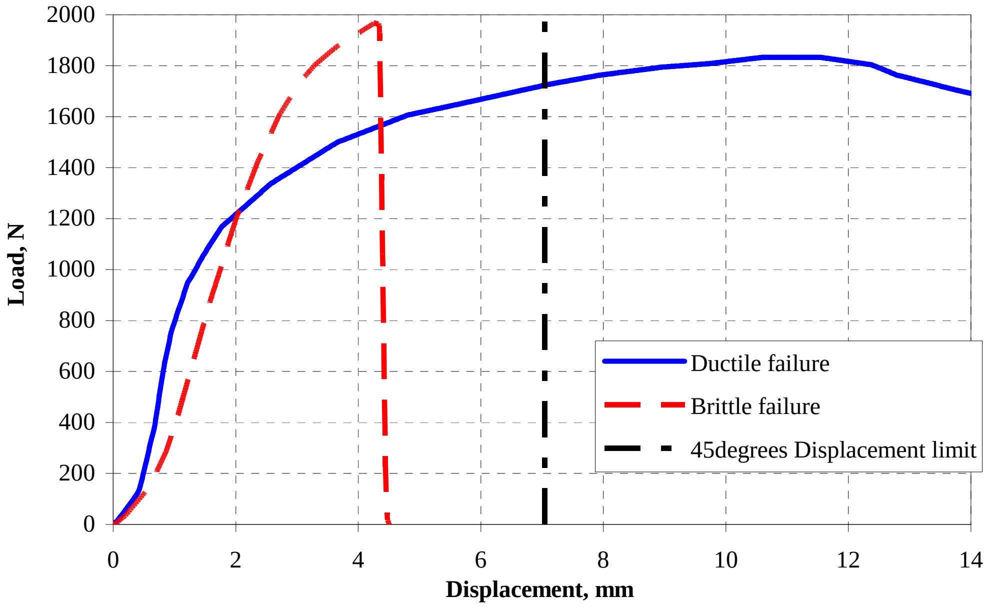


Figure 3.7: Typical four points bending test ductile and brittle failures, and 45° limit for 51mm span test set up.

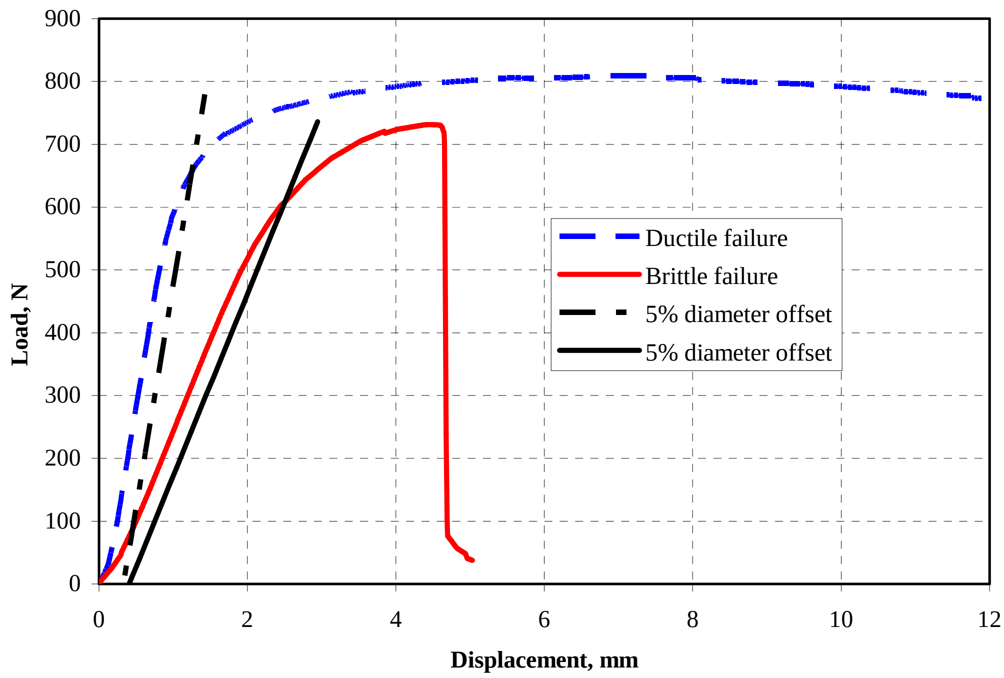


Figure 3.8: Typical three points bending test ductile and brittle failures

In both set of tests helically shaped fasteners behaved in a ductile manner, Figure 3.9. The loads used in the four points bending tests to determine the fasteners' yield moment were the load at the bent limit allowed by the different European standards. For the American standard the fasteners root diameter was used for determining the yield load for calculating the fasteners yield moment. Woodscrews behaved in a brittle manner under both sets of loading conditions, Figure 3.9.

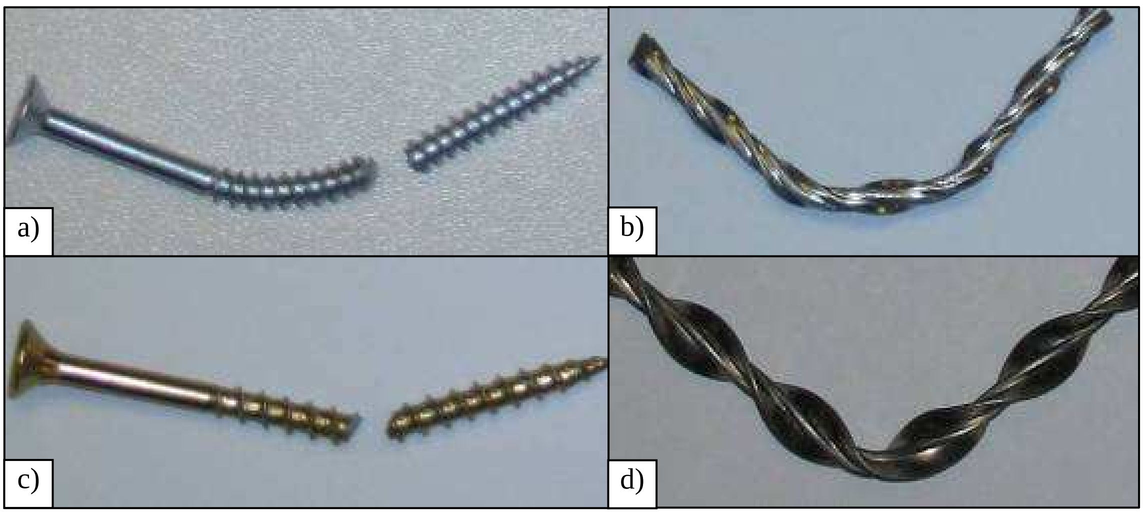


Figure 3.9: Bending tests a) four-point brittle failure; b) four-points ductile failure; c) three-point brittle failure and d) three-point ductile failure.

As mentioned above, differences between the two European and the American standards are significant, in terms of test set up, but also in terms of determining the load at which it can be considered that the fastener has yielded in order to determine the yield moment. Consequently, from the tests data the following yield moment values were determined for each fastener according to the different standards:

- **$M_{y,14592}$:** Yield moment determined according to BS EN 14592:2008 (BSI,2009a), where the angle α was limited for Helically shaped fasteners and screws to $\alpha = (45/d^{0.7})^o$, with d the fastener nominal diameter,
- **$M_{y,409}$:** Yield moment determined according to EN 409:2009 (BSI, 2009b), using the minimum of the ultimate load achieved during tests or load for $\alpha = 45^o$;
- **$M_{y,US}$:** Yield moment calculated from three points bending tests according to ASTM F 1575-01 (ASTM, 2001), using the root diameter of fasteners;
- **$M_{y,EC5}$:** Yield moment calculated from the design equations given in Eurocode 5 (BSI, 2004), and the fasteners ultimate tensile strength from Table 3.5.

$M_{y,EC5}$ was determined using Equation 3.2:

$$M_{y,EC5} = 0.3 \cdot f_u \cdot d^{2.6} \qquad \textbf{...(3.2)}$$

The results of the tests performed on the fasteners to determine their yield moment are given in Table 3.5. For Helically shaped fasteners the root diameter was used in the calculations for $M_{y,EC5}$ as using the thread diameter would lead to greatly overestimated results. For the screws and nails the requirements of Eurocode 5 and BS EN 14592 were observed – i.e. the effective diameter of screws taken as 1.1 × root diameter and nominal diameter used for nails. The results are illustrated in Figure 3.10.

Table 3.5: Yield moment tests results and EC5 calculations

Fastener	$M_{y,14592}$	$M_{y,409}$	$M_{y,US}$	$M_{y,EC5}$
StarTie 10	4939.2	16918.2	8519.0	13169.8
StarTie 8	5240.1	10044.1	4551.7	10519.0
InSkew	3774.5	8549.4	3806.3	8516.7
TimTie	3549.2	6470.5	2933.5	5900.6
RWN 4.50	15221.8	15221.8	10211.0	10935.5
RWN 6.00	34507.5	34507.5	13614.0	19256.7
HTN 3.10	6164.5	6164.5	3886.5	4889.6
ATN 3.75	9236.8	9236.8	5432.0	7563.1
ATN 5.00	17460.4	17460.4	10710.2	13354.7
UMW 5	7551.5	12407.5	8616.4	12021.5
UMW 6	12629.3	22613.9	14102.8	19237.2
BZPNo 10	6349.1	15612.0	5796.9	10428.9
BZPNo 12	11286.8	18930.0	11438.3	16540.6

Note: All values in N.mm

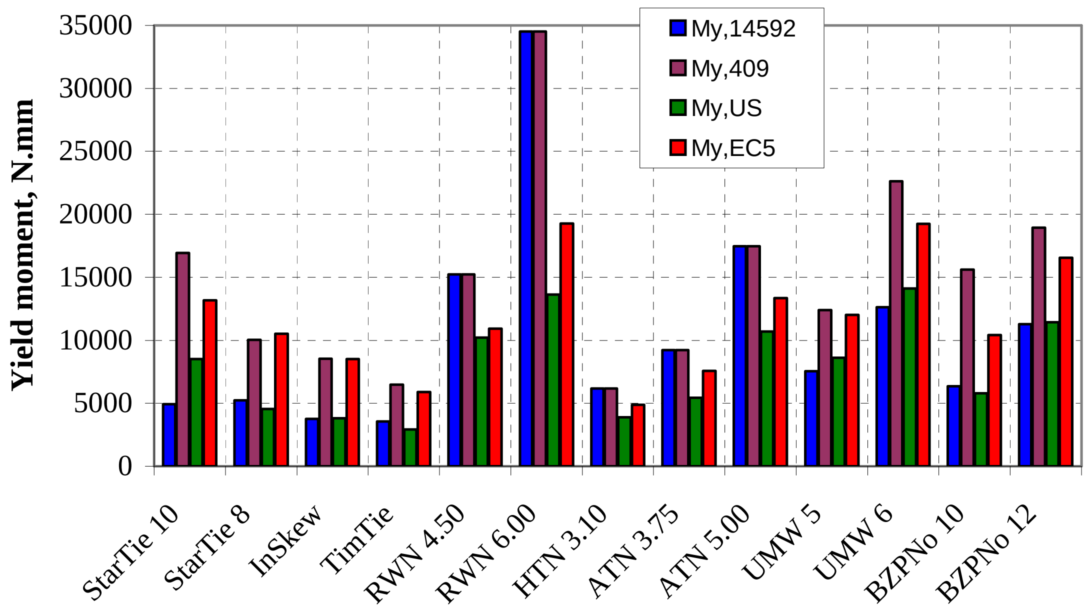


Figure 3.10: Yield moment results

From the results the following observations can be made:

- Using the limiting factor of $(45/d^{0.7})^{o}$ of the bent angle as mentioned in BS EN 14592:2008 (BSI, 2009a) greatly reduces the yield moment determined by tests, in comparison with the results determined using the maximum load achieved during testing or a limiting angle of 45º in accordance to BS EN 409:2009 (BSI, 2009b).

- Eurocode 5 design equations for nails and screws result in conservative values compared with results to BS EN 409.
- Eurocode 5 design equations overestimate yield moment values for screws and helically shaped fasteners compared to results to BS EN 14592.
- Using the root diameter in Eurocode 5 design equations for helically shaped fasteners gave results comparable or conservative depending on fastener diameter with results to BS EN 409, and greatly overestimated results compared to those calculated with BS EN 14592.
- Results obtained from the American method are comparable to those obtained to BS EN 14592 for screws.

The tests results clearly show that the use of the limiting factor for screws and helically shaped connectors would reduce greatly the yield moment values determined by tests for design purposes. Also, if this limiting factor is to be implemented there is a need for re-evaluating the design equations in Eurocode 5 as they would overestimate the yield moment of fasteners. In contrast, the equations of Eurocode 5 were derived from extensive test data, and they result in conservative values compared to test results when determined to BS EN 409 for all types of fasteners. The design equation from Eurocode 5 predicts the yield moment of Helically shaped fasteners with an average error of -16.4%, +4.3%, -9.2% and -10.1% for StarTie 10, StarTie 8, InSkew and TimTie respectively when tested to EN 409, with an absolute average difference between test results and predicted value from Eurocode 5 design equation of 10%.

3.3.3 Determination of Helically shaped yield moment

From previous research it was shown that the yield moment of fasteners is directly related to the fastener diameter and tensile strength, as Equation 3.2 shows (Hairstans, 2007). Figure 3.11 represents the relationship between Helically shaped root diameters and characteristic yield moment according to BS EN 409 ($M_{y,k,409}$) normalised to the average tensile strength, and Eurocode 5 ($M_{y,k,ec5}$) using Equation 3.2. The average characteristic tensile strength of helically shaped fasteners was found to be 957 N/mm^2. The characteristic values from tests and Equation 3.2 are given in Table 3.6, the prediction error is also calculated.

Table 3.6: *Characteristic values and prediction error of helically shaped fastener's yield moment*

Fastener	Root Diameter	Yield moment		Prediction
	d	My,k,409	My,k,ec5	error
	mm	N/mm^2	N/mm^2	%
StarTie 10	4.25	15874.7	12355.0	-22.17
StarTie 8	3.75	8522.3	8923.1	4.70
InSkew	3.35	6659.0	6655.0	-0.06
TimTie	3	4997.2	4995.1	-0.04

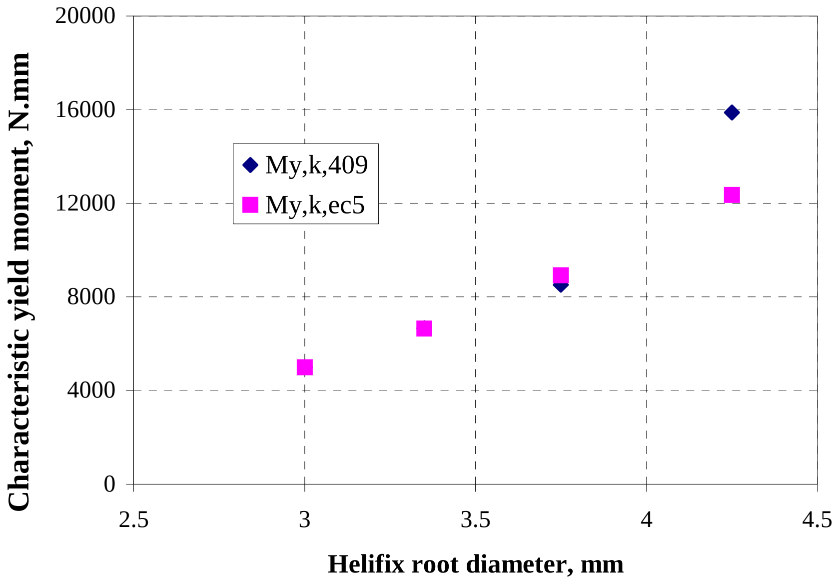


Figure 3.11: Relationship between Helically shaped root diameter and characteristic yield moments.

This shows that the design equation given in Eurocode 5 (BSI, 2004) predicts accurately the yield moment of helically shaped fasteners with smaller root diameters. For StarTie 10 fasteners Eurocode 5 underestimates the yield moment value by 22%. Such a digression can affect the results when using the equations given in Eurocode 5 when calculating the lateral shear strength of a connection. Even if this would yield conservative results, it would be beneficial for design purposes to predict the yield moment of helically shaped fasteners accurately.

To achieve this, two empirical equations types were developed for deriving accurate model for helically shaped fasteners, Equations 3.3 and 3.4. The former is of the form used for conventional timber fasteners, while the latter is a power function but taking into consideration the intercept value.

$$M_{y1} = f_u \cdot a \cdot d^b \qquad \textbf{...(3.3)}$$

$$M_{y2} = f_u \cdot x \cdot d^y + z \qquad \textbf{...(3.4)}$$

Where, a, b, x, y and z were constants determined using the function *Genfit* in the software MathCAD. Replacing the values equations 3.4 and 3.5 can be written as follows, the results of the two equations are detailed in Table 3.7:

$$M_{y1} = 0.066 \cdot f_u \cdot d^{3.688} \qquad \textbf{...(3.5)}$$

$$M_{y2} = 0.000114 \cdot f_u \cdot d^{7.87} + 4499 \qquad \textbf{...(3.6)}$$

Table 3.7: *Results of helically shaped fasteners yield moment models*

Fastener	$M_{y,k,409}$	$M_{y,k,ec5}$	Prediction error	Equation 3.6	Prediction error	Equation 3.7	Prediction error
	N/mm^2	N/mm^2	%	N/mm^2	%	N/mm^2	%
StarTie 10	15874.7	12355.0	-22.17	15423.1	-2.84	15843.9	-0.19
StarTie 8	8522.3	8923.1	4.70	9721.0	14.07	8735.7	2.50
InSkew	6659.0	6655.0	-0.06	6412.9	-3.70	6242.9	-6.25
TimTie	4997.2	4995.1	-0.04	4268.9	-14.57	5230.7	4.67

Table 3.7 shows that Equation 3.6 Yields to prediction error up to 14%, which is higher than would be recommended. Equation 3.7 is the best compromise between all the prediction equations, as the highest prediction error underestimates the yield moment by just over 6%.

3.4 Embedment strength

The embedment strength is not a fastener property but a system property as it depends on the type and shape of fastener, the joint geometry and the manufacturing process of the connection and the timber or wood based material properties (Ehlbeck, 1992). The test method described in BS EN 383:2007 (BSI, 2007) for determining the embedment strength of a fastener was developed after intensive work carried out at the Timber Research And Development Association (TRADA) in the UK by Rodd et al (1987). As this test method became accepted as a suitable mean to determine the embedding strength of a fastener, numerous studies were carried out collecting sufficient data to enable the development of design equations.

The aim of this part of the research was to determine and characterize the embedment strength of helically shaped fasteners and to evaluate the compatibility of the design equations given in Eurocode 5 for these fasteners.

3.4.1 Embedment tests set-up and procedures

The embedment tests on helically shaped fasteners and common timber fasteners were performed in accordance with BS EN 383:2007 – Timber Structures – Determination of embedding strength and foundation values for dowel type fasteners (BSI, 2007). The embedment test aims to determine the behaviour of the system fastener-timber under loading perpendicular to the fastener's axis. In order to have the best possible representation of this interaction bending of the fastener should be prevented. In order to avoid yielding of the fastener the standard recommends that "*the thickness of the timber should be in the range of 1.5d to 4d in order to comply with the principle of the test*".

As the test programme included a large variety of fasteners and to provide results that could be compared it was decided to perform all the tests using only one size of timber sample. Preliminary tests were performed on all fasteners to ensure that the principle of the test was respected – i.e. no bending of the fastener.

The samples were made of timber grade C24, in accordance to BS EN 338:2003 Structural Timber – Strength Classes (BSI, 2003), the dimensions were 140 x 50 mm. Following preliminary tests the thickness was determined to be 12mm. A "U frame" was screwed to the timber sample, supporting two Linear Variable Differential Transducers (LVDT) placed either side of the sample recording the displacement of the sample; this allowed the recording of the average displacement in case the sample rotated during the test. A 50kN load cell was used to record the load applied to the specimens, Figure 3.12. Pilot holes of 0.8 times the root diameter of profiled fasteners, and 0.8 times the actual diameter for round fasteners were drilled before the fasteners were inserted, Table 3.8.

To obtain comparable results between the range of fasteners, the rate of loading was kept constant, this was determined after the preliminary tests, as is shown in Figure 3.13.

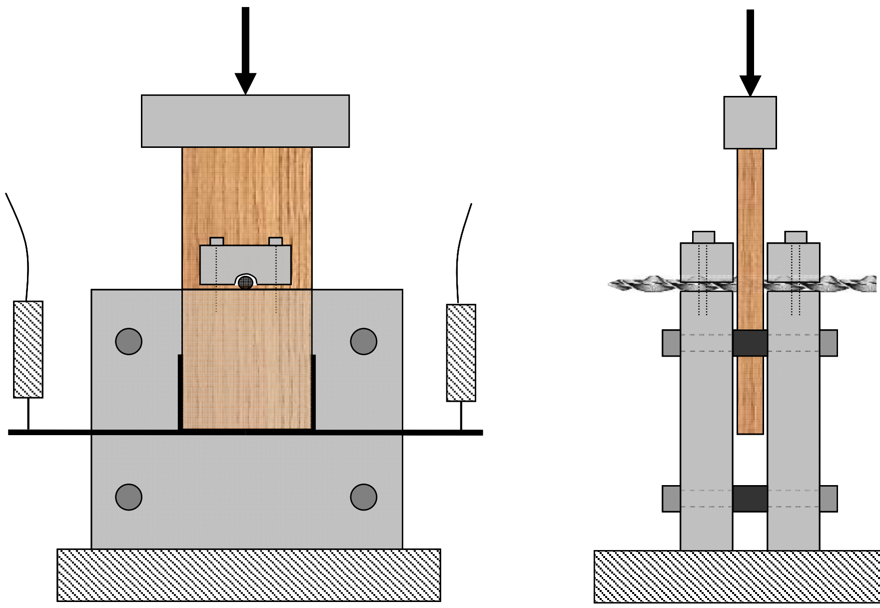


Figure 3.12: Embedment test set up.

Table 3.8: *Pilot hole sizes for the different fasteners.*

Fastener	Pilot hole diameter mm	Fastener	Pilot hole diameter mm
StarTie 10	3.50	ATN 3.75	2.80
StarTie 8	3.00	ATN 5.00	3.60
InSkew	2.80	UMW 5	2.80
TimTie	2.40	UMW 6	3.40
RWN 4.50	3.60	BZPNo 10	2.50
RWN 6.00	4.80	BZPNo 12	3.20
HTN 3.10	2.40		

For each set of fastener 5 specimens were tested, the moisture content and density of the samples were recorded.

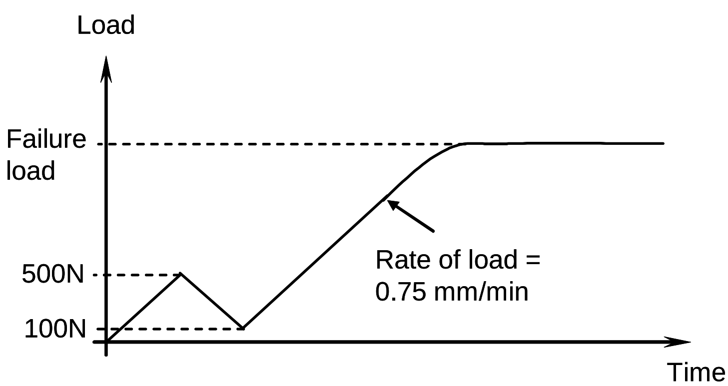


Figure 3.13: Rate of loading for embedment tests.

3.4.2 Tests results

As recommended in BS EN 383:2007 (BSI, 2007)), calibration tests were performed on the test rig in order to adjust the load displacement relationships accordingly to the set up stiffness. 3.75mm and 4.50mm diameter steel pins inserted in a steel specimen that was placed in the rig. The results of the calibration tests showed that the stiffness of the rig was of 13426 N/mm and 12800 N/mm for the steel pins of diameter 3.75 and 4.5mm respectively. The results show that there was no significant effect from the fastener diameter on the rig stiffness. Therefore an average value of 13113 N/mm was used to represent the stiffness of the rig, to obtain the corrected load displacement curves of the fasteners embedment tests. Typical load displacement curves from tests for helically shaped fasteners and woodscrews along with the corrected curves due to the rig stiffness are shown in Figure 3.14.

The load displacement curves show that woodscrews exhibit a more elastic and stiffer behaviour than helically shaped fasteners under similar loading conditions. For screws the load increases linearly with the increasing deflection until yielding, and then the load decreases slowly with increased displacement. For Helically shaped fasteners the linear part of the load displacement relationship is much shorter, then the load increase with increase in displacement in a non linear manner until a maximum is reached, at which point the load decreases with increasing deformation.

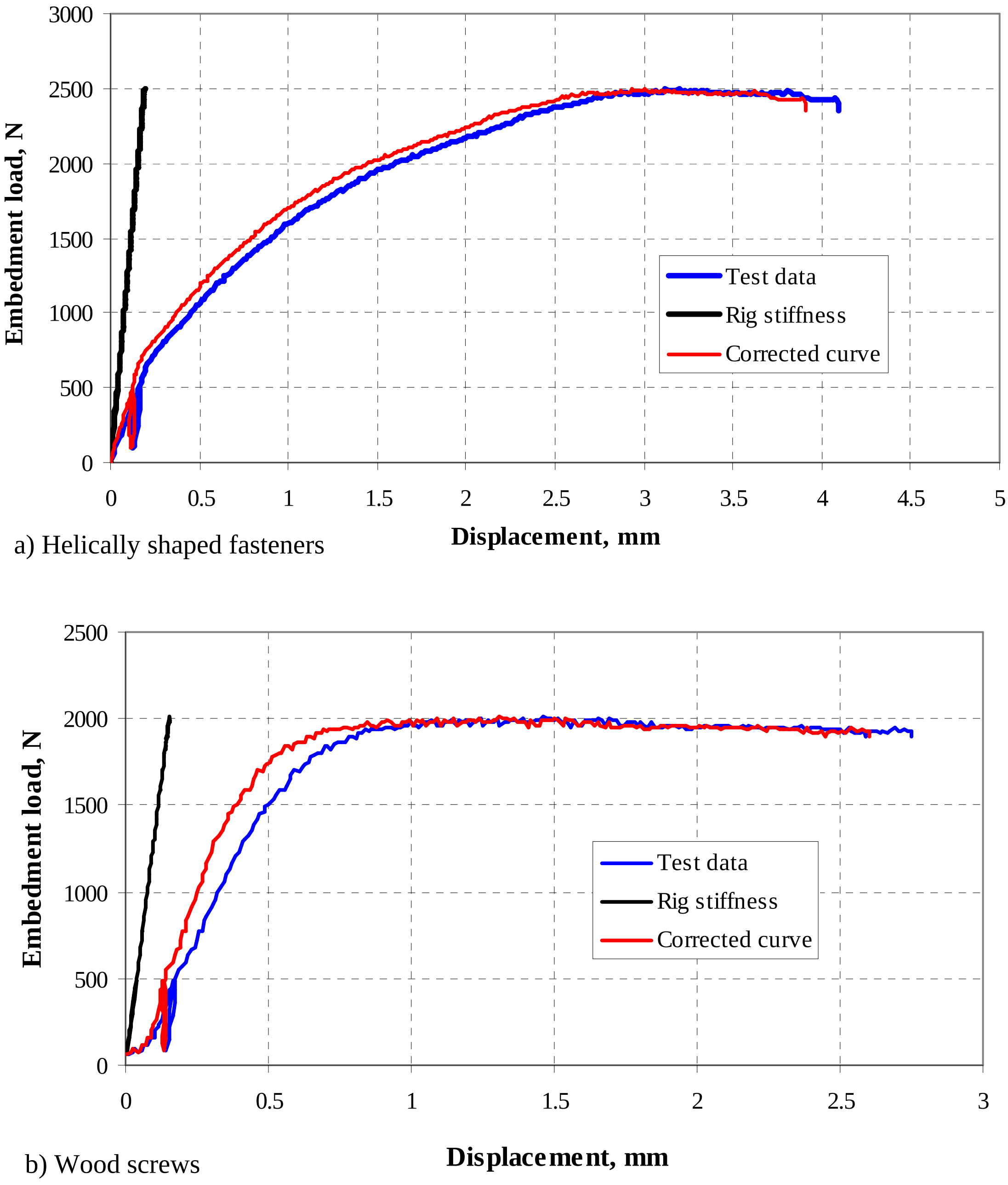


Figure 3.14: Typical load displacement curves from embedment test results and corrected curves for a) Helically shaped fasteners, and b) Woodscrews.

The embedment strength of the fasteners was calculated according to the following equation given in BS EN 383:2007 (BSI, 2007).

$$f_{h,EN383} = \frac{F_{\max}}{d \cdot t} \qquad \text{...(3.7)}$$

Where F_{max} is the maximum load recorded during test (in N), d is the fastener diameter (in mm), for profiled fasteners BS EN 383 recommends that the shank diameter is used, and t is the thickness of the timber sample (in mm).

Eurocode 5 (BSI, 2004) allows the calculation of the characteristic embedment strength of fasteners with predrilled holes using the following equation:

$$f_{h,EC5} = 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k \qquad \textbf{...(3.8)}$$

Where ρ_k is the characteristic timber density – for timber grade C24 ρ_k = 350kg/m^3 – and d is the fastener diameter, for screws this is taken as 1.1 × root diameter, and for nails as the nominal diameter according to BS EN 14592:2008 (BSI, 2009a).

The results of the tests and the design values according to Eurocode 5 with the corresponding diameters used in the calculations are detailed in Table 3.9 and shown in Figure 3.15.

Table 3.9: *Embedment tests results and Eurocode 5 design values*

Fastener	**d_{383}**	**$f_{h,EN383}$**	**$d_{E.C.5}$**	**$f_{h,EC5}$**
	mm	**N/mm^2**	**mm**	**N/mm^2**
StarTie 10	10.00	19.08	10.00	25.83
StarTie 8	8.00	22.09	8.00	26.40
InSkew	6.00	30.09	6.00	26.98
TimTie	4.50	30.97	4.50	27.41
RWN 4.50	4.50	27.50	4.50	27.41
RWN 6.00	6.00	26.66	6.00	26.98
HTN 3.10	3.20	42.74	3.10	27.81
ATN 3.75	4.20	34.94	3.75	27.62
ATN 5.00	5.60	29.13	5.00	27.27
UMW 5	4.90	24.43	3.85	27.60
UMW 6	5.90	26.06	4.62	27.37
BZPNo 10	4.90	32.41	3.52	27.69
BZPNo 12	5.50	33.79	4.29	27.47

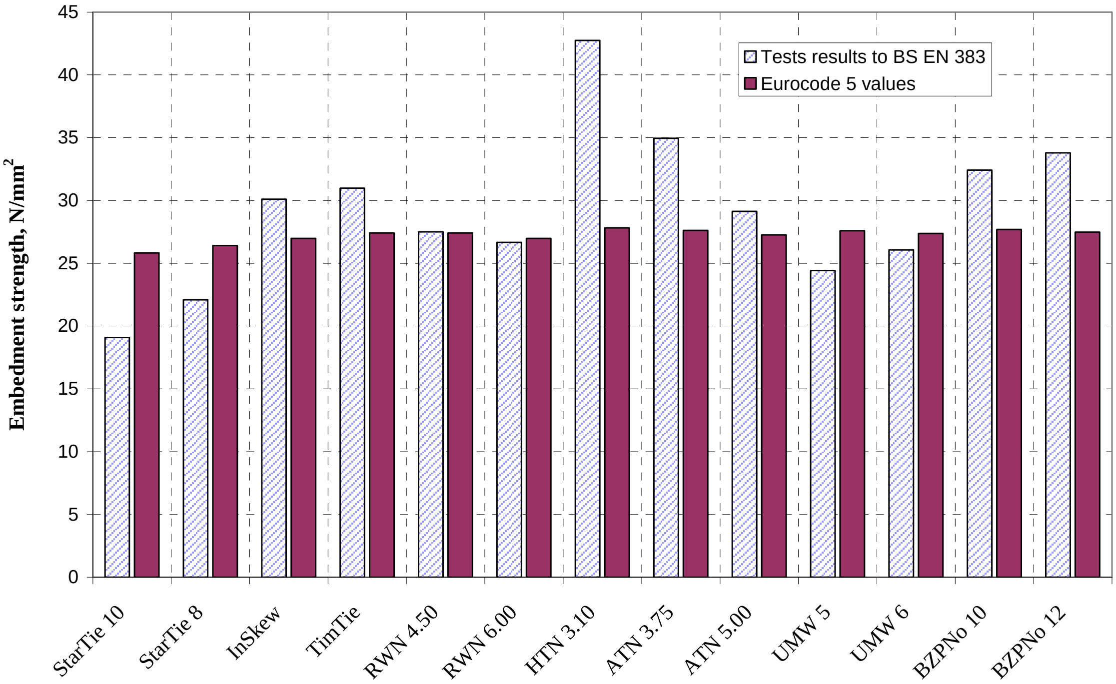


Figure 3.15: Embedment tests results to BS EN 383 and Eurocode 5 design values

The results show that for screws and nails the embedding strength is decreasing with an increase of the fastener diameter, which is also the case for helically shaped fasteners. This relationship between diameter and embedment strength is widely accepted as being true for dowel type fasteners, and the results therefore show that helically shaped fastener behave as common timber fasteners.

However Figure 3.15 also shows that the design equation form Eurocode 5 does not result in conservative values for all the fasteners. While in the case of woodscrews, this is due to using different diameters in Equations 3.8 and 3.9, which results in test values being lower than design values, the same cannot be said for helically shaped fasteners.

In addition, it can be noted that despite the variety of fasteners tested in the study the characteristic embedment values calculated using Equation 3.9 range between 25.8 and 27.8 N/mm^2; while embedment tests values range from 19.1 to 42.7 N/mm^2. The design equation originally developed for round fastener does not give a true representation of the embedment behaviour of helically shaped fasteners; consequently a new design relationship is necessary, as the existing equation from Eurocode 5 overestimates the embedment strength of helically shaped fasteners by about 25% for large diameters.

3.4.3 Representation of Helically shaped fasteners embedment strength

Ehlbeck & Werner (1992) tested various round fasteners in different timber species and concluded that the embedding strength may be assumed to increase linearly with increasing timber density. In order to verify the validity of this assumption for helically shaped fasteners, the maximum load achieved during testing has been divided by the average load for each size of fastener, and plotted against the corresponding timber density measured from the tests sample, Figure 3.16.

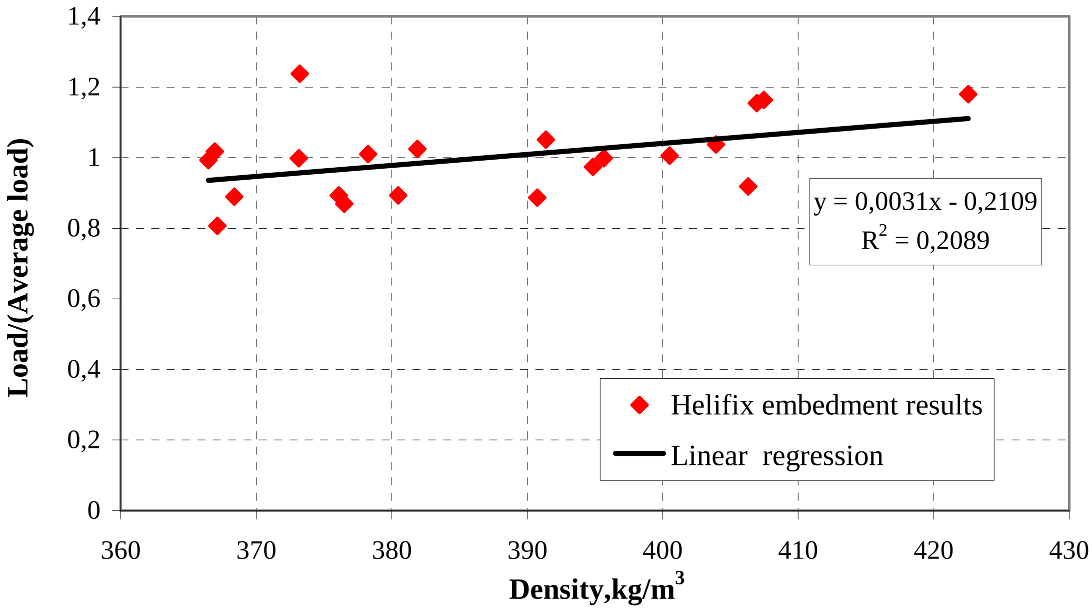


Figure 3.16: Influence of timber density on helically shaped fasteners embedment strength

The results show that, independent of the fastener diameter, the embedment strength of helically shaped fasteners increase with increase in timber density. As the embedment strength is directly proportional to the timber density, to determine the effect of fastener diameter, the characteristic embedment strength divided by the timber density was plotted against helically shaped thread diameters, Figure 3.17.

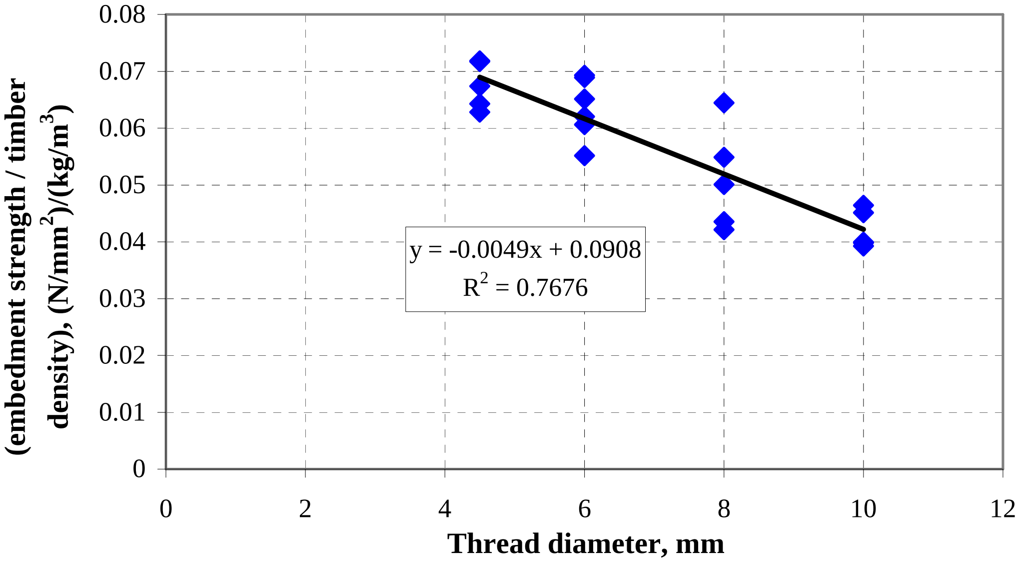


Figure 3.17: Influence of fastener diameter on helically shaped fasteners embedment strength

From Figures 3.16 and 3.17 the following relationship can be derived to determine the characteristic embedment strength of helically shaped fasteners in terms of thread diameter and timber density:

$$f_{helifix} = (-0.0049 \cdot d_t + 0.0908)\rho \qquad \textbf{...(3.9)}$$

In Table 3.10 the characteristic values determined using Equation 3.9 from Eurocode 5, from testing in accordance to BS EN 383 and using Equation 3.10 are shown. The prediction error from equations 3.9 and 3.10 are also given.

Table 3.10: *Prediction of Helically shaped fasteners embedment strength*

Fastener	d_{shank}	$f_{h,EC5}$	$f_{h,k,EN383}$	**Error**	$f_{helifix}$	**Error**
	mm	**N/mm^2**	**N/mm^2**	**%**	**N/mm^2**	**%**
StarTie 10	10	25.83	16.25	-37.09	16.18	-0.46
StarTie 8	8	26.40	19.25	-27.09	19.45	1.04
InSkew	6	26.98	25.06	-7.11	24.19	-3.46
TimTie	4.5	27.41	26.22	-4.33	26.61	1.47

The results given in Table 3.10 show that helically shaped fasteners' embedment strength can be predicted using Equation 3.10. The use of the design equation given in Eurocode 5 would result in greatly overestimated values, especially for large diameters.

3.5 Summary and conclusions

This part of the study focused on the mechanical properties of helically shaped fasteners that influence the resistance and behaviour of timber connections. The properties investigated were the tensile strength, the yield moment and embedment strength.

The tensile strength tests performed showed that helically shaped fasteners were in line with the recommendations of Eurocode 5 where the fasteners need to show a minimum tensile strength of 600 N/mm². All other fasteners tested also respected the minimum criterion except round nails made from steel wire.

The yield moment of timber fasteners, as a critical factor in timber joint resistance, is to be determined by testing according to BS EN 409:2009. The principle of the test method described involves the loading of the fastener in such a manner that "*the loading points do not move along the nail and the loads remain normal to the axis of the fastener during the test.*" The review of various studies showed that determining the yield moment accurately is critical, however manufacturing a test rig and performing yield moment tests which respect the test principle as mentioned above is a great difficulty. To overcome the difficulty, the American Society for Testing Materials recommends that the yield moment of fasteners should be tested in a standard three point bending test. Due to the uncertainty of the test method, and the inability to perform in house a test which would respect the test principle defined in BS EN 409, the fasteners used in this study were tested on two occasions: four points and three points standards bending tests – i.e. with the loading points remaining vertical during the tests. In addition to the complexity of the test set up, BS EN 14592 introduced the notion of angle limit of the test in order to evaluate the yield moment of the fasteners at angles which could be witnessed in practise as opposed to the standard 45° limit of BS EN 409. This evolution shows that the issue of yield moment is critical, but difficult to appreciate. In both tests helically shaped fasteners exhibited a very ductile behaviour mainly due to their high slenderness ratio, while most profiled fasteners failed in a brittle manner.

The results of the tests performed on helically shaped fasteners exhibit a very ductile behaviour when subjected to either three or four points loading bending tests. However the design method as recommended in Eurocode 5 did not predict accurately helically shaped fasteners characteristic yield moment, especially the larger diameter fasteners.

Therefore, a specific design equation for calculating the yield moment of helically shaped fasteners was developed. It was found to predict the yield moment to an average error of 6%.

The embedment strength and behaviour of helically shaped fasteners was investigated in accordance with BS EN 383:2007 (BSI, 2007). The results showed that helically shaped fasteners achieved similar embedment strength to common timber fasteners; however they exhibited less stiff behaviour than common fasteners. In addition the results showed that, as for nails and screws, the embedment strength of helically shaped fasteners decreases with increasing fastener diameter.

The results were then compared to the design method of Eurocode 5. It was found that the design equation, which was developed for common nails, did not accurately predict the embedment strength of helically shaped fasteners; hence a specific design equation was developed. The embedment strength of helically shaped fasteners was shown to be directly proportional to the timber density and fastener diameter. Therefore, a design equation was developed including these two parameters and it was found to predict the embedment strength of helically shaped fasteners with an average error of 2%.

Chapter 4 Axially loaded helically shaped fasteners in timber

4.1 Introduction

Threaded fasteners were originally developed to provide increased resistance to loads applied parallel to their axis, as common round wire nails only resist relatively important withdrawal forces when the load is applied soon after driving. The withdrawal capacity of smooth nails is a function of the friction between the timber and shank of the nail. Helically shaped fasteners were created to offer increased bond between the cement or concrete, and are now also used as cavity wall ties in timber frame structures.

As wall ties, helically shaped fasteners often resist tension loads applied parallel to their axis, as a link between the timber frame and masonry leaf. However, their direct withdrawal performance in structural timber compared to common timber nails is not known. In this chapter the withdrawal performance and behaviour of helically shaped compared to conventional timber nails and screws are investigated. The tests results are analysed with the design equations form Eurocode 5 (BSI, 2004).

When Eurocode 5 was being developed, the withdrawal resistance of the fastener and its contribution to the load carrying capacity of a connection was first overlooked, however research has shown that a fastener with greater withdrawal capacity exhibited an increase of the lateral shear capacity of a joint. Since an allowance has been added to Eurocode 5 for the effect of pull out capacity in the design calculations of timber connections.

In addition, the chapter reports on the investigation of parameters influencing the load displacement characteristics and ultimate strength of helically shaped fasteners when subjected to axial loads in timber. From this experimental programme a semi empirical model is developed for simulating and predicting helically shaped fasteners withdrawal behaviour. The analysis considered the effects of the timber, and the installation of the fasteners in the timber.

4.2 Withdrawal of Helically shaped fasteners and timber fasteners

4.2.1 Tests set up and procedures

The withdrawal capacity of helically shaped and common timber fasteners was determined in accordance with BS EN 1382:1999 – Timber structures – Test methods – Withdrawal capacity of timber fasteners (BSI, 1999). The fasteners used in this research are shown in Figure 3.1 and their characteristics given in Table 3.1.

Preliminary tests were performed with helically shaped fastener and common fasteners to determine the rate of loading during the tests. The rate of loading should be constant and such that the time taken to reach the maximum load is 90 ± 30seconds (BSI, 1999). Due to the diversity of fasteners tested the results of the preliminary tests showed that two rates of loading were necessary to comply with the test procedure described in BS EN 1382 (BSI, 1999). The nail and screws needed tested at a rate of loading of 1.0mm per minute and helically shaped fasteners needed tested at a rate of loading of 4.0 mm per minute. As the difference between the two rates of loading required by the standard was such, tests with helically shaped fasteners tested at a rate of 1.0mm per minute were performed. The results of these tests showed that the load displacement characteristics and withdrawal resistance was not influenced by the rate of loading. Therefore the rates of loading complying with BS EN 1382 (BSI, 1999) were used during the experimental programme – 1.0mm/min for common fasteners and 4.0mm/min for helically shaped fasteners.

In addition, it is recommended in BS EN 1382 (BSI, 1999) that the fasteners are tested in the direction parallel and perpendicular to the grain, as this has a significant influence on the withdrawal strength. When tested in solid timber, the tests perpendicular to the grain half of the fasteners should be inserted radially to the growth rings and half tangentially. Preliminary tests were performed on the fasteners inserted radially and tangentially perpendicular to the grain. The results of the preliminary withdrawal tests showed that the perpendicular direction from which the nails are inserted into the timber did not have a significant influence on the withdrawal capacity of the fastener. Therefore the decision was taken to ignore the direction of the fibres (radial or tangential) for the tests perpendicular to the grain. The same results were found with helically shaped fasteners.

Five specimens were tested for each fastener to determine their withdrawal strength. A preload of 100N was applied before the tests to eliminate the initial slack in the loading system. The nails and screws were tested using a steel sleeve placed around the fastener's head and attached to the testing machine, as shown in Figure 4.1.

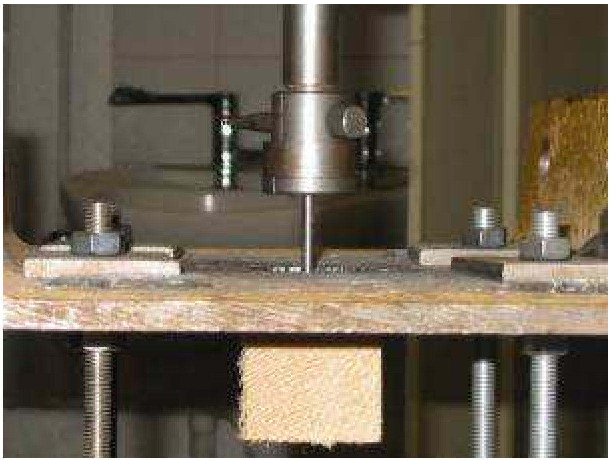


Figure 4.1 Withdrawal test set up for common timber fasteners.

Because Helically shaped fasteners do not have a head and could not be pulled directly by the travelling head of the testing machine as the common timber fasteners, the fasteners were driven into two pieces of timber and were pulled apart as shown on Figure 4.2 and illustrated in Figure 4.3 This method also prevented the fasteners from unscrewing.

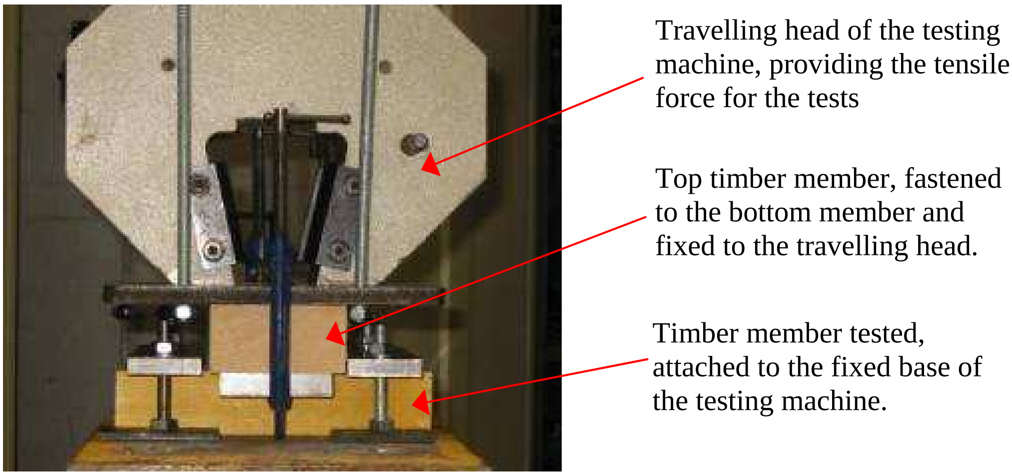


Figure 4.2: Withdrawal test set up for helically shaped fasteners

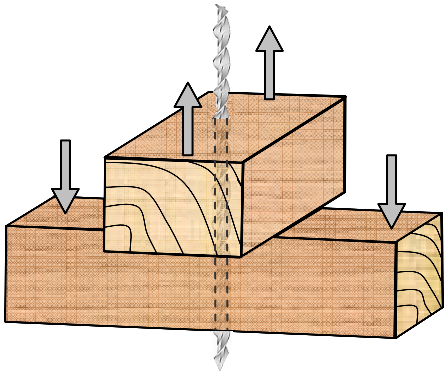


Figure 4.3: Withdrawal test specimen with helically shaped fastener

The samples were fabricated, as recommended by the European standard, representing site practices; pilot holes of 0.8 times the diameter of the fasteners were drilled before the fasteners were inserted into the timber. The pilot holes for withdrawal tests are similar to those for embedment tests, details are given in Table 3.8. All nails were manually hammered, and the screws inserted with an electrical drill. Helically shaped fasteners were hammered into the timber using a hand-held tool acting as a sleeve and transmitting the impact force. This tool was provided by Helifix Ltd, and is used for standard installation into masonry or timber. It also offers the advantage of restraining the free length of the fastener and prevents bending that might occur when using a hammer alone for inserting helically shaped fasteners.

The timber used in the tests was stored for a period of two months before the tests to achieve constant moisture content. Samples were cut, and clear specimens chosen for the tests, however within a specimen, small knots and variation in the slope of the timber fibres were permitted provided they were unlikely to significantly reduce the specimen strength, or have any influence on the test behaviour or result. The samples were fabricated and tested within one hour.

4.2.2 Modes of failure

The tests results show that when subjected to load parallel to their axis, timber fasteners behave differently depending on the geometry of the fastener shank. Typical load displacement relationships for each fastener tested are shown in Figure 4.4. While Figure 4.5 shows details the load displacement relationships of the different types of fasteners.

The results show that conventional timber fasteners and helically shaped fasteners exhibit different load-displacement characteristics. Helically shaped fasteners show a much more ductile behaviour, starting with a sharp increase in the load until the fastener starts to yield, followed by a less steep but steady increase in load. Once the maximum load is attained the decrease in load is slow and steady.

Screws and annularly threaded nails displayed similar withdrawal behaviour – however woodscrews show greater stiffness. The load increases at a steady rate until maximum load is attained, between 2 and 4mm displacement, at a point which the load decreases rapidly with increase displacement.

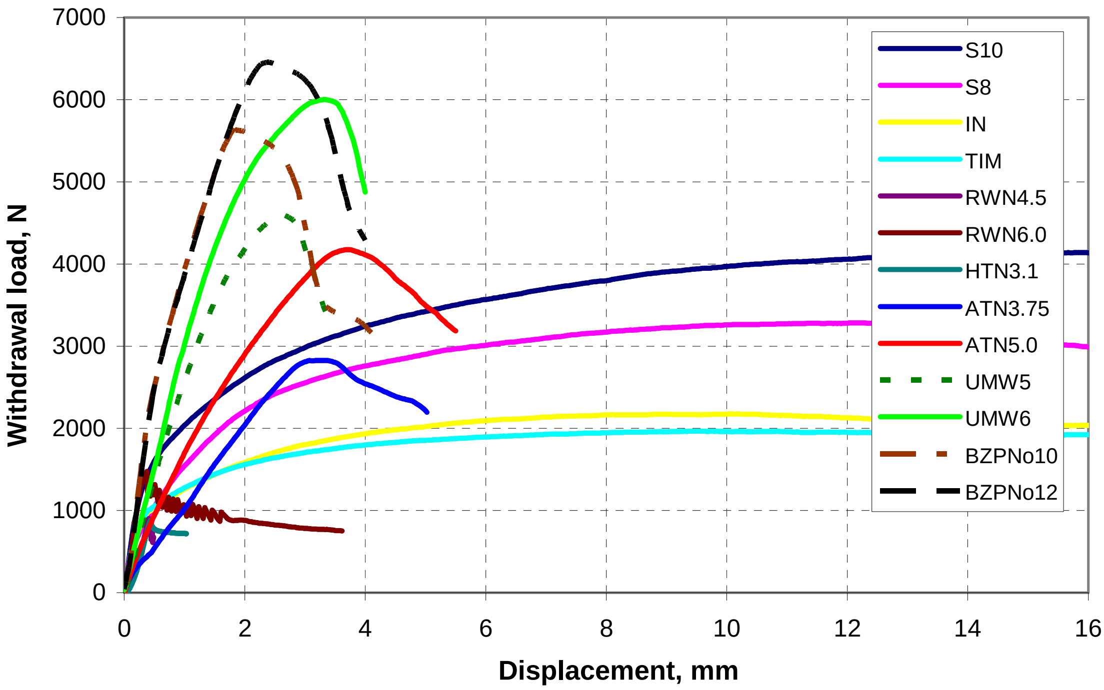


Figure 4.4: Typical withdrawal behaviour of timber fasteners.

Round and helically threaded nails exhibited similar behaviour up to failure; where the load increases sharply until the maximum load is attained which results in a brittle failure as the load decreases sharply. After this sharp decrease, in the case of round wire nails, the load increases again until the friction between the timber and shank of the nails is overcome and the nail is slowly pulled out of the timber. As the contact area between timber and nails shank reduces the load required for overcoming the friction is reduced. On the other hand, for helically threaded nail, after a sharp decrease the load decreases at a slow and constant rate.

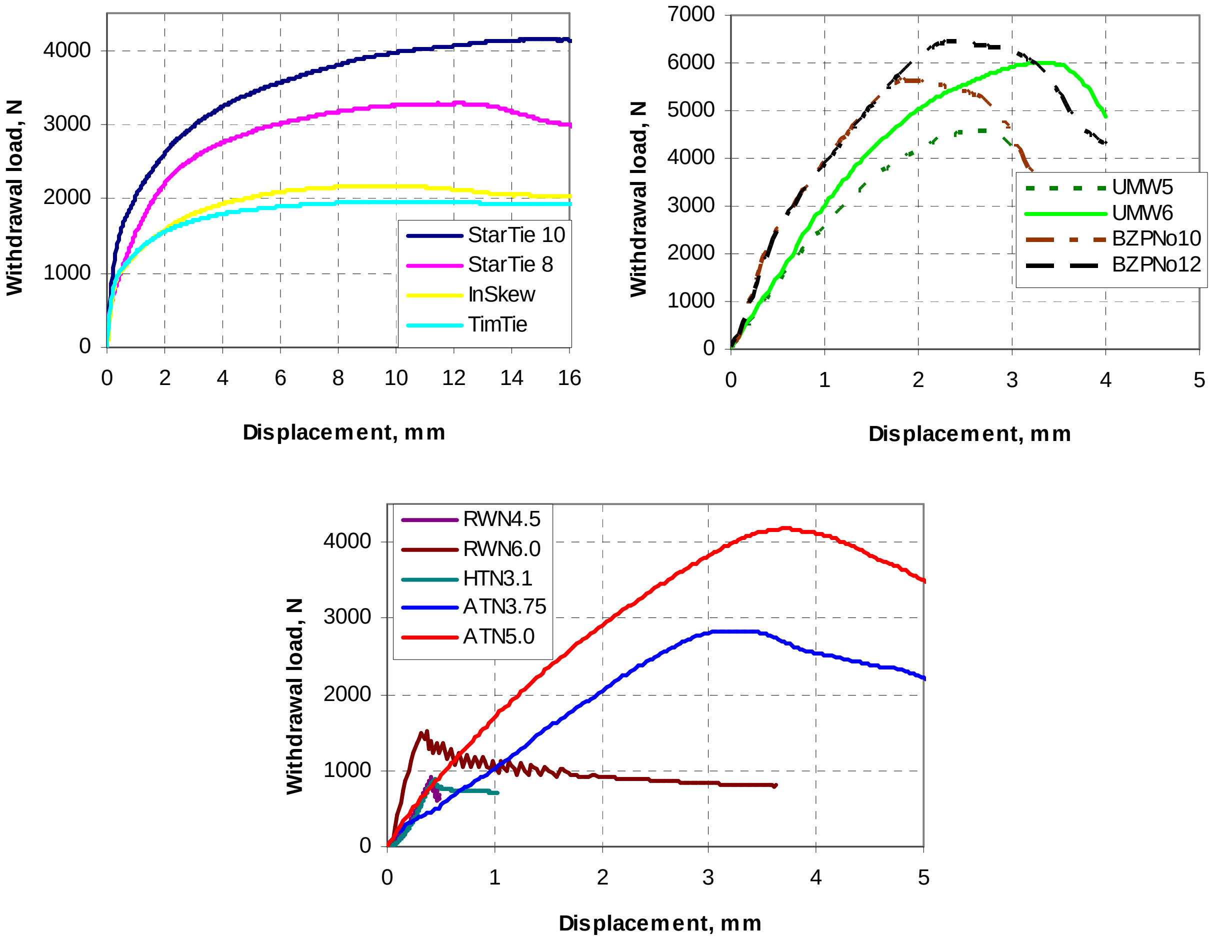


Figure 4.5: Withdrawal behaviour of a) Helically shaped fasteners, b) Woodscrews, and c) Nails.

While for conventional fasteners, withdrawal tests did not have any effect on their shape or geometry; after testing helically shaped fasteners presented flattened helixes when tested perpendicular to the grain. As the test set up included two pieces of timber maintained in the testing machine, with increasing displacement between the timber members the fastener require to rotate in the opposite direction to its helixes as it is pulled out. This phenomenon is shown in Figure 4.6 (b). When tested in end grain helically shaped fasteners did not unwind, their geometry was not altered by the forces acting in the test setup; the fibres caught in the helixes of the fasteners were sheared off as the fasteners were pulled out of the samples Figure 4.6 (c).

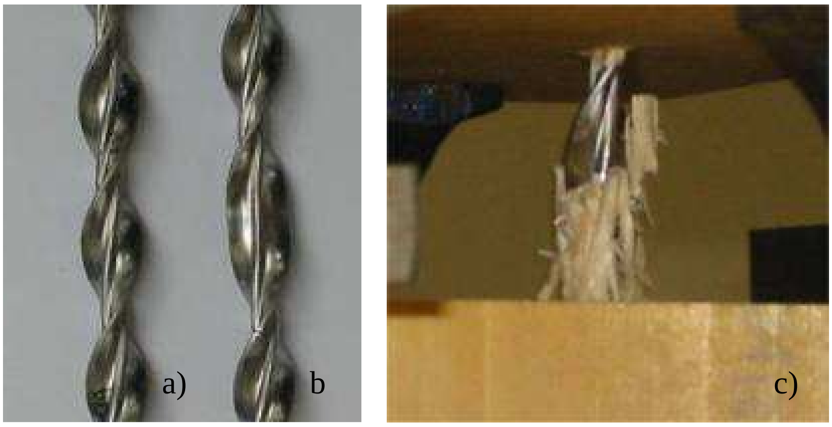


Figure 4.6: a) Helically shaped fastener; b) failure perpendicular to the timber grain; c) failure parallel to the timber grain.

4.2.3 Results

The withdrawal strength, or parameter, of a fastener is defined in BS EN 1382 (BSI, 1999) as the "*parameter measuring the resistance of a timber piece to the withdrawal of a timber fastener*", it is determined as follows:

$$f_{ax} = \frac{F_{\max}}{d \cdot l_p} \qquad \textbf{...(4.1)}$$

Where f_{ax} is the withdrawal parameter, N/mm^2,
F_{max} the maximum load achieved during testing, N
d the fastener nominal diameter, mm
l_p the depth of penetration in the timber, mm.

Contained in Table 4.1 are the withdrawal loads, and withdrawal strengths, as calculated using Equation 4.1, and withdrawal stiffness achieved for each fastener.

Table 4.1: *Withdrawal loads and strength for different timber fasteners.*

Fastener	Withdrawal load	Nominal diameter	Depth of penetration	Withdrawal strength	Withdrawal stiffness
	N	**mm**	**mm**	**N/mm^2**	**N/mm**
StarTie 10	4268.51	10.00	44.81	9.53	4344.16
StarTie 8	3111.50	8.00	44.34	8.77	4576.17
InSkew	2147.40	6.00	44.16	8.11	3109.09
TimTie	2081.33	4.50	44.73	10.34	4703.70
RWN 4.50	782.45	4.50	44.53	3.90	2051.89
RWN 6.00	2044.12	6.00	43.68	7.80	3456.56
HTN 3.10	831.93	3.10	44.59	6.02	1534.30
ATN 3.75	2769.94	3.75	44.09	16.75	1315.44
ATN 5.00	3697.17	5.00	43.88	16.85	1376.85
UMW 5	4932.93	4.90	43.67	23.05	2519.62
UMW 6	6028.94	5.90	43.66	23.40	2738.68
BZPNo 10	5811.75	4.90	43.72	27.13	4986.10
BZPNo 12	5993.80	5.50	43.84	24.86	4428.75

Table 4.1 shows that helically shaped fasteners achieve higher withdrawal loads than most common timber nails; while the maximum withdrawal loads were attained by wood screws. As it can be predicted, larger diameter fasteners tend to achieve higher loads than similar fasteners of smaller diameter. Also it can be noticed that nails with deformed shank perform much better in direct pull out than nails with smooth shank; helically threaded nails performed similarly to round wire nails of 4.50mm diameter despite a cross sectional area smaller by 50%. It can also be noticed that helically shaped fasteners have similar withdrawal stiffness to woodscrews achieving the highest withdrawal loads, and higher stiffness to all nails.

However, the withdrawal strength of helically shaped fasteners, calculated using Equation (4.1), shows that annularly threaded nails result in higher strength despite attaining lower withdrawal loads. The tests showed that for similar fasteners with different diameters – e.g. annularly threaded nails – that larger diameter fasteners achieve higher loads but have a similar withdrawal strength and stiffness. This tends to show that while the equation given in BS EN 1382 (BSI, 1999) is valid for common timber fasteners which generally have a circular cross section; however this equation does not represent the performance of Helically shaped fasteners accurately.

Equation (4.1), used to determine the withdrawal strength, does not effectively consider the friction between the timber and the helically shaped fasteners as their shape is not of a circular form. In this regard, using Equation (4.1) for helically shaped fasteners leads to an underestimation of the total surface area of the fastener in contact with the timber, and in turns to an underestimation of their withdrawal performances.

4.2.4 Eurocode 5 design equations

The withdrawal strength of round wire nails and screws can be determined in Eurocode 5 (BSI, 2004) using the following equations:

For smooth nails:

$$F_{ax,Rk} = (20\times10^{-6} \cdot \rho_k^2) \cdot d \cdot t_{pen} \qquad \textbf{...(4.2)}$$

Where ρk is the timber characteristic density, in kg/m^3, d is the nominal fastener diameter according to BS EN 14592:2008 (BSI, 2009a), in mm and t_{pen} is the fastener pointside penetration length, in mm.

For screws:

$$F_{ax,\alpha,RK} = n_{ef}\,(\pi \cdot d_t \cdot l_{ef})^{0.8} \cdot f_{ax,\alpha,k} \qquad \textbf{...(4.3)}$$

With $$f_{ax,\alpha,k} = \frac{3.6\times10^{-3} \cdot \rho_k^{1.5}}{\sin^2\alpha + 1.5\cdot\cos^2} \qquad \textbf{...(4.4)}$$

Where $f_{ax,\alpha,k}$ is the characteristic withdrawal strength at an angle α to the grain, n_{ef} the effective number of screws, d_t is the outer diameter measured on the threaded part (in mm), l_{ef} is the pointside penetration length of the threaded part minus one screw diameter (in mm).

The characteristic withdrawal strength perpendicular to the timber grain, according to Eurocode 5 (BSI, 2004) equations detailed above, was calculated for the fasteners used in this study, assuming a characteristic density for timber class C24 of 350 kg/m^3. The results are detailed in Table 4.2, and shown in Figure 4.7; along with the characteristic withdrawal strength derived from the tests performed according to BS EN 1382 (BSI, 1999). For Helically shaped fasteners, two values were determined according to Equation (4.2) and Equation (4.3).

Table 4.2: *Characteristic withdrawal loads*

Fastener	F_{test}	d or d_t	t_{pen} or l_{ef}	F_{EC5}	
				Eq. (4.2)	Eq. (4.3)
	N	mm	mm	N	N
StarTie 10	3628.23	10.00	44.81	1097.72	7783.69
StarTie 8	2644.78	8.00	44.34	868.97	6456.45
InSkew	1825.29	6.00	44.16	649.08	5112.45
TimTie	1769.13	4.50	44.73	493.18	4103.86
RWN 4.50	665.08	4.50	44.53	490.94	N/A
RWN 6.00	1737.51	6.00	43.68	642.13	N/A
HTN 3.10	707.14	3.10	44.59	338.66	N/A
ATN 3.75	2354.45	3.75	44.09	405.08	N/A
ATN 5.00	3142.59	5.00	43.88	537.48	N/A
UMW 5	4192.99	4.90	38.77	N/A	3918.12
UMW 6	5124.60	5.90	37.76	N/A	4451.11
BZPNo 10	4939.99	4.90	38.82	N/A	3921.84
BZPNo 12	5094.73	5.50	38.34	N/A	4259.10

F_{test} : Characteristic withdrawal load from tests according to BS EN 1382

F_{EC5} : Characteristic withdrawal load calculated to EC5

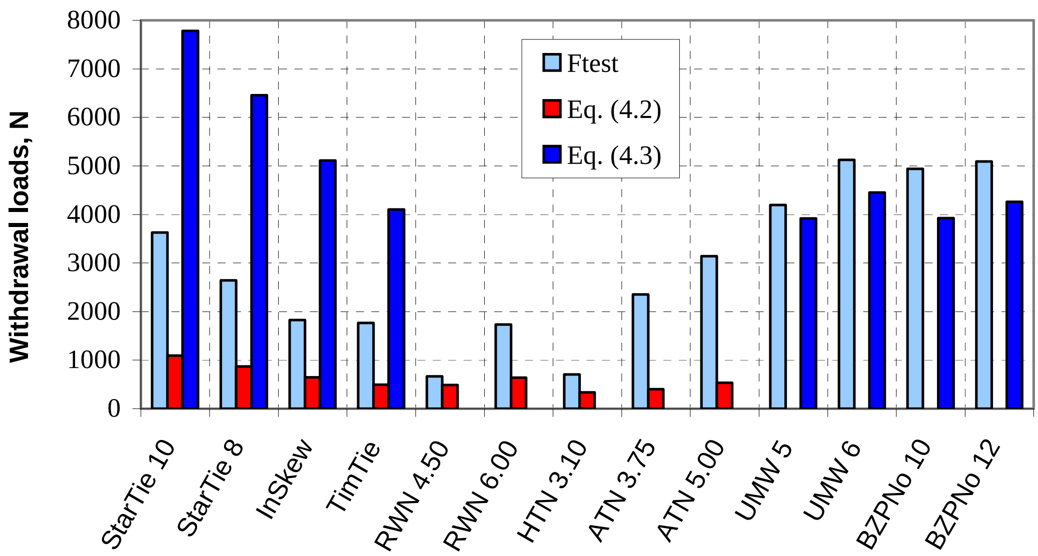


Figure 4.7: Characteristic withdrawal values from tests and EC5

The results show that for conventional fasteners, the equations given in Eurocode 5 result in conservative values for screws and small diameter nails, and over conservative values for large diameter round nails and annularly threaded nails, compared to the results from tests according to BS EN 1382 (BSI, 1999). For Helically shaped fasteners, Equation (4.2) resulted in conservative values with an average prediction error of 68%; while Equation (4.3) resulted in overestimated values with an average prediction error of 142%.

This shows that the design equations provided in Eurocode 5 are not suitable for helically shaped fasteners. It can also be argued that Equation (4.2) is suitable for small diameter nails only as it yields over conservative results for large diameter round nails ($d \geq 6mm$) and annularly threaded nails, with prediction errors of 68% and 82% respectively.

4.3 Extended experimental programme

As mentioned in Section 2.4.1, the withdrawal strength of fasteners in timber is influenced by a number of parameters. A comprehensive experimental programme was developed with the aim to investigate the influence of the parameters that were considered to be significant and could affect the withdrawal behaviour of helically shaped fasteners.

4.3.1 Datum tests

To have a basis for analysis of the influence of these parameters on the withdrawal strength and behaviour of helically shaped fasteners, the different parameters investigated were set to a datum value. This allowed variation of only one parameter at a time while keeping the others to their datum value. The effect of each individual parameter was therefore investigated.

Eurocode 5 recommends that the pilot hole through which the nails are inserted in the timber should not exceed 0.8 times the nominal diameter, as defined in BS EN 14592:2008 (BSI, 2009a). For Helically shaped fasteners using predrilling with a pilot hole of 0.8 times the nominal diameter lead to large diameters; from 8mm for StarTie 10 to 3.6mm for TimTie. Due to the cross section varying along the length and the general geometry of the fasteners these pilot holes tended to remove too much timber and minimise drastically the surface area for the fasteners to be in contact with the timber, and result in an adverse effect to the rationale for predrilling the timber. Therefore it was decided that the timber samples were to be predrilled for the datum tests to pilot holes of diameters measuring the root diameter of the fasteners. However, obtaining drill sizes of the exact dimensions of the root diameters proved impossible, thus the nearest smaller drill sizes available was used. The pilot hole diameters used for the datum tests were 4.0, 3.5, 3.2, 3.0mm for StarTie 10, StarTie 8, InSkew and TimTie respectively. Using these drill sizes resulted in ratios of pilot hole to root diameters ranging from 0.94 to 1.0.

The datum tests had the following characteristics:

- Timber of grade C24,
- Predrilling to 0.94 – 1.0 times the root diameter,
- Inserted at 90° to the timber grain,
- Moisture content of the timber of 10±1%,
- Timber samples of thickness 45mm.

The timber used in the tests was stored for a period of two months before the tests to achieve constant moisture content. Samples were cut, and clear specimens chosen for the tests, however within a specimen, small knots and variation in the slope of the timber fibres were permitted provided they were unlikely to significantly reduce the specimen strength, or have any influence on the test behaviour or result. The samples were fabricated and tested within one hour.

4.3.2 Factors investigated

The experimental programme was developed for investigating the factors affecting the withdrawal strength independently, on the assumption that there is no significant interaction between the factors. The factors investigated, that were thought to have a significant influence on the behaviour and performance of helically shaped fasteners in withdrawal were as follows:

- *Diameter of pilot hole*. In addition to the datum value, samples without a pilot hole were first tested perpendicular to the grain. As the profile of helically shaped fastener varies around the perimeter, a series of pilot hole diameters was also considered. These included a 2.0mm diameter hole as a nominal pilot hole size, 0.8 × the root diameter, 0.8 × the effective diameter (where the effective diameter is defined in Eurocode 5 (BSI, 2004) as 1.1 × the root diameter), and 0.8 × average of the root and shank diameters.

- *Timber density*. Two other timber strength classes were considered, a grade C16 softwood and a hardwood grade D30 in accordance with BS EN 338:2003 (BSI, 2003).

- *Depth of penetration*. Tests were performed on samples with the following thicknesses: 20mm, 30mm, and 60mm.

- *Angle of penetration to the timber grain*. Tests were performed with the fasteners inserted at 0°, 23°, 45° and 67° to the direction of the grain, Figure 4.8.

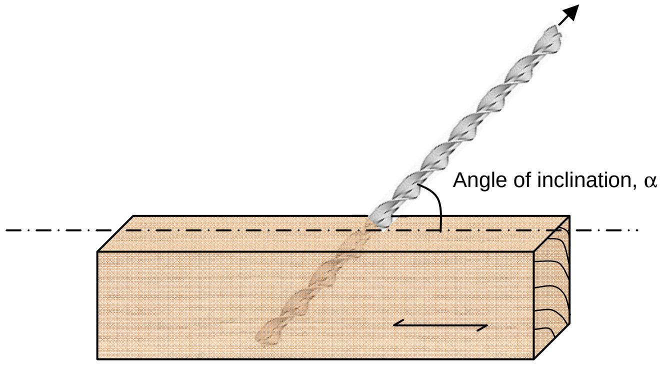


Figure 4.8: Fastener angle of penetration to the timber grain

The timber moisture content is an important parameter influencing the withdrawal behaviour of fasteners, however in order to investigate its effect on the load displacement characteristics and withdrawal strength it would be necessary to perform the tests for a range of moisture content. The equipment available in the laboratory did not allow the regulation of the moisture content with sufficient accuracy for a large enough range, for the tests to result in data that represent the true effect of the moisture content. Therefore the tests were performed for timber that had been stored to achieve constant moisture content of ± 10%.

As discussed in Section 4.2.3, when determining the withdrawal strength of a fastener the friction between the timber and the fastener shank should be effectively considered. As the profile of helically shaped fasteners vary around the perimeter, and form helixes around the length of the fastener, a projected length and the fastener perimeter should be considered. The projected length of the fastener measured at the top of the shank diameter will be greater than the projected length measured at the root diameter. Average projected lengths were calculated for each diameter in order to represent the total area of fastener in contact with the timber. The ratios of effective penetrated length of the fasteners to the depth of timber were found to be 1.13, 1.12, 1.13 and 1.11 for StarTie 10, StarTie 8, InSkew and TimTie fasteners respectively. The fastener perimeters were also measured and were found to be 28.5 mm, 23.5 mm, 18.7 mm and 15.0 mm for StarTie 10, StarTie 8, InSkew and TimTie fasteners respectively.

4.3.3 Results and observations

The results of extended experimental program tests are given in Table 4.3. The average maximum withdrawal loads are given for each set of tests. The general trend of the effects of the parameters investigated on the withdrawal load of helically shaped fasteners can be observed, Figure 4.9.

Table 4.3: *Effect of the variation of the pilot hole, angle, depth and density on the withdrawal load of helically shaped fasteners.*

Fastener	Pilot hole	Load	Angle	Load	Depth	Load	Timber grade	Load
	mm	N	°	N	mm	N		N
StarTie 10	0.00	3583.65	0.00	2292.95	20.00	1821.05	C16	3093.25
	2.00	3265.68	23.00	2668.31	30.00	2521.85	C24	3169.60
	3.50	3938.83	45.00	2914.79	45.00	3169.60	D30	5997.23
	3.80	3335.95	67.00	2926.74	60.00	3919.65		
	4.00	3169.60	90.00	3169.60				
	5.70	3031.87						
StarTie 8	0.00	3376.96	0.00	1701.34	20.00	1604.26	C16	2208.14
	2.00	2892.43	23.00	2102.34	30.00	2004.96	C24	2241.48
	3.00	3054.29	45.00	2218.03	45.00	2241.48	D30	4397.92
	3.20	2272.87	67.00	2295.65	60.00	2707.64		
	3.50	2241.48	90.00	2241.48				
	4.70	2060.69						
InSkew	0.00	2220.59	0.00	1102.98	20.00	1167.24	C16	1729.40
	2.00	1914.04	23.00	1681.91	30.00	1454.07	C24	1799.24
	2.80	1803.22	45.00	1918.87	45.00	1799.24	D30	3152.33
	3.00	1786.40	67.00	1765.33	60.00	1837.05		
	3.20	1799.24	90.00	1799.24				
	3.80	1620.33						
TimTie	0.00	2061.02	0.00	778.47	20.00	1096.57	C16	1526.40
	2.00	1392.15	23.00	1217.86	30.00	1326.72	C24	1573.47
	2.40	1973.25	45.00	1467.78	45.00	1573.47	D30	2415.80
	2.70	1517.52	67.00	1360.24	60.00	1603.67		
	3.00	1573.47	90.00	1573.47				
	3.60	1460.33						

From these results it can be seen that the factors investigated influence the withdrawal load carrying capacity of helically shaped fasteners in timber. The factors investigated were identified from the literature, where previous research on nails or screws. In figure 4.9, it can be seen that the conclusions of these researches can be applied to helically shaped fasteners in direct withdrawal from timber:

a) The size of pilot hole has a negative effect on the withdrawal capacity of helically shaped fasteners. As they rely on the shear resistance of the timber fibres, with increase pilot hole diameter reduces the withdrawal capacity; however the pilot holes present the advantage of making it easier for the insertion of the fastener;
b) The withdrawal strength increases with timber density. While the difference between the C24 and C16 cannot be fully appreciated based on these results due to the relative low variation in density; it is however clear that the withdrawal load increases with density between timber grade C24 and D30;

c) The withdrawal capacity increases with the depth of penetration of the fastener in timber;

d) As the angle of loading relative to the timber grain increases the withdrawal load increases.

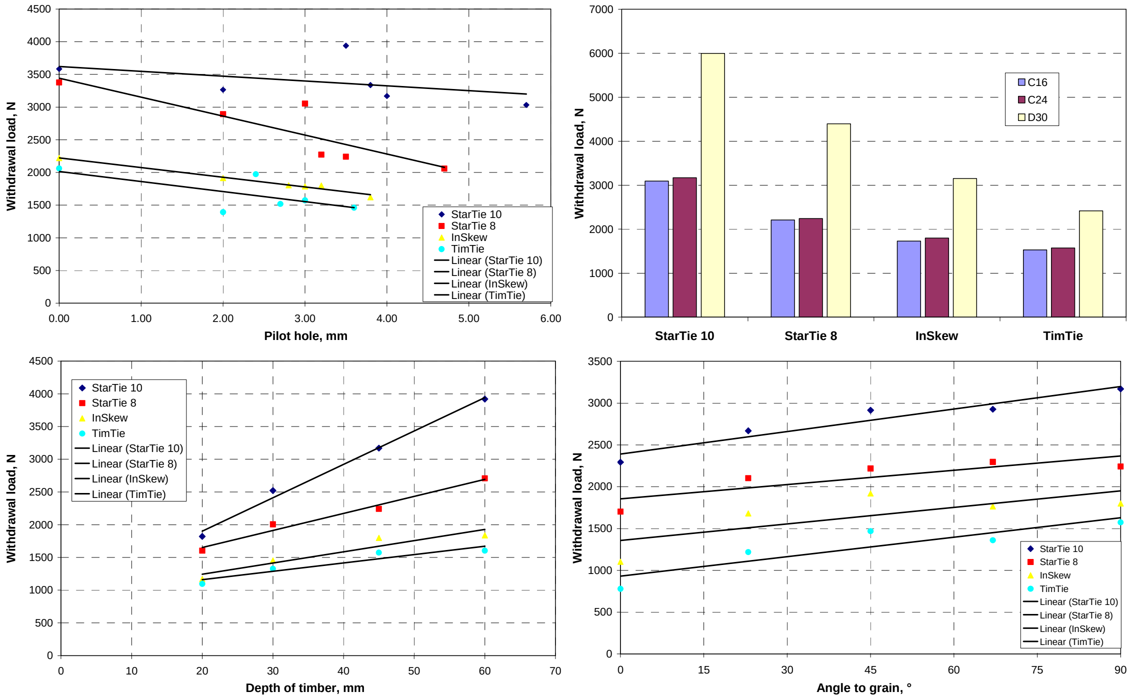


Figure 4.9: Effect of pilot hole, timber grade, depth of penetration and angle to grain, on the withdrawal load of helically shaped fasteners.

4.4 Semi empirical model for axially loaded helically shaped fasteners in timber

The only numerical model developed for predicting the withdrawal of a fastener in timber relates to the fastener diameter, the timber density and the penetration length; see Section 2.4.2. Such models predict the withdrawal load of a fastener in timber; however as can be seen in Figure 4.4, the withdrawal behaviour of helically shaped fasteners in timber is radically different than of conventional timber fasteners. Therefore, to represent the true load-displacement behaviour of helically shaped fasteners in direct pull-out, a semi-empirical model was developed based on a method described by Porteous and Kermani (2005) and first used by Mack in 1966 for laterally loaded timber joints. Mack showed that the parameters investigated did not significantly interact and that the relationship between the load and displacement was a function of the product of each of the parameters.

Thus, the withdrawal load of helically shaped fasteners in timber can be expressed as follow:

$$W = f_1(\delta)\cdot f_2(p_e)\cdot f_3(l_p)\cdot f_4(D)\cdot f_5(r_d)\cdot f_6(\alpha)\cdot f_7(g)\cdot f_8(v) \qquad \textbf{…(4.5)}$$

Where: W = withdrawal load of the fastener,

$f_1(\delta)$ = Displacement function,

$f_2(p_e)$ = Perimeter function,

$f_3(l_p)$ = Depth of penetration function,

$f_4(D)$ = Density function,

$f_5(r_d)$ = Pilot hole function,

$f_6(\alpha)$ = Angle of the grain function,

$f_7(g)$ = Generic function,

$f_8(v)$ = Function of remaining variables.

The function $f_8(v)$ allows for other variables that may influence the behaviour of axially loaded Helically shaped fasteners to be considered in the model- e.g. method of insertion (manual or mechanical), time between fabrication and testing, etc…. However, as their influence was not studied in the test programme the function $f_8(v)$ is taken as unity. The functions f_1 to f_7 are addressed in the following sections.

4.4.1 Displacement function, $f_1(\delta)$

The load displacement behaviour of helically shaped fasteners results in the maximum load being attained at a displacement of 7.0 to 14.0mm. However, such displacements represent failure regarding serviceability limit states. A joint slip of 2.50mm was chosen as an appropriate limit for structural purposes and was used to derive the displacement function. At this slip, the load reached by each helically shaped fastener was, on average, approximately 70% of its maximum withdrawal load.

As it is assumed that there is no interaction between the parameters investigated, the load displacement relationship can be rewritten from Eq. (4.5) as:

$$W = f_1(\delta)\cdot k \qquad \textbf{…(4.6)}$$

If k is the load at the slip limit of 2.50mm, the function will be unity at this limit, and at any intermediate load the function can be written:

$$f_1(\delta) = \frac{(W)_\delta}{(W)_{2.50}} \qquad \textbf{...(4.7)}$$

$(W)_\delta / (W)_{2.50}$ is referred as the reduced load and over the range 0-2.50mm it will define the displacement function. The concept of reduced load was first introduced by Mack in 1966, and has since been widely used in timber research to develop semi-empirical models. To represent the load behaviour, many forms of displacement functions have been developed (Porteous, 2003). Various forms were tried to represent the displacement function of helically shaped fasteners in direct withdrawal, the one that achieved the best fit to the test data was generalised four parameters non linear exponential equation developed by Mack (1966):

$$f_1(\delta) = (A \cdot 0.4\delta + B)(1 - e^{(C \cdot 0.4\delta)})^D \qquad \textbf{...(4.8)}$$

The test results were processed in the software Excel; for each test the load at displacement δ was divided by the load achieved at 2.50mm, to obtain the reduced load curve. The parameters A, B, C and D were then calculated for each fastener diameter and for the full range of diameters. These parameters were determined using the commercial software MathCAD and its least square non-linear regression analysis function Genfit as it allows the user to create its own equation type. An example of the MathCAD worksheet used for determining the parameters in Equation (4.8) is given in Appendix C. The regression analysis results from MathCAD for each diameter size and for the full range of helically shaped fasteners diameters are detailed below:

StarTie 10 $(0.2164\delta + 0.476) \cdot (1 - e^{(-4.1388\delta)})^{0.821}$ **...(4.9a)**

StarTie 8 $(0.1732\delta + 0.583) \cdot (1 - e^{(-3.074\delta)})^{0.722}$ **...(4.9b)**

InSkew $(0.1644\delta + 0.607) \cdot (1 - e^{(-3.3112\delta)})^{0.659}$ **...(4.9c)**

TimTie $(0.1668\delta + 0.598) \cdot (1 - e^{(-3.4716\delta)})^{0.658}$ **...(4.9d)**

All $(0.1804\delta + 0.565) \cdot (1 - e^{(-3.4348\delta)})^{0.704}$ **...(4.9e)**

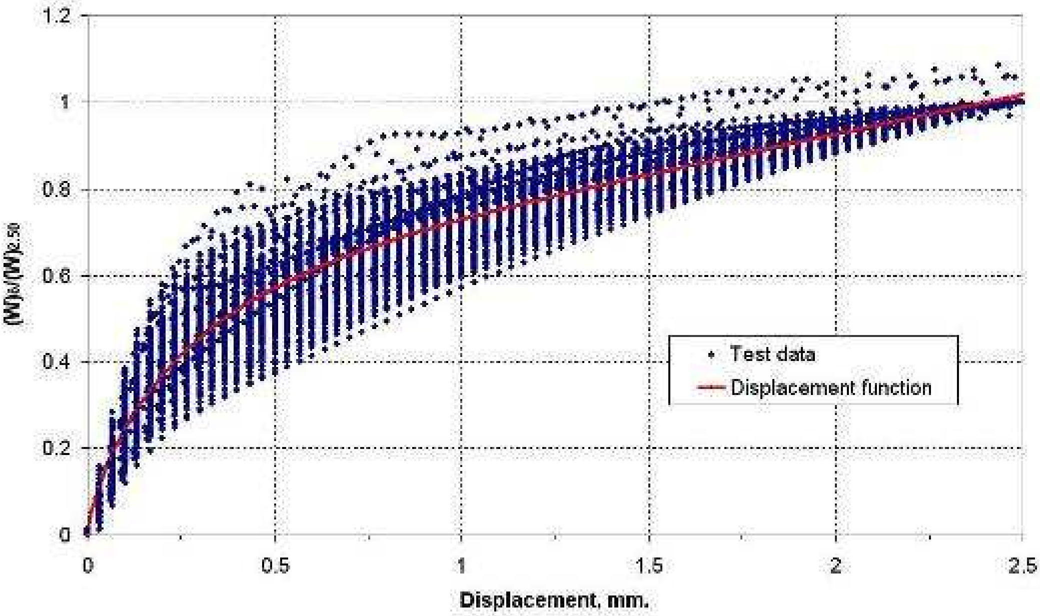


Figure 4.10: Regression graph for all helically shaped fasteners and corresponding displacement function $f_1(\delta)$.

4.4.2 Perimeter function, $f_2(p_e)$

Rammer et al. (2001) tested round, annularly threaded and helically threaded nails and showed that the withdrawal strength is directly proportional to the diameter of the nails. However, with helically shaped fasteners the perimeter length of the fastener is the appropriate variable and this has been used in the analysis to determine the perimeter function.

The perimeter function $f_2(p_e)$ was evaluated using the tests results with timber grades C16, C24 and D30. Figure 15 shows the withdrawal loads for each fastener at 2.50mm displacement. In this analysis, the fasteners all had the same depth of penetration.

For the three timber densities the relationship between the withdrawal loads and the fasteners perimeters was found to be linear, the function $f_2(p_e)$ can therefore be expressed as:

$$f_2(p_e) = \textit{Fastener perimeter} \qquad \textbf{...(4.10)}$$

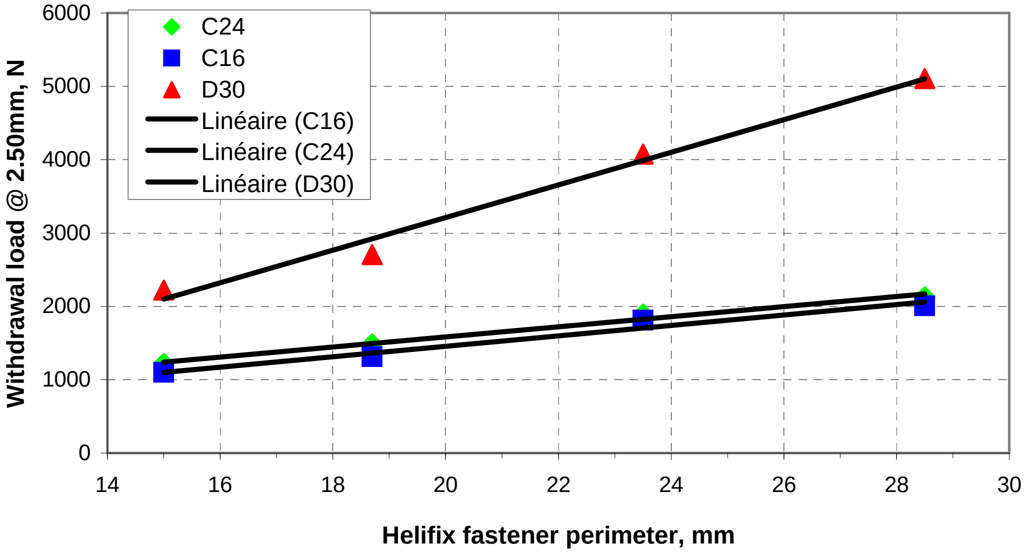


Figure 4.11: Relationship between withdrawal load and fastener perimeter.

4.4.3 Depth of penetration function, $f_3(l_P)$

The effect of the depth of penetration of the fasteners on the withdrawal strength was investigated by inserting the fasteners in samples with nominal thicknesses of 20mm, 30mm, 45mm, and 60mm. The withdrawal loads at 2.50mm displacement were plotted against the projected penetration depth for the four fasteners and the results are shown in Figure 4.12. It shows that the withdrawal load is increasing linearly with the projected depth of penetration allowing the function to be represented as:

$$f_3(l_p) = \textit{projected depth of penetration} \qquad \textbf{...(4.11)}$$

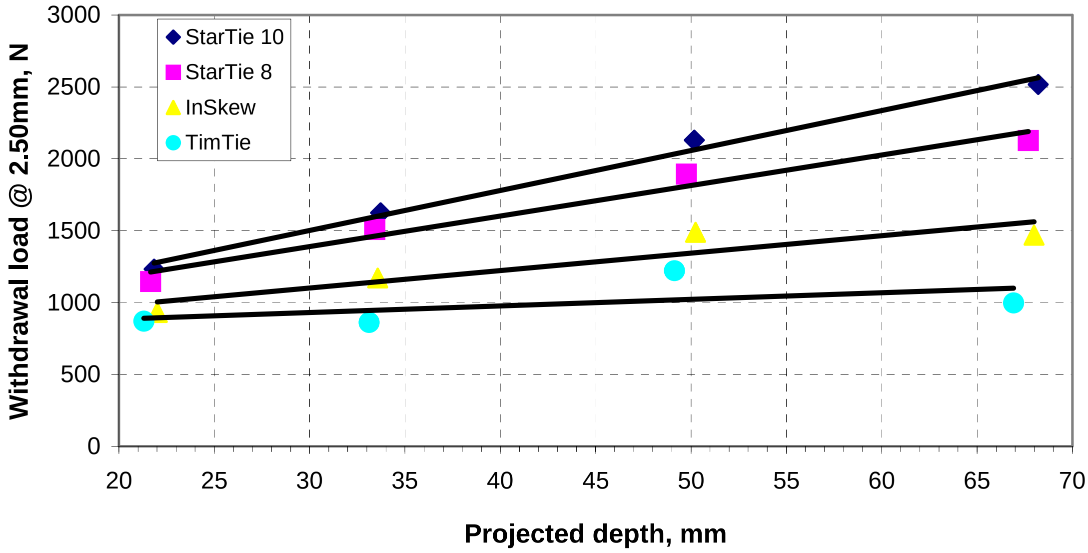


Figure 4.12: Withdrawal load vs projected depth of penetration

4.4.4 Density function, $f_4(D)$

The withdrawal of the fastener results in the fastener being twisted to a plastic state, as the failure modes show, and the effect of this action has been incorporated in the evaluation of the Density function.

As the perimeter function and the depth of penetration function were found to be linear, the density function $f_4(D)$ was able to be developed for the four fasteners in the form described by Equation (2.5). The withdrawal strength at a displacement of 2.50mm was calculated using the fasteners perimeter and projected depth of penetration in timber, it was then plotted against timber density (D). All of the samples had similar pilot hole to root diameters ratios. The constants a and b of the equation were determined using the non-linear least squares regression function *Genfit* in MathCAD. This form of equation also resulted in the best fit to test data. From the analysis the density function was found to be:

$$f_4(D) = 3.7268 \times 10^{-5} \cdot D^{1.7892} \qquad \textbf{...(4.12)}$$

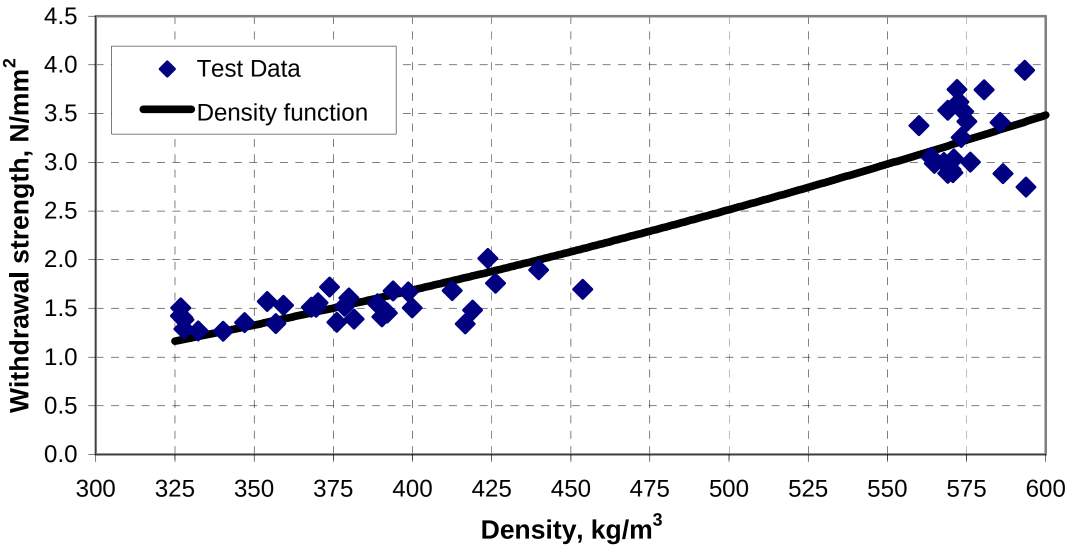


Figure 4.13: Density function, $f_4(D)$

Three grades of timber were used in determining the effect of density on the withdrawal strength of helically shaped fasteners. Using the historical form of equation for determining the pull out force of helically shaped fastener in timber, shows that the relationship between timber density and withdrawal load is similar to that of conventional nails. This is in the form of a power equation.

4.4.5 Pilot hole function, $f_5(r_d)$

The pilot holes diameters used in this experimental programme were chosen as factors of the various diameters of helically shaped fasteners, as mentioned in Section 4.3.2. In order to evaluate the influence of the pilot hole diameters on the withdrawal strength of the four sizes of Helically shaped fasteners used in the study, the ratio, r_d, of the pilot hole diameter to the corresponding root diameter was used. The ratio, r_d, ranged from 0.00 to 1.34; the range was estimated to be wide enough for the effect of the pilot home diameter to be analysed. It is known that the increase in pilot hole diameter has an adverse effect on the withdrawal strength of conventional fasteners, and therefore it is often limited in design standards. The same trend was expected with helically shaped fasteners.

The withdrawal strength of each fastener was plotted against the associated value of the ratio r_d, as shown in Figure 4.14. The plot shows that the withdrawal strength decreases as the ratio of the pilot hole to root diameter increases and from a regression analysis the relationship between the withdrawal strength and r_d can be represented as follows:

$$f_5(r_d) = -0.5862 r_d + 2.1652 \qquad \textbf{...(4.13)}$$

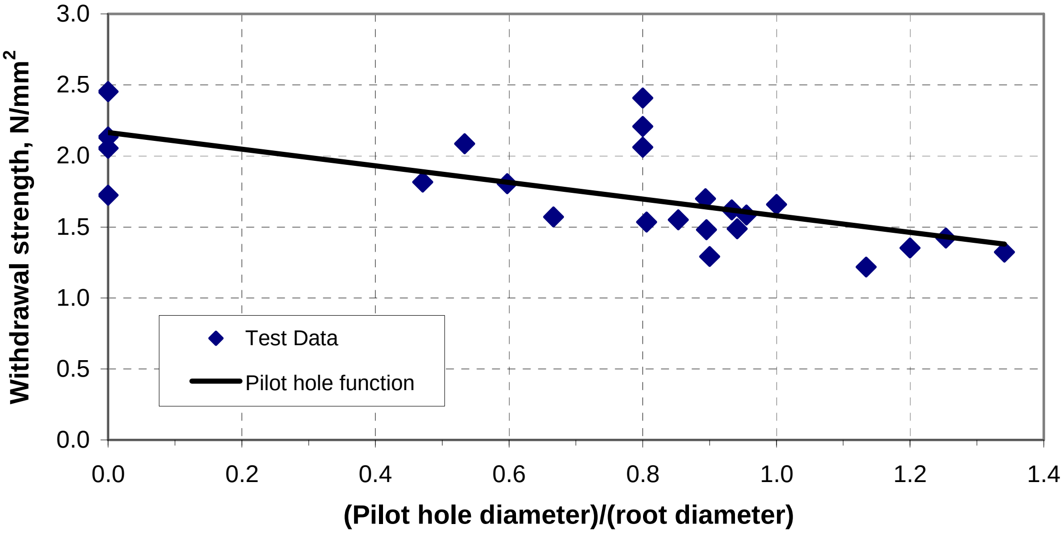


Figure 4.14: Pilot hole function, $f_5(r_d)$

4.4.6 Angle to the grain function, $f_6(\alpha)$

In the case of laterally loaded timber joints with dowels types fasteners, researchers concluded that the grain orientation had little influence on the joints mechanical properties (Smith & Whale, 1985). However, axially loaded fasteners are greatly influenced by the angle of insertion to the timber fibres (Rammer & Zelinka, 2004), with ratios of end grain to side grain withdrawal ranging from 0.5 to 0.8. They also concluded that in the case of threaded nails, due to increase withdrawal performance perpendicular to the grain, the ratio was significantly lower. Therefore, to investigate the effect of the fastener angle to the grain on the withdrawal performance, tests were performed with fasteners inserted at angles α = 0°, 23°, 45°, 67° and 90° to the timber fibres.

The results confirmed that maximum resistance was achieved perpendicular to the grain, i.e. at 90°, with the withdrawal resistance decreasing as the angle of insertion reduced. Thus, the function $f_6(\alpha)$ was developed in such a way that it is taken to be unity for fasteners inserted at right angle to the grain. The ratios of α/90°, and of the withdrawal loads at the different angles to the withdrawal load at 90° were computed for the fasteners and the averages are plotted in Figure 4.15. A least squares regression analysis was performed on the results to determine $f_6(\alpha)$, resulting in the following relationship:

$$f_6(\alpha) = 0.3129(\frac{\alpha}{90}) + 0.682 \qquad \textbf{...(4.14)}$$

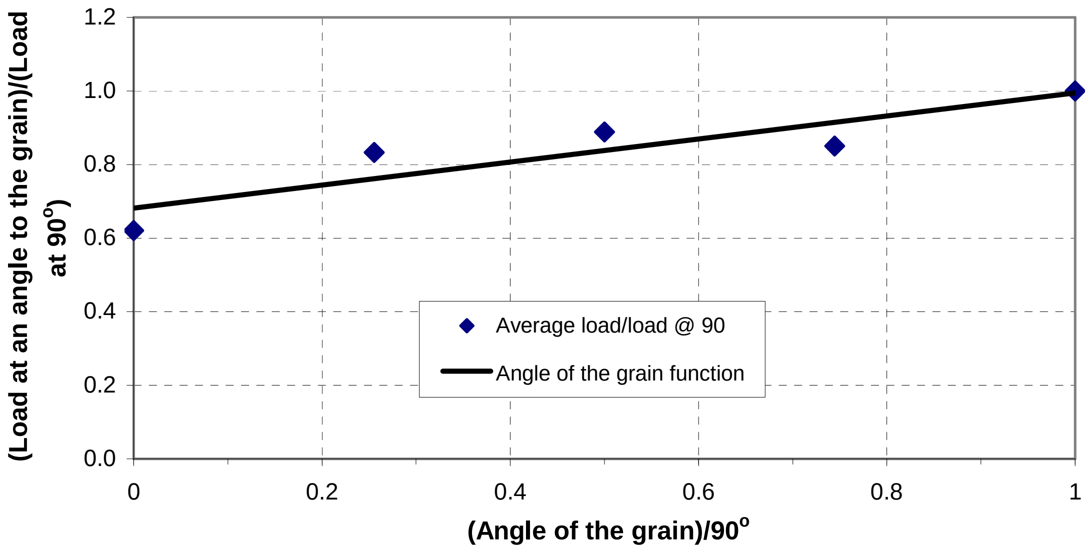


Figure 4.15: Angle to the grain function, $f_6(\alpha)$

4.4.7 Generic function, $f_7(g)$

The generic function is the function that takes into account all the parameters not included in the function f_1 to f_6. The function $f_7(g)$ for each test performed was determined by rearranging Equation (4.5) as follow:

$$f_7(g) = \frac{W}{f_1(\delta)\times f_2(p_e)\times f_3(l_p)\times f_4(D)\times f_5(r_d)\times f_6(\alpha)} \qquad \textbf{...(4.15)}$$

For each test performed a generic function was calculated, and the average was calculated for each fastener diameter and for the full range of helically shaped fasteners. The results were as follows:

StarTie 10 $f_7(g) = 0.5740139$ **…(4.16a)**

StarTie 8 $f_7(g) = 0.5967899$ **…(4.16b)**

InSkew $f_7(g) = 0.5719884$ **…(4.16c)**

TimTie $f_7(g) = 0.5792401$ **…(4.16d)**

All $f_7(g) = 0.5732783$ **…(4.16e)**

The equations (4.16a) to (4.16e) were determined using the corresponding displacement functions detailed in Equation (4.9) calculated in Section 4.4.1.

4.4.8 Semi empirical model

The influence of the different parameters that affect the load withdrawal behaviour of helically shaped fasteners in timber has been studied in the sections above. The displacement, perimeter and generic functions can be determined for each fastener size and for the range of diameters; therefore a semi empirical model can be derived using the corresponding functions for each fastener diameter. A general model including the functions calculated for all fastener diameters was also derived. Substituting for the relevant functions in Equation (4.5) the load displacement relationship of helically shaped fasteners in direct withdrawal becomes:

- For StarTie 10:

$$W = 0.5740139 \cdot [(0.2164\delta + 0.476) \cdot (1 - e^{-4.1388\delta)})^{0.821})] \cdot p_e \cdot l_p \cdot (3.7268 \times 10^{-5} \cdot D^{1.7892})$$
$$\cdot (-0.5862 \cdot r_d + 2.1652) \cdot (0.3129 \cdot (\frac{\alpha}{90}) + 0.682) \qquad \textbf{…(4.17a)}$$

- For StarTie 8:

$$W = 0.5967899 \cdot [(0.1732\delta + 0.583) \cdot (1 - e^{-3.074\delta)})^{0.722})] \cdot p_e \cdot l_p \cdot (3.7268 \times 10^{-5} \cdot D^{1.7892})$$
$$\cdot (-0.5862 \cdot r_d + 2.1652) \cdot (0.3129 \cdot (\frac{\alpha}{90}) + 0.682) \qquad \textbf{…(4.17b)}$$

- For InSkew:

$$W = 0.5719884 \cdot [(0.1644\delta + 0.607) \cdot (1 - e^{-3.3112\delta)})^{0.659})] \cdot p_e \cdot l_p \cdot (3.7268 \times 10^{-5} \cdot D^{1.7892})$$
$$\cdot (-0.5862 \cdot r_d + 2.1652) \cdot (0.3129 \cdot (\frac{\alpha}{90}) + 0.682) \qquad \textbf{…(4.17c)}$$

- For TimTie:

$$W = 0.5492401 \cdot [(0.1668\delta + 0.598) \cdot (1 - e^{-3.4716\delta)})^{0.658})] \cdot p_e \cdot l_p \cdot (3.7268 \times 10^{-5} \cdot D^{1.7892})$$
$$\cdot (-0.5862 \cdot r_d + 2.1652) \cdot (0.3129 \cdot (\frac{\alpha}{90}) + 0.682) \qquad \textbf{…(4.17d)}$$

- For all diameters:

$$W = 0.5732783 \cdot [(0.1804\delta + 0.565) \cdot (1 - e^{-3.4348\delta)})^{0.704})] \cdot p_e \cdot l_p \cdot (3.7268 \times 10^{-5} \cdot D^{1.7892})$$
$$\cdot (-0.5862 \cdot r_d + 2.1652) \cdot (0.3129 \cdot (\frac{\alpha}{90}) + 0.682) \qquad \textbf{…(4.17e)}$$

Where W = withdrawal load of a helically shaped fastener at a slip δ (N),

δ = is the displacement at which the load is calculated (mm), - noting that this is the summation of the slip of the fastener in the connected members.

D = timber density (kg/m^3) at a moisture content of 10±1%,

p_e = perimeter of the fastener (mm),

r_d = ratio of the pilot hole diameter to the fastener root diameter,

l_p = projected depth of penetration in the timber (mm),

α = angle of the fastener to the grain orientation (degrees).

Rearranging and simplifying the Equations (4.17a) to (4.17e) become:

- For StarTie 10:

$$W = 9.4x10^{-9} \cdot p_e \cdot l_p \cdot D^{1.79} \cdot (\alpha + 196) \cdot (3.69 - r_d) \cdot (\delta + 2.2) \cdot (1 - e^{-4.14\delta)})^{0.82} \quad \textbf{...(4.18a)}$$

- For StarTie 8:

$$W = 7.8x10^{-9} \cdot p_e \cdot l_p \cdot D^{1.79} \cdot (\alpha + 196) \cdot (3.69 - r_d) \cdot (\delta + 3.4) \cdot (1 - e^{-3.07\delta)})^{0.72} \quad \textbf{...(4.18b)}$$

- For InSkew:

$$W = 7.1x10^{-9} \cdot p_e \cdot l_p \cdot D^{1.79} \cdot (\alpha + 196) \cdot (3.69 - r_d) \cdot (\delta + 3.7) \cdot (1 - e^{-3.31\delta)})^{0.66} \quad \textbf{...(4.18c)}$$

- For TimTie:

$$W = 6.9x10^{-9} \cdot p_e \cdot l_p \cdot D^{1.79} \cdot (\alpha + 196) \cdot (3.69 - r_d) \cdot (\delta + 3.6) \cdot (1 - e^{-3.47\delta)})^{0.66} \quad \textbf{...(4.18d)}$$

- For all diameters:

$$W = 7.8x10^{-9} \cdot p_e \cdot l_p \cdot D^{1.79} \cdot (\alpha + 196) \cdot (3.69 - r_d) \cdot (\delta + 3.1) \cdot (1 - e^{-3.45\delta)})^{0.79} \quad \textbf{...(4.18e)}$$

Equation (4.18) allows the determination of the withdrawal behaviour and performance of helically shaped fasteners of shank diameter ranging between 4.50 to 10.00mm in timber at a moisture content of 10±1%.

4.4.9 Comparison of test data and model

Using the average parameters in each test series – timber density, depth of penetration, pilot hole diameter – in the Equation (4.18a to d) and in Equation (4.18e), two predicted values were calculated for each test series, one corresponding to the model customised to the fastener diameter, and one corresponding to the model which is independent of fastener diameter; The results are given in Table 4.4 (a & b respectively). From Table 4.4(b) it can be seen that for some test series the general model (Equation 4.18e) results in a reasonable fit, while for others the prediction error is over 20%., however on average the prediction error is 10.44%. The results of the withdrawal tests and the models (generalised and customised) have been plotted together in Figure 4.16 To 4.19 for StarTie 10, StarTie 8, InSkew and TimTie respectively. The models use the average density, depth of penetration of the test series as detailed in Table 4.4.

The ratio of the percentage error of the general model to the percentage error of customised models was calculated for each test series. The results show that the ratio between the two prediction errors varies between -2.57 and +6.56, with an absolute average ratio of 1.15. This shows that the model customised per fastener does not provide significantly improved predictions values for the withdrawal of helically shaped fasteners. Therefore a unique model for all size of diameter for helically shaped fasteners is preferred.

Table 4.4 (a): *Comparison of test results and Equations (4.18a) to (4.18d)*

Samples	Perimeter, mm	Projected depth, mm	Pilot hole, mm	Density, kg/m^3	Angle, ^{o}	Test Load, N	Model load, N	Error, %
S10-NOPP	28.5	49.55	0.0	397.36	90	2433.17	2959.58	17.79
S10-2mm	28.5	49.83	2.0	403.08	90	2578.42	2664.27	3.22
S10-CO	28.5	50.63	3.4	432.75	90	2974.67	2759.53	-7.80
S10-EF	28.5	50.22	3.8	406.49	90	2433.34	2367.47	-2.78
S10-BF	28.5	50.18	4.0	405.41	90	2129.19	2314.92	8.02
S10-AV	28.5	49.81	5.7	400.96	90	1879.03	1925.64	2.42
S10-C16	28.5	50.50	4.0	339.13	90	2008.51	1692.71	-18.66
S10-HW	28.5	49.98	4.0	571.13	90	5106.43	4257.33	-19.94
S10-D20	28.5	21.85	4.0	444.13	90	1233.72	1186.58	-3.97
S10-D30	28.5	33.71	4.0	430.70	90	1625.48	1733.14	6.21
S10-D60	28.5	68.21	4.0	361.52	90	2517.37	2563.38	1.79
S10-AL23	28.5	56.50	4.0	414.59	23	2293.64	2077.83	-10.39
S10-AL45	28.5	56.50	4.0	461.54	45	2407.54	2770.16	13.09
S10-AL67	28.5	56.50	4.0	400.07	67	2084.95	2340.75	10.93
S10-PAR	28.5	56.50	4.0	391.93	0	1680.78	1681.80	0.06
S8-NOPP	23.5	49.99	0.0	434.89	90	2883.68	3004.08	4.01
S8-2mm	23.5	49.48	2.0	428.94	90	2424.83	2482.39	2.32
S8-CO	23.5	49.83	3.0	435.33	90	2585.16	2350.14	-10.00
S8-EF	23.5	49.50	3.2	408.83	90	1804.41	2048.02	11.89
S8-BF	23.5	49.76	3.5	398.92	90	1893.92	1914.80	1.09
S8-AV	23.5	49.91	4.7	402.09	90	1670.42	1722.12	3.00
S8-C16	23.5	50.11	3.5	351.68	90	1810.48	1539.18	-17.63
S8-HW	23.5	49.73	3.5	578.06	90	4077.36	3716.41	-9.71
S8-D20	23.5	21.66	3.5	440.48	90	1146.55	995.25	-15.20
S8-D30	23.5	33.42	3.5	449.55	90	1509.83	1592.71	5.20
S8-D60	23.5	67.68	3.5	371.62	90	2127.98	2294.42	7.25
S8-AL23	23.5	56.20	3.5	384.24	23	1755.49	1548.74	-13.35
S8-AL45	23.5	56.20	3.5	453.04	45	1921.86	2288.26	16.01
S8-AL67	23.5	56.20	3.5	442.26	67	2014.22	2391.69	15.78
S8-PAR	23.5	56.20	3.5	381.40	0	1377.16	1367.91	-0.68
IN-NOPP	18.7	50.08	0.0	409.13	90	1926.72	2062.29	6.57
IN-2mm	18.7	49.56	2.0	423.40	90	1672.87	1818.98	8.03
IN-CO	18.7	49.81	2.7	397.75	90	1430.23	1524.44	6.18
IN-EF	18.7	50.13	3.0	380.49	90	1387.29	1373.43	-1.01
IN-BF	18.7	50.24	3.2	399.91	90	1487.88	1472.39	-1.05
IN-AV	18.7	50.02	3.8	387.34	90	1140.77	1294.04	11.84
IN-C16	18.7	50.12	3.2	348.87	90	1311.49	1150.64	-13.98
IN-HW	18.7	49.86	3.2	574.92	90	2706.87	2797.49	3.24
IN-D20	18.7	22.01	3.2	441.19	90	931.12	768.86	-21.10
IN-D30	18.7	33.57	3.2	412.65	90	1169.65	1040.61	-12.40
IN-D60	18.7	67.98	3.2	357.54	90	1469.81	1630.68	9.86
IN-AL23	18.7	56.32	3.2	382.01	23	1393.94	1164.78	-19.67
IN-AL45	18.7	56.32	3.2	450.63	45	1541.22	1722.46	10.52
IN-AL67	18.7	56.32	3.2	393.78	67	1388.56	1476.64	5.97
IN-PAR	18.7	56.32	3.2	387.65	0	995.39	1070.27	7.00
TIM-NOPP	15	49.26	0.0	417.71	90	1577.32	1616.78	2.44
TIM-2mm	15	48.76	2.0	412.55	90	1149.33	1282.75	10.40
TIM-CO	15	49.59	2.4	424.19	90	1790.83	1310.63	-36.64
TIM-EF	15	49.71	2.7	384.92	90	962.91	1066.16	9.68
TIM-BF	15	49.13	3.0	406.55	90	1221.67	1120.41	-9.04
TIM-AV	15	49.21	3.6	433.13	90	998.43	1163.45	14.18
TIM-C16	15	49.67	3.0	369.66	90	1099.87	955.46	-15.11
TIM-HW	15	49.02	3.0	573.70	90	2220.62	2070.08	-7.27
TIM-D20	15	21.31	3.0	488.37	90	871.08	674.59	-29.13
TIM-D30	15	33.12	3.0	409.16	90	862.26	763.88	-12.88
TIM-D60	15	66.91	3.0	354.57	90	997.49	1194.58	16.50
TIM-AL23	15	55.43	3.0	418.69	23	988.11	1020.33	3.16
TIM-AL45	15	55.43	3.0	456.52	45	1002.46	1310.67	23.52
TIM-AL67	15	55.43	3.0	440.90	67	1042.31	1343.88	22.44
TIM-PAR	15	55.43	3.0	391.16	0	745.97	808.60	7.75

Table 4.4 (b): *Comparison of test results and Equation (4.18e)*

Samples	Perimeter, mm	Projected depth, mm	Pilot hole, mm	Density, kg/m^3	Angle, ^{o}	Test Load, N	Model load, N	Error, %
S10-NOPP	28.5	49.55	0.0	397.36	90	2433.17	2964.02	17.91
S10-2mm	28.5	49.83	2.0	403.08	90	2578.42	2667.84	3.35
S10-CO	28.5	50.63	3.4	432.75	90	2974.67	2762.64	-7.68
S10-EF	28.5	50.22	3.8	406.49	90	2433.34	2370.22	-2.66
S10-BF	28.5	50.18	4.0	405.41	90	2129.19	2317.56	8.13
S10-AV	28.5	49.81	5.7	400.96	90	1879.03	1927.43	2.51
S10-C16	28.5	50.50	4.0	339.13	90	2008.51	1695.00	-18.50
S10-HW	28.5	49.98	4.0	571.13	90	5106.43	4260.43	-19.86
S10-D20	28.5	21.85	4.0	444.13	90	1233.72	1187.80	-3.87
S10-D30	28.5	33.71	4.0	430.70	90	1625.48	1735.00	6.31
S10-D60	28.5	68.21	4.0	361.52	90	2517.37	2566.65	1.92
S10-AL23	28.5	56.50	4.0	414.59	23	2293.64	2079.78	-10.28
S10-AL45	28.5	56.50	4.0	461.54	45	2407.54	2772.59	13.17
S10-AL67	28.5	56.50	4.0	400.07	67	2084.95	2343.34	11.03
S10-PAR	28.5	56.50	4.0	391.93	0	1680.78	1683.34	0.15
S8-NOPP	23.5	49.99	0.0	434.89	90	2883.68	2897.19	0.47
S8-2mm	23.5	49.48	2.0	428.94	90	2424.83	2393.71	-1.30
S8-CO	23.5	49.83	3.0	435.33	90	2585.16	2265.90	-14.09
S8-EF	23.5	49.50	3.2	408.83	90	1804.41	1974.72	8.62
S8-BF	23.5	49.76	3.5	398.92	90	1893.92	1846.25	-2.58
S8-AV	23.5	49.91	4.7	402.09	90	1670.42	1660.17	-0.62
S8-C16	23.5	50.11	3.5	351.68	90	1810.48	1484.30	-21.98
S8-HW	23.5	49.73	3.5	578.06	90	4077.36	3581.76	-13.84
S8-D20	23.5	21.66	3.5	440.48	90	1146.55	959.51	-19.49
S8-D30	23.5	33.42	3.5	449.55	90	1509.83	1535.47	1.67
S8-D60	23.5	67.68	3.5	371.62	90	2127.98	2212.46	3.82
S8-AL23	23.5	56.20	3.5	384.24	23	1755.49	1493.09	-17.57
S8-AL45	23.5	56.20	3.5	453.04	45	1921.86	2205.76	12.87
S8-AL67	23.5	56.20	3.5	442.26	67	2014.22	2305.66	12.64
S8-PAR	23.5	56.20	3.5	381.40	0	1377.16	1318.65	-4.44
IN-NOPP	18.7	50.08	0.0	409.13	90	1926.72	2070.88	6.96
IN-2mm	18.7	49.56	2.0	423.40	90	1672.87	1826.14	8.39
IN-CO	18.7	49.81	2.7	397.75	90	1430.23	1530.43	6.55
IN-EF	18.7	50.13	3.0	380.49	90	1387.29	1378.84	-0.61
IN-BF	18.7	50.24	3.2	399.91	90	1487.88	1478.05	-0.66
IN-AV	18.7	50.02	3.8	387.34	90	1140.77	1298.96	12.18
IN-C16	18.7	50.12	3.2	348.87	90	1311.49	1155.26	-13.52
IN-HW	18.7	49.86	3.2	574.92	90	2706.87	2807.04	3.57
IN-D20	18.7	22.01	3.2	441.19	90	931.12	771.73	-20.65
IN-D30	18.7	33.57	3.2	412.65	90	1169.65	1044.57	-11.97
IN-D60	18.7	67.98	3.2	357.54	90	1469.81	1637.18	10.22
IN-AL23	18.7	56.32	3.2	382.01	23	1393.94	1169.13	-19.23
IN-AL45	18.7	56.32	3.2	450.63	45	1541.22	1728.66	10.84
IN-AL67	18.7	56.32	3.2	393.78	67	1388.56	1482.28	6.32
IN-PAR	18.7	56.32	3.2	387.65	0	995.39	1074.15	7.33
TIM-NOPP	15	49.26	0.0	417.71	90	1577.32	1695.62	6.98
TIM-2mm	15	48.76	2.0	412.55	90	1149.33	1345.03	14.55
TIM-CO	15	49.59	2.4	424.19	90	1790.83	1374.15	-30.32
TIM-EF	15	49.71	2.7	384.92	90	962.91	1117.91	13.87
TIM-BF	15	49.13	3.0	406.55	90	1221.67	1174.66	-4.00
TIM-AV	15	49.21	3.6	433.13	90	998.43	1219.56	18.13
TIM-C16	15	49.67	3.0	369.66	90	1099.87	1001.83	-9.79
TIM-HW	15	49.02	3.0	573.70	90	2220.62	2169.41	-2.36
TIM-D20	15	21.31	3.0	488.37	90	871.08	707.09	-23.19
TIM-D30	15	33.12	3.0	409.16	90	862.26	800.85	-7.67
TIM-D60	15	66.91	3.0	354.57	90	997.49	1252.62	20.37
TIM-AL23	15	55.43	3.0	418.69	23	988.11	1069.51	7.61
TIM-AL45	15	55.43	3.0	456.52	45	1002.46	1373.79	27.03
TIM-AL67	15	55.43	3.0	440.90	67	1042.31	1408.74	26.01
TIM-PAR	15	55.43	3.0	391.16	0	745.97	847.56	11.99

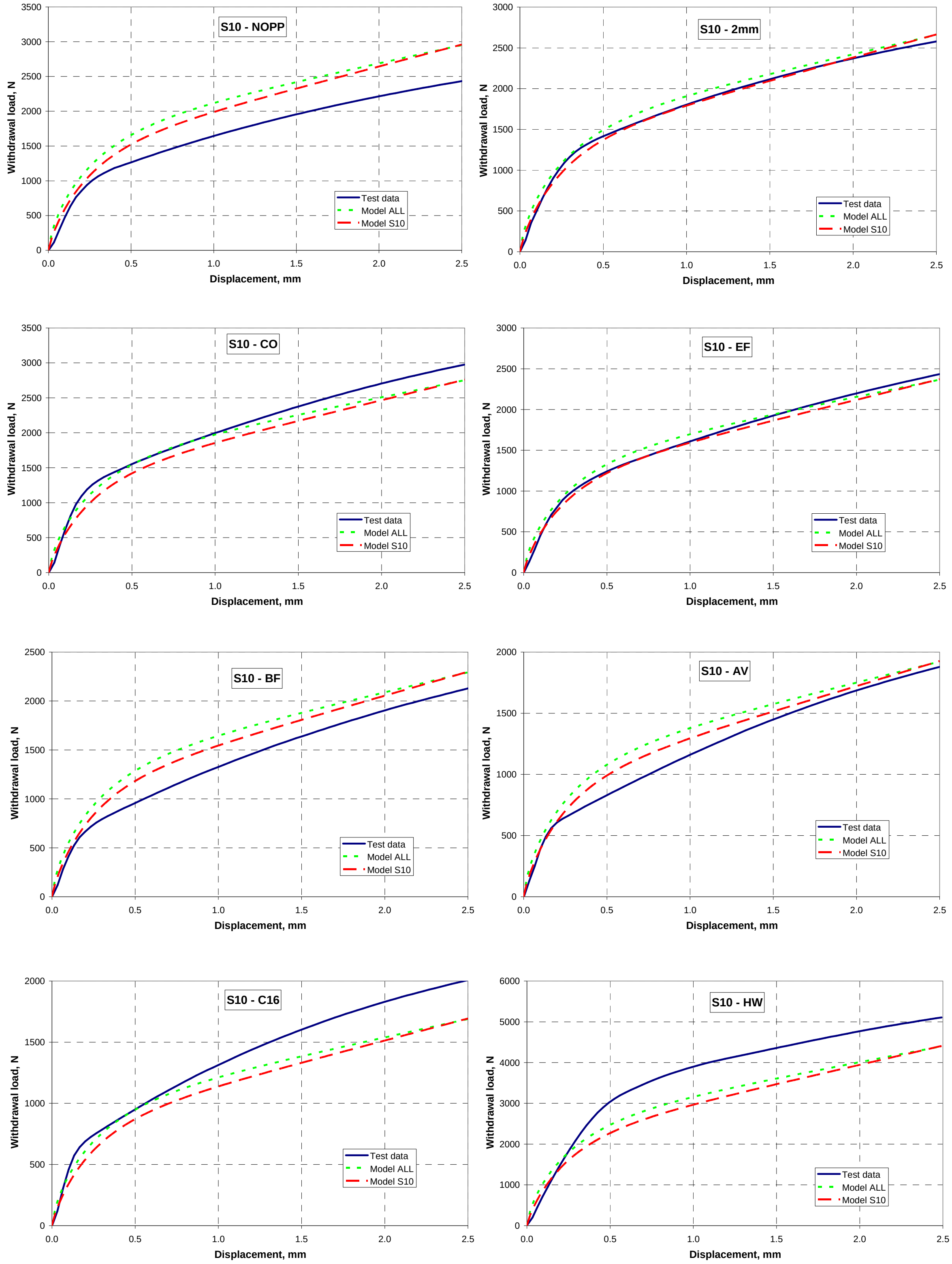


Figure 4.16: Withdrawal load displacement relationship from test and predicted from semi empirical models for StarTie 10 fasteners.

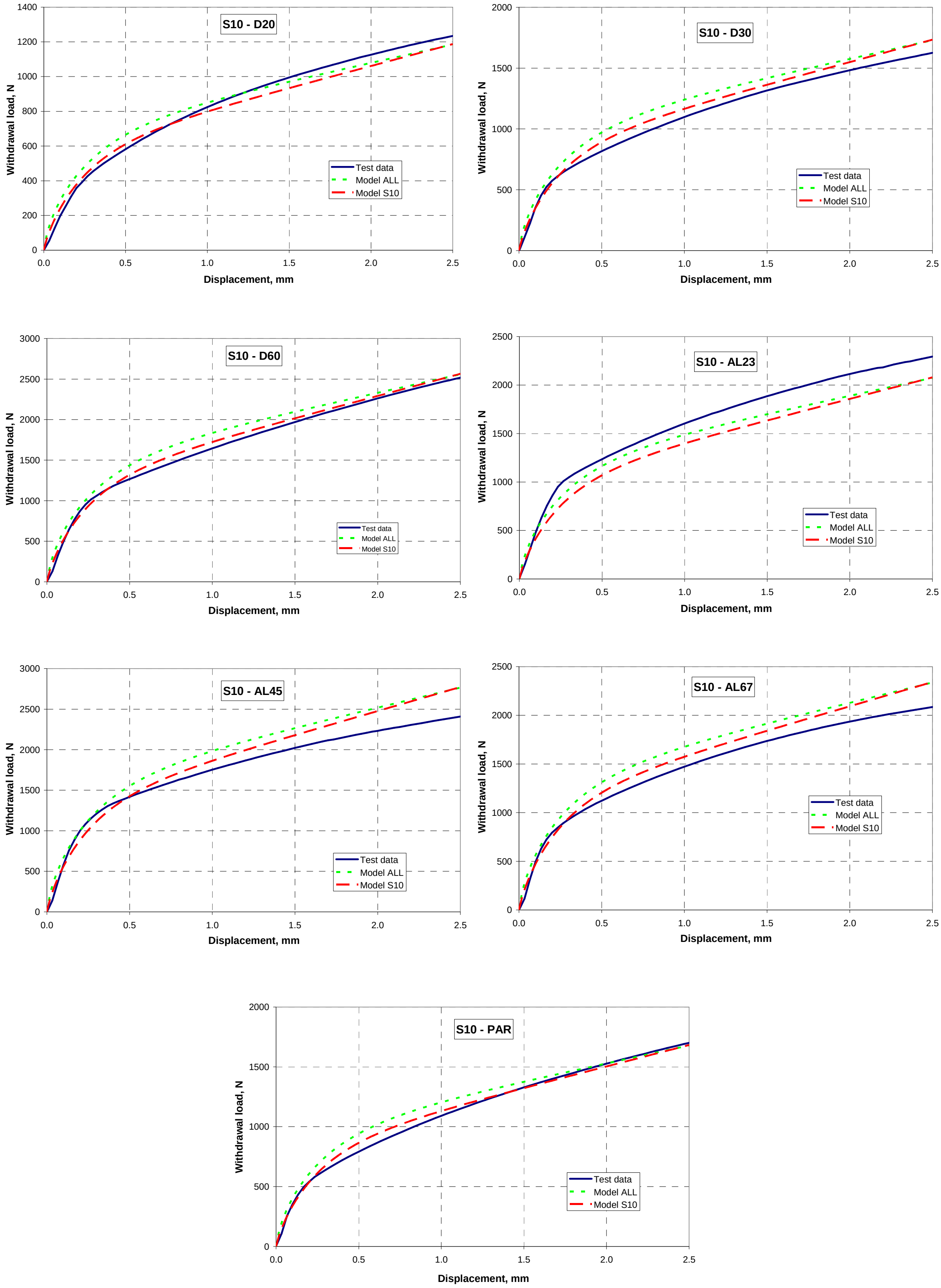


Figure 4.16 continued: Withdrawal load displacement relationship from test and predicted from semi empirical models for StarTie 10 fasteners.

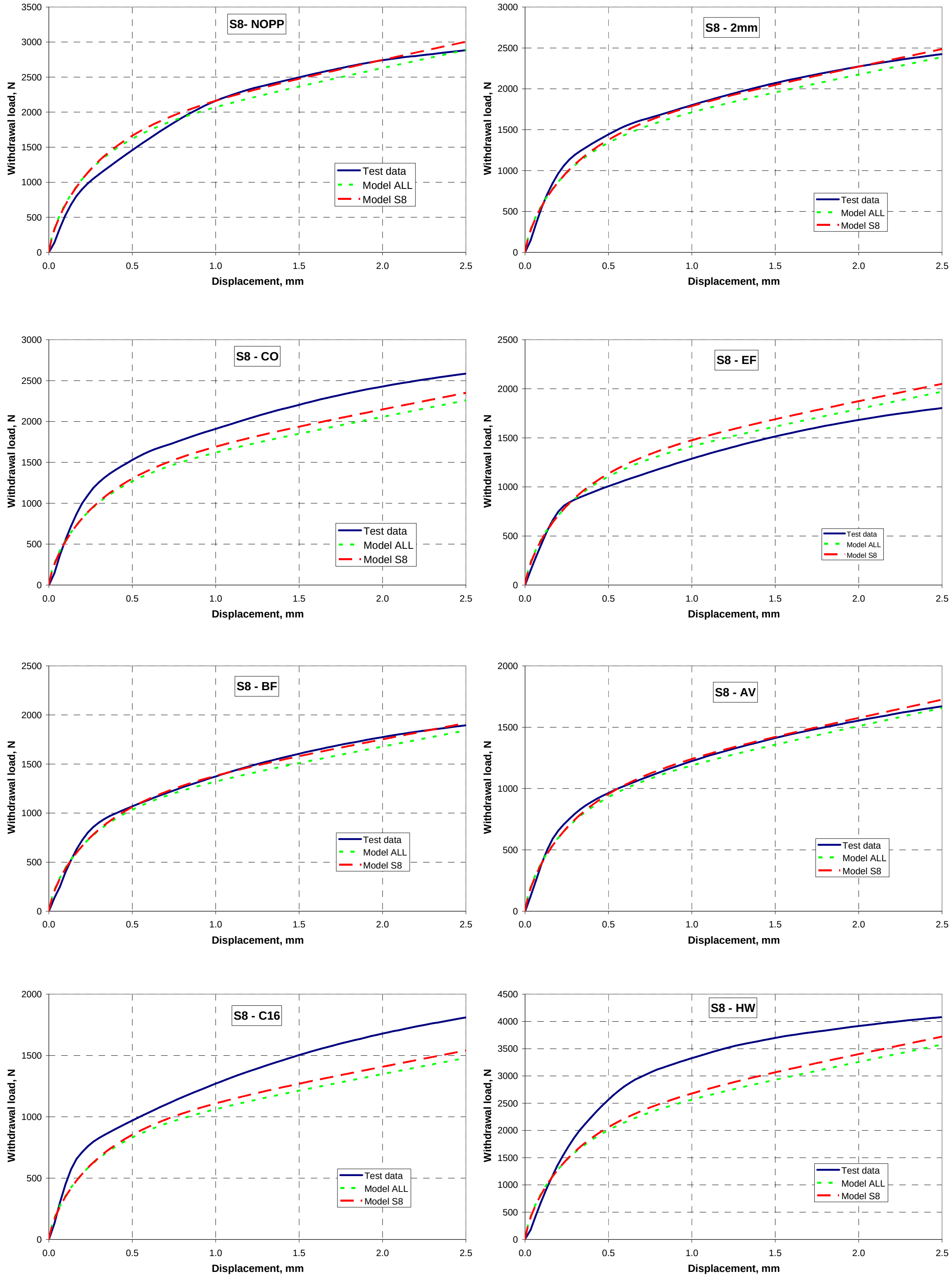


Figure 4.17: Withdrawal load displacement relationship from test and predicted from semi empirical models for StarTie 8 fasteners.

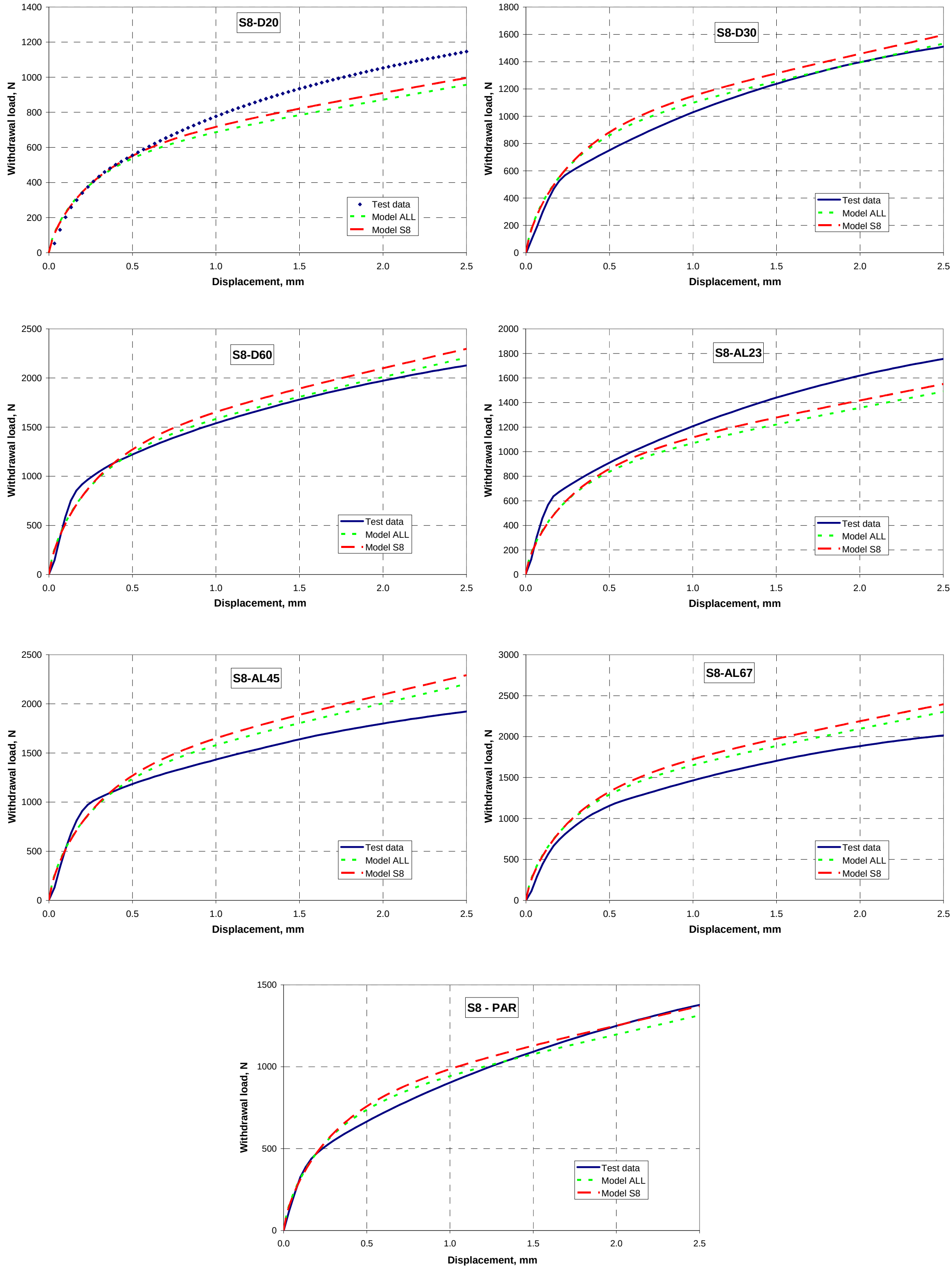


Figure 4.17 continued: Withdrawal load displacement relationship from test and predicted from semi empirical models for StarTie 8 fasteners.

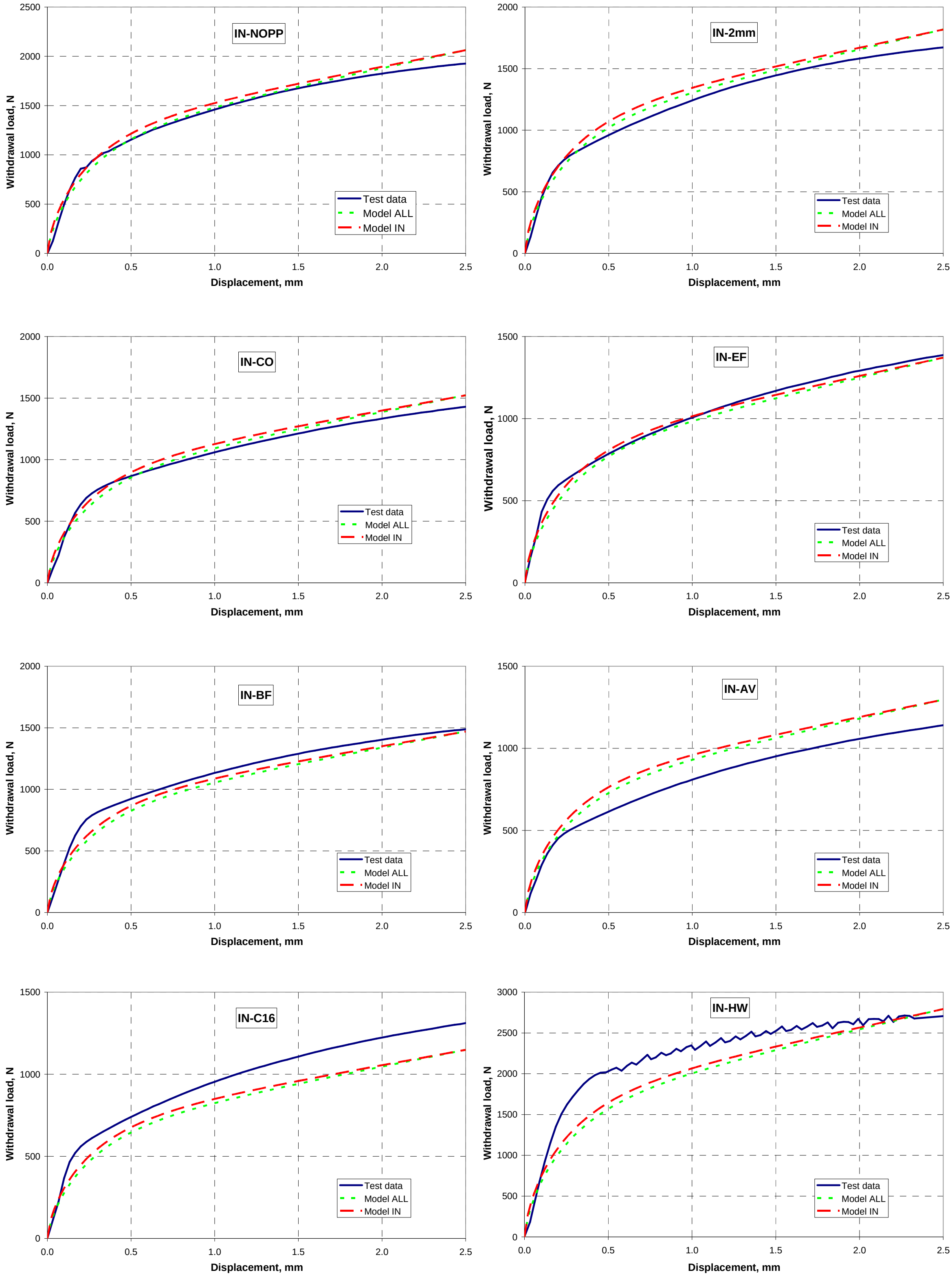


Figure 4.18: Withdrawal load displacement relationship from test and predicted from semi empirical models for InSkew fasteners.

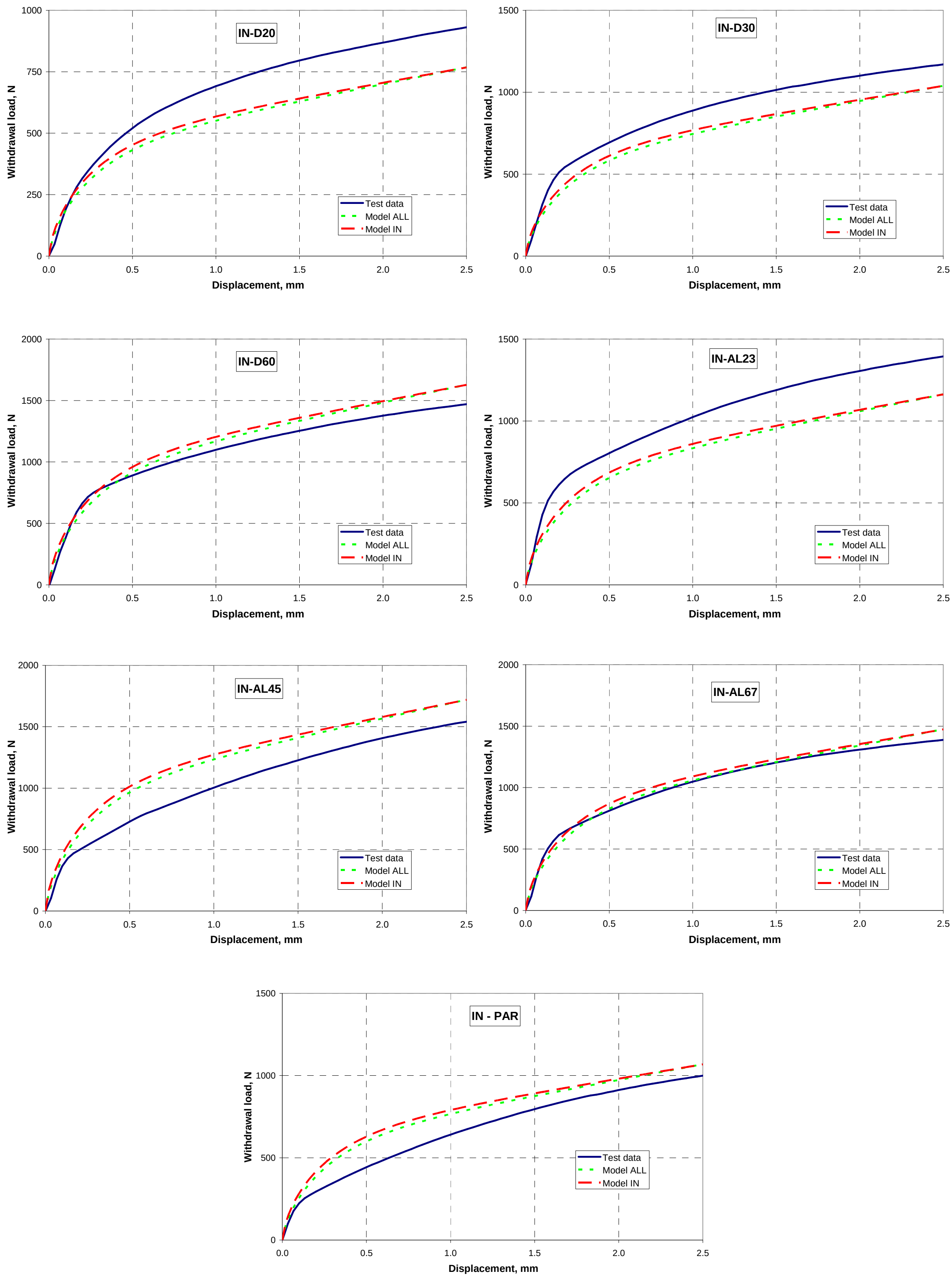


Figure 4.18 continued: Withdrawal load displacement relationship from test and predicted from semi empirical models for InSkew fasteners.

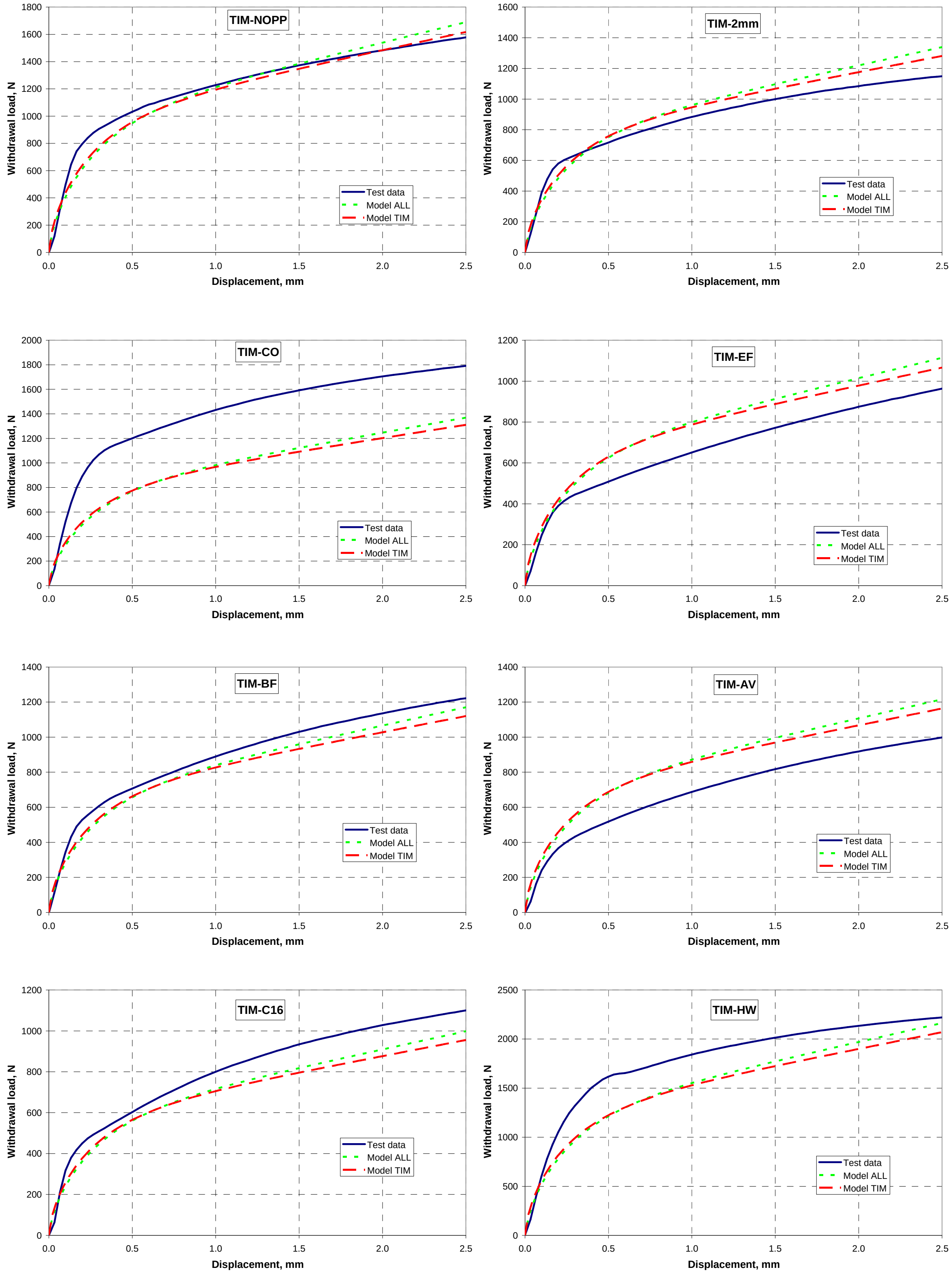


Figure 4.19: Withdrawal load displacement relationship from test and predicted from semi empirical models for TimTie fasteners.

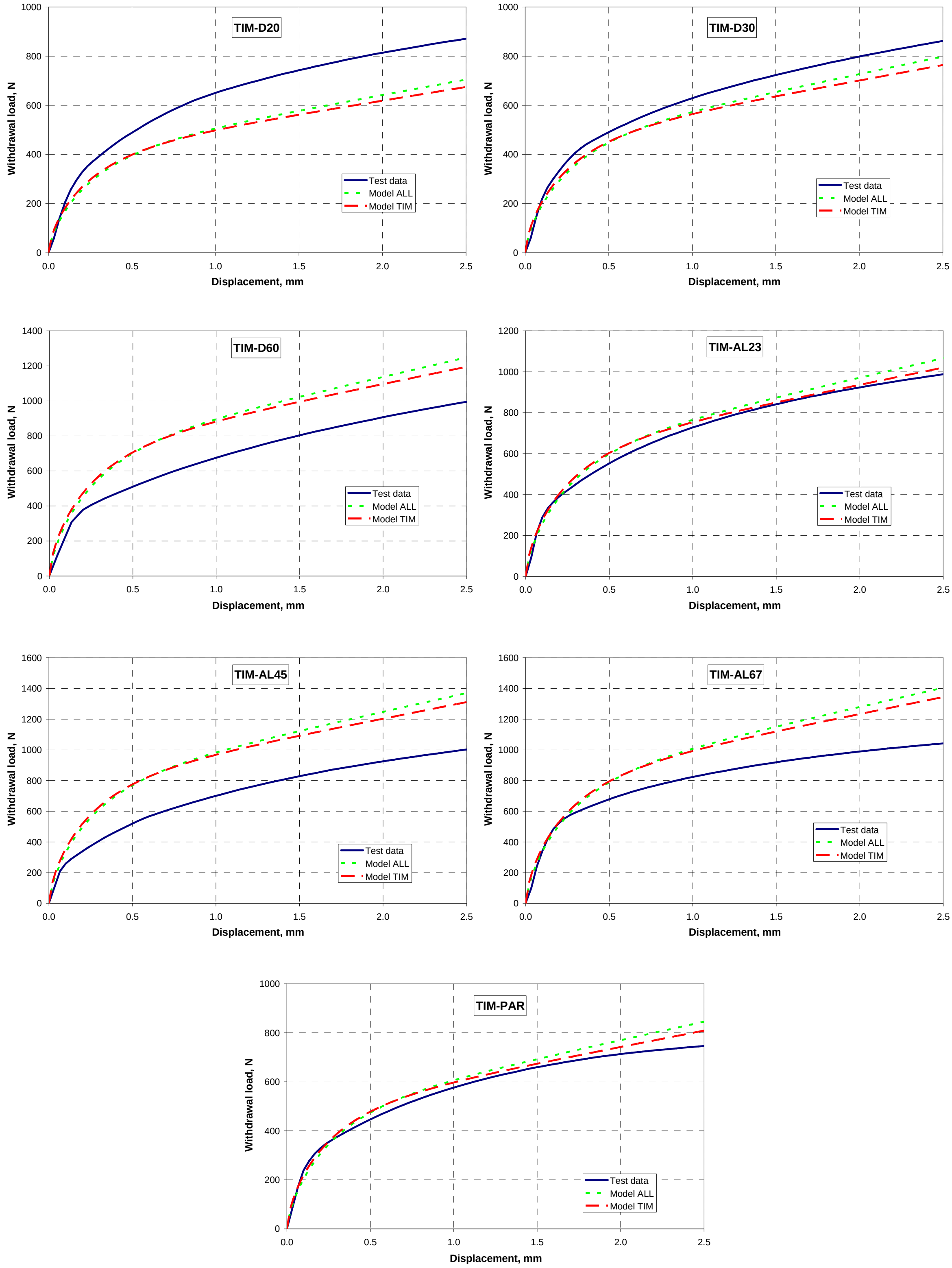


Figure 4.19 continued: Withdrawal load displacement relationship from test and predicted from semi empirical models for TimTie fasteners.

4.5 Summary and conclusions

In this chapter the withdrawal behaviour and resistance of helically shaped fasteners in timber was investigated, and compared to those of conventional timber fasteners.

It has been shown that, compared to conventional nails (threaded and smooth) helically shaped fasteners can attain higher withdrawal loads, but the maximum loads were achieved by woodscrews. However results also showed that helically shaped fasteners had similar stiffness to woodscrews.

The withdrawal strength was calculated according to BS EN 1382 (BSI, 1999); the results showed that, when the results are compared for the different types of fasteners used in this study, the equation given did not reflect the withdrawal capacity of helically shaped fasteners. A better representation of the withdrawal behaviour of helically shaped fasteners was found to include the fastener perimeter and actual (or projected) depth of penetration in timber, in order to accurately use the contact area between the fastener and timber in the calculations. The test results were also compared to the design equations given in Eurocode 5 (BSI, 2004) for screws and nails. The comparison demonstrates that the equations in the various standards for determining or predicting the withdrawal strength of fasteners cannot be applied to helically shaped fasteners.

In view of these observations, an extended experimental programme was developed to investigate the parameters that were considered important on the withdrawal behaviour and resistance of helically shaped fasteners. The factors investigated were the diameter of pilot hole, timber density, depth of penetration in timber and angle to the timber grain. From the results of the extended test programme semi-empirical models were developed for helically shaped fasteners in timber to a displacement of 2.50mm, on the assumption that there is no significant interaction between the parameters. No evidence was found that the factors have significant interaction.

The results show that the withdrawal strength of helically shaped fasteners is directly proportional to their perimeter and depth of penetration in timber. As for conventional timber fasteners, as the timber density increases the withdrawal resistance of helically shaped fasteners increase, in a power function; and the maximum withdrawal resistance was attained when the fasteners were inserted perpendicular to the timber grain. The results also confirmed that maximum withdrawal resistance was achieved without predrilling. However it should be noted that the insertion of helically shaped fasteners is improved with predrilling.

Based on these results semi-empirical models were developed for each size of helically shaped fasteners, and regrouping all diameters studied. By inputting the test properties in the models, the strength and load displacement behaviour up to 2.50mm displacement can be computed. This showed that customised model per fastener diameter did not improve on the generic model regrouping all diameters, and therefore the generic model should be used. The model predicts withdrawal loads for helically shaped fasteners to a displacement of 2.50mm and gave an average error of 10.44%.

Chapter 5 Laterally loaded connections with helically shaped fasteners

5.1 Introduction

Dowel type fasteners are mostly used for connecting members in the same plane and therefore are loaded in shear. Over the past decades researchers have used similar arrangements. The samples usually consist of a timber member sandwiched between two other members that are of timber, timber based material or steel, with connectors penetrating the members and acting in single or double shear under lateral loading. In order to investigate and evaluate the performance of laterally loaded timber joints with helically shaped fasteners, lateral shear tests were performed with helically shaped fasteners and common timber fasteners.

The first test series aimed to compare the behaviour and performance of helically shaped and conventional timber fasteners, when loaded in single and double shear. The subsequent test series were then developed to investigate the factors that may influence the behaviour and performance of helically shaped fasteners laterally loaded in single and double shear. From this experimental programme a semi empirical model is developed for simulating and predicting the lateral shear performance and behaviour of helically shaped fasteners. The analysis considered the effects of the timber, and nailing configuration.

5.2 Tests set up and procedures

The lateral capacity of joints with dowel type fasteners was determined in accordance to BS EN 26891:1991 – Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics (BSI, 1991). The fasteners used in this research are shown in Table 3.1 and their characteristic detailed in Table 3.1.

The research programme was divided into three stages:

- Comparison between helically shaped and conventional timber fasteners laterally loaded in timber,
- Investigation of timber joints with helically shaped fasteners loaded in single shear,

- Investigation of timber joints with helically shaped fasteners loaded in double shear.

The first stage of the experimental programme aimed at evaluating the performance and load displacement behaviour of laterally loaded helically shaped fasteners in timber compared to conventional fasteners. It also aimed at evaluating the recommendation of Eurocode 5 on the geometry of timber joints: Loaded/Unloaded end or edge, pointside penetration and overlapping length.

The second and third stages aimed at investigating the parameters that may respectively influence the single and double shear capacity and load displacement behaviour in timber. The factors that may influence timber connection with helically shaped fasteners include joint configuration variables, material properties and dimensions of the fastener and timber. The loading conditions were kept constant during the experimental programme.

5.2.1 Sample fabrication procedure

Preliminary tests performed with helically shaped fasteners showed that when they are inserted through pilot holes in multiple timber elements, the alignment of the pilot holes and the possible deviation of the pilot hole are likely to influence the performance of the connection. Misalignment of the pilot hole will create difficulties during insertion and may in turn influence the experimental results. In order to avoid any problems a drilling procedure for pilot holes was put in place.

The timber was cut by the laboratory staff with great care in order to produce samples that presented the minimum possible defects. On the samples the pilot hole positions were marked before drilling with a vertical drill. To avoid any deviation of the drill bit, the pilot holes were first drilled with a starter drill bit, to mark the position precisely. When all the pilot holes were marked, the pilot holes were then drilled to the relevant diameter pilot hole. In order to keep the spacing between the pilot holes constant a drilling jig was built and used for drilling all the samples, Figure 5.1.

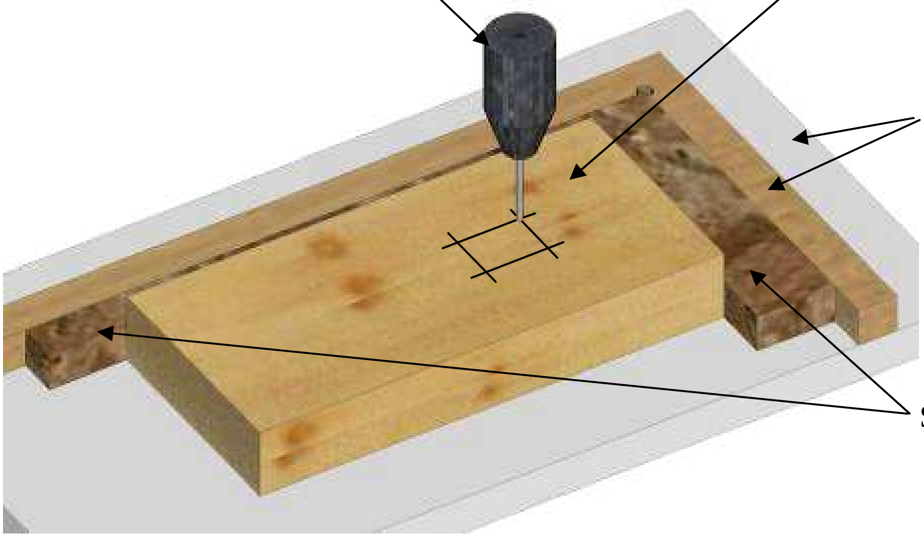


Figure 5.1: Drill set up of timber samples.

The square frame was glued on to a base support in order to keep the same frame for all the samples. The spacing blocs were cut from particle board with high content of resin which greatly reduced the dimensional variation due to the varying temperature and humidity. For each set of samples the support was clamped to the vertical drill base, and all the samples were drilled in one lot to avoid set up variations. This set up and procedure made the drilling phase a long one, but proved very effective in eliminating set up variations within a set of samples, in eliminating any misalignment of the pilot holes, and in eliminating any deviation of the pilot hole.

5.2.2 Test procedure

The timber used in the tests was stored for a period of two months before the tests to achieve constant moisture content. Samples were cut, and clear specimens were chosen for the tests, however within a specimen, small knots and variation in the slope of the timber fibres were permitted provided they were unlikely to significantly reduce the specimen strength, or have any influence on the test behaviour or result. When cut to the relevant dimensions for the research, the samples where predrilled according to the procedure described above with pilot holes of diameter 3.50mm for StarTie 10, 3.00mm for StarTie 8, 2.70mm for InSkew and 2.40mm for TimTie fasteners, or if the number of samples was too large for testing during the day, they were put back in the storage area to be tested at a later date. This procedure allowed for the samples to be cut in one operation which reduced the risk of dimensional differences, and drilling occurred before testing, in order to minimise fibre relaxation.

For each test series four samples were tested. The samples were fabricated just before testing. Only the samples of a same test series were fabricated and then tested within one hour, in order to avoid fibre relaxation around the fasteners. All nails were manually hammered, and the screws inserted with an electrical drill. Helically shaped fasteners were hammered into the timber using a hand-held tool acting as a sleeve and transmitting the impact force. This tool was provided by Helifix Ltd, and is used for standard installation into masonry or timber. It also offers the advantage of restraining the free length of the fastener and prevents bending that might occur when using a hammer alone for inserting helically shaped fasteners.

The samples in this part of the research were tested according to BS EN 26891:1991 (BSI, 1991), or to industry standard when necessary. As the range of samples and fixings vary greatly it was decided to set the value of the estimated load capacity of all samples, F_{est}, mentioned in BS EN 26891:1991 to 500N. The value of 500N was chosen as it is sufficiently high to eliminate the slack in the testing sample and machine. Also it is sufficiently low to ensure that for samples with predicted lateral shear capacities relatively small – i.e. samples with one fastener only in single or double shear – the estimated load was not in the plastic stage of the connection load displacement behaviour.

As the standard allows the pre-cycle load to 40% of the estimated load, and the period of constant loading were omitted from the load cycle. The value of 500N was chosen as the most relevant value, which would be representative for all samples and eliminate the slack in the testing machine. The loading procedure for all samples is shown in Figure 5.2.

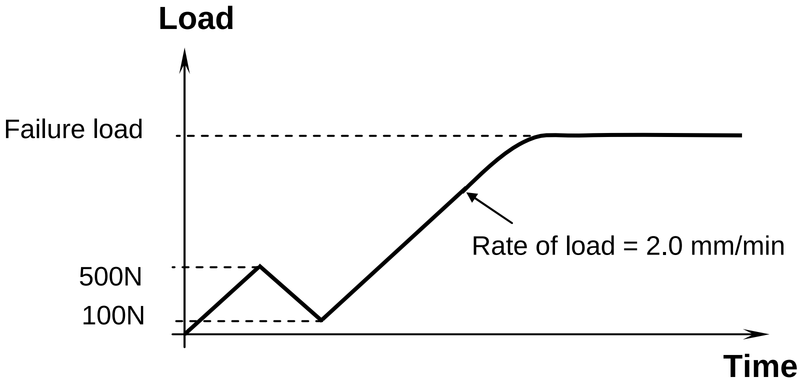


Figure 5.2: Loading procedure for laterally loaded joints

The displacement was recorded using two brackets screwed on the side of the middle timber member on which displacement transducers were positioned, with the joint displacement taken as the average of the two measurements. The load was recoded using a 50 kN load cell, placed between the travelling head of the testing machine and the sample, steel plates were placed on top of the samples to act as load spreader. A typical test set up is shown in Figure 5.3. Following testing, small clear samples were taken out of the timber members of the joint in order to measure the sample density and moisture content.

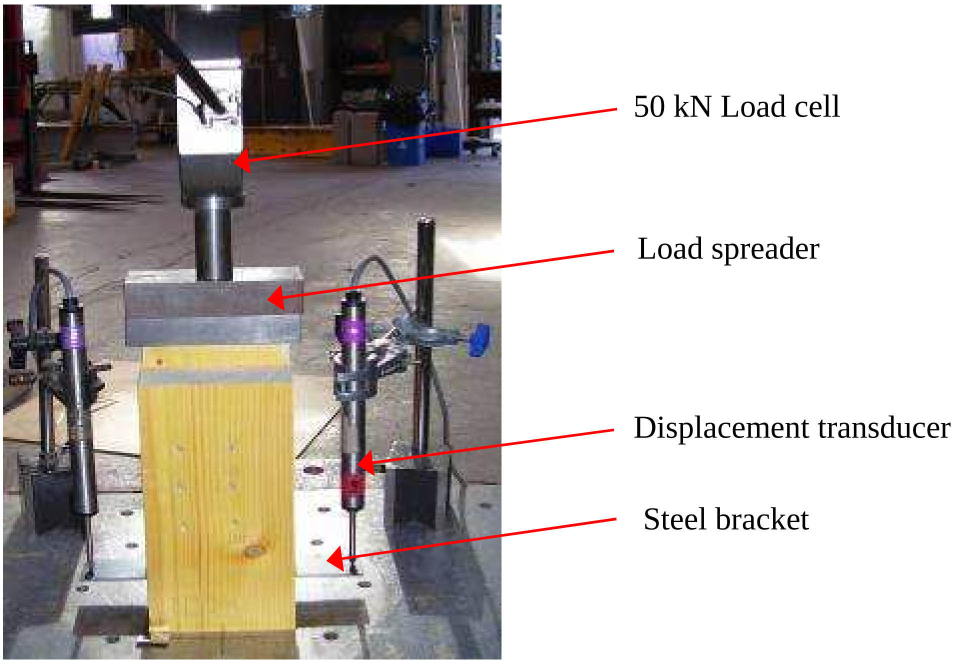


Figure 5.3: Lateral shear tests set up

Test data was recorded using a numerical data logger, with load and displacement reading logged every second. The data was then processed on the software Excel, with the start of the load displacement relationship starting at the beginning of the reload cycle.

The experimental programme developed for the investigation of the behaviour and performance of helically shaped fasteners included tests on single and double shear joints. Due to the testing machine used in the experimental research, samples constituted of only two timber members, for single shear tests loaded in tension or compression, can not be tested as such set up creates an eccentricity in the sample, Figure 5.4. In order to avoid eccentricity in the samples it was decided to investigate the single shear performance of helically shaped fasteners in timber using a “double single shear” set up. This set up, shown in Figure 5.3, consisted of three timber members, with the middle one used for two sets of single shear joints with the side members. As the joints are tested in “double single” shear the nails inserted in the side members were staggered in order to permit the minimum spacing in the middle member according to Eurocode 5 (BSI, 2004).

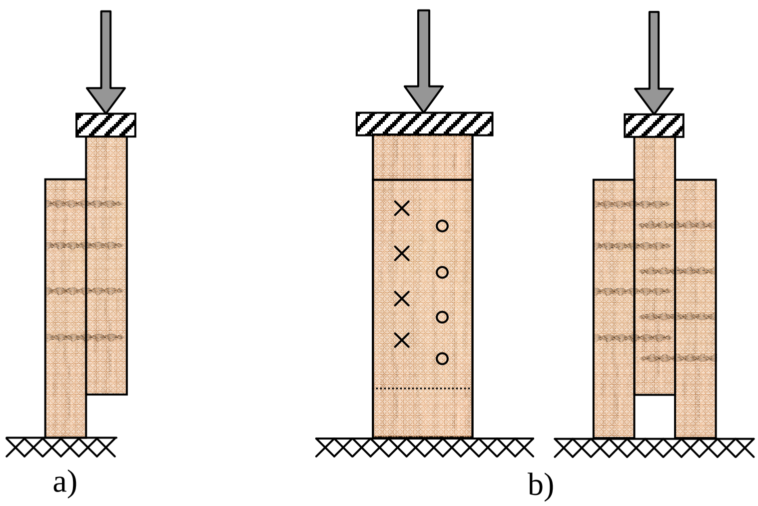


Figure 5.4: a) Single shear test; b) “Double single shear” set up used in the experimental programme.

5.2.3 Minimum spacing and distances

Eurocode 5 (BSI, 2004) recommends the following minimum distances and spacings for nails, and screws with a diameter of 6mm or less, with predrilled holes, shown in Figure 5.5:

- Spacing parallel to the grain, a_1 5d
- Spacing perpendicular to the grain, a_2 4d
- Distance to loaded end, $a_{3,t}$ 12d
- Distance to unloaded end, $a_{3,c}$ 7d
- Distance to loaded edge, $a_{4,t}$ 5d
- Distance to unloaded edge, $a_{4,c}$ 3d

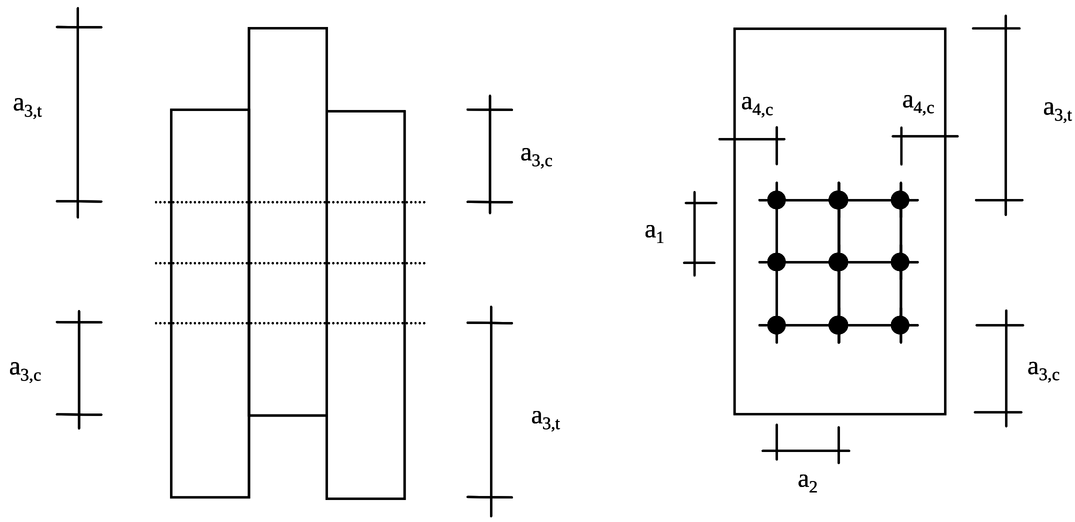


Figure 5.5: Spacings and end/edges distances according to EC5 (BSI, 2004)

As the diameter of helically shaped fasteners varies around its perimeter, the diameter to be used as reference for determining the minimum spacing and distances needed to be determined. Samples made of timber of strength class C24 were tested. First the root diameter of helically shaped fasteners was used for calculating the minimum spacings according to the factors recommended in Eurocode 5 (BSI, 2004). The samples nailing pattern was as shown in Figure 5.5; once fabricated the samples showed that the timber was split along the grain and between the fasteners, under the internal forces created with their insertion. Splits were observed on all samples. This suggests that when using the root diameter of helically shaped fasteners the minimum spacing recommended by EC5 is not sufficient.

Further tests were conducted in order to determine a suitable parallel to the grain spacing for helically shaped fasteners. As the spacing recommended in EC5 of 5d was not adequate as it induced splitting of the timber, a new spacing of 8d was tested. Four fasteners were inserted in the timber in a row parallel to the timber grain, with a minimum of spacing of 8d, with predrilling pilot holes of 0.8 times the root diameter of the fasteners, resulting in parallel to the grain spacing of 21mm for TimTie, 24mm for InSkew, 27mm for StarTie 8 and 30mm for StarTie 10 Helically shaped fasteners respectively. These samples showed no signs of slipping along the timber fibres, and therefore spacing parallel to the grain of 7d, with d the root diameter, for helically shaped fasteners was adopted.

Perpendicular to the grain, the minimum recommended spacing by Eurocode 5, seemed sufficient following inspection of the samples fabricated during the preliminary tests. A spacing perpendicular to the grain of 5 times the root diameter was used during the experimental programme. The minimum distance to unloaded edge from Eurocode 5 is 3d, it was assumed that the distance to the edge does not influence the behaviour and load carrying capacity of laterally loaded joints, the distance to unloaded edge was kept constant for all samples to 25mm. The minimum distances to loaded and unloaded end were first chosen to respect the Eurocode 5 criteria for the largest diameter (i.e.: StarTie 10), respectively 60mm and 30mm. However preliminary samples have shown that the maximum load occur at large displacements, often over 40mm. So a minimum clearance of 50mm was chosen for the samples, allowing the tests to reach the maximum loads, resulting in larger loaded end distance, as it is shown that it is more critical than the unloaded end distance.

In summary the minimum spacings and distances used in the experimental programme with helically shaped fasteners were as follow:

- Min spacing parallel to the grain, a_1	8d
- Min spacing perpendicular to the grain, a_2	5d
- Distance to loaded end, $a_{3,t}$	80mm
- Distance to unloaded end, $a_{3,c}$	30mm
- Distance to loaded edge, $a_{4,t}$	25mm
- Distance to unloaded edge, $a_{4,c}$	25mm

Where d is the root diameter of the fastener, the spacings between fasteners were rounded to the nearest millimetre.

Using the minimum spacings and distances summarised above, several test series were designed for investigating the behaviour and resistance of helically shaped fasteners in timber to timber connections. Details of the various tests series are shown in Appendix

B. The first test aimed to study the connection behaviour of single fastener connection in double and single shear, and comparing helically shaped fasteners with commonly used timber connectors. The subsequent series investigate the connection parameters influencing the connection behaviour in single and double shear.

It is important to note, that the spacing used and observation described in the above paragraphs have not been investigated fully, and were used in the experimental programme as spacing and distances which did not influence the sample integrity before tests. A full investigation may be necessary on the spacing and edge and end distances of helically shaped fasteners in different grades of timber in order to fully analyse their influence.

5.3 Single fastener joints

In this first part of the research on timber joints with helically shaped fasteners, samples with a single fastener, loaded in single and double shear were investigated; helically shaped fasteners were tested along with common timber fasteners. The aim of this investigation was to compare the fasteners behaviour and performances, but also with to investigate the diameter effect of helically shaped fasteners. The nailing configurations used in this test series are detailed in Appendix B.

5.3.1 Comparison between timber fasteners

Helically shaped and commonly used timber fasteners were tested in single fastener joints in single shear as detailed in test series AB shown in Appendix 5.1. The samples were fabricated as described in the section above, and were predrilled with pilot holes of diameter 3.50mm for StarTie 10, 3.00mm for StarTie 8, 2.70mm for InSkew and 2.40mm for TimTie fasteners. The timber used in this test programme was of grade C24, the samples measured 45mm in thickness, helically shaped fasteners measured 90mm for tests in single shear. The results of the tests with helically shaped fasteners loaded in single shear were obtained by dividing the load on the test samples by half, in order to obtain the load per shear plane. For comparison purposes common timber fasteners were laterally tested in single shear. The samples were of timber grade C24 – in accordance with DS EN 338:2003 Structural Timber – Strength Classes (BSI, 2003), and were of the same dimensions than for helically shaped fasteners. The fasteners tested were UMW-5 screws, BZP-10 screws, BZP-12 screws, ATN375 ring shank nails and HTN3 helical nails; dimensional details and pilot hole diameters are given in Chapter 3.

For each set of tests the maximum load per shear plane, P_{max}, was determined and the slip at maximum load, δ_{max} noted. The yield loads from the load displacement curves were also determined for the fasteners; the yield load was taken as the intersection of the two tangents of the linear parts of the curves, Figure 5.6. The ductility ratio, R_D, of the joints was also calculated, it is taken as the ratio of slip at maximum load, δ_{max}, to slip at yield load, δ_y (Smith et al., 2005).

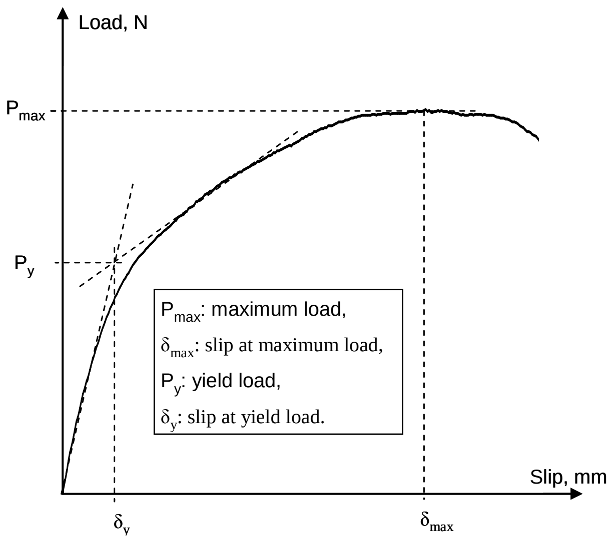


Figure 5.6: Test data load and displacement notation

Table 5.1 contains the single fasteners tests results for helically shaped and common timber fasteners tested in single shear.

Table 5.1: Single fastener test results

Single shear test results					
Fasteners	P_{max}	δ_{max}	P_y	δ_y	R_D
	N	**mm**	**N**	**mm**	
StarTie 10	4597.8	33.36	2814.5	3.50	9.52
StarTie 8	3828.1	33.56	2532.3	3.64	9.21
InSkew	3522.4	30.93	1969.7	2.57	12.03
TimTie	3260.0	29.25	1877.4	1.80	16.23
UMW5	3099.1	5.19	2001.0	0.97	5.33
BZP-10	3011.9	7.94	1674.3	1.07	7.44
BZP-12	4379.3	7.57	2291.1	0.84	8.98
ATN375	2619.1	12.79	1676.8	1.92	6.65
HTN3	2080.6	15.26	1671.0	1.95	7.83

The typical load displacement relationships for the fasteners tested in single shear are shown in Figure 5.7. It shows that helically shaped fasteners behave in a more ductile behaviour than common timber fasteners, particularly compared to wood screws which exhibited a brittle behaviour. The modes of failure, shown in Figure 5.8, also reflect the load displacement relationship of the fasteners.

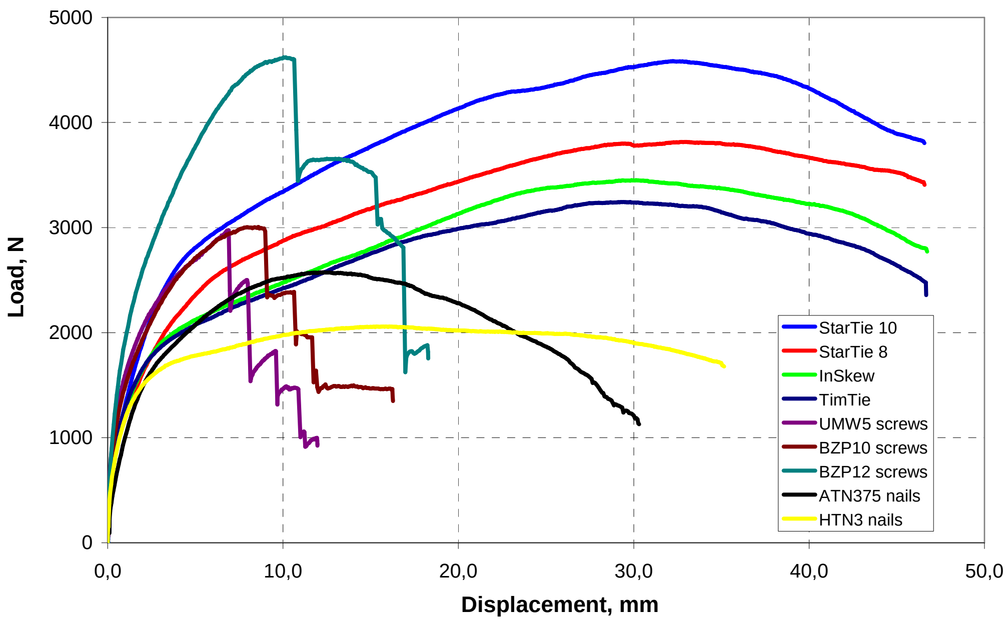


Figure 5.7: Typical load displacement relationships of single fastener tests

The modes of failure show that the brittle failure of the wood screws connections is due to bending failure of the screws. Yield moment tests showed that when loaded in bending – for which details were given in Chapter 3 – the screws used in this experimental programme exhibited a brittle failure, therefore such failure in laterally loaded joints was to be expected. Threaded nails exhibited a ductile behaviour when laterally loaded; the mode of failure observed for such samples was crushing of the timber at the interface of the members under the nail, and a yield point in the nail in the head point member. Helically shaped fasteners exhibit similar failure modes to treaded nails, with yield points in both timber members and crushing of the timber under the fastener at the interface of timber members. However, samples with helically shaped fastener joints showed horizontal displacement of the fastener in the head and point side members due to the vertical displacement of the joint.

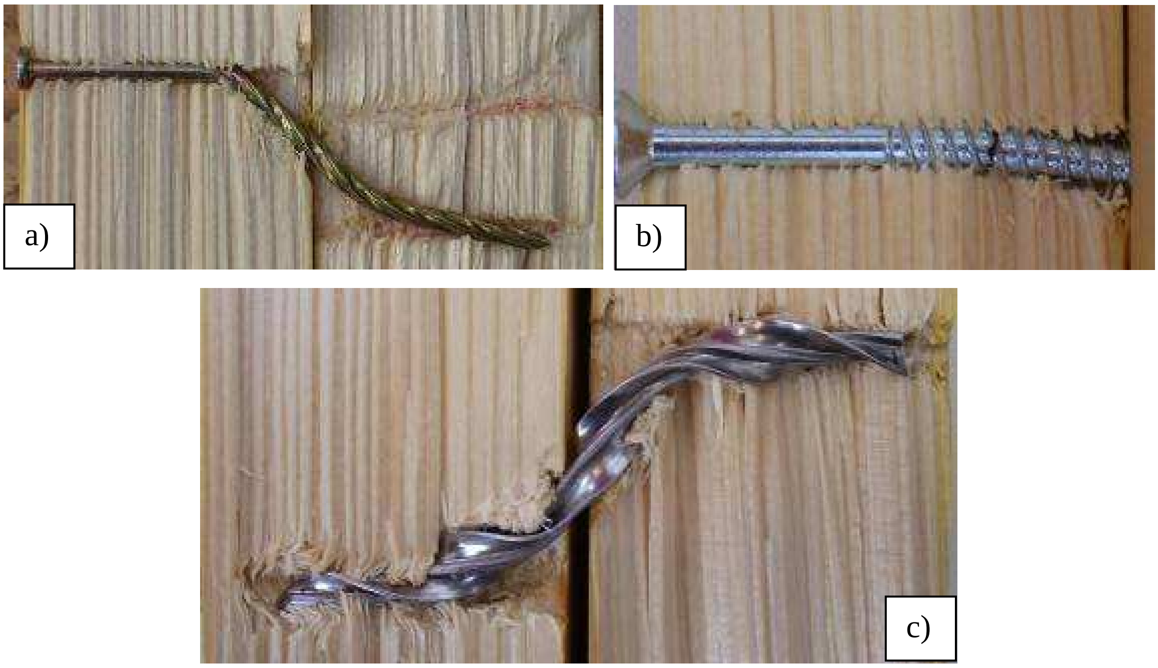


Figure 5.8: Typical failures for a) threaded nails; b) wood screw, c) Helically shaped fasteners

From a structural engineering point of view, the yield load corresponds to a stage of transition between elastic and plastic behaviour. At yield load irreversible damage is caused to the timber joint. The results presented in Table 5.1 show that the yield point is achieved at similar or greater loads for helically shaped fasteners; they also confirm that the larger helically shaped fasteners (StarTie 10 and 8, and InSkew) reached the yield loads at relatively large displacements compared to screws, and threaded nails to a smaller extent. In addition, the results show that helically shaped fasteners joints result in greater ductility ratios, which can be explained by the joints reaching maximum loads at large displacement. Smith et al. (2005) found that the ductility ratio is related to the fastener slenderness is confirmed for helically shaped fasteners.

These results demonstrate that helically shaped connections can achieve similar loads to common timber fasteners of similar length and diameter, but also that helically shaped fasteners compare favourably to these timber connectors for the following reasons:

- Failure loads are achieved at very large displacement,
- Failures are ductile compared to brittle screws failures,
- Yield points are reached at larger displacement

5.3.2 Double shear connections and fastener overlap

Test series AB described above consisted of single helically shaped fasteners being tested in single shear. In order to fully understand the behaviour of single fastener connections with helically shaped fasteners further tests were performed on single fastener connections. Details are shown in Appendix B.

- ***Test series AC:*** Fastener in double shear, predrilled as for series AB, the fasteners measured 135mm in length,
- ***Test series AD:*** Fasteners in single shear, overlapping in the middle member in accordance with Eurocode 5;
- ***Test series AE:*** Fasteners in single shear, overlapping in the middle member over its full thickness;
- ***Test series AF:*** As series AB with pilot holes of 4.80mmfor StarTie 10, 4.00mm for StarTie 8, 3.50mm for InSkew and 3.30mm for TimTie fasteners;
- ***Test series AH:*** As series AC with pilot holes of 4.80mmfor StarTie 10, 4.00mm for StarTie 8, 3.50mm for InSkew and 3.30mm for TimTie fasteners.

The results of the tests on connections with single helically shaped fasteners are detailed in Table 5.2; the load per shear plane is given for test series AB, AD, AE and AF.

Table 5.2: *Single fastener tests results*

Test series	Fastener	Overlapp (1)	Pilot hole (2)	Shear planes	P_{max} N	δ_{max} mm	P_y N	δ_y mm	R_D
Single shear samples									
AB	StarTie 10	N.A.	$0.8*d_r$	1	4597,8	33,36	2814,5	3,50	9,52
	StarTie 8	N.A.	$0.8*d_r$	1	3828,1	33,56	2532,3	3,64	9,21
	InSkew	N.A.	$0.8*d_r$	1	3522,4	30,93	1969,7	2,57	12,03
	TimTie	N.A.	$0.8*d_r$	1	3260,0	29,25	1877,4	1,80	16,23
AD	StarTie 10	EC5	$0.8*d_r$	1	2949,2	17,99	2180,0	3,08	5,84
	StarTie 8	EC5	$0.8*d_r$	1	2667	19,35	2420,6	4,16	4,66
	InSkew	EC5	$0.8*d_r$	1	2176,8	20,37	1863,6	2,82	7,21
	TimTie	EC5	$0.8*d_r$	1	1895,0	20,27	1550,1	2,85	7,12
AE	StarTie 10	FULL	$0.8*d_r$	1	5712,3	35,90	3114,6	3,08	11,64
	StarTie 8	FULL	$0.8*d_r$	1	4967,0	36,50	3209,4	3,87	9,42
	InSkew	FULL	$0.8*d_r$	1	4154,4	35,37	2418,9	2,61	13,58
	TimTie	FULL	$0.8*d_r$	1	3226,5	34,95	1994,2	2,49	14,04
AF	StarTie 10	N.A.	$1.0*d_r$	1	4288,6	40,94	3090,9	5,25	7,80
	StarTie 8	N.A.	$1.0*d_r$	1	3741,6	41,34	2220,2	4,62	8,94
	InSkew	N.A.	$1.0*d_r$	1	2805,0	38,02	1701,1	3,76	10,12
	TimTie	N.A.	$1.0*d_r$	1	2548,3	37,35	1488,7	3,46	10,79
Double shear samples									
Test series	**Fastener**	**Overlapp (1)**	**Pilot hole (2)**	**Shear planes**	**P_{max} N**	**δ_{max} mm**	**P_y N**	**δ_y mm**	**R_D**
AC	StarTie 10	N.A.	$0.8*d_r$	2	9361,3	26,84	5792,9	3,50	7,67
	StarTie 8	N.A.	$0.8*d_r$	2	6674,3	28,83	4681,1	3,80	7,59
	InSkew	N.A.	$0.8*d_r$	2	6121,0	28,63	4327,2	3,02	9,50
	TimTie	N.A.	$0.8*d_r$	2	4712,8	28,41	2940,9	2,64	10,76
AH	StarTie 10	N.A.	$1.0*d_r$	2	9525,0	30,48	5479,6	4,34	7,03
	StarTie 8	N.A.	$1.0*d_r$	2	7658,0	28,63	5669,6	4,71	6,09
	InSkew	N.A.	$1.0*d_r$	2	5702,0	29,67	3611,8	3,86	7,68
	TimTie	N.A.	$1.0*d_r$	2	4887,5	27,66	3218,1	3,72	7,43

(1) The fasteners overlapp in the middle member according to EC5 design rules or over the full thickness of the timber member

(2) The pilot holes are factors of the fasteners root diameter (d_r)

Test series AB, AD and AE aimed to investigate the fastener overlap in the middle member. In test series AB the fasteners were not overlapping in the middle member in order to investigate the behaviour of single shear connections, in test series AD the fastener overlapping recommendations of Eurocode 5 were respected (the distance from the point of the nail to the end of the member should be at least 4d, with d the fastener diameter). In test series AE the fasteners overlapped over the full length of the timber member.

The results of the investigation on fastener overlap show that for the test series following the recommendation of Eurocode 5 (BSI, 2004), the joint capacity was greatly reduced, and as shown in Table 5.2 the maximum load was reached at lower displacement. The fastener overlap rule of Eurocode 5 was designed for fasteners which, when overlapping occurred, provoked early splitting of the timber, and therefore reduced joint capacity. The results with helically shaped fasteners show that when the fasteners overlap over the full length of the timber member no premature timber splitting occur, and the load displacement behaviour was not affected by such a nailing configuration, see Figure 5.9. This may be explained by the fact that the cross section of helically shaped fasteners is not constant over its perimeter, and the splitting forces created by driving the fastener act in different directions over the length of the fasteners, when common round fasteners impose forces to the timber fibres in the same direction over their length. This distribution of the forces with helically shaped fasteners reduces the risk of splitting, and allows for full overlapping of the fasteners in connections. This finding is also confirmed by the fact that the results of test series AE are greater or equivalent in the case of TimTie fasteners, than the results for test series AB.

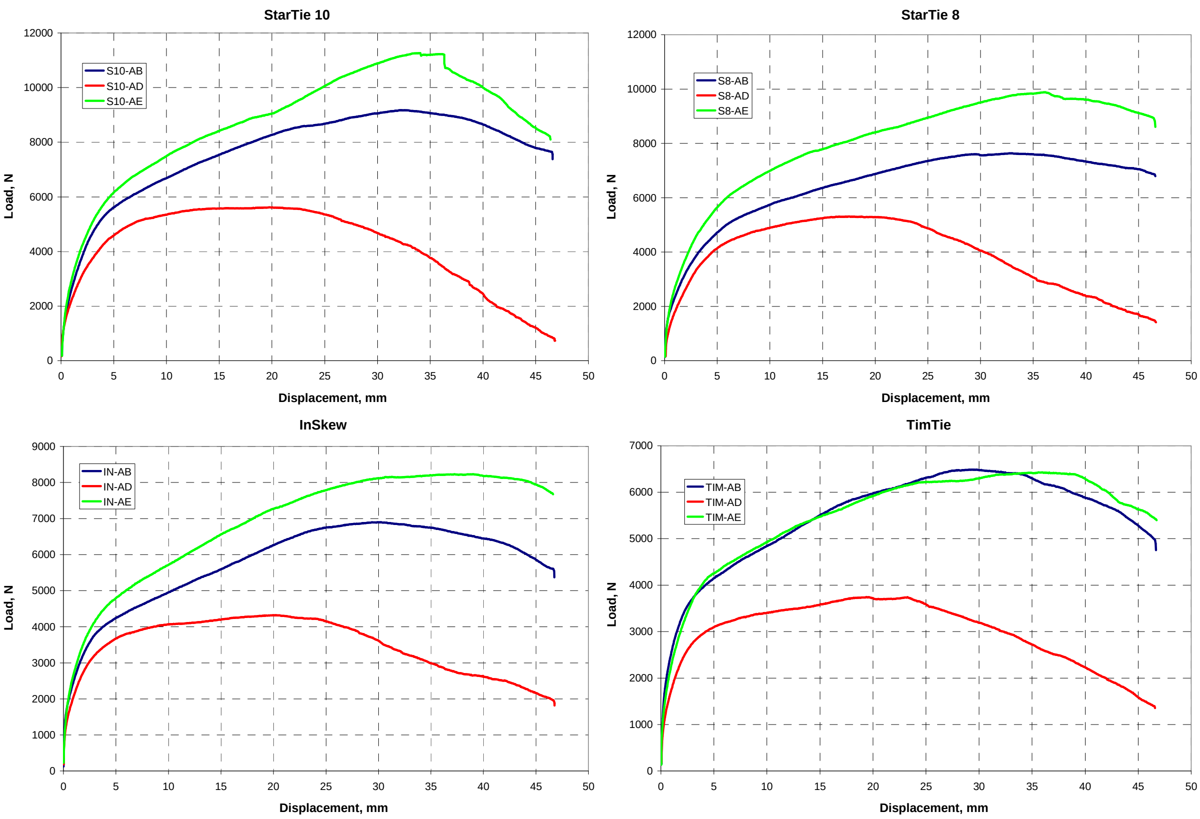


Figure 5.9: Effect of fastener overlap on joint behaviour

Numerous researches have shown that the joint resistance is affected by predrilling of the timber, with the joint resistance decreasing with increasing predrilling diameter. For Helically shaped fasteners, tests on the effect of pilot holes diameters on their withdrawal behaviour showed that with increasing pilot hole diameter the withdrawal load decreases. Following these observations, a similar behaviour is expected with helically shaped fasteners, therefore the test series AF and AH aimed to investigate the performance of joints with pilot holes equivalent to the root diameter of the fasteners.

A comparison between the tests results with pilot holes of 0.8*d and pilot holes of 1.0*d (with d the fastener root diameter) is shown in Figure 5.10 for all fasteners.

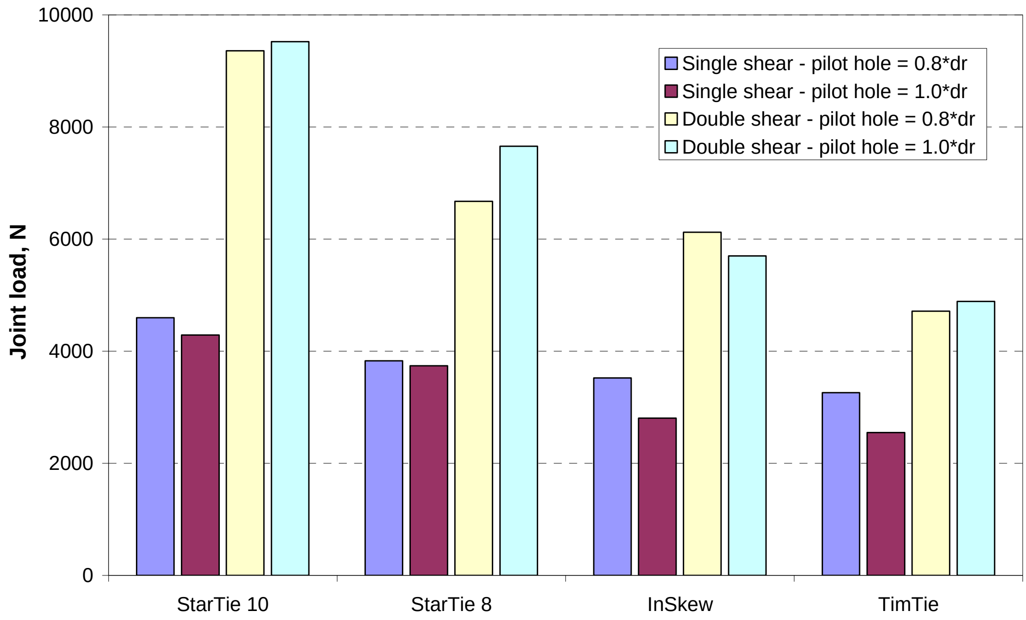


Figure 5.10: Effect of pilot hole on joint load

The results show that increasing the pilot hole diameter has an adverse effect on the joint resistance, on average the difference for connection with increased pilot hole is a 4.7% reduction in strength. This is particularly evident for connection with fasteners loaded in single shear, and more so for fasteners of smaller diameter. For connections with fasteners loaded in double shear the tests results do not show a trend, with for StarTie 10, StarTie 8 and TimTie fasteners the connection load increases with increasing pilot hole, and for InSkew fasteners the connection load is decreased with increasing pilot hole. In view of these observations, it can be concluded, as expected, that with increase in pilot hole size the connection resistance is decreasing, however no clear trend can be defined based on these results.

5.3.3 Influence of Helically shaped fastener diameter on connection behaviour

This part of the research programme investigated the behaviour of timber to timber connections with helically shaped fasteners. Four sizes of helically shaped fasteners have been tested in a variety of configurations. In order to study the effect of the fastener diameter on the connection resistance, the root and thread diameters were plotted against the joint load for the various joint configurations detailed in Appendix B, see Figure 5.11.

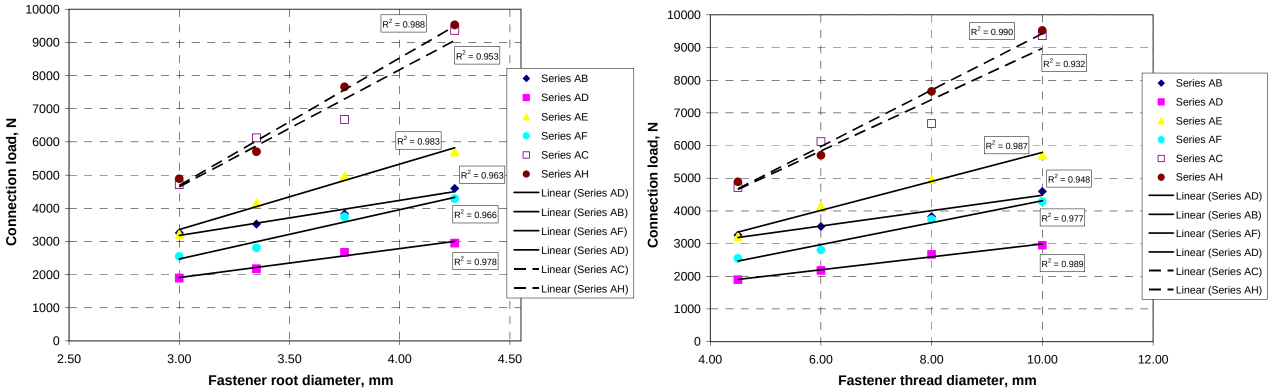


Figure 5.11: Load connection vs. fastener root and thread diameter

Figure 5.11 shows the connection load to fastener root and thread diameter relationship, for the different connections tested. It is clear from the results that the performance of a joint with helically shaped fastener is directly proportional to the fastener diameter, with the R^2 values ranging for the different test series from 0.932 to 0.990. This finding correspond to the general understanding of timber connections with dowel type fasteners, with most design models taking the fastener diameter as a direct factor to determine the connection performance.

5.4 Multiple fasteners shear tests

Following lateral shear tests on single Helically shaped fasteners, and comparison with common timber fasteners, detailed in the section above, the experimental programme explored multiple fastener Helically shaped connections. The aim was to investigate the joint geometry parameters that may influence the connection behaviour and performance.

Single fasteners tests showed that the performance of helically shaped fasteners was directly proportional to the fastener diameter. In view of this, the experimental programme was designed with one fastener diameter for the single and double shear tests as to reduce the number of replicas needed for each nailing configuration. In order to avoid unrepresentative results due to the relatively large range of diameters, it was decided to use fasteners of diameter as close as possible to the average helically shaped fasteners diameter. Therefore single shear tests were performed with the helically shaped fasteners InSkew and double shear tests performed with StarTie 8 fasteners.

5.4.1 Factors investigated

The experimental programme on laterally loaded helically shaped fasteners in connections aimed to investigate the joint geometry parameters that may influence their performances and behaviour; the parameters included:

- Effective number of fasteners in a line;
- Number of fasteners in a row;
- Row of fastener spacing;
- Number of fasteners in a line;
- Line of fasteners spacing;
- Timber density;
- Nailing geometry.

The lines and rows in a joint are shown in Figure 5.12. The material properties and loading conditions were kept constant as detailed in the sections above during the testing programme. The nailing configurations are shown in Appendix 5.2 and 5.3. Except when investigating the material properties the timber used in this test programme was of grade C24 with moisture content of 10±1%, the samples measured 45mm in thickness, and the fasteners measured 90mm and 135mm for tests in single and double shear respectively. During the investigation of a parameter, all other factors that were thought to have an influence on the connection behaviour and performance, were kept constant.

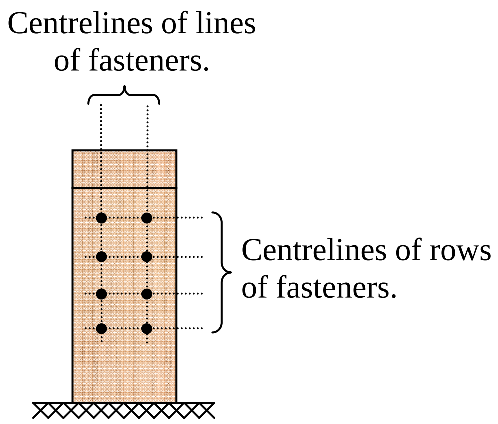


Figure 5.12 : Centrelines of rows and lines of fasteners in a connection

The experimental programme first investigated the effective number of fasteners in a line parallel to the timber grain. As detailed above, the spacing between fasteners was 8d, with d the fastener root diameter. In order to minimise the number of tests the samples tested comprised two, four, six eight and ten fasteners in a line. Previous research on effective number of fasteners showed that joint load is not uniformly distributed between the fasteners in a line with the fasteners at the extreme of the lines taking greater share of the connection load (Blass, 1990). Therefore the connection resistance for such nailing configuration is dependent on the effective number of fastener. Zahn (1991) showed that due to the unequal load sharing between fasteners in a line, the connection load reaches an upper limit, Figure 5.13.

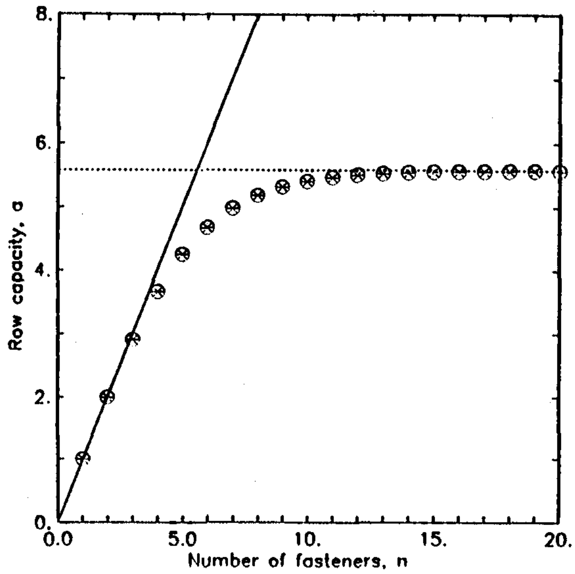


Figure 5.13: Effective number of fasteners in a line (Zahn, 1991)

In Eurocode 5, the effective number of fasteners in a joint is also taken into account for connection design with the added factor that the effective number of fasteners is dependent on the fastener spacing. When inserted with minimum spacing of 14d the code allows for the effective number of fasteners to be equal to the actual number of fasteners. And on the opposite, with reducing spacing between fasteners, the effective number of fasteners is also reducing. A spacing of 8d was therefore deemed suitable for investigating this parameter with helically shaped fasteners in timber to timber joints.

The effect of number of lines and line spacing was then studied. To study the number of lines, samples with two, three and four lines were tested. For double shear connection with StarTie 8 fasteners three line spacings were tested to investigate the line spacing, however, due to the testing method used in the research for single shear connection, five line spacings were tested. The line spacing was investigated for line spacing greater than the minimum of 5d that was chosen for helically shaped fasteners.

For fasteners inserted in a line with minimum spacing of 14d, in Eurocode 5, the connection load is directly proportional to the number of fasteners, therefore the number of rows was investigated with the fasteners spacing equal to 14d; samples with two, three and four rows of two lines of fasteners were tested. The row spacing was investigated by testing samples with three rows of two lines of fasteners. As for line spacing, the investigation of row spacing concentrated above the minimum spacing that was found suitable for helically shaped fasteners – i.e. 8d.

Following the investigation of these joint geometry factors the experimental programme was directed towards studying material factors such as timber dimensions, density and moisture content. To study the timber dimension factor tests with reduced side timber members were performed, along with samples with the side and middle member reduced. The density was studied by testing connection with timber of grade C16 and D30. The later proved to be an issue for double shear connections, as predrilling was to 0.8 times the root diameter inserting the fasteners through the three timber members was impossible with the fasteners bending under the impact load of the hammer. Therefore only single shear samples were tested in timber grade D30.

Due to the complexity of the effect of the moisture content on the joint resistance and behaviour a full experimental research of its influence on connections with helically shaped fasteners could not be performed. However, in order to appreciate the moisture content influence below the timber fibres saturation point it was decided to test samples with timber moisture content of 12%. This also represented the only moisture contents controllable during the experimentation.

While incomplete in view of the many factors that may influence the resistance and behaviour of timber to timber connections with helically shaped fasteners, this experimental programme was developed to investigate the factors that were controllable, and concentrated on the joint geometry factors. This allowed for the development of semi empirical models for predicting the connection behaviour and capacity.

5.4.2 Modes of failure and discussion

Lateral shear tests with helically shaped fasteners loaded in single and double shear were tested as described above in a variety of nailing configurations. The typical load displacement exhibited for the entirety of the samples was similar to that of single fastener samples shown in Figure 5.7. All the samples exhibited a very ductile behaviour, with the maximum load attained at displacement between 6.51mm (samples CG) and 42.02mm (samples NA).

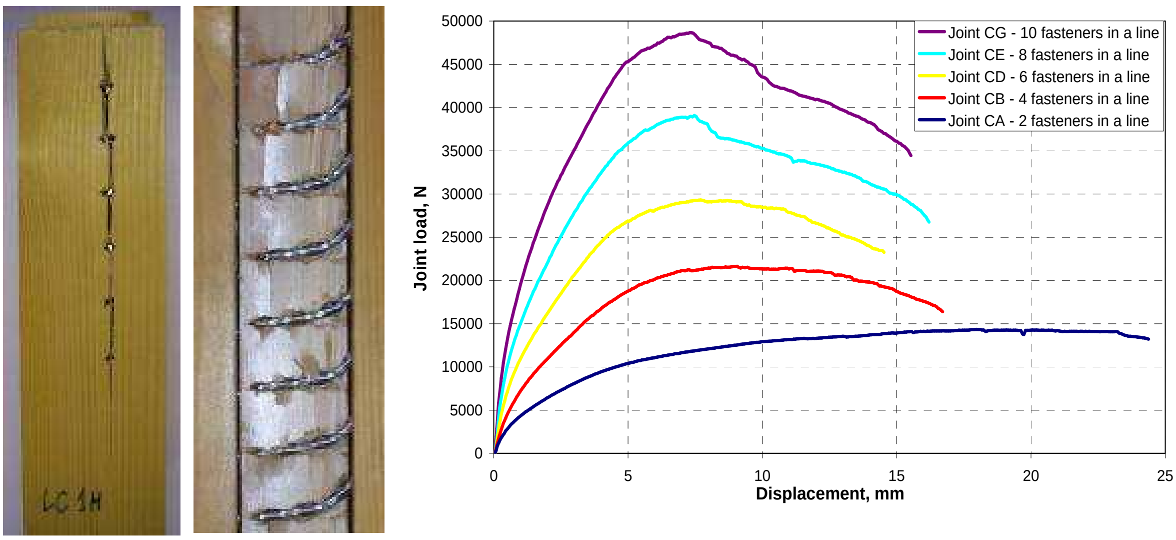
Due to the ductility of the samples, and the large displacements reached during testing, the modes of failure for the different nailing patterns could not be properly identified. After the tests the samples were split open to examine the failure modes of the fasteners and timber members. The following observations were made:

- The fasteners are being pulled by the relative displacement between the timber members; both sides of the fasteners loaded in single and double shear fasteners are pulled;
- At the interface of the timber members the timber fibres are crushed under the fastener;
- The fasteners show the formation of plastic hinges near the timber members' interfaces, for fasteners loaded in single and double shear.

Figure 5.14: Mode of failure of multiple fastener samples

Eurocode 5 defines for six and four ductile modes of failures types for laterally loaded fasteners in single and double shear respectively, as shown in Section 2.5. It allows for embedment failure of the timber members (Mode I), combined embedment failure of the timber and partial yielding of the fastener (Mode II), and combined embedment failure of the timber and full yielding of the fastener (Mode III). Even though identifying the exact mode of failure for the samples tested proved difficult, it can be assumed that failure of the type II and III would occur in view of the observation of the samples. To gain a better understanding of the connection behaviour, and in order to try to identify the exact mode of failure for joints with helically shaped fasteners, a detailed theoretical analysis of the joint behaviour was undertaken and is described in Chapter 6.

These observations on ductile failure were made on the majority of the nailing patterns, however in the case of tests investigating the effective number of fasteners in a line, the samples exhibited a ductile behaviour with brittle failure due to splitting of the timber members. This mode of failure was observed for samples with six, eight and ten fasteners in a line and a parallel to the grain spacing of 8d, Figure 5.15.



a) b)

Figure 5.15: a) Brittle failure due to wood splitting; b) Ductile behaviour

It is to be noted that for samples with six, eight and ten fasteners in a line, the maximum loads were attained at an average displacement of 17.23mm for fasteners loaded in single shear and 8.37 for fasteners loaded in double shear. Also the load displacement relationships for these samples show that the brittle failure occurred after the yield point of the connection.

5.5 Semi empirical models for laterally loaded joints

As mentioned in Chapter 2, various methods have been developed and used over the years in order to predict the stiffness behaviour and overall performance of connections with dowel type fasteners. Following the experimental programme detailed above for connections with helically shaped fasteners, and the investigation of the factors that influence their behaviour, semi-empirical models are developed for predicting the connections load displacement behaviour and load carrying capacity.

The semi-empirical models were developed based on a method described by Porteous and Kermani (2005) and first used by Mack in 1966; and also used earlier in this research for simulating the withdrawal behaviour and resistance of helically shaped fasteners. Mack showed that the parameters investigated did not significantly interact and that the relationship between the load and displacement was a function of the product of each of the parameters; therefore the load displacement behaviour could be simulated by analysing the factors that influence the connection individually.

The analysis of the data for the samples tested show that on average yield of the joint occurred at a displacement of 3.07mm and 3.11mm and failure occurred at slip of 32.19mm and 22.36mm for single and double shear connection respectively. The semi empirical models were developed to predict the strength and behaviour of timber to timber connections with helically shaped fasteners in single and double shear in the elastic range of the connection behaviour. Therefore, it was decided that the slip limit for which the models were developed should be based on a displacement of 3.2mm. This slip limit of 3.20mm represents loads on the connections of 86% and 81% of the yield loads for single and double shear joints respectively.

As the factors that influence the connection behaviour do not interact, and as shown by previous research that the relationship between the load and displacement was a function of the product of each of the parameters, the load displacement relationship for timber to timber connections with helically shaped fasteners can be written of the following form:

$$P = f_1(\delta)\cdot f_2(d)\cdot f_3(D)\cdot f_4(mc)\cdot f_5(N_L)\cdot f_6(L_S)\cdot f_7(N_R)\cdot f_8(R_S)\cdot f_9(g)\cdot f_{10}(v) \qquad \textbf{…(5.1)}$$

Where: P = Connection load,

$f_1(\delta)$ = Displacement function,

$f_2(d)$ = Fastener diameter function,

$f_3(D)$ = Timber density function,

$f_4(mc)$ = Timber moisture content function,

$f_5(N_L)$ = Number of lines of fasteners function,

$f_6(L_S)$ = Lines of fasteners spacing function,

$f_7(N_R)$ = Effective number of fasteners, number of rows function,

$f_8(R_S)$ = Rows of fasteners spacing function,

$f_9(g)$ = Generic function and,

$f_{10}(v)$ = Function of remaining variables.

The function $f_{10}(v)$ allows for other variables that may influence the behaviour of timber to timber connections with Helically shaped fasteners to be considered in the model- e.g. method of insertion (manual or mechanical), time between fabrication and testing, angle between fasteners and timber fibres, etc…. However, as their influence was not studied in the test programme the function $f_{10}(v)$ is taken as unity. The functions f_1 to f_9 are addressed in the following sections.

5.5.1 Model for Helically shaped fasteners loaded in single shear

Using test data for connections tested with helically shaped fasteners loaded in single shear, a semi empirical model is developed on the form of Equation 5.1.

$f_1(\delta)$ = Displacement function:

As it is assumed that the factors that influence the load displacement behaviour of connections with helically shaped fasteners Equation 5.1 can be written as follow:

$$P = f_1(\delta) \cdot K \qquad \textbf{…(5.2)}$$

If K is the load at the slip limit of 3.20mm, the function will be unity at this limit, and at any intermediate load the function can be written:

$$f_1(\delta) = \frac{(P)_\delta}{(P)_{3.20}} \qquad \textbf{…(5.3)}$$

$(P)_\delta /(P)_{3.20}$ is referred to as the reduced load; and over the range 0-3.20mm it will define the displacement function. The concept of reduced load was first introduced by Mack in 1966, and has since been widely used in timber research to develop semi-empirical models. To represent the load displacement behaviour, many forms of displacement functions have been developed. The two most common forms were developed by Mack (1966) and McLain (1976), Equations 5.4 and 5.5.

$$f_{Mack}(\delta) = (A \cdot 0.3125\delta + B)(1 - e^{0.3125C\delta})^{D} \qquad \textbf{…(5.4)}$$

$$f_{McLain}(\delta) = E \cdot \log(1 + F \cdot \delta) \qquad \textbf{…(5.5)}$$

Where A, B, C, D, E and F are constants to fit test data.

The test results were processed in the software Excel; for each test the load at displacement δ was divided by the load achieved at 3.20mm, to obtain the reduced load curve. The reduced loads were then compiled and the commercial software MathCAD, and its least square non-linear regression analysis function Genfit, was used to fit the test data using the format of the above equations. A detailed MathCAD analysis is shown in Appendix D. The data analysis resulted in the following equations:

$$f_{Mack}(\delta) = (0.144 \cdot \delta + 0.552) \cdot (1 - e^{(-1.353 \cdot \delta)})^{0.969} \qquad \textbf{…(5.6)}$$

$$f_{Mclain}(\delta) = 1.323 \cdot \log(1 + 1.465 \cdot \delta) \qquad \textbf{…(5.7)}$$

Both equations resulted in high coefficients of determination R^2, 0.982 and 0.968 respectively. The data analysis for double shear tests also resulted in high coefficients of determination R^2, 0.987 and 0.966 for the two equations types. These results indicate that both equation forms could be used for characterising the load displacement behaviour of timber connections with helically shaped fasteners to a slip limit of 3.20mm. A review of previous research on both forms used concluded that Mack's equation was the most used and adaptable for timber to timber connections. In view of this the displacement function for single shear connections with helically shaped fasteners can be written as follows and is shown in Figure 5.16:

$$f_1(\delta) = (0.144 \cdot \delta + 0.552) \cdot (1 - e^{(-1.353 \cdot \delta)})^{0.969} \qquad \textbf{…(5.8)}$$

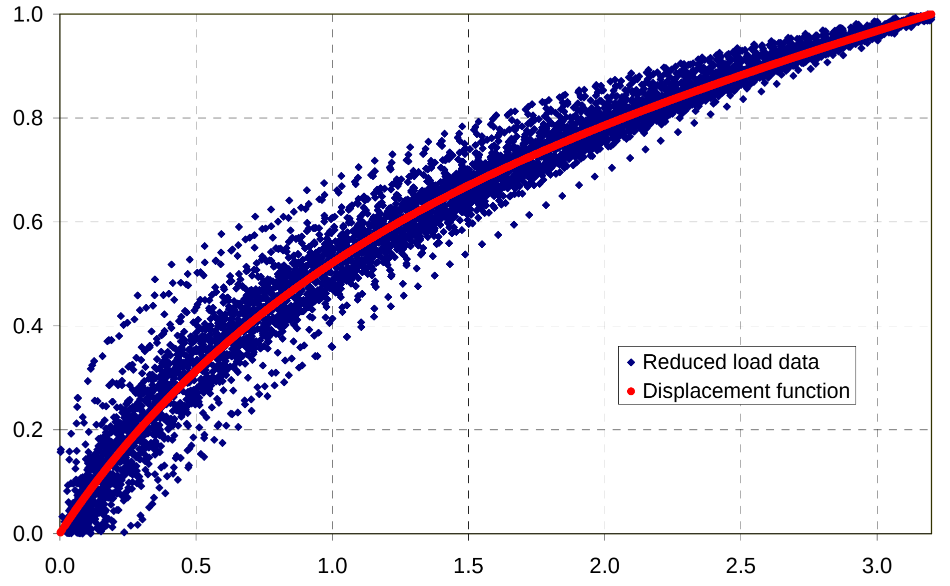


Figure 5.16: Displacement function f_1

$f_2(d)$ = Fastener diameter function:

As mentioned in the above sections, and shown in Figure 5.11, the connection load is directly proportional to the fastener root diameter d_r. Therefore the fastener diameter function can be written:

$$f_2(d) = d_r \qquad \textbf{...(5.9)}$$

$f_3(D)$ **= Timber density function:**

As mentioned in chapter 2 the timber density has been shown to be correlated to the timber strength and stiffness. Also, in the case of timber connections, various studies used a linear relationship between the connection strength and the timber density. To study the effect of timber density on the strength of joints with helically shaped fasteners, similar joints were tested using three grades of timber: C16, C24 and D30. The results of the tests are plotted against the timber density (D) of the samples in Figure 5.17.

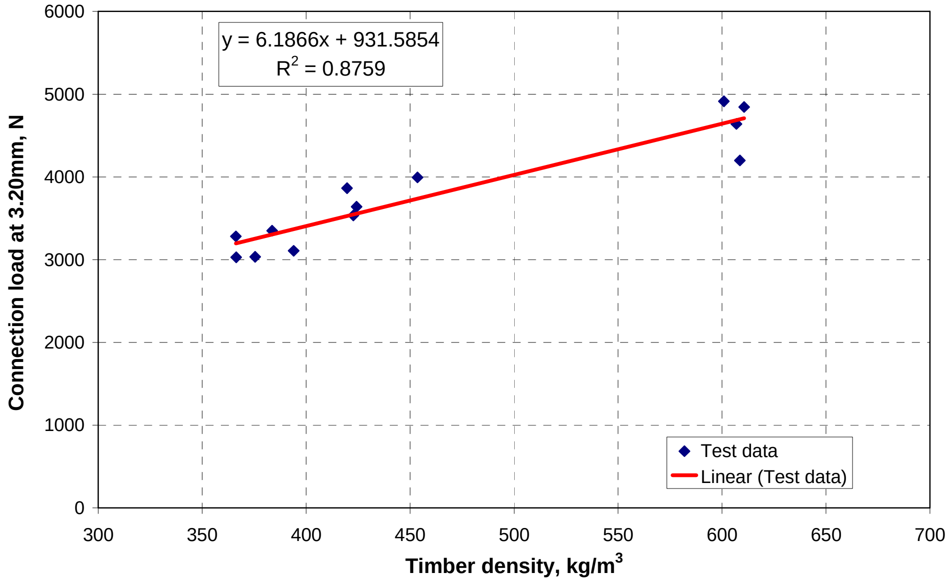


Figure 5.17: Connection load vs. timber density

In view of the results shown in the Figure 5.17, and conclusions of previous research, the density function for timber connection with helically shaped fasteners is written as follows:

$$f_3 = D \qquad \textbf{...(5.10)}$$

***$f_4(mc)$* = Timber moisture content function:**

The effect of moisture content was investigated by testing connections at different moisture content which remained under the timber saturation point. Tests were carried out with a moisture content of 10±1% and 12±1%. This range, however small, was deemed sufficient to investigate the effect of moisture content, as previous research showed that a change of 1% of moisture content can provide changes up to 5% on various timber mechanical properties; and for the range of 8% to 20% moisture content the relationship between moisture content and mechanical properties is linear (STEP1, 1995). The results of tests with different moisture content are shown in Table 5.3.

__Table 5.3:__ effect of moisture content on connection load

Joints	**Load**	**Moisture content**
	N	**%**
MA	3758.5	8.75
MA-MC12	2706.5	12.31
Ratios	1.39	1.41
% difference	1.339	

The ratios of the connections load at 3.20mm and moisture content were calculated. It is to be noted that the inverse of the ratio of the moisture content is shown in Table 5.3; the increase in moisture content has an inverse effect on the joint strength. The percentage difference between the load and moisture content ratios was calculated to be 1.34%; hence the moisture content function can be written:

$$f_4(mc) = \frac{1}{mc} \qquad \textbf{...(5.11)}$$

***$f_5(N_L)$* = Number of lines of fasteners function:**

Joints AB, MA, MB and MC with two, three and four lines of fasteners were used in the determination of the number of lines function. The test data, to the slip limit of 3.20mm was plotted for the three sets and the best fit for the data calculated. The form of the best fit was of the form of Equation 5.5; the best fit equations were as follows:

Joint AB	y = 1148*log(1+13.191x)	$R^2 = 0.93$
Joint MA	y = 5542*log(1+1.179x)	$R^2 = 0.979$
Joint MB	y = 5624*log(1+2.088x)	$R^2 = 0.982$
Joint MC	y = 7911*log(1+1.358x)	$R^2 = 0.986$

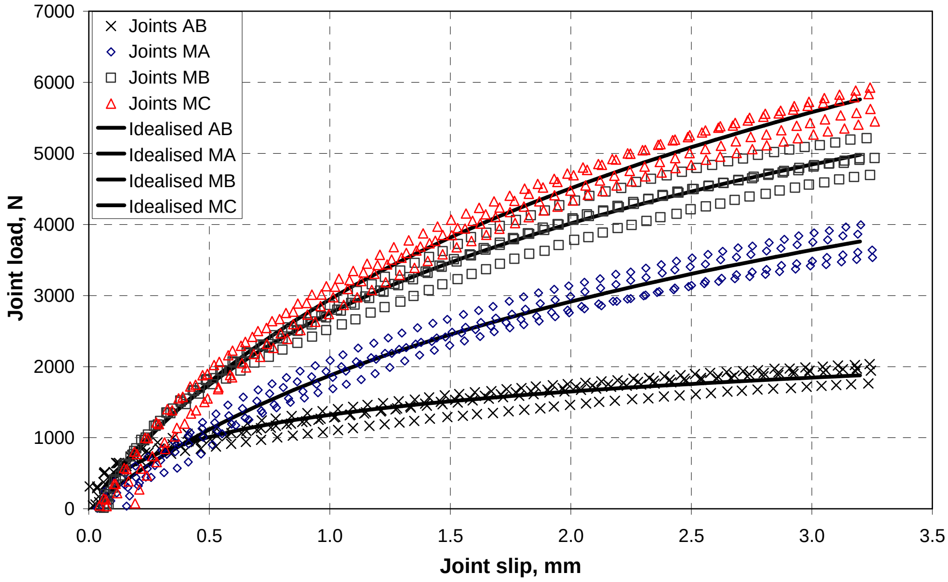


Figure 5.18: Joints AB, MA, MB MC test data, and idealised curves

Using the equations above for the respective joints, the load displacement curves were compared in order to determine the multiplying factor between joints. To achieve this, two curves are analysed using the percentage mean deviation (md) until it becomes zero by adjusting the factor "i" in the following equation:

$$md = \sum_{1}^{n} \frac{(i \cdot P_1 - P_2)}{P_2 \cdot n} \cdot 100 \qquad \textbf{...(5.12)}$$

The multiplying factors found from the analysis, and the theoretic factors between joints are shown in Table 5.4 below.

***Table 5.4**: Number of lines multiplying factors*

Joints	Multiplying Factors	
	Actual	Theoretic
MA/AB	1.422	2.0
MB/AB	2.089	3.0
MC/AB	2.235	4.0
MB/MA	1.433	1.5
MC/MA	1.562	2.0
MB/MA	1.087	1.33

The results shown in Table 5.4, and illustrated in Figure 5.19, indicate that the joint load is not directly proportional to the number of lines of fasteners.

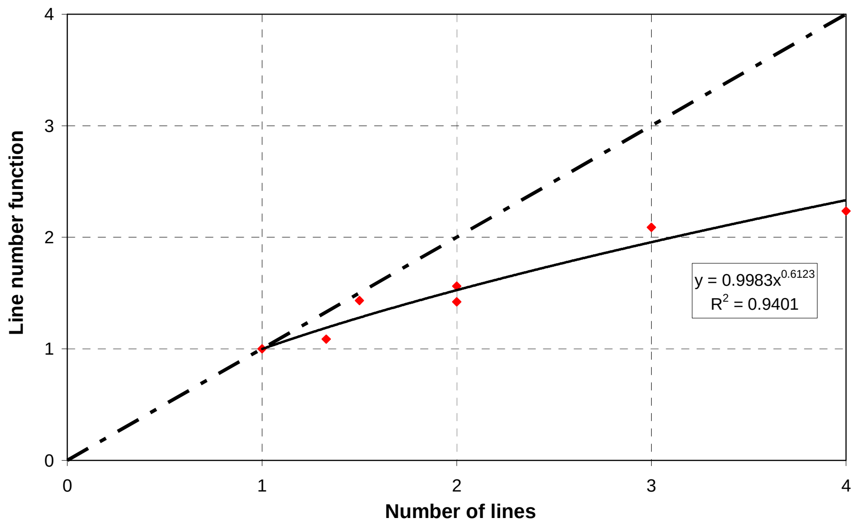


Figure 5.19: Number of lines function

In view of these results a best fit function was calculated for determining the number of lines function. Therefore the function $f_5(N_L)$ can be written:

$$f_5(N_L) = 0.998 \cdot (N_L)^{0.612} \quad \textbf{...(5.13)}$$

***$f_6(L_S)$* = Lines of fasteners spacing function:**

The line spacing function was investigated by testing the same nailing configuration of two rows and two lines of fasteners using five line spacings, multiples of the fastener root diameter. The results of the five nailing configurations and the corresponding line spacing are detailed in Table 5.5.

Table 5.5: *Effect of line spacing on joint strength*

Joint	Line Spacing		Joint Load
	x*d	mm	N
MK	5d	17	6276
ML	8d	27	6379
MG	10d	34	5919
MF	12d	40	6503
MH	14d	47	6375

The results shown in the table above show that the load on the connection is similar for all spacings tested, with the exception of the joint MG with a spacing of 10d. The joint MG resulted in a lower load, however the percentage difference to the average load of the other joints is 7.8%. In view of this, and considering the conclusions from results of previous research, notably from Porteous (2003), the effect of line of fastener spacing can be written:

$$f_6(L_S) = 1 \qquad \textbf{…(5.14)}$$

***$f_7(N_R)$* = Number of rows of fasteners function:**

The samples tested in investigating the number of rows constituted of joints with 1, 2, 3 and 4 rows of two lines of fasteners. The rows spacing was 14d. A close analysis of the results showed that the samples presented large differences in terms of densities. As the function ***$f_3(D)$*** showed, the joint load is directly proportional to the timber density. Therefore in order to eliminate the effect of density from the investigation the load divided by the sample density was used.

The results of the four sets of joints were compared in order to determine the multiplying factors between connections. The factors found have been compared to the theoretical factors between the joints, and the percentage difference between the factors calculated in Table 5.6. The theoretical factors were obtained by calculating the ratio of number of rows of fasteners between the different joints configurations.

Table 5.6: *Number of rows multiplying factors*

Joints	Multiplying factors		
	Actual	Theorical	% difference
NA/MA	2,070	2,0	-3,52
NB/MA	3,162	3,0	-5,40
NC/MA	4,515	4,0	-12,88
NB/NA	1,527	1,5	-1,81
NC/NA	2,181	2,0	-9,04
NC/NB	1,428	1,33	-7,36

The results shown in Table 5.6 show that the percentage difference between factors is relatively low; consequently the number of rows of fasteners function can be written:

$$f_7(N_R) = N_R \qquad \textbf{…(5.15)}$$

***$f_8(R_S)$* = Rows of fasteners spacing function:**

The results of the number of rows of fasteners function, *$f_7(N_R)$*, indicate that for a spacing of 14 times the fastener root diameter the connection load is directly proportional to the number of rows of fasteners. Thus, the row of fasteners spacing needs to be investigated between the minimum spacing recommended by Eurocode 5 (BSI, 2004) and 14 times the fastener root diameter. From a spacing of 14 times the fastener root diameter the row of fasteners spacing function will be unity.

For row spacings between $8*d_r$ and $14*d_r$, similar joints with varying spacings have been tested. The load at the slip limit of 3.20mm was divided by timber density and number of rows and plotted against the row spacing, expressed in terms of multiple of the root diameter, see Figure 5.20.

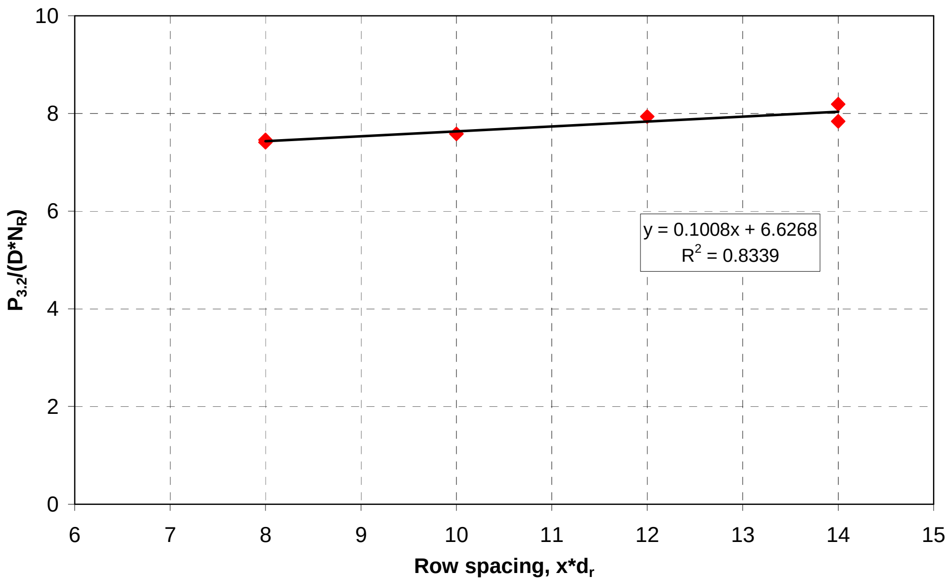


Figure 5.20: Effect of row spacing on joint strength

As mentioned above the row spacing function is unity for spacing of $14*d_r$ or more and also for joints with one row of fasteners. Therefore the function from Figure 5.20 was re-written to incorporate the boundary conditions of unity for spacings of $14*d_r$ and zero. Hence, the function *$f_8(R_S)$* can be written:

- For row spacing $8*d_r \leq R_S \leq 14*d_r$:

$$f_8(R_S) = 0.0015 \cdot R_S^2 - 0.0201 \cdot R_S + 0.9992 \qquad \textbf{...(5.16a)}$$

- For row spacing $R_S \geq 14^* d_r$:

$$f_8(R_S) = 1$$ **…(5.16b)**

***f₉(g)* = Generic function:**

The generic function is the fit function, it takes into account all the parameters not included in the function f_1 to f_8. The function $f_9(g)$ for each test performed was determined by using the load at slip limit of 3.20mm, and rearranging Equation (5.1) as follows:

$$f_9(g) = \frac{P_{3.2}}{f_1(\delta)\cdot f_2(d_r)\cdot f_3(D)\cdot f_4(mc)\cdot f_5(N_L)\cdot f_6(L_S)\cdot f_7(N_R)\cdot f_8(R_S)}$$

…(5.17)

For each test performed Equation (5.17) was evaluated, and the average calculated to determine the generic function:

$$f_9(g) = 15.264$$ **…(5.18)**

Semi-empirical model

Substituting for the relevant functions determined in the sections above in equation 5.1, the load displacement relationship for timber to timber joints with helically shaped fasteners loaded in single shear becomes:

$$P = (0.144\cdot\delta + 0.552)(1 - e^{-1.353\cdot\delta})^{0.969}\, d_r \cdot D \cdot (1/mc)\cdot 0.998 N_L^{0.612} \cdot N_R \cdot (0.0015 R_S^2 - 0.0201 R_S + 0.9992)\cdot 15.264$$

…(5.19)

Where P = Lateral shear load of single shear joint with helically shaped fasteners at a slip δ (N),

δ = The joint slip at which the load is calculated (mm),

d_r = Fastener root diameter (mm),

D = Timber density (kg/m^3),

mc = Timber moisture content (%) – noting that the model was developed for moisture content below saturation point,

N_L = Number of lines of fasteners in the connection,

N_R = Number of rows of fasteners in the connection,

R_S = Row spacing, expressed as a multiple of the fastener root diameter.

5.5.2 Model for Helically shaped fasteners loaded in double shear

Using test data for connections tested with helically shaped fasteners loaded in double shear, a semi empirical model is developed in the form of Equation 5.1. The development of the function f_1 to f_9 for fasteners loaded in double shear followed the same analytical method as for fasteners loaded in single shear. The variables functions were found to be as follows:

$$f_1(\delta) = (0.203 \cdot \delta + 0.354) \cdot (1 - e^{-2.589 \cdot \delta})^{0.986} \qquad \text{…(5.20)}$$

$$f_2(d) = d_r \qquad \text{…(5.21)}$$

$$f_3(D) = D \qquad \text{…(5.22)}$$

$$f_4(mc) = \frac{1}{mc} \qquad \text{…(5.23)}$$

$$f_5(N_L) = 0.9671 \cdot N_L^{0.9609} \qquad \text{…(5.24)}$$

$$f_6(L_S) = 1 \qquad \text{…(5.25)}$$

$$f_7(N_R) = N_R \qquad \text{…(5.26)}$$

$$f_8(R_S) = 0.0014 \cdot R_S^2 - 0.019 \cdot R_S + 0.9992 \qquad \text{…(5.27)}$$

$$f_9(g) = 25.5925 \qquad \text{…(5.28)}$$

Substituting for the relevant functions above in equation 5.1, the load displacement relationship for timber to timber joints with helically shaped fasteners loaded in double shear becomes:

$$P = (0.203 \cdot \delta + 0.354)(1 - e^{-2.589 \cdot \delta})^{1.164} d_r \cdot D \cdot (1/mc) \cdot 0.9671 N_L^{0.9609} \cdot N_R \cdot (0.0014 R_S^2 - 0.019 R_S + 0.9992) \cdot 25.5925$$

…(5.29)

5.5.3 Comparison between semi-empirical models and test data

The semi-empirical models developed for connections with helically shaped fasteners loaded in single and double shear above are compared to the tests data. The average test data for the connection tested is used in the models for determining the model load at 3.20mm. The results are presented in Table 5.7 and 5.8 for single and double shear connections respectively. The percentage error between the model and test data is also calculated; a positive error indicates that the model overestimates the connection load, and a negative error underestimates the connection load.

Table 5.7: *Comparison between test data and model for single shear connections*

Joint	d_r	D	mc	N_L	N_R	R_S	Test load	Model load	Error
	mm	kg/m^3	%	nbre	nbre	x*d_r	N	N	%
AB	3.35	412.24	10.22	1	1	0	1929.4	2057.3	6.22
LA	3.35	390.21	9.64	1	2	8	3031.3	3857.9	21.43
LB	3.35	389.79	9.57	1	4	8	5551.5	7766.6	28.52
LD	3.35	365.21	9.43	1	6	8	8705.3	11079.9	21.43
LE	3.35	393.19	9.41	1	8	8	11058.5	15933.3	30.59
LG	3.35	395.25	9.43	1	10	8	14150.1	19974.3	29.16
MA	3.35	430.07	8.70	2	1	0	3758.5	3851.3	2.41
MB	3.35	393.52	9.58	3	1	0	4848.3	4102.8	-18.17
MC	3.35	380.96	9.61	4	1	0	5709.3	4723.3	-20.87
MG	3.35	385.78	9.45	2	2	8	5830.1	5947.9	1.98
MF	3.35	397.77	9.44	2	2	8	6503.6	6144.1	-5.85
MH	3.35	397.39	9.26	2	2	8	6219.5	6257.0	0.60
ML	3.35	361.12	9.19	2	2	8	6378.6	5727.9	-11.36
MK	3.35	358.10	9.27	2	2	8	6276.0	5632.2	-11.43
NA	3.35	359.65	10.46	2	2	14	5527.3	5360.0	-3.12
NB	3.35	381.54	10.57	2	3	14	9222.0	8435.3	-9.33
NC	3.35	380.37	11.28	2	4	14	12549.1	10506.7	-19.44
ND	3.35	391.41	10.20	2	3	8	8948.9	8390.5	-6.65
NG	3.35	394.26	10.18	2	3	10	9065.3	8588.3	-5.55
NH	3.35	404.65	10.87	2	3	12	9385.1	8485.4	-10.60
MAC16	3.35	377.12	9.44	2	1	0	3191.4	3111.5	-2.57
MAD30	3.35	606.76	9.38	2	1	0	4649.9	5040.3	7.75
MAMC12	3.35	345.26	12.35	2	1	0	2822.0	2178.0	-29.57

Table 5.8: *Comparison between test data and model for double shear connections*

Joint	d_r	D	mc	N_L	N_R	R_S	Test load	Model load	Error
	mm	kg/m^3	%	nbre	nbre	$x*d_r$	N	N	%
AC	3.75	417.50	10.30	1	1	0	3686.8	3771.0	2.23
CA	3.75	396.48	9.19	1	2	8	8380.5	7525.6	-11.36
CB	3.75	405.53	8.56	1	4	8	14695.5	16533.0	11.11
CD	3.75	392.46	8.87	1	6	8	21464.0	23151.2	7.29
CE	3.75	429.47	8.95	1	8	8	28854.7	33486.5	13.83
CG	3.75	385.86	9.36	1	10	8	36512.3	35969.9	-1.51
DA	3.75	376.90	9.53	2	1	0	8108.8	7163.0	-13.20
DB	3.75	382.02	9.51	3	1	0	10906.3	10739.0	-1.56
DC	3.75	358.43	9.38	4	1	0	14412.8	13474.9	-6.96
DE	3.75	410.24	8.71	2	2	8	14369.3	15989.8	10.13
DG	3.75	406.05	8.38	2	2	8	13543.0	16448.4	17.66
DH	3.75	350.68	9.37	2	2	8	14251.0	12710.2	-12.12
EA	3.75	399.85	9.50	2	2	14	15106.8	15368.0	1.70
EB	3.75	391.30	9.21	2	3	14	23268.3	23273.0	0.02
EC	3.75	334.94	8.69	2	4	14	25517.3	28143.2	9.33
ED	3.75	386.73	9.57	2	3	8	22384.0	20590.2	-8.71
EG	3.75	362.93	9.45	2	3	10	20902.0	19820.5	-5.46
EH	3.75	374.36	9.18	2	3	12	20894.3	21579.1	3.17
DA-C16	3.75	367.67	8.76	2	1	0	7203.3	7602.8	5.25
DA-MC12	3.75	337.04	12.11	2	1	0	6094.5	5042.4	-20.87

The results show that the models developed can predict the connection load reasonably well with the absolute average error being 13% and 8% for connections in single and double shear respectively.

However, a close inspection of the results of Table 5.7a shows that the model for single shear connections overestimated the joint strength of samples LA, LB, LD, LG and LE by over 26% on average. These samples exhibited brittle failures due to wood splitting, as shown in Figure 5.15. It was assumed that up to a slip of 3.20mm the effect of wood splitting was in the early stages and would not influence the connection load displacement behaviour. However when analysing the results and difference between the models and tests loads, the results showed that the model over predicted the connection strength for those samples. This may indicate that timber splitting may occur in the early stage of the load displacement curves. Consequently the generic function for connections in single shear was re-evaluated without the samples LA, LB, LD, LG and LE. The new value of the generic function for fasteners loaded in single shear was found to be:

$$f_9(g) = 16.3785 \quad \textbf{...(5.30)}$$

Substituting Equation 5.30 for Equation 5.18; the load displacement relationship for timber to timber joints with helically shaped fasteners in single shear in Equation 5.19 becomes:

$$P = (0.144 \cdot \delta + 0.552)(1 - e^{-1.353 \cdot \delta})^{0.969} d_r \cdot D \cdot (1/mc) \cdot 0.998 N_L^{0.612} \cdot N_R \cdot (0.0015 R_S^2 - 0.0201 R_S + 0.9992) \cdot 16.3785$$

…(5.31)

Equation 5.31 was used for determining a new model load for connections with helically shaped fasteners in single shear, detailed in Table 5.9. The results show that the new single shear model can predict the connection load with greater accuracy, with the absolute average error calculated to be 7%.

Table 5.9: *Comparison between test data and new model for single shear connections*

Joint	dr	D	mc	NL	NR	RS	Test Load	Model Load	Error
	mm	kg/m3	%	nbre	nbre	x*d	N	N	%
AB	3.35	412.24	10.22	1	1	0	1929.4	2207.6	12.60
MA	3.35	430.07	8.70	2	1	0	3758.5	4132.5	9.05
MB	3.35	393.52	9.58	3	1	0	4848.3	4402.3	-10.13
MC	3.35	380.96	9.61	4	1	0	5709.3	5068.2	-12.65
MG	3.35	385.78	9.45	2	2	8	5830.1	6382.2	8.65
MF	3.35	397.77	9.44	2	2	8	6503.6	6592.7	1.35
MH	3.35	397.39	9.26	2	2	8	6219.5	6713.8	7.36
ML	3.35	361.12	9.19	2	2	8	6378.6	6146.1	-3.78
MK	3.35	358.10	9.27	2	2	8	6276.0	6043.4	-3.85
NA	3.35	359.65	10.46	2	2	14	5527.3	5823.9	5.09
NB	3.35	381.54	10.57	2	3	14	9222.0	9165.3	-0.62
NC	3.35	380.37	11.28	2	4	14	12549.1	11416.1	-9.92
ND	3.35	391.41	10.20	2	3	8	8948.9	9003.2	0.60
NG	3.35	394.26	10.18	2	3	10	9065.3	9215.5	1.63
NH	3.35	404.65	10.87	2	3	12	9385.1	9105.0	-3.08
MAC16	3.35	377.12	9.44	2	1	0	3191.4	3338.7	4.41
MAD30	3.35	606.76	9.38	2	1	0	4649.9	5408.3	14.02
MAMC12	3.35	345.26	12.35	2	1	0	2822.0	2337.0	-20.75

Similarly to connections in single shear, the double shear model was re-evaluated omitting the results of the test samples CA, CB, CD CE and CG as the load displacement relationship showed that the samples exhibited ductile behaviour but with brittle failures as shown in Figure 5.15. As both sets of samples exhibited similar behaviour it was concluded that the brittle failure could influence the behaviour of the samples in the elastic range in double shear. The new value of the generic function for fasteners loaded in single shear was found to be:

$$f_9(g) = 25.9229 \qquad \textbf{…(5.32)}$$

Substituting Equation 5.32 for Equation 5.28; the load displacement relationship for timber to timber joints with helically shaped fasteners in single shear in Equation 5.29 becomes:

$$P = (0.203 \cdot \delta + 0.354)(1 - e^{-2.589 \cdot \delta})^{1.164} d_r \cdot D \cdot (1/mc) \cdot 0.9671 N_L^{0.9609} \cdot N_R \cdot (0.0014 R_S^2 - 0.019 R_S + 0.9992) \cdot 25.9229 \qquad \textbf{…(5.33)}$$

Equation 5.33 was used for determining a new model load for connections with helically shaped fasteners in double shear, detailed in Table 5.10. The results show that the new double shear model can predict the connection load with greater accuracy, with the absolute average error calculated to be 6%.

Table 5.10: *Comparison between test data and new model for double shear connections*

Joint	d_r	D	mc	N_L	N_R	R_S	Test load	Model load	Error
	mm	kg/m^3	%	nbre	nbre	x*d_r	N	N	%
AC	3,75	417,50	10,30	1	1	0	3686,8	3819,7	3%
DA	3,75	376,90	9,53	2	1	0	8108,8	7255,5	-12%
DB	3,75	382,02	9,51	3	1	0	10906,3	10877,7	0%
DC	3,75	358,43	9,38	4	1	0	14412,8	13648,9	-6%
DE	3,75	410,24	8,71	2	2	8	14369,3	16196,2	11%
DG	3,75	406,05	8,38	2	2	8	13543,0	16660,8	19%
DH	3,75	350,68	9,37	2	2	8	14251,0	12874,3	-11%
EA	3,75	399,85	9,50	2	2	14	15106,8	15566,4	3%
EB	3,75	391,30	9,21	2	3	14	23268,3	23573,5	1%
EC	3,75	334,94	8,69	2	4	14	25517,3	28506,5	10%
ED	3,75	386,73	9,57	2	3	8	22384,0	20856,0	-7%
EG	3,75	362,93	9,45	2	3	10	20902,0	20076,4	-4%
EH	3,75	374,36	9,18	2	3	12	20894,3	21857,7	4%
DA-C16	3,75	367,67	8,76	2	1	0	7203,3	7701,0	6%
DA-MC12	3,75	337,04	12,11	2	1	0	6094,5	5107,5	-19%

The average results of the lateral shear tests have been plotted against the semi empirical models from equations 5.31 and 5.33 in Figure 5.21 and 5.22 for single and double shear connections respectively. The models use the average values for the test series as detailed in Table 5.9 and 5.10.

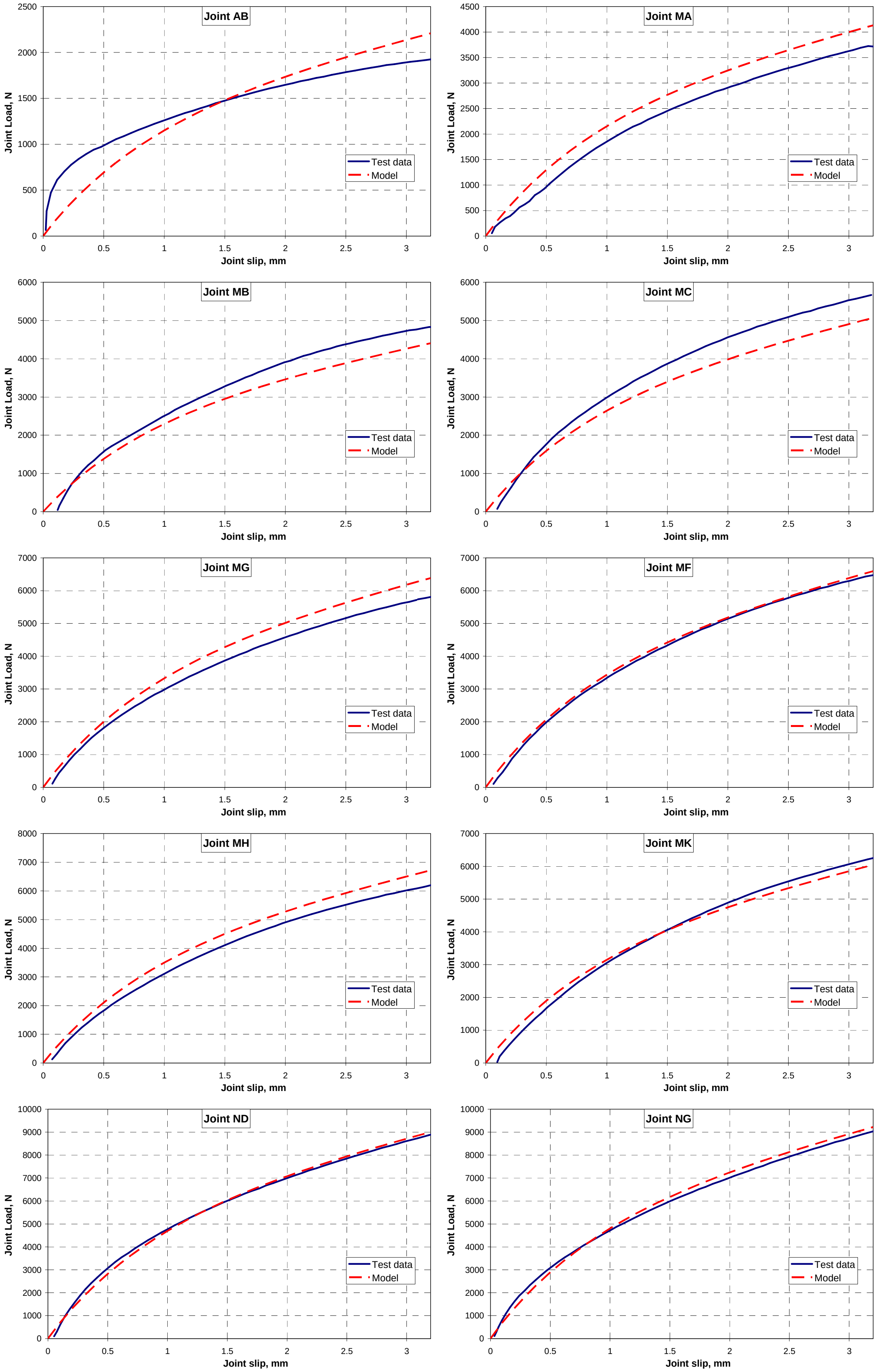


Figure 5.21: Comparison between load displacement behaviour of timber connections with helically shaped fasteners loaded in single shear and semi-empirical model

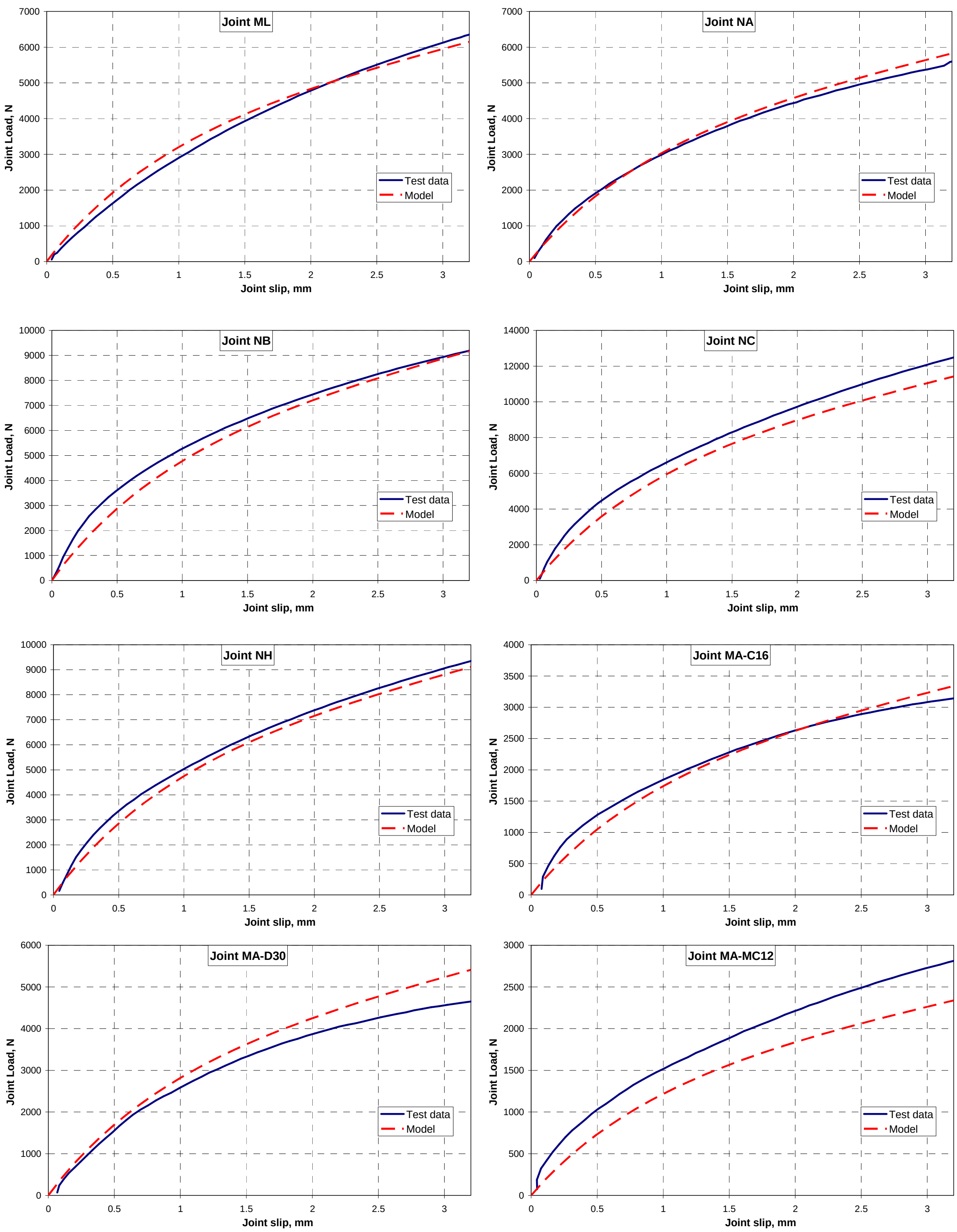


Figure 5.21 (Continued): Comparison between load displacement behaviour of timber connections with helically shaped fasteners loaded in single shear and semi-empirical model

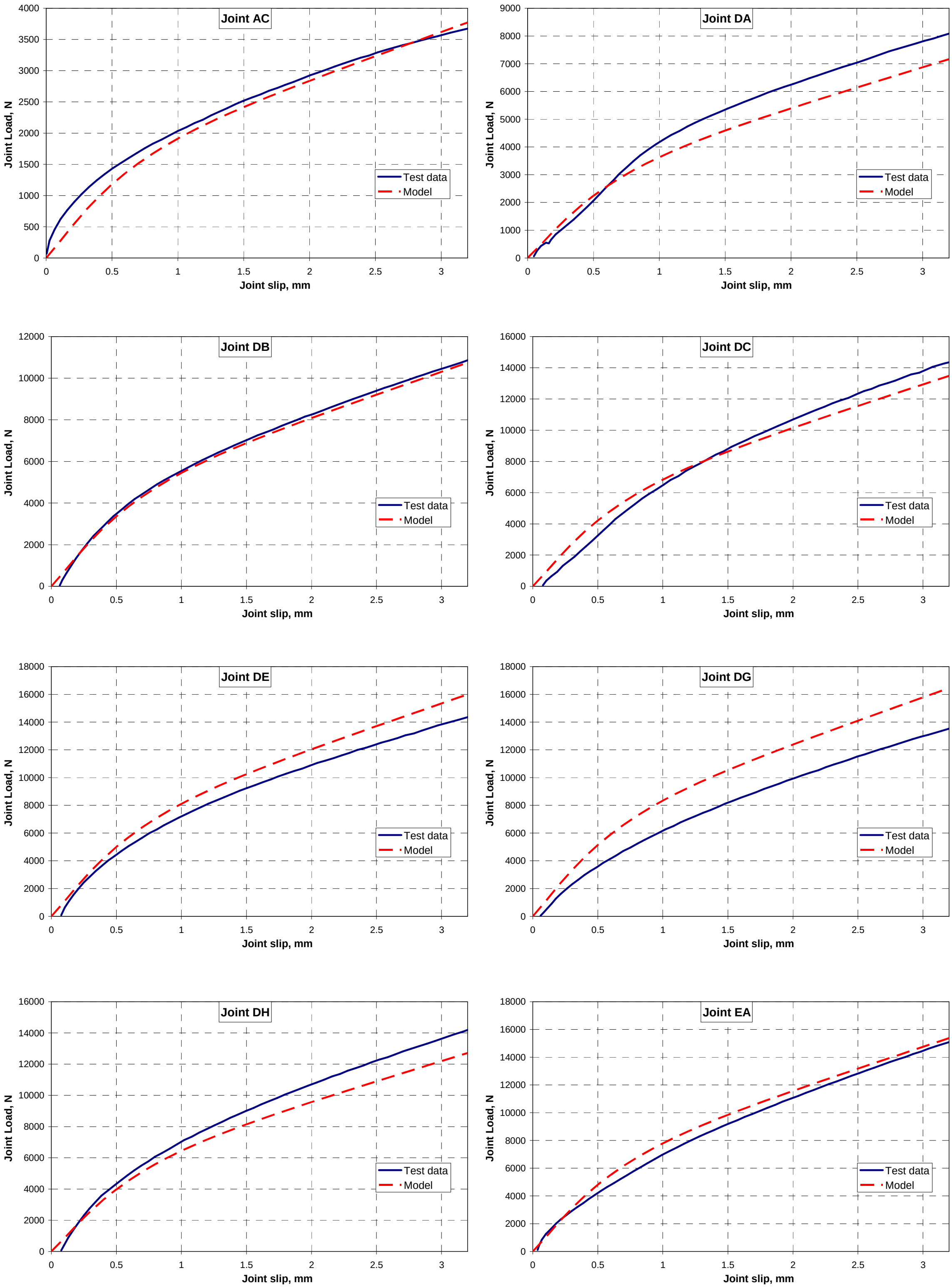


Figure 5.22: Comparison between load displacement behaviour of timber connections with helically shaped fasteners loaded in double shear and semi-empirical model

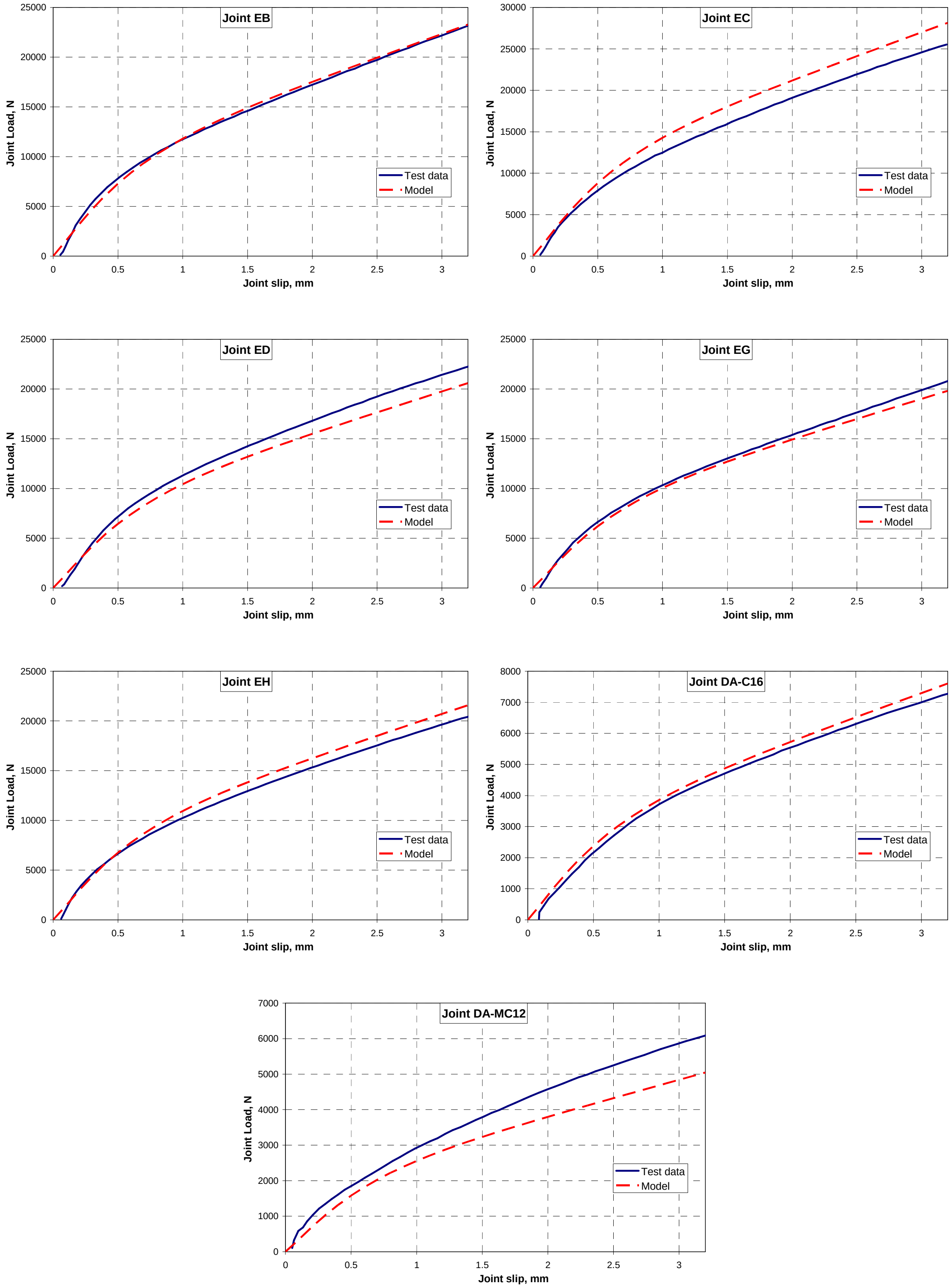


Figure 5.22 (continued): Comparison between load displacement behaviour of timber connections with helically shaped fasteners loaded in double shear and semi-empirical model

The results presented in Figure 5.20 and 5.21 for timber to timber connections with helically shaped fasteners loaded in single and double shear show relatively good fit between the semi –empirical models and the average tests results over the full range of joint slip to the slip limit of 3.20mm. The semi-empirical models also predicted with good accuracy the joint stiffness in the early stages of loading. For samples with average moisture content of 12 ±1% it is to be noted that the model underestimated the load at slip limit by more than 20%. While in the analysis the moisture content function was found to be directly proportional to the joint strength with an inverse effect, the influence of moisture content on timber connections with Helically shaped fasteners may actually follow a different relationship. As mentioned above, these tests were performed in order to appreciate the influence of the moisture content on connection strength and behaviour. The results of the tests and semi-empirical models indicate that a full experimental programme may need to be conducted in order to understand and appreciate its influence on connections with helically shaped fasteners.

5.6 Summary and conclusion

The load displacement behaviour and strength of timber to timber connections with helically shaped fasteners were studied and evaluated in this chapter.

In the first stages of this study, the preliminary results showed that the minimum spacings and distances mentioned in Eurocode 5 could not all be applied to helically shaped fasteners as recommended due to the shape and definition of diameter. Using the fasteners root diameter with Eurocode 5 recommendation resulted in somewhat small distances and spacing, whereas using the thread diameter resulted in overly large values. Using results of preliminary tests minimum spacings were defined for helically shaped fasteners using the fasteners root diameter.

Timber connections with common timber connectors – woodscrews and threaded nails – and helically shaped fasteners were tested for comparison purposes. The results indicate that connections with Helically shaped fasteners can achieve similar loads to common connectors while exhibiting a much more ductile behaviour; offering overall a good compromise between the strength of screws and ductile behaviour of threaded nails.

The connection behaviour of joints with helically shaped fasteners was then investigated in detail by evaluating the connections factors that may influence the joint strength and load displacement relationship. An extensive test programme was performed on multiple fastener joints in order to develop semi-empirical models for connections with helically shaped fasteners loaded in single and double shear. The models were developed to a slip limit of 3.20mm; they include the investigation of nailing configuration (number and spacing of lines or rows of fasteners), fastener diameter and timber density and moisture content. The comparison between the load and slip curves predicted by the models and test showed good fit, with the average error between test and model loads at the slip limit of 3.20mm being 8%.

However the results also highlighted the brittle behaviour of connections with multiple fasteners in a row in the early stages of the slip curve for single shear connections. Even if the results of double shear connections with multiple fasteners in a row could be used in the semi-empirical model as opposed to single shear connections, brittle failure was also witnessed for those samples. The minimum spacing for Helically shaped fasteners parallel to the grain was evaluated to be at 8 times the root diameter; however due to the results of samples with up to ten fasteners in a row exhibiting brittle failure it may be the case that this value should be increased for high load joints and for joints with large number of fasteners in a row.

Chapter 6 Design methods for timber joints with helically shaped fasteners

6.1 Introduction

The structural behaviour of timber to timber connections with helically shaped fasteners and the joint configuration parameters that may influence their load displacement and strength have been investigated in the previous chapters. The mechanical properties of the fasteners were also evaluated to the relevant European standards. In this chapter, the helically shaped fasteners properties and single and double shear timber to timber connections are compared with the design recommendations in accordance with the latest draft of Eurocode 5. Using the results of the tests, and analyses detailed in chapters 3, 4 and 5 helically shaped fasteners are evaluated to the timber design rules and compared to common timber fasteners.

Joints with dowel type fasteners can fail in ductile or brittle manner, however due to the unexpected loss of strength generally witnessed in brittle failures Eurocode 5 requirements were developed with the aim to ensure that only ductile failures would occur. To achieve this, the design code was based on the connection design of the yield theory, first developed by Johansen (1947).

In the first part of this chapter, the design of connections based on Eurocode 5 is detailed. Then, the experimental results are compared to the design values obtained from the yield theory. The embedment and yield moment of fasteners design equations for helically shaped fasteners which were evaluated in previous chapters are summarised. Then the axial resistance design method for helically shaped fasteners is investigated as it was shown that the tools from Eurocode 5 did not accurately predict the withdrawal capacity. Finally, the load carrying capacity of helically shaped fastener loaded in single or double shear is investigated in comparison to the yield theory as used in Eurocode 5 and as it was developed by Johansen.

6.2 Eurocode 5 connection design

During the creation and development of the European Economic Community in the 1970s and 1980s the existence of different national structural codes and standards was seen as a “barrier of free trade” which was the fundamental idea for the EEC. To remedy this, the member states, through the Commission of the European Communities, issued a Construction Products Directive with the intention to draft a new set of unified design codes and material standards to cover all building materials (Page, 2005). The new unified code (the Eurocodes) had the purpose to promote the functioning of the common market, to remove obstacles to free movement of services and products by providing common rules for structural design, to reinforce the competitive position of the European construction industry through advanced concepts of design. These ambitious objectives were realised through the work of committees over a period of 30 years, with the last of the Eurocodes being adopted as national standards in the member states over the next few years.

For timber structural systems EN 1995 – Design of Timber Structures, or Eurocode 5, was first drafted on the basis of the 1983 "CIB Structural Timber" Code from the CIB Working Commission 18. Through the working commission changes were brought to the code and design standards, with the first draft of the Eurocode 5 published in 1987 for comments; and the first formal publication as DDENV 1995:1994 in 1994 (Porteous, 2003). The latest version, adopted as national standards within the member states, was published as EN 1995-1-1:2004 + A1:2008 (BSI, 2008). It can be noted that the latest version of Eurocode 5 differs only from the 2004 version in additions that were included in the National Annex which did not affect the work in this study.

The Eurocodes are limit states codes; meaning that the design is related to defined states beyond which the structure no longer satisfies the design performance requirements. Two types of requirements are defined in the Eurocodes: Ultimate Limit States (ULS), and Serviceability Limit States (SLS) (BSI, 2002).

Ultimate Limit States are associated with collapse or any type of structural failure that may endanger safety. They include, amongst others, failure through excessive deformations, loss of stability, rupture and loss of equilibrium. Serviceability Limit States correspond to states beyond which specified service criteria are no longer met. They include, amongst others, deformations that affect appearance or effective use of the structure, damage to finishes and discomfort to users.

The structural verifications to ULS and SLS is based on the partial coefficient method, which applies factors to loads to increase the value of the applied load, and factors to material to reduce the value of material properties or strength. In the simplest case the Eurocodes require that the following is verified:

$$S_d \leq R_d \qquad \textbf{...(6.1)}$$

Where S_d is the design action effect, calculated from the combination of actions and partial factors on loads, and R_d is the design load carrying capacity. The design load carrying capacity is calculated as follows:

$$R_d = \frac{k_{\text{mod}}}{\gamma_m} \cdot R_k \qquad \textbf{...(6.2)}$$

Where:

- k_{mod} is a modification factor taking into account the combined effect of moisture content and the duration of load;
- γ_m is the partial safety factor for the material property;
- R_k is the characteristic value of material property or strength.

The characteristic value is defined as the fifth percentile, derived from statistical analysis or results of tests performed in accordance with the relevant European standards - for timber connections the tests are to be performed according to BS EN 26891:1991 (BSI, 1991). For properties or strength characteristics of timber, timber based materials or products for use in timber the characteristic value should be determined in accordance with BS EN 14358:2006 (BSI, 2007).

For connection with dowel type fasteners two types of failures can arise: ductile and brittle. However due to the associated loss of strength with brittle failures, the Eurocode was developed with the aim to prevent such failure types. Ductile failures include a combination of wood crushing under the fasteners and partial or full fastener yielding. The design basis of ductile failures for timber connections were first introduced by Johansen in 1947, who derived design equations for timber connection with dowel type fasteners in single and double shear assuming that both the fastener and timber were ideal rigid-plastic materials. In the current version of Eurocode 5 the characteristic values, $F_{v,Rk}$, for dowel type fasteners are obtained for a single fastener joint per shear plane using the following equations (BSI, 2004):

For single shear connection:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d & \textbf{...(6.3a)} \\ f_{h,2,k} \cdot t_2 \cdot d & \textbf{...(6.3b)} \\ \dfrac{f_{h,1,k} \cdot d \cdot t_1}{1+\beta} \left[\sqrt{\beta + 2\beta^2 \cdot \left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1}\right)^2 \right] + \beta^3 \cdot \left(\dfrac{t_2}{t_1}\right)^2} - \beta \cdot (1 + \dfrac{t_2}{t_1}) \right] + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.3c)} \\ 1.05 \cdot \dfrac{f_{h,1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right] + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.3d)} \\ 1.05 \cdot \dfrac{f_{h,1,k} t_2 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_2^2}} - \beta \right] + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.3e)} \\ 1.15 \cdot \sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.3f)} \end{cases}$$

For double shear connection

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d & \textbf{...(6.4a)} \\ 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d & \textbf{...(6.4b)} \\ 1.05 \cdot \dfrac{f_{h,1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right] + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.4c)} \\ 1.15 \cdot \sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} & \textbf{...(6.4d)} \end{cases}$$

Where $F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;
$f_{h,i,k}$ is the characteristic embedment strength in timber member i;
d is the fastener nominal diameter;
t_i is the timber thickness or fastener penetration depth;
$M_{y,Rk}$ is the fastener characteristic yield moment;
β is the ratio between the timber members embedment strength;
$F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener.

The characteristic embedment strength, yield moment and withdrawal values can be derived from standard tests or calculated using the relevant equations in Eurocode 5 which were derived from extensive testing over the years. For nails, and screws with a diameter less than 6mm, the yield moment, embedment and axial withdrawal characteristic strength, with predrilling and at an angle of 90° to the timber fibres, can be calculated using the equations shown in Table 6.1.

Table 6.1: *Dowel type fasteners characteristic structural properties from Eurocode 5*

	Notation	Round Nails	Other nails	Screws
Yield moment	$M_{y,Rk}$	$0.3 \cdot f_u \cdot d^{2.6}$	$0.45 \cdot f_u \cdot d^{2.6}$	$0.45 \cdot f_u \cdot d^{2.6}$
Embedment strength	$f_{h,k}$	$0.082 \cdot (1\text{-}0.01 \cdot d) \cdot \rho_k$		
Withdrawal capacity	$F_{ax,Rk}$	$f_{ax,k} \cdot d \cdot t_{pen}$	$f_{ax,k} \cdot d \cdot t_{pen}$	$n_{ef} \cdot (\pi \cdot d_t \cdot l_{ef})^{0.8} \cdot f_{ax,\alpha,k}$
Withdrawal strength	$f_{ax,k}$ **or** $f_{ax,\alpha,k}$	$20 \times 10^{-6} \cdot \rho_k^{\ 2}$	BS EN 1382	$3.6 \times 10^{-3} \cdot \rho_k^{\ 1.5}$

In the Table 6.1, f_u is the fastener tensile strength (N/mm²), d is the nominal diameter for nails and the effective diameter for screws (mm), d_t is the screw thread diameter (mm), t_{pen} the fastener penetration depth in timber (mm), n_{ef} the effective number of fasteners, l_{ef} the pointside penetration length minus one screw diameter, and ρ_k is the timber characteristic density (kg/m^3).

For connections with multiple fasteners in a row parallel to the timber grain, the effective number of fastener needs to be determined as it was shown in previous research works that the connection strength is not directly proportional to the number of fasteners for spacing between fasteners less than 14 times the fastener diameter. For nails and screws the effective number of fasteners in a row parallel to the timber grain is calculated as follows:

$$n_{ef} = n^{k_{ef}} \qquad \textbf{…(6.5)}$$

Where n is the number of fasteners in a row, and k_{ef} is given in the table below, and is a function of the fasteners spacing parallel to the timber grain, a_1, and predrilling.

Table 6.2: *Values of factor k_{ef} (BSI, 2004)*

Spacing[a]	k_{ef}	
	Not predrilled	Predrilled
$a_1 \geq 14d$	1,0	1,0
$a_1 = 10d$	0,85	0,85
$a_1 = 7d$	0,7	0,7
$a_1 = 4d$	-	0,5
[a] For intermediate spacings, linear interpolation of k_{ef} is permitted		

The characteristic load carrying capacity of a joint is then calculated by multiplying $F_{v,Rk}$, calculated as above, by the number of lines of fasteners and the effective number of fasteners in a row in the connection while respecting the sets of rules for spacings between fasteners, distances to the timber members edges and connection details. It has to be noted that in Eurocode 5 the contribution of the rope effect, factor ($F_{ax,Rk}/4$) in Equations (6.3) and (6.4), is limited to 15%, 25% and 50% for round, grooved and other nails respectively, and for screw the contribution is limited to 100%.

While Equations (6.3) and (6.4) were developed for dowel type fasteners, the equations given in Table (6.1) and Table (6.2) were derived from extensive testing on round or threaded nails and screws.

6.3 Helically shaped fastener properties

In the previous sections of this study, tensile, yield moment and embedment tests were performed on helically shaped fasteners for determining their structural behaviour in timber. The results were analysed to determine their characteristic values and compared to the current version of Eurocode 5. As the design equations of Eurocode 5 did not compare favourably to the tests results, new equations were developed for determining helically shaped fasteners' yield moment and embedment strength. The detailed analysis, described in Chapter 3, shows that the characteristic yield moment, $M_{y,helically\ shaped}$, and embedment strength, $f_{h,helically\ shaped}$, of Helically shaped can be determined as follows:

$$M_{y,helifix} = 0.000114 \cdot f_u \cdot d_r^{7.87} + 4499 \qquad \textbf{…(3.7)}$$

$$f_{h,helifix} = (-0.0049 \cdot d_t + 0.0908) \cdot \rho \qquad \textbf{…(3.10)}$$

The equations developed for helically shaped fasteners were of the same form as those developed for common timber fasteners. This shows that, while the helically shaped fasteners exhibit different behaviour to common fasteners, they follow a similar pattern in which the parameters that have an influence are the same to those influencing the behaviour of common fasteners.

Also, it is to be noted that the yield moment function is represented by the fastener root diameter, while the embedment function is represented by the fastener thread diameter.

While this is not ideal for design purposes, the parameters used in the equations provide a more realistic representation of the behaviour of the connectors as observed during testing. For practical reasons the diameters used in the design process can be either the root diameter or thread diameter using the following equations:

$$d_r = 0.224 \cdot d_t + 1.989 \qquad \textbf{…(6.6)}$$

$$d_t = 4.448 \cdot d_r - 8.834 \qquad \textbf{…(6.7)}$$

Tensile tests were performed on the four sizes of helically shaped fasteners used in this study, the results showed that the tensile strength values were constant across the range of diameters. Therefore, if quality procedures are in place it can be assumed that the tensile strength of all helically shaped fasteners can be taken as the characteristic tensile strength determined from the tests, which is 957 N/mm².

6.4 Axially loaded fastener design

In chapter 4 the withdrawal behaviour of helically shaped fasteners in timber was investigated. The study identified the parameters that influence the behaviour and strength of axially loaded helically shaped fasteners. It was also shown that the design equations of Eurocode 5 do not accurately predict the withdrawal capacity of axially loaded helically shaped fasteners; as the withdrawal capacity is greatly underestimated using the design equation for nails, and greatly overestimated using the design equation for screws.

A semi empirical model was developed by analysing individually the parameters that affect the withdrawal load displacement behaviour when axially loaded. However for design purposes the results showed that a specific design equation should be developed for predicting the characteristic withdrawal capacity of helically shaped fasteners.

The results of the experimental tests showed that the factors had a positive or negative influence on the withdrawal capacity of helically shaped fastener. A detailed analysis of the results was undertaken in order to evaluate the influence of the parameters and level of their influence. The influence is taken as positive when for an increase in the parameter value, the withdrawal capacity increases; while the influence was taken as negative when for an increase in the parameter value the withdrawal resistance decreases. The level of influence of the parameters is evaluated by measuring the level of increase or decrease in the withdrawal capacity induced by an increase of the parameter value. A summary of the factors and their influence on the withdrawal capacity is shown in Table 6.3.

Table 6.3: *Influence of the connection parameters*

Parameter	Influence	Level
Pilot hole diameter	Negative	1
Angle to timber grain	Positive	1
Depth of penetration	Positive	2
Fastener diameter	Positive	2
Timber density	Positive	3

As mentioned in Chapter 4, it was shown that the factors do not interact and therefore the withdrawal resistance is a function of the product of the parameters. For that reason and in order to avoid values which could sway the final product (for the pilot hole and angle to fibres), the parameters were evaluated with a reference value. The individual products were evaluated as follows:

Pilot Hole: $\left(\frac{d_t - p_h}{d_t}\right)$

Angle to fibres: $\left(\frac{1}{180-\alpha}\right)$

Depth of penetration l_p

Fastener thread diameter d_t

Timber density ρ_k

As the factors do not affect the pull out resistance of helically shaped fasteners with similar levels of influence, as detailed in Table 6.3, they were affected power coefficients when developing the design equations. This method for calculating the withdrawal factor was based on the analysis of previous work on withdrawal resistance of dowel type fasteners, and on the back of the analysis of helically shaped fasteners axially loaded, as detailed in Chapter 4. In view of all these observations, a withdrawal factor, $f_{ax,Helically\ shaped}$, was developed from the tests configurations.

$$f_{ax,Helifix} = \left(\frac{d_t - p_h}{d_t}\right) \cdot \left(\frac{1}{180-\alpha}\right) \cdot l_p^{1.5} \cdot d_t^{1.5} \cdot \rho_k^2 \qquad \textbf{...(6.8)}$$

Where: d_t is the thread diameter of Helically shaped fasteners, mm

p_h is the pilot hole diameter, mm

α Is the angle of the fastener with the timber fibres in degrees, °

l_p is the depth of penetration, mm

ρ_k Is the timber characteristic density, kg/m^3

For the four sizes of helically shaped fasteners the withdrawal factor calculated was plotted against the characteristic pull out load obtained from the experimental programme described in Chapter 4, Figure 6.1.

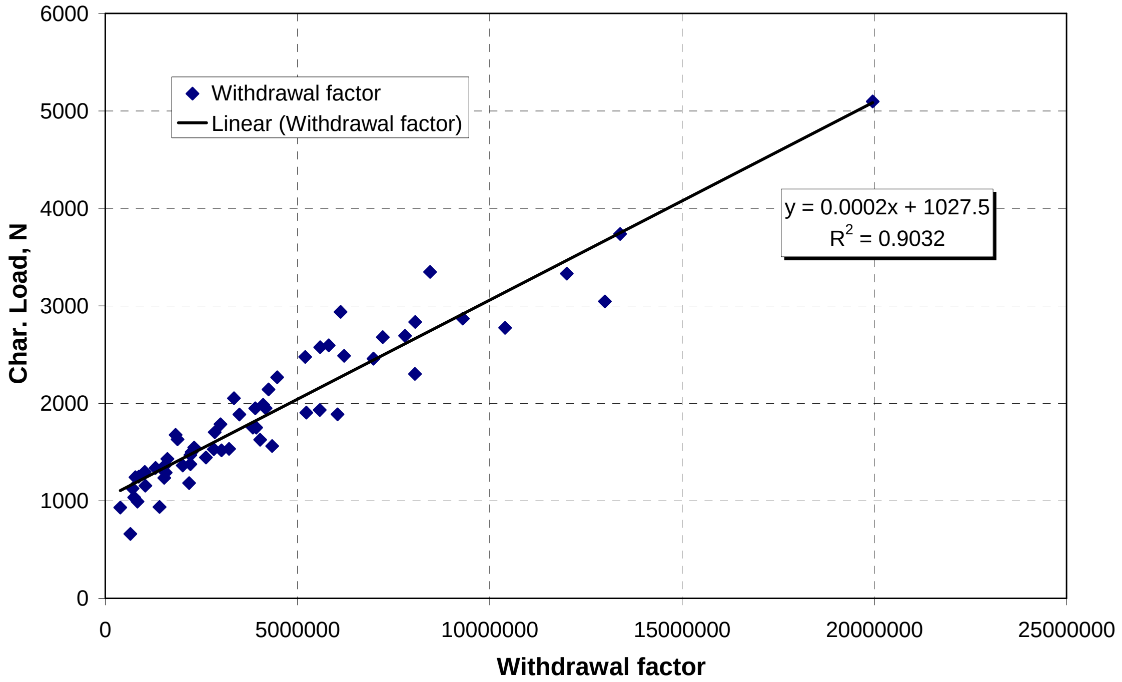


Figure 6.1: Relationship between withdrawal factor and characteristic load

A relationship between the withdrawal factor and the test characteristic load was developed for helically shaped fasteners, and is detailed below.

$$W_k = 0.0002 \cdot f_{ax,helifix} + 1027.5 \qquad \textbf{...(6.9)}$$

Using equations 6.9 the characteristic withdrawal load, W_k in N, for helically shaped fasteners was calculated and compared to the test characteristic load. The results showed that the withdrawal design equations predict the pull out resistance of helically shaped fasteners accurately with an absolute average error of 8.5%, however in some cases the model overestimate the characteristic withdrawal resistance of helically shaped fasteners. As such predictions should be limited and kept within an acceptable range; an arbitrary factor of 0.9 was added to the design equations to reduce the model values. The resulting equation is as follows:

$$W_k = 0.00018 \cdot f_{w,helifix} + 924.71 \qquad \textbf{…(6.10)}$$

The results of equation 6.10 and the calculated percentage error are shown in Table 6.4. Using equation 6.10, the characteristic withdrawal resistance of helically shaped fasteners can be predicted for use in design, providing that the pilot hole diameter, depth of penetration angle to the timber fibres and timber characteristic density are known.

In practice most fasteners are inserted with a pilot hole as recommended in Eurocode 5 with a pilot hole of 0.8 times the fasteners – in the case of helically shaped fasteners the root diameter is the reference for the pilot holes – and perpendicular to the timber fibres. In this general case, the method can be simplified for design purposes.

The pilot hole factor can be written: $\left(\dfrac{d_t - (0.8 \cdot (0.224 \cdot d_t + 1.5912))}{d_t} \right)$

Replacing in equation 6.8 and finally 6.10 the equation becomes:

$$W_k = 0.203 \times 10^{-5} \cdot (0.776 \cdot d_t - 1.5912) \cdot d_t^{0.5} \cdot l_p^{1.5} \cdot \rho_k^2 + 924.71 \qquad \textbf{…(6.11)}$$

Table 6.4: *Helically shaped fastener design characteristic withdrawal loads from equation 6.10*

Fastener	d_{thread}	l_p	p_h	α	ρ_k	Test Char. load	Withdrawal factor	Model Char. Load	Error
	mm	mm	mm	°	kg/m^3	N	$f_{w,helifix}$	N	%
StarTie 10	10	45	0.0	90	350	3046.10	12993087.10	3263.51	6.7
	10	45	2.0	90	350	2775.82	10394469.68	2795.75	0.7
	10	45	3.5	90	350	3348.01	8445506.62	2444.94	-36.9
	10	45	3.8	90	350	2835.56	8055714.00	2374.78	-19.4
	10	45	4.0	90	350	2694.16	7795852.26	2328.00	-15.7
	10	45	5.7	90	350	2577.09	5587027.45	1930.41	-33.5
	10	45	4.0	0	350	1949.01	3897926.13	1626.38	-19.8
	10	45	4.0	23	350	2268.06	4468959.90	1729.16	-31.2
	10	45	4.0	45	350	2477.57	5197234.84	1860.25	-33.2
	10	45	4.0	67	350	2487.73	6209085.87	2042.39	-21.8
	10	20	4.0	90	350	1547.89	2309882.15	1340.53	-15.5
	10	30	4.0	90	350	2143.57	4243524.48	1688.58	-26.9
	10	60	4.0	90	350	3331.70	12002499.74	3085.20	-8.0
	10	45	4.0	90	310	2937.91	6115766.55	2025.59	-45.0
	10	45	4.0	90	560	5097.64	19957381.79	4517.08	-12.9
StarTie 8	8	45	0.0	90	350	2870.42	9297096.32	2598.23	-10.5
	8	45	2.0	90	350	2458.57	6972822.24	2179.86	-12.8
	8	45	3.0	90	350	2596.15	5810685.20	1970.67	-31.7
	8	45	3.2	90	350	1931.94	5578257.79	1928.84	-0.2
	8	45	3.5	90	350	1905.26	5229616.68	1866.08	-2.1
	8	45	4.7	90	350	1751.58	3835052.23	1615.06	-8.5
	8	45	3.5	0	350	1446.14	2614808.34	1395.42	-3.6
	8	45	3.5	23	350	1786.98	2997869.43	1464.37	-22.0
	8	45	3.5	45	350	1885.33	3486411.12	1552.30	-21.5
	8	45	3.5	67	350	1951.30	4165181.43	1674.48	-16.5
	8	20	3.5	90	350	1363.62	1549516.05	1203.66	-13.3
	8	30	3.5	90	350	1704.22	2846642.76	1437.15	-18.6
	8	60	3.5	90	350	2301.49	8051521.60	2374.02	3.1
	8	45	3.5	90	310	1985.66	4102580.92	1663.21	-19.4
	8	45	3.5	90	560	3738.23	13387818.70	3334.56	-12.1
InSkew	6	45	0.0	90	350	1887.50	6038641.20	2011.71	6.2
	6	45	2.0	90	350	1626.93	4025760.80	1649.39	1.4
	6	45	2.8	90	350	1532.74	3220608.64	1504.46	-1.9
	6	45	3.0	90	350	1518.44	3019320.60	1468.23	-3.4
	6	45	3.2	90	350	1529.35	2818032.56	1432.00	-6.8
	6	45	3.8	90	350	1377.28	2214168.44	1323.30	-4.1
	6	45	3.2	0	350	937.53	1409016.28	1178.37	20.4
	6	45	3.2	23	350	1429.62	1615432.68	1215.53	-17.6
	6	45	3.2	45	350	1631.04	1878688.37	1262.91	-29.1
	6	45	3.2	67	350	1500.53	2244450.71	1328.75	-12.9
	6	20	3.2	90	350	992.15	834972.61	1075.05	7.7
	6	30	3.2	90	350	1235.96	1533942.63	1200.86	-2.9
	6	60	3.2	90	350	1561.49	4338644.95	1705.71	8.5
	6	45	3.2	90	310	1469.99	2210717.79	1322.68	-11.1
	6	45	3.2	90	560	2679.48	7214163.35	2223.30	-20.5
TimTie	4.50	45	0.0	90	350	1751.86	3922212.51	1630.75	-7.4
	4.50	45	2.0	90	350	1183.32	2179006.95	1316.97	10.1
	4.50	45	2.4	90	350	1677.26	1830365.84	1254.22	-33.7
	4.50	45	2.7	90	350	1289.89	1568885.00	1207.15	-6.9
	4.50	45	3.0	90	350	1337.45	1307404.17	1160.08	-15.3
	4.50	45	3.6	90	350	1241.28	784442.50	1065.95	-16.4
	4.50	45	3.0	0	350	661.70	653702.09	1042.42	36.5
	4.50	45	3.0	23	350	1035.18	749467.36	1059.65	2.3
	4.50	45	3.0	45	350	1247.61	871602.78	1081.64	-15.3
	4.50	45	3.0	67	350	1156.20	1041295.36	1112.18	-4.0
	4.50	20	3.0	90	350	932.09	387379.01	994.48	6.3
	4.50	30	3.0	90	350	1127.71	711660.69	1052.85	-7.1
	4.50	60	3.0	90	350	1363.12	2012880.40	1287.07	-5.9
	4.50	45	3.0	90	310	1297.44	1025645.23	1109.37	-17.0
	4.50	45	3.0	90	560	2053.43	3346954.68	1527.20	-34.5

6.5 Lateral shear capacity

The lateral shear capacity of a timber joint connection derived from test is taken in Eurocode 5 as the characteristic load multiplied by a moisture content and duration factor (k_{mod}) and divided by a partial factor for material property (γ_m). The characteristic values can be calculated from the equations given in the code or, in their absence by deriving them from tests in accordance to the relevant standards. In case of timber connections test can be performed in accordance to BS EN 26891:1991 (BSI, 1991). From the tests, where the maximum test loads are recorded, the characteristic values is taken as the fifth percentile values calculated in accordance to BS EN 14538:2006 (BSI, 2007).

6.5.1 Load carrying capacity per fastener

The load carrying capacity of a fastener in single or double shear was first developed by Johansen in 1949. It was assumed that both the fastener and timber are perfect rigid-plastic materials to derive the equations corresponding to the possible failure modes. The equations derived by Johansen using the joint geometry are:

For single shear connection:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d & \text{...(6.12a)} \\ f_{h,2,k} \cdot t_2 \cdot d & \text{...(6.12b)} \\ \dfrac{f_{h,1,k} \cdot d \cdot t_1}{1+\beta}\left[\sqrt{\beta + 2\beta^2 \cdot \left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1}\right)^2\right] + \beta^3 \cdot \left(\dfrac{t_2}{t_1}\right)^2} - \beta \cdot \left(1 + \dfrac{t_2}{t_1}\right)\right] & \text{...(6.12c)} \\ \dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta\right] & \text{...(6.12d)} \\ \dfrac{f_{h,1,k} t_2 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_2^2}} - \beta\right] & \text{...(6.12e)} \\ \sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Rk} f_{h,1,k} d} & \text{...(6.12f)} \end{cases}$$

For double shear connection

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d & \text{...(6.13a)} \\ 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d & \text{...(6.13b)} \\ \dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}}-\beta\right] & \text{...(6.13c)} \\ \sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Rk} f_{h,1,k} d} & \text{...(6.13d)} \end{cases}$$

Where $F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;
$f_{h,i,k}$ is the characteristic embedment strength in timber member i;
d is the fastener nominal diameter;
t_i is the timber thickness or fastener penetration depth;
$M_{y,Rk}$ is the fastener characteristic yield moment;
β is the ratio between the timber members embedment strength;

As mentioned above, the characteristic load per fastener per shear plane of a timber to timber joint can be calculated in Eurocode 5 using Equation (6.3). The joint capacity is then calculated from the load per fastener per shear load in accordance to Eurocode 5. From the original equations (6.12 and 6.13) Eurocode 5 included additional resistance due to axial forces in the joints and factors to include the friction between members to enhance the resistance for failure modes 2 and 3.

The experimental programme described in the previous chapter was performed in accordance to those standards, and therefore the characteristic tests values can be calculated for joints with helically shaped fasteners loaded in single and double shear.

To evaluate the design method from Johansen and Eurocode 5 on timber joints with helically shaped fasteners, the following characteristic lateral shear capacities per fastener per shear planes were calculated for all fastener sizes:

- Using equations (6.12) and (6.13) for single and double shear joints, and using the property equations from EC5 for the fastener yield moment and embedment strength (**$F_{k,1}$**);
- Using equations (6.3) and (6.4) for single and double shear joints, and using the property equations from EC5 for the fastener yield moment, embedment and axial strength (**$F_{k,2}$**);

- Using equations (6.12) and (6.13) for single and double shear joints, and using the property equations derived for Helically shaped fastener (3.7 and 3.10) for the fastener yield moment and embedment strength (**$F_{k,3}$**);
- Using equations (6.3) and (6.4) for single and double shear joints, and using the property equations derived for Helically shaped fastener (3.7, 3.10 and 6.10) for the fastener yield moment, embedment and axial strength (**$F_{k,4}$**)

In the calculations described above the following parameters are used:

- Timber characteristic density of C24: 350 kg/m^3;
- Fastener characteristic tensile strength: 957 N/mm^2;
- Fastener root diameter.

Due to the joint geometry, timber thickness and joint geometry, the calculations for single and double shear result in equal values. As the experimental programme also used a symmetrical timber connection an average value can be taken for the experimental characteristic load per fastener per shear plane from tests in single and double shear. The results are then compared to the characteristic tests values (**$F_{k,exp}$**) obtained in single fasteners connections described in Section 5.3; as shown in Figure 6.2 and Table 6.5.

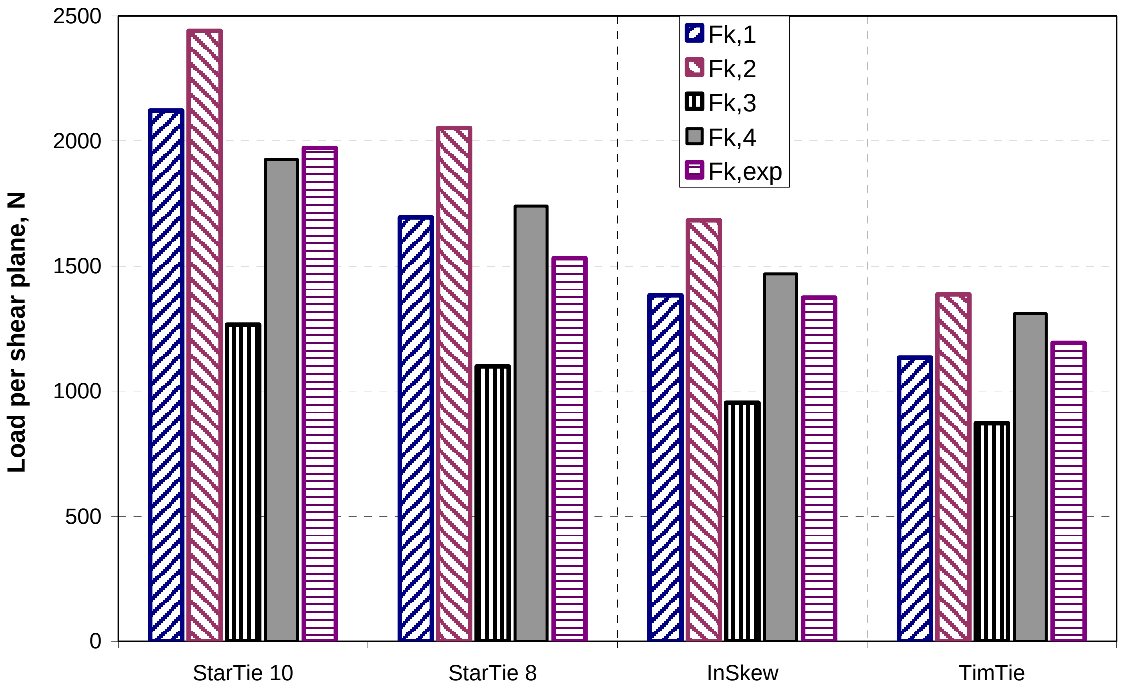


Figure 6.2: Characteristic load carrying capacities from Johansen, Eurocode 5 and tests

Table 6.5: *Load carrying capacity per fastener from design and tests, in N.*

Fastener	$F_{k,1}$	$F_{k,2}$	$F_{k,3}$	$F_{k,4}$	$F_{k,exp}$
StarTie 10	2121.75	2440.50	1265.51	1926.06	1971.65
StarTie 8	1694.18	2051.66	1098.65	1739.69	1531.09
InSkew	1383.15	1682.96	953.56	1468.65	1374.10
TimTie	1134.19	1387.00	871.53	1309.11	1193.48

Table 6.6 shows the average error between the predicted values and values obtained from the experimental programme; with a positive error showing overestimation of the test value, and negative error showing conservative results to the test values.

Table 6.6: *Percentage prediction errors from calculations methods 1) to 4)*

Fastener	$F_{k,exp}$	$F_{k,1}$	$F_{k,2}$	$F_{k,3}$	$F_{k,4}$
StarTie 10	1971.65	8%	24%	-36%	-2%
StarTie 8	1531.09	11%	34%	-28%	14%
InSkew	1374.10	1%	22%	-31%	7%
TimTie	1193.48	-5%	16%	-27%	10%

The results from the calculations as described above were given by the equations of the modes of failure 2 or 3 where there is bedding timber failure in conjunction with partial (mode 2) or full (mode 3) plastic failure of the fastener. From the experimental results, despite the fact that the modes of failures could not properly be identified, the samples showed that the timber was crushed at the interface of members under the fasteners and that the fasteners yielded to a plastic stage; Figure 6.3. This shows the calculation method predicted the failure modes relatively accurately on the evidence from tests.

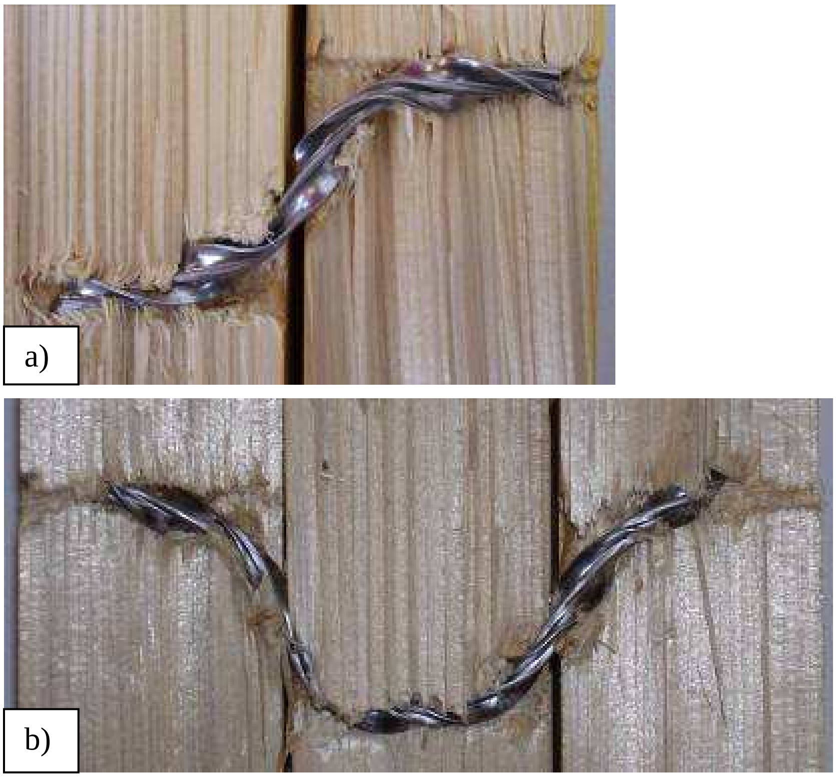


Figure 6.3: Typical failure modes of joints with helically shaped fasteners loaded in single (a) and double (b) shear

The calculation Method 1 and 2 predicts the characteristic load per shear plane with an average absolute error of 6% and 24% respectively. However, reservations can be made on this method of calculations as it was shown in the previous chapter that the equations from Eurocode 5 for determining the yield moment, embedment and axial strength do not predict accurately the characteristic values for helically shaped fasteners. The accuracy of the results from method 1 may be fortunate as the equations do not include the various parameters that can affect the connection resistance. This is confirmed by the fact that the results of calculation method 2 overestimate the characteristic load capacity of joints with helically shaped fasteners.

On the other hand, the results obtained from method 3 and 4 show that the yield theory can be applied to helically shaped fasteners as the results are in line of what could be expected. Indeed, using the equations developed by Johansen the characteristic values calculated are on average 30% below the experimental characteristic values as the factors for friction and rope effect are not included. Hence, when the factors are included as in calculation method 4, the absolute average error is 8%.

Following these observations, an intermediate model between method 1 and method 4 is to be considered for predicting load carrying capacity of timber connections with helically shaped fasteners loaded in single or double shear. The rope effect factor is to be included as in method 4, however slight adjustments on the factors are required so the design calculation method do not overestimate the lateral shear capacity of Helically shaped fasteners in timber connections. This is especially true as helically shaped fasteners were shown to exhibit high withdrawal strength which may introduce unusually high loads in the calculations method in addition to the factors introduced to account for factors such as friction between members.

In accordance with these conclusions, the calculations were carried out to compare results from method 4 without the factors of 1.05 and 1.15 in Equations 6.3 and 6.4 with experimental characteristic loads. The results show that the characteristic lateral shear capacity of Helically shaped fasteners ($F_{v,Rk}$) is predicted with an average error of -2%, Table 6.7.

Table 6.7: *Predicted load capacities for helically shaped fasteners*

Fastener	$F_{k,exp}$	$F_{v,Rk}$	Error (%)
StarTie 10	1971.65	1862.79	-6%
StarTie 8	1531.09	1574.89	3%
InSkew	1374.10	1325.62	-4%
TimTie	1193.48	1178.38	-1%

The analysis carried out demonstrates that the yield theory can be applied to connections with helically shaped fasteners. However the specific equations need to be used in the calculations in order to avoid results that cannot be related to experimental reality; as in the case of calculation method 1. Also, compared to other common timber fasteners, and due to their high withdrawal strength, for helically shaped fasteners the effect of other parameters in the connection such as friction may be ignored by removing the factors from the design equations in Eurocode 5. The lateral shear capacity per fastener per shear plane of timber connections with helically shaped fasteners can thus be determined using the following equations:

- For single shear connection:

$$
F_{v,Rk} = \min \begin{cases}
f_{h,1,k,Helifix} \cdot t_1 \cdot d_r & \textbf{…(6.14a)} \\
f_{h,2,k,Helifix} \cdot t_2 \cdot d_r & \textbf{…(6.14b)} \\
\dfrac{f_{h,1,k,Helifix} \cdot d_r \cdot t_1}{1+\beta}\left[\sqrt{\beta + 2\beta^2 \cdot \left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1}\right)^2\right] + \beta^3 \cdot \left(\dfrac{t_2}{t_1}\right)^2} - \beta \cdot (1 + \dfrac{t_2}{t_1})\right] + \dfrac{W_k}{4} & \textbf{…(6.14c)} \\
\dfrac{f_{h,1,k,Helifix} t_1 d_r}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Helifix}}{f_{h,1,k,Helifix} d_r t_1^2}} - \beta\right] + \dfrac{W_k}{4} & \textbf{…(6.14d)} \\
\dfrac{f_{h,1,k,Helifix} t_2 d_r}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Helifix}}{f_{h,1,k,Helifix} d_r t_2^2}} - \beta\right] + \dfrac{W_k}{4} & \textbf{…(6.14e)} \\
\sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Helifix} f_{h,1,k,Helifix} d} + \dfrac{W_k}{4} & \textbf{…(6.14f)}
\end{cases}
$$

For double shear connection

$$
F_{v,Rk} = \min \begin{cases}
f_{h,1,k,Helifix} \cdot t_1 \cdot d_r & \textbf{…(6.15a)} \\
0.5 \cdot f_{h,2,k,Helifix} \cdot t_2 \cdot d_r & \textbf{…(6.15b)} \\
\dfrac{f_{h,1,k,Helifix} t_1 d_r}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Helifix}}{f_{h,1,k,Helifix} d_r t_1^2}} - \beta\right] + \dfrac{W_k}{4} & \textbf{…(6.15c)} \\
\sqrt{\dfrac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Helifix} f_{h,1,k,Helifix} d_r} + \dfrac{W_k}{4} & \textbf{…(6.15d)}
\end{cases}
$$

With the characteristic yield moment, embedment and axial strengths are determined as follows:

$$M_{y,Helifix} = 0.000114 \cdot f_u \cdot d_r^{7.87} + 4499 \qquad \textbf{…(3.7)}$$

$$f_{h,k,Helifix} = (-0.0049 \cdot d_t + 0.0908) \cdot \rho_k \qquad \textbf{…(3.10)}$$

$$W_k = 0.00018 \cdot f_{ax,Helifix} + 924.71 \qquad \textbf{…(6.10)}$$

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

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

d_r is the fastener root diameter;

d_t is the fastener thread diameter;

t_i is the timber thickness or fastener penetration depth;

$M_{y,Helically\ shaped}$ is the fastener characteristic yield moment;

β is the ratio between the timber members embedment strength;

W_k is the characteristic axial withdrawal capacity of the fastener;

f_u is the characteristic tensile strength of Helically shaped fasteners (957 N/mm^2);

ρ_k is the characteristic timber density;

$f_{ax,Helically\ shaped}$ is the withdrawal factor as defined in section 6.3.

6.5.2 Multiple fasteners connections

For joints with multiple fasteners the characteristic load carrying capacity is determined from the load per shear plane per fastener multiplied by the effective number of fasteners in a row and the number of lines in the connection. The effective number of fasteners is determined for rows of fasteners when inserted parallel to the timber grain. Previous research on timber connection showed that the load capacity of connections with multiple common timber fasteners in a row is not equal to the load per fastener multiplied by the number of fasteners. It was shown that depending on the spacing between fasteners the effective number of fasteners is increasing with increasing spacing, with the effective number of fastener equal to the number of fasteners for spacing equal or greater than 14 times the fasteners diameter.

Hence, the characteristic load carrying capacities of multiple fasteners connections can be determined in accordance to Eurocode 5 for all joint patterns used in the experimental programme. The characteristic loads obtained for multiple fasteners connections ($F_{v,ef,Rk}$) are then compared to the characteristic loads obtained from the tests ($F_{k,exp}$) as determined in accordance to BS EN 26891:1991 (BSI, 1991). The results are detailed in Tables 6.8 and 6.9 for connections with multiple fasteners loaded in single and double shear respectively, with a positive error showing overestimation of the test value, and negative error showing conservative results to the test values.

Table 6.8: *Multiple fasteners joints characteristic loads in single shear*

Joint	Number of Rows	Number of Lines	Row sapcing	k_{ef}	n_{ef}	$F_{v,rk}$	$F_{v,ef,Rk}$	$F_{k,exp}$	Error
			x*d			N	N	N	%
LA	2	1	8	0.74	1.67	1325.62	2214.02	4062.58	-46%
LB	4	1	8	0.74	2.79	1325.62	3697.80	7433.04	-50%
LC	6	1	8	0.74	3.77	1325.62	4991.73	11557.45	-57%
LE	8	1	8	0.74	4.66	1325.62	6175.98	14202.54	-57%
LG	10	1	8	0.74	5.50	1325.62	7284.82	18155.68	-60%
MA	1	2	-	1	1.00	1325.62	2651.24	5292.31	-50%
MB	1	3	-	1	1.00	1325.62	3976.86	5992.93	-34%
MC	1	4	-	1	1.00	1325.62	5302.48	7319.99	-28%
MG	2	2	8	0.74	1.67	1325.62	4428.04	7728.41	-43%
MF	2	2	8	0.74	1.67	1325.62	4428.04	8397.58	-47%
MH	2	2	8	0.74	1.67	1325.62	4428.04	8083.93	-45%
ML	2	2	8	0.74	1.67	1325.62	4428.04	8450.06	-48%
MK	2	2	8	0.74	1.67	1325.62	4428.04	8172.54	-46%
NA	2	2	14	1	2.00	1325.62	5302.48	6863.96	-23%
NB	3	2	14	1	3.00	1325.62	7953.72	11604.63	-31%
NC	4	2	14	1	4.00	1325.62	10604.96	16064.15	-34%
ND	3	2	8	0.74	2.25	1325.62	5977.49	11843.48	-50%
NG	3	2	10	0.85	2.54	1325.62	6745.31	12655.86	-47%
NH	3	2	12	0.925	2.76	1325.62	7324.64	11763.79	-38%

Table 6.9: *Multiple fasteners joints characteristic loads in double shear*

Joint	Number of Rows	Number of Lines	Row sapcing	k_{ef}	n_{ef}	$F_{v,rk}$	$F_{v,ef,Rk}$	$F_{k,exp}$	Error
			x*d			N	N	N	%
CA	2	1	8	0.74	1.67	1574.89	5260.69	11848.36	-56%
CB	4	1	8	0.74	2.79	1574.89	8786.27	18864.48	-53%
CD	6	1	8	0.74	3.77	1574.89	11860.75	25188.05	-53%
CE	8	1	8	0.74	4.66	1574.89	14674.62	33506.72	-56%
CG	10	1	8	0.74	5.50	1574.89	17309.33	41832.18	-59%
DA	1	2	-	1	1.00	1574.89	6299.56	13173.02	-52%
DB	1	3	-	1	1.00	1574.89	9449.34	17723.14	-47%
DC	1	4	-	1	1.00	1574.89	12599.12	23107.68	-45%
DE	2	2	8	0.74	1.67	1574.89	10521.37	22439.58	-53%
DG	2	2	8	0.74	1.67	1574.89	10521.37	21336.70	-51%
DH	2	2	8	0.74	1.67	1574.89	10521.37	22170.13	-53%
EA	2	2	14	1	2.00	1574.89	12599.12	24138.94	-48%
EB	3	2	14	1	3.00	1574.89	18898.68	36768.24	-49%
EC	4	2	14	1	4.00	1574.89	25198.24	39593.21	-36%
ED	3	2	8	0.74	2.25	1574.89	14203.00	33750.10	-58%
EG	3	2	10	0.85	2.54	1574.89	16027.41	31384.34	-49%
EH	3	2	12	0.925	2.76	1574.89	17403.93	30735.15	-43%

The results shows that the predicted characteristic loads are underestimated compared to the experimental characteristic joints load carrying capacity with an average error of -44% (with standard deviation of 10%) and -51% (with standard deviations of 6%) for connections with fasteners in single and double shear respectively.

When analysed in details the results show that:

1. For connections with one line of multiple fasteners in a row, the prediction error is increasing with increasing number of fasteners in a row for joints in single and

double shear. This shows that the calculation method for the effective number of fasteners does not reflect the distribution of load between helically shaped fasteners.

2. This finding is also confirmed by results of similar joints with increasing row spacing, with connections with smaller spacing resulting in greater prediction error.
3. The prediction error for connections with varying line spacings is reasonably constant for connections with fasteners loaded in single and double shear. This confirms the results from Chapter 5 which showed that the line spacing did not affect the connection capacity, and is in line with Eurocode 5.
4. For connections with fasteners loaded in double shear the prediction error is relatively constant for connections with varying number of lines. Whereas, for similar joints with fasteners loaded in single shear the results show that the prediction error is decreasing with increasing number of lines. This is in line with the findings of Chapter 5 where in the semi empirical models developed the number of lines is taken as an effective number smaller than the actual line number; with the effective number of lines being smaller for connections in single shear compared to that of connections in double shear. These findings go against most previous research on timber joints with common timber fasteners. While joint resistance does not seem to be directly proportional to the number of lines, the design method yields conservative results.

As the prediction resulted in overly conservative results, the calculation method was adapted to determine the joints load carrying capacity with the effective number of fasteners in a row equal to the actual number of fasteners (thus k_{ef} equal to 1.0), Table 6.10 and 6.11.

Table 6.10: *Multiple fasteners joints characteristic loads in single shear with* k_{ef} *=1.0*

Joint	Number of Rows	Number of Lines	Row sapcing	k_{ef}	n_{ef}	Fv,rk	Fv,ef,Rk	Fk,exp	Error
			x*d			N	N	N	%
LA	2	1	8	1	2	1325.62	2651.24	4062.58	-35%
LB	4	1	8	1	4	1325.62	5302.48	7433.04	-29%
LC	6	1	8	1	6	1325.62	7953.72	11557.45	-31%
LE	8	1	8	1	8	1325.62	10604.96	14202.54	-25%
LG	10	1	8	1	10	1325.62	13256.20	18155.68	-27%
MA	1	2	-	1	1	1325.62	2651.24	5292.31	-50%
MB	1	3	-	1	1	1325.62	3976.86	5992.93	-34%
MC	1	4	-	1	1	1325.62	5302.48	7319.99	-28%
MG	2	2	8	1	2	1325.62	5302.48	7728.41	-31%
MF	2	2	8	1	2	1325.62	5302.48	8397.58	-37%
MH	2	2	8	1	2	1325.62	5302.48	8083.93	-34%
ML	2	2	8	1	2	1325.62	5302.48	8450.06	-37%
MK	2	2	8	1	2	1325.62	5302.48	8172.54	-35%
NA	2	2	14	1	2	1325.62	5302.48	6863.96	-23%
NB	3	2	14	1	3	1325.62	7953.72	11604.63	-31%
NC	4	2	14	1	4	1325.62	10604.96	16064.15	-34%
ND	3	2	8	1	3	1325.62	7953.72	11843.48	-33%
NG	3	2	10	1	3	1325.62	7953.72	12655.86	-37%
NH	3	2	12	1	3	1325.62	7953.72	11763.79	-32%

Table 6.11: *Multiple fasteners joints characteristic loads in double shear with* k_{ef} *=1.0*

Joint	Number of Rows	Number of Lines	Row sapcing	k_{ef}	n_{ef}	Fv,rk	Fv,ef,Rk	Fk,exp	Error
			x*d			N	N	N	%
CA	2	1	8	1	2	1574.89	6299.56	11848.36	-47%
CB	4	1	8	1	4	1574.89	12599.12	18864.48	-33%
CD	6	1	8	1	6	1574.89	18898.68	25188.05	-25%
CE	8	1	8	1	8	1574.89	25198.24	33506.72	-25%
CG	10	1	8	1	10	1574.89	31497.80	41832.18	-25%
DA	1	2	-	1	1	1574.89	6299.56	13173.02	-52%
DB	1	3	-	1	1	1574.89	9449.34	17723.14	-47%
DC	1	4	-	1	1	1574.89	12599.12	23107.68	-45%
DE	2	2	8	1	2	1574.89	12599.12	22439.58	-44%
DG	2	2	8	1	2	1574.89	12599.12	21336.70	-41%
DH	2	2	8	1	2	1574.89	12599.12	22170.13	-43%
EA	2	2	14	1	2	1574.89	12599.12	24138.94	-48%
EB	3	2	14	1	3	1574.89	18898.68	36768.24	-49%
EC	4	2	14	1	4	1574.89	25198.24	39593.21	-36%
ED	3	2	8	1	3	1574.89	18898.68	33750.10	-44%
EG	3	2	10	1	3	1574.89	18898.68	31384.34	-40%
EH	3	2	12	1	3	1574.89	18898.68	30735.15	-39%

With k_{ef} equal to 1.0 the characteristic load capacity of joints from calculations yield conservative values. However the average error is lower than with k_{ef} calculated as recommended in Eurocode 5. For single and double shear connections the average error is -33% and -40% respectively. Even as there is a reduction in the average error from the calculated and experimental characteristic loads the error is not negligible as for some connections patterns the design value represent only 50% of the experimental characteristic loads obtained. Such difference between the characteristic loads can be explained by the following causes:

1. The characteristic design load capacity per fastener per shear plane is conservative compared to that obtained in tests. Therefore the multiplication of error is emphasised for connections with multiple fasteners and shear planes.
2. The calculations from Eurocode 5 limit the rope effect to 25% of the axial strength of the fasteners. As Helically shaped fasteners exhibit high withdrawal capacity this limitation factor can induce important errors in the calculations of the load capacity of multiple fasteners connections.
3. Multiple fastener connections loaded in single or double shear will inevitably introduce in the connection friction loads that can increase the load carrying capacity of the connections artificially compared to single fastener connections. The multiplication of fasteners increase the friction and as this factor is not included in the calculations; the resulting error between design and experimental values is thus increased.

Three main causes could explain the error between the experimental and design characteristic load carrying capacities obtained. Also, as the possibilities for inducing the error in the calculations compared to the experimental values are multiple and may be interdependent (such as the rope effect and friction) it is difficult to analyse the findings in more depth. However in is important to notice that the design values are conservative which is recommended in design, and the results show that the design method described in this section can be used for structural timber connections with helically shaped fasteners.

6.6 Summary and conclusions

In this chapter the experimental results of the various tests performed on helically shaped fasteners or connections with helically shaped fasteners are analysed in comparison to the design equations of Eurocode 5 (BSI, 2004). The design method is called the yield theory and is derived from the theory developed by Johansen in 1949, the equations were developed using the connection geometry and assuming that the timber and fasteners are ideal rigid-plastic materials. By analysing the internal forces in the connections it was concluded that the connection resistance was a function of the embedment strength, the fastener yield moment, withdrawal strength and the connection geometrical dimensions. With the years and numerous researches the design equations for the connection parameters were developed.

The yield moment and embedment strength of Helically shaped fasteners were investigated in Chapter 3. The experimental results were compared to the design Equations in Eurocode 5 and this showed that specific equations were needed for helically shaped fasteners embedment strength and yield moment for predicting accurately the characteristic capacities.

The resistance and behaviour of axially loaded helically shaped fasteners in timber have been investigated with the results reported in Chapter 4 and compared to the design Equations in Eurocode 5. The results showed that as for yield moment and embedment strength, specific equations were necessary for helically shaped fasteners. The experimental programme investigated the parameters that affect the withdrawal resistance of helically shaped fasteners, an in depth analysis was carried out to evaluate the level of influence of the various parameters. Using such levels of influence a withdrawal factor was calculated for all tests and then an equation for axially loaded helically shaped fastener in timber was developed. This equation predicts the characteristic withdrawal loads with an average error of 7%. In addition, it was noted that a very high percentage of dowel type fasteners in structural timber systems are inserted with a pilot hole of 0.8 times the fastener nominal diameter and perpendicular to the timber grain. Therefore the general withdrawal equation developed was adapted for such input, and a specific equation developed.

The lateral shear capacity of connections is predicted using the Eurocode 5 equations from Johansen’s work. Connections with Helically shaped fasteners experimental characteristic load carrying capacities were evaluated in comparison to Johansen original equations and Eurocode 5 equations. In addition, for both methods, the input parameters (yield moment, embedment and axial strengths) were calculated using the recommended Eurocode 5 methods and the specific equations developed for helically shaped fasteners. The results showed that the load carrying capacity per shear plane per fastener was best predicted using Eurocode 5 equations in conjunction with the specific input equations for helically shaped fasteners. This result is rational with all the findings of the experimental programme in which helically shaped fasteners in timber exhibited load resistance capacity and behaviour that varied from common timber connectors. Following the determination of the load carrying capacity per shear plane per fastener, multiple fasteners connections were investigated and analysed in comparison to Eurocode 5 design method. The results showed that contrary to other timber fasteners the effective number of rows in a connection is equal to the number of rows for a minimum spacing between rows of seven times the root diameter. Also the analysis showed that the joint capacity is not directly proportional to the number of lines in the connection; with the error between the predicted and experimental values decreasing with increasing number of lines. Still, the prediction was conservative for all connection patterns investigated. Following these findings it was shown that the load carrying capacity of timber joints with Helically shaped fasteners could be calculated using Eurocode 5 method in combination with the specific yield moment, embedment and axial strengths equations using an effective number of fastener factor k_{ef} equal to 1.0, such method resulting in characteristic predicted values on average 37% lower than experimental characteristic loads.

Chapter 7 Helically shaped fasteners as shear connectors in timber concrete composite systems

7.1 Introduction

As building techniques evolve and aim to use the material to the maximum of their possibilities composites systems are more and more investigated and developed. In this view, timber-concrete composite structural systems have increasingly been used across Europe, as they exploit the advantages of both concrete and timber in compression and bending respectively. The main drawback of this building technique lies in the connection between the materials and the transfer of the load in order to obtain a composite action with continuity.

The main use in the UK for such connections is for sole plate fixings in timber-framed construction where water can be present in the wall footing through capillarity. In addition, the development of composite structures for external uses, such as bridges, implies that the fasteners need resistance to corrosion. Therefore the main requirements for the fixings between the concrete sub structure and sole plates are as follow:

- Axial load carrying capacity (withdrawal and/or head pull through resistance) in both timber and concrete substrates;
- Water resistant fasteners, due to the possible capillarity effect of the concrete, external applications;
- Lateral shear capacity to sustain horizontal loads, and shear loads in composites systems such as floors.

Helically shaped fasteners have successfully been used for two decades now as remedial crack fixings for masonry and stone structures as well as wall ties in the housing construction market. In the previous chapters of this research, it was shown that their lateral shear and withdrawal behaviour and capacities made them structurally competitive fasteners for timber connections. These characteristics of helically shaped fasteners, combined with their natural resistance to water as stainless steel fixings, respond to the main requirements for fixings in timber-concrete composite systems. Therefore, considering the characteristics of helically shaped fasteners in concrete and masonry materials, as main intended use, and in timber, as seen in the previous chapters; an experimental and analytical programme was developed with the aim to investigate the behaviour and characteristics of Helically shaped connectors as shear connectors for timber-concrete composite structural systems. As detailed in the following section there are two main uses for timber-concrete shear connectors: timber to concrete blocks, and connectors for composite flooring systems. In the following sections both uses are investigated with helically shaped fasteners as timber to concrete shear connectors.

7.2 Background

For over twenty years timber-concrete composite systems have been used in a variety of applications with their use widely increasing over continental Europe (Mettem, 2003). In principle the combination of the two materials appears improbable due to the different hygroscopic and mechanical properties of each of the materials. However such combinations have now been in use, without collapses or serious serviceability issues being reported. The main application of timber to concrete system is in flooring systems, where the use of timber in the tension side and concrete as the slab and compression side result in a structural system generally improving the floor characteristics compared to either all concrete or all timber floors. (STEP 2, 1995). In the United Kingdom such flooring systems are not commonly used due to a lack of knowledge or awareness from engineers on the behaviour and design methods of such flooring systems. However, timber-concrete composites are commonly used in the connection of sole plates to the wall footing in the construction of timber plateform frame building construction (Hairstans, 2007).

In addition to improving the floor characteristics compared to single material floorings systems, timber-concrete flooring systems present several advantages (STEP 2, 1995):

- Creation of a light, rigid and structurally efficient structure;
- Reduction of spring effect compared to timber floors;
- Improvement of sound insulation: 1) the increase mass of the floor compared to timber reduces the transmission of air transmitted noises, 2) higher damping compared to concrete floors improve the impact noise transmission;
- Reduced cost compared to all-concrete floor;
- Improvement of the fire protection of buildings.

In timber to concrete floors, the shear resistance of the connection and the shear transfer has been the subject of numerous research programmes, as the components of the system need to be acting together in order to create a structural system. Various connectors have therefore been investigated; the most common are shown in Figure 7.1.

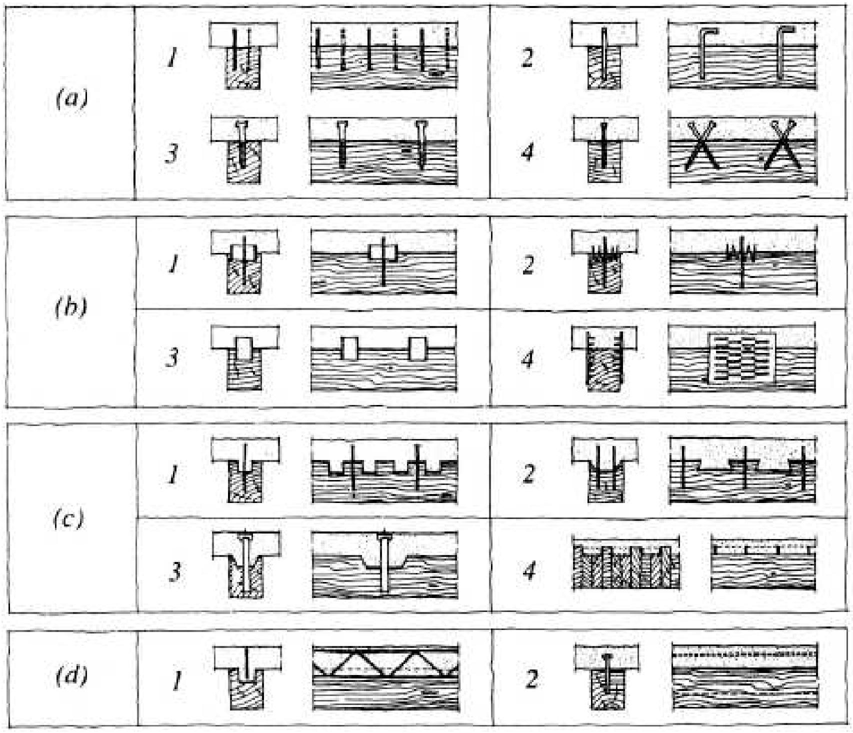


Figure 7.1: Examples of different timber-concrete connections systems: a) dowel type connectors; b) surface connections; c) notched connections; d) bonded connections (STEP 2, 1995)

Connectors as shown in Figure 7.1, are traditionally used or where developed for timber to timber connections. In addition to these, researchers developed shear connectors specifically for timber-concrete composite systems, Figure 7.2. Connectors A and B are screws of diameter 6mm and are usually installed at an angle of 45° to the timber surface; Connector C consist of a steel plate and a large diameter dowel encaged in the concrete fixed to the timber members using two coach screws; Connector D was developed for timber-concrete composite systems requiring high stiffness and high resistance, it consists of a steel sheet anchored in the timber using common timber screws (Steinberg et al, 2003).

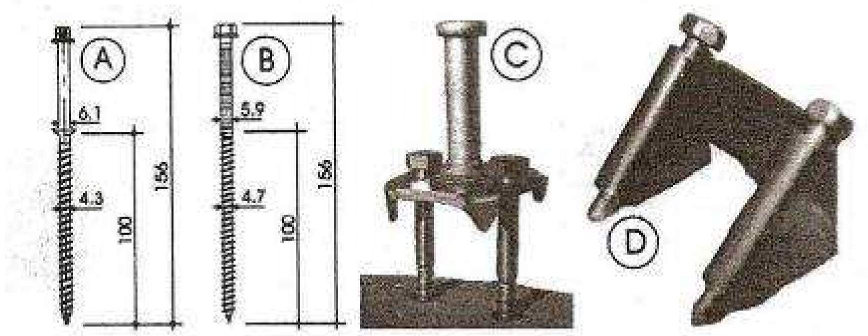
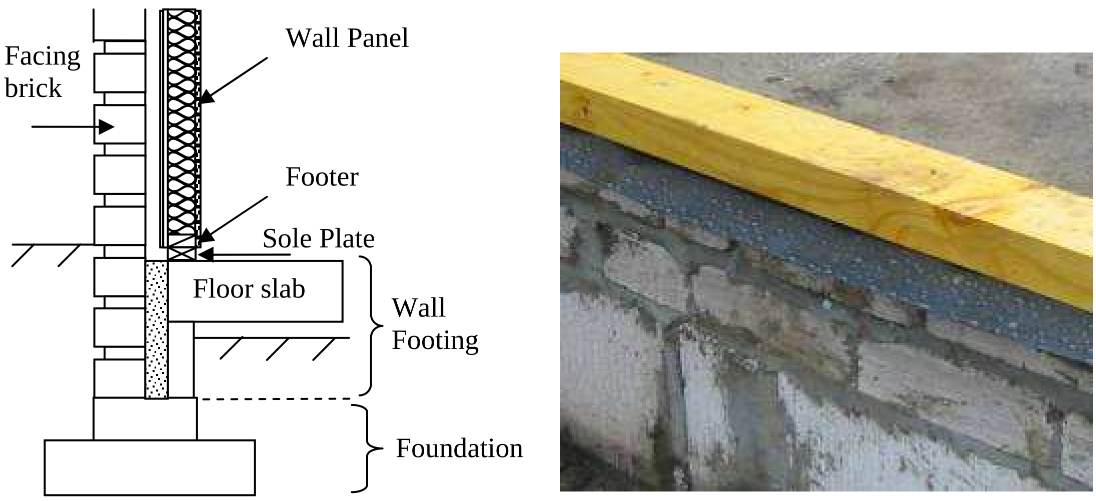


Figure 7.2: Connectors developed for Timber-concrete composite systems (Steinberg et al, 2003).

In timber plateform frame construction the load transfer of horizontal forces to the sub structure is mainly realised by shear resistance of connectors between the sole plate and wall footing, generally constituted of common bricks or concrete bricks, Figure 7.3.



a) Typical foundation detail

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

Figure 7.3: Timber-concrete sole plate connection (Hairstans, 2007)

In this connection detail, the transfer of horizontal forces, mainly due to the wind forces acting on the structure, from the timber panels to the sub-structure is only realised by the connectors between the two materials. The potential friction effect that could exist between the timber and concrete is being eliminated by the introduction of a damp proof coursing between the two layers in order to prevent water migration in the timber members.

The most commonly used fasteners for sole plates connections in the UK are:

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

With the development of timber-concrete composites systems across continental Europe, and the increasing number of timber frame buildings in the United Kingdom, researchers have set to develop design rules and models for timber-concrete composite systems. The first models were developed following extensive testing on floor systems using various connection methods (Van Der Linden, 1999); the different studies developed linear elastic analytical models, with the effect of joint slip taken into account in the majority of the cases. Nowadays, finite elements methods are being used for analysing and modelling the behaviour of timber-concrete composite systems (Dias et al, 2006).

The development of design rules for timber-concrete composite systems has also been investigated following the various researches. In 1995, Ceccotti suggested that timber-concrete floor systems could be designed according to Eurocode 5 Annex B formulas. While such formulas can be used for short term calculations, numbers of time dependent phenomena differ between timber to timber composite systems and timber to concrete composite systems (such as inelastic strains in the concrete slab and difference in creep coefficients) may lead to significant approximations when designing to long term loading (Schanzlin, 2007). For the design of the shear connection between the two materials research demonstrated in 2003 that smooth and threaded nails the timber to concrete connections showed the same load carrying capacity to timber to thick steel as calculated using Eurocode 5 design formulas for lateral shear connections (Dias, 2005), Equation 7.1.

$$F_{v,Rk} = \min \begin{cases} f_{h,k} \cdot t_1 \cdot d \left[\sqrt{2 + \dfrac{4 \cdot M_{y,Rk}}{f_{h,k} \cdot d \cdot t_1^2}} - 1 \right] + \dfrac{F_{ax,Rk}}{4} & \textbf{...(7.1a)} \\ 2.3\sqrt{M_{y,Rk} \cdot f_{h,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} & \textbf{...(7.1b)} \\ f_{h,k} \cdot t_1 \cdot d & \textbf{...(7.1c)} \end{cases}$$

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.

The failure modes and corresponding equations were first developed by Johansen (1949), when studying the behaviour of timber joints. As for timber to timber joints, the equations predict the ultimate strength of connections with dowel type fasteners due to bearing failure in the timber member or simultaneous development of bearing failure in the timber and yield point in the fastener. The mode of failure varies with the joint geometry and properties of the timber and fastener.

7.3 Sole plate anchoring systems

In the UK timber plateform frame construction accounts for 20% of the housing construction market, with this percentage likely to grow with the new regulations planned. As shown in Figure 7.3, the wall panels are fixed to the substructure via a sole plate which in turn is fixed to the foundations. In this part of the research the viability of helically shaped fasteners were studied along with common fasteners for sole plate connections.

7.3.1 Experimental investigation

The fasteners were tested in 7 N/mm^2 concrete blocks. The specimens were assembled with the substrate and timber predrilled according to the fixings specifications, and the fasteners inserted to a depth equal or superior to the minimum required depth of penetration in the substrate. Damp proof coursing was placed at the interface between the two materials according to site practises. The timber used in the experimental programme was of grade C24 according to BS EN 338:2003 (BSI, 2003).

A symmetrical arrangement, comprising two shear planes with each containing two fasteners, was used in the testing programme. Four specimens were tested for each fastener in accordance with BS EN 1380:2009 (BSI, 2009c) and BS EN 26891:1991 (BSI, 1991) requirements. A typical test arrangement is shown in Figure 7.4.

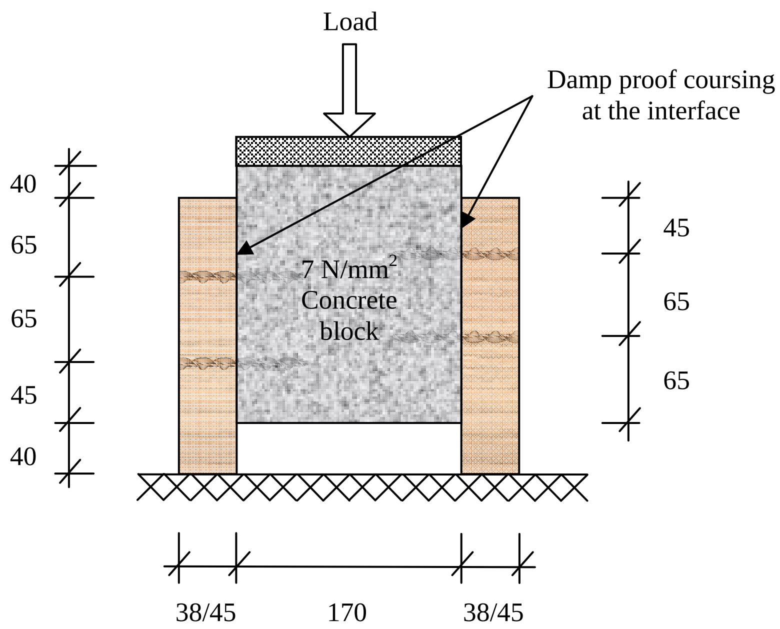


Figure 7.4: Typical sole plate connection test set up

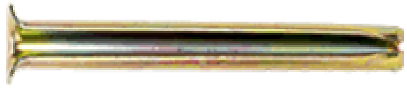
Details of the fasteners used in the experimental programme are shown in Table 7.1. Along with Helically shaped fasteners masonry screws, masonry anchors and Express nails were tested, Figure 7.5.

Table 7.1: *Details of fasteners tested as shear fixings for sole plates*

Fixing Type	Specification	Length	Root diameter	Thread diameter	Smooth Shank Diameter
		mm	mm	mm	mm
Helifix fasteners	StarTie 10	N/A	4.25	10.0	N/A
	StarTie 8	N/A	3.75	8.0	N/A
	InSkew	N/A	3.35	6.0	N/A
Masonry screws	MSC36082	82	3.8	5.4	3.8
	BTB4C82	82	4.4	6.4	4.7
Masonry anchors	KF7.5x80	80	5.2	7.4	N/A
	KF7.5x100	100	5.2	7.4	N/A
Express nails	EXPN6x100	100	N/A	N/A	6
	EXPN8x70	70	N/A	N/A	8
	EXPN8x90	90	N/A	N/A	8

a) MSC & BTB masonry screw

b) KF masonry anchor



c) EXPN express nail

Figure 7.5: Common sole plate fixings

According to Helically shaped documentation the minimum depth of penetration in light concrete blocks (5-20N/mm^2) is 70mm, which would provide lengths for helically shaped fasteners of 115mm and 108mm for tests in 45 and 38mm timber respectively. However the recommendations are for wall ties specifications, where a gap between the two substrates exists. As the tests represent a different use for helically shaped fasteners, and to compare them with common sole plate anchors, it was decided to insert Helically shaped fastener to a depth equivalent to that of commonly used sole plate to foundation fixings.

Three tests series were performed in this study. In the first test series all fasteners described in Table 7.1 were tested in 45mm sole plates; helically shaped fasteners were inserted in the concrete blocks to a depth of 50mm. In test series 2 and 3, only StarTie fasteners were tested. In Series 2 the sole plate was of thickness 38mm; and in series 3 helically shaped fasteners were inserted to a depth of 70mm. A summary of the sole plate and fixing depth of penetration in the concrete blocks is given in Table 7.2.

Table 7.2: *Test programmes fixings and sole plates dimensions*

Test Series 1			
Fastener	Timber thickness	Penetration depth in block	Fastener length
	mm	mm	mm
StarTie 10	45	50	95
StarTie 8	45	50	95
InSkew	45	50	95
MSC36082	45	37	82
BTB4C82	45	37	82
KF7.5x80	45	35	80
KF7.5x100	45	55	100
EXPN6x100	45	55	100
EXPN8x90	45	45	90
EXPN8x70	45	25	70
Test Series 2			
StarTie 10	38	50	88
StarTie 8	38	50	88
Test Series 3			
StarTie 10	45	70	115
StarTie 8	45	70	115

The testing programme was developed and conducted using standard 7N/mm² concrete blocks. From each block two tests samples and two 100mm cubes were cut. The cubes were tested in compression in order to determine density and compressive strength of the concrete blocks. Also, after each tests samples were cut from the timber to measure and record the density and moisture content of each test specimen.

7.3.2 Results and analysis

For comparison purposes, and considering results which showed that timber-concrete samples could be designed using Eurocode 5 equations for thick steel to timber connections, design calculations have been carried out and the results are shown along with experimental results in Figure 7.7. Two methods were used for calculating the Eurocode 5 design values:

a) The average experimental data was used for input in the equations (fastener yield strength according to BS EN 14592:2008 (BSI, 2009a), timber density of samples),
b) The characteristic values from Eurocode 5 design equation are used for fasteners yield moment, and C24 characteristic density from BS EN 338:2003 (BSI, 2003).

To calculate design values using Eurocode 5 equations the tensile strength and yield moments of the fasteners were determined by testing. The tests carried out were similar to those described in Chapter 3 for the tensile strength and yield moment of helically shaped fasteners. Due to the unusual shape of Express nails, yield moment tests were carried out in three configurations, the conservative value was used in the design equations, Figure 7.6. It has to be noted that the presence of damp proof coursing limits the friction between the timber and concrete.

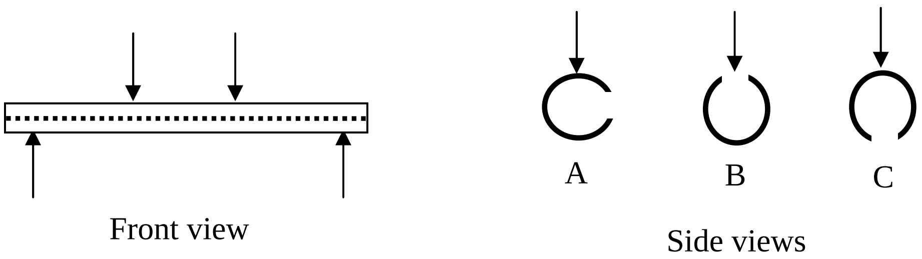


Figure 7.6: Yield moment tests, configuration for minimum yield moment of Express nails

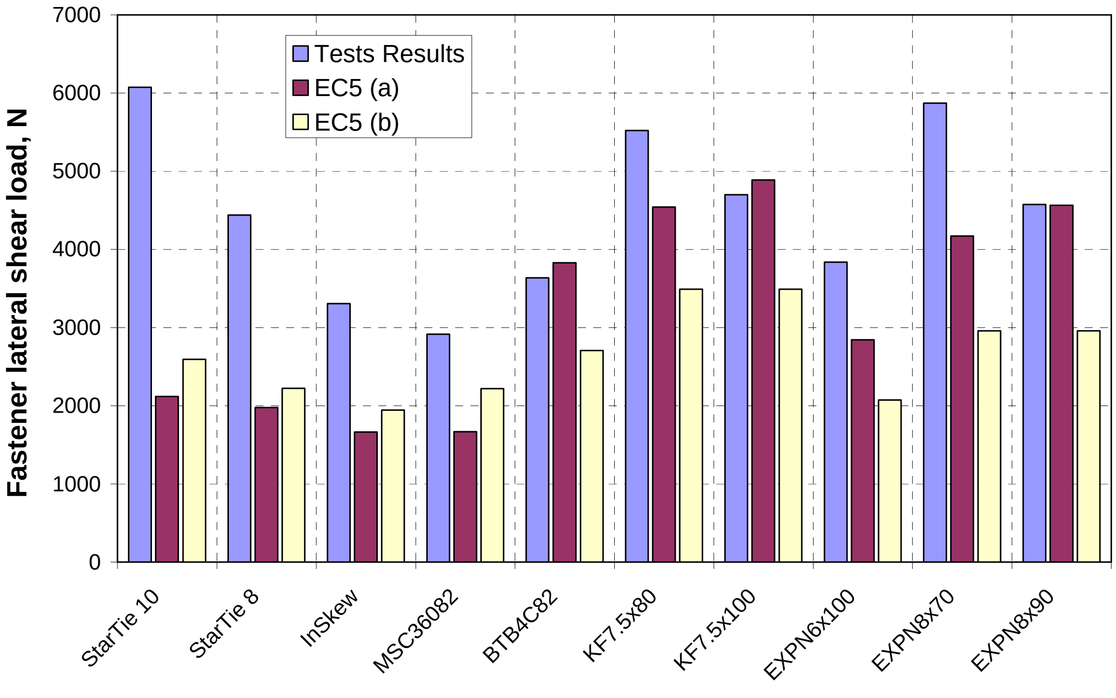


Figure 7.7: Timber-concrete characteristic lateral shear fixing capacity in 7N/mm² blocks

Shown in Table 7.3 are the characteristic test results of the experimental investigation and the results of the design calculations, using the two methods described above. The results show that:

- Helically shaped fasteners achieve similar load carrying capacities to commonly used fasteners for sole plate connections,
- For all fasteners, Eurocode 5 method b) design calculation result in conservative design values, and for Helically shaped StarTie 10 and StarTie 8 fasteners the design values are overly-conservative,
- Eurocode 5 method a) design calculation results in some cases (BTB4C82, KF7.5x100, EXPN 8x90) in over estimation of load capacities,
- For Helically shaped fasteners the lateral shear capacity increases with increasing fastener diameter,
- For same diameter fasteners higher resistance was achieved for fasteners least embedded in the concrete – shortest fasteners (KF and EXPN8 fasteners).

Design calculations resulted in equation 7.1(a) being the minima for method b) for all fasteners, and for method a) half the fasteners resulted in equation 7.1(a) being the least favourable (in grey in Table 7.3), and for the other half equation 7.1(b) was decisive. Equation 7.1(a) corresponds to bearing failure of the timber member under the fixing, and Equation 7.1(b) corresponds to simultaneous bearing failure of the timber member and yielding of the fixing. Observations on the samples with common timber-concrete fixings, following testing, showed bearing failures of the timber in most cases; however, yielding of the fasteners was often difficult to assess as the samples were allowed to reach large displacements. However in the case of helically shaped fasteners yielding of the fasteners could be observed in the samples.

Table 7.3: *Tests results comparison between helically shaped and commonly used fasteners*

Fasteners	**Tests Results**	**EC5 (a)**	**EC5 (b)**
	N	**N**	**N**
StarTie 10	6074,81	2330,51	2710,09
StarTie 8	4439,83	2138,84	2326,96
InSkew	3306,67	1834,30	2036,00
MSC36082	2915,95	1840,35	2323,03
BTB4C82	3636,19	4035,92	2827,96
KF7.5x80	5521,90	4743,95	3634,12
KF7.5x100	4700,60	5130,66	3634,12
EXPN6x100	3836,45	2997,39	2182,36
EXPN8x70	5871,31	4385,15	3111,16
EXPN8x90	4573,32	4831,49	3111,16

During testing two modes of failures were observed: ductile and brittle failures due to shearing of the fixing see Figure 7.8. Fixings which exhibited lateral shear brittle failures were those that exhibited such failure when determining experimental yield moments. Also, brittle failure was observed in the case of KF7.5x100 fasteners which may explain the lower values achieved compared to FF7.5x80 fasteners.

In 2005, Dias mentioned that the design equations developed for thick steel to timber connections, for use in timber to concrete connections result in overestimated values, due to the consideration that perfect clamping is assured by the concrete which proves unrealistic, as the bearing capacity of the concrete is overestimated. However, configurations such as those described in this experimental programme result in conservative results for all types of fasteners tested.

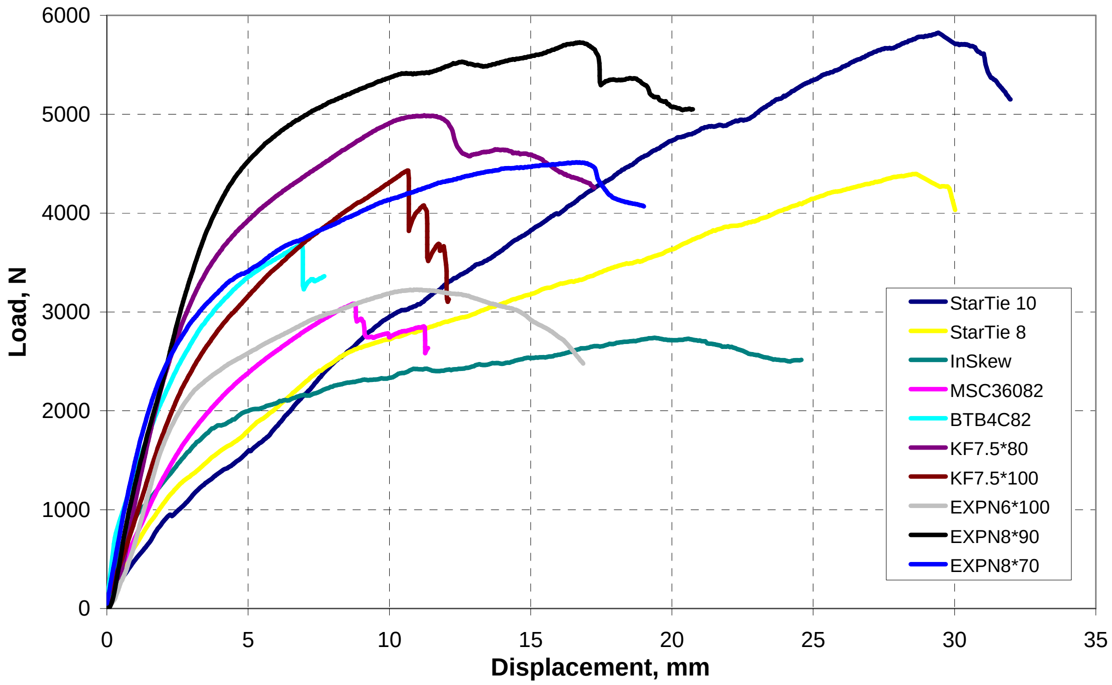


Figure 7.8: Fasteners typical load displacement curves in Timber-concrete connections

Figure 7.8 shows that helically shaped fasteners behave in a more ductile behaviour than common fixings, however they achieve relatively low stiffness compared to other fasteners. The stiffness of timber-concrete sole plate connection is important for sustaining instant or impact loads. As the connection detail is aimed at sustaining loads from wind loads and improbable seismic loads, ductility and elasticity could be preferred over connection stiffness in these situations in order to respect serviceability in extreme cases and to avoid brittle failures and collapse of the structure.

In 2005 Dias observed that using the design equations for timber to thick steel connections was the most realistic and available tool compared to timber to timber or timber to thin steel design equations. However this model considers that the clamp of fastener in the concrete is infinitely rigid; which has been shown to be untrue. As concrete is an elasto-plastic material crushing also occurs in the concrete under the bearing area of the fastener. The problem is often bypassed by introducing a gap between the two substrates in the model which can be assumed to correspond to the damaged area in the concrete.

Other studies have focused on using similar design equations for determining the load carrying capacity of such connections by developing models of timber to thick steel substrates with an interlayer (Dias, 2005). Such a model may be more accurate as it takes into account the bearing strength of the interlayer in the design. However both methods require that the thickness of the gap or interlayer is evaluated efficiently and accurately.

The results of tests Series 2 and 3 on helically shaped fasteners StarTie 10 and StarTie 8 fasteners are shown in Figure 7.9 along with design values calculated according to the two methods described above.

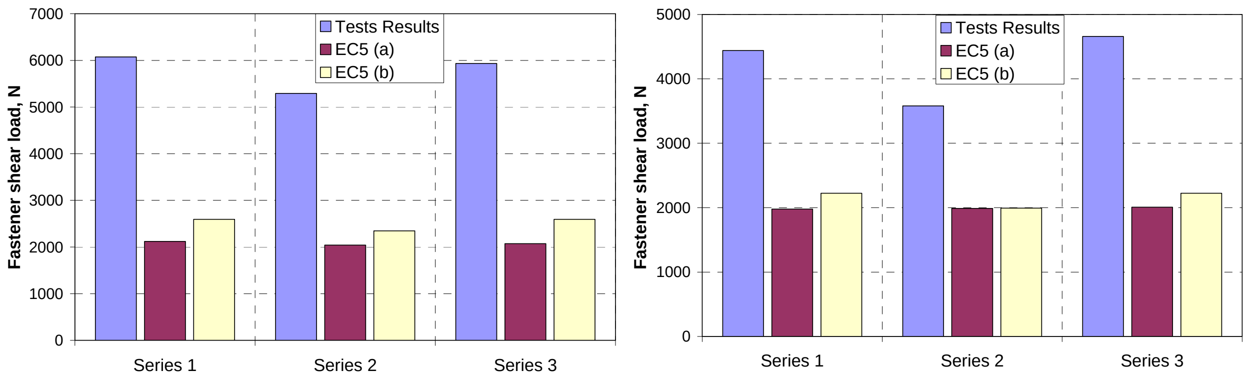


Figure 7.9: Results of Helically shaped fasteners test series 1, 2 and 3 for Helically shaped StarTie 10 (left) and StarTie 8 (right) fasteners

While the tests performed on helically shaped fasteners StarTie 10 and 8 fasteners over the three test series cannot be used for in depth analysis due to the number of samples and for the low range of investigation for each variable, preliminary observations can be made:

- Lower load carrying capacity per fastener is achieved in testing with sole plate of thickness 38mm compared to sole plates of thickness 45mm. A reduction of 12.8% and 19.3% is witnessed for StarTie 10 and StarTie 8 fasteners respectively.
- Tests results for fasteners with greater penetration depth in concrete substrate are not very conclusive. For StarTie 10 fasteners results of series 3 (greater penetration depth) lower than for series 1 while the opposite happened for StarTie 8 fasteners. However the percentage difference between the two test series (1 and 3) is only of +4.9% and -2.3% for StarTie 10 and StarTie 8 fasteners respectively.
- Eurocode 5 design calculations resulted in values on average 60.1 % and 50.5% lower than test results for StarTie 10 and StarTie 8 fasteners respectively between method (a) and (b).
- It can be noticed that design values are equal for method b) in test series 1 and 3 as the numerical model does not take into account the depth of penetration in the concrete substrate.
- Design calculation method b) result in slightly higher values than design calculation method a) for all three series.

Such results may suggest that while a minimum depth of penetration is required for design, the load carrying capacity of a fastener in timber-concrete block connection may reach a maximum even with increasing depth of penetration. In addition, as could be expected higher loads are achieved when greater thickness of timber are used as sole plates. Comparing the test results for the three test series of helically shaped fasteners to the results of the design equations from both method a) and b) showed that the design method is over conservative. While conservative results from design equations are generally expected, such difference between actual values and design characteristic values can be considered too great. It has to be noticed that the design calculations used above were initially developed for timber to steel plates with round nails, which may explain the gap between design and tests results. This also confirms that the design equations need to be adapted for helically shaped fasteners for connections between timber and concrete.

7.3.3 Design of timber to concrete blocks connections

In the above section, the design values were calculated using either the characteristic test values from existing standards or the existing design equations from Eurocode 5. However, it was shown in the previous chapters that while the standards and design equations result in predictive results for commonly used timber fasteners, specific design equations should be developed and used for designing timber connections with helically shaped fasteners. Considering these previous findings, the following equations were used for input in the timber to thick steel connection design equations of Eurocode 5 (Equations 7.1 a, b and c):

$$M_{y,helifix} = 0.000114 \cdot f_u \cdot d_r^{7.87} + 4499 \qquad \textbf{...(3.7)}$$

$$f_{h,helifix} = (-0.0049 \cdot d_t + 0.0908) \cdot \rho_k \qquad \textbf{...(3.10)}$$

$$W_k = 0.203 \times 10^{-5} \cdot (0.776 \cdot d_t - 1.5912) \cdot d_t^{0.5} \cdot l_p^{1.5} \cdot \rho_k^2 + 924.71 \qquad \textbf{...(6.11)}$$

Where: $M_{y,Helically\ shaped}$ is the fastener characteristic yield moment, N.mm;

$f_{h,Helically\ shaped}$ is the fastener characteristic embedment strength, N/mm²;

W_k is the withdrawal capacity in timber, in N

d_r is Helically shaped fastener root diameter, mm;

d_t is Helically shaped fastener thread diameter, mm;

l_p is the fastener length in timber, mm

f_u is Helically shaped fastener characteristic tensile strength, N/mm²;

ρ_k is the timber density, kg/m^3.

Using the characteristic timber density of C16 and the dimensions from test in the above equations the minimum design characteristic shear capacity is given from Equation (7.1b) where the fastener presents two yield points; the results are given in Table 7.4. Table 7.4 shows the average error between the predicted values and values obtained from the experimental programme; with a positive error showing overestimation of the test value, and negative error showing conservative results to the test values

Table 7.4: *Characteristic test and design values*

Series	Fastener	Characteristic tests values	Characteristic design values	
		N	**N**	**Error (%)**
1	StarTie 10	6074,81	3903,05	-36%
	StarTie 8	4439,83	2963,13	-33%
	InSkew	3306,67	2391,33	-28%
2	StarTie 10	5293,61	3821,04	-28%
	StarTie 8	3579,62	2908,23	-19%
3	StarTie 10	5934,38	3903,05	-34%
	StarTie 8	4659,99	2963,13	-36%

The results show that the design equations yield conservative values of the lateral shear capacity of Helically shaped fasteners in concrete blocks with an average error of -31%. While such error can be deemed too conservative and uneconomical, it can lay the foundations for deeper research to be performed on the subject and shows that this proposed design method can be applied for structural purposes.

It can be noted that in Equation (7.1), to incorporate the effect of parameters that can affect the connection strength but are not incorporated in the formulae (such as friction between members) a factor is introduced to artificially increase the characteristic design values. As the average error between the experimental and design values is relatively large, such a factor could be introduced for predicting Helically shaped fasteners lateral shear capacity in timber to concrete blocks capacity. However, in order to assess such factor, an extensive experimental programme, investigating the various parameters affecting the connection behaviour and strength should be performed.

As mentioned above design equation models with a gap or interlayer have been developed for predicting the load carrying capacity of timber to concrete connections. However it was noted by Dias (2005) that such models should be used in the case of overestimation of the characteristic load carrying capacity of the timber by thick steel model. As for Helically shaped fasteners the model underestimates the characteristic loads such models should not be used.

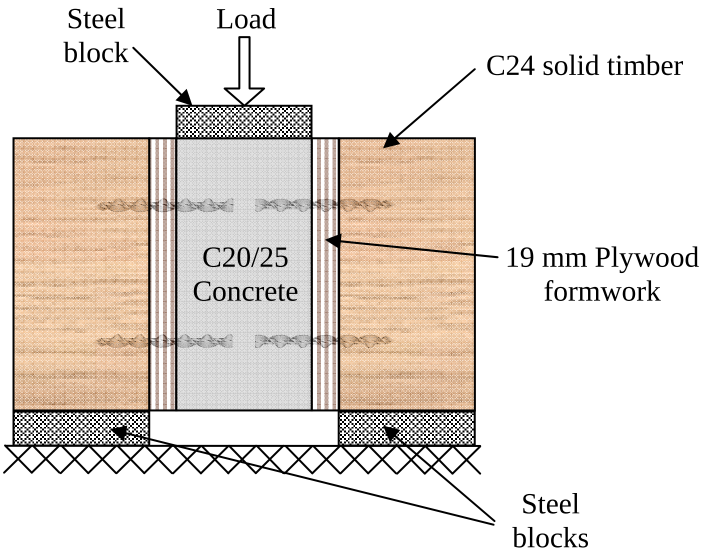
7.4 Timber-concrete composites shear connectors

The development of timber concrete composite structural elements started when a shortage of steel emerged between the two world wars (Dias, 2005), and their uses are now spreading due to the advantages such elements can provide compared to single material structural elements, specifically in refurbishment of timber floors. Such technique is often used as flooring systems in order to create light weight floors in residential buildings while keeping the advantages of the timber in bending and of concrete in compression. As refurbishment or for creating new flooring, various techniques can be used during construction. The concrete slab can be casted directly on top of the timber joists where the shear connectors are placed, and formwork need to be placed between the joists. Alternatively, for easier and faster construction to avoid formwork, precast concrete slabs have also been used between joists, or in most cases plywood or floor boards are placed on top of the timber joists. Both methods present the advantages of eliminating installation of formwork and to create a cleaner finish to the floor (STEP 2, 1995). Nowadays, most timber concrete floors use plywood or floor boards as formwork.

As efficient timber connectors, and with a history of successful concrete and brick connections, helically shaped fasteners have been investigated as connectors to transfer shear in timber to concrete floorings. As full scale floorings could not be tested for various reasons, only the connection details were investigated in isolation. Helically shaped fasteners and common timber connectors were used in the study. As mentioned above the most commonly used timber concrete system is with floorboard used as formwork for the concrete slab; therefore the experimental programme aimed at investigating the behaviour and performance of the fasteners as shear connectors in such flooring types.

7.4.1 Experimental programme

The experimental programme was developed following an extensive review of previous research on the subject. As for concrete blocks to timber connections, the experimental tests were performed using a sandwich connection, in a symmetrical arrangement, comprising two shear planes with each containing two fasteners. Four specimens were tested for each fastener in accordance with BS EN 1380:1999 (BSI, 1999 (b)) and BS EN 26891:1991 (BSI, 1991) requirements. A typical test arrangement is shown in Figure 7.10.



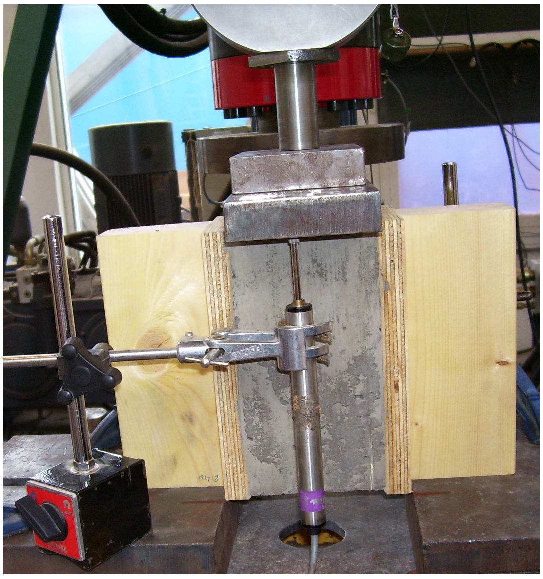


Figure 7.10: Typical timber to cast concrete shear test arrangement

The test arrangement was aimed to study the shear connection between the timber joist and concrete slab. As plywood boards are placed at the interface of the two materials the shear connections can be assimilated as including a large gap between the two substrates. The plywood used in the experimental programme was obtained form a local build centre, with a characteristic density of 460 kg/m^3.

The concrete was mixed within the university laboratories. To achieve the target resistance of 20 to 25 MPa, the concrete was composed of the following materials in the following quantities for one cubic meter:

- Cement 350 kg;
- Dry fine Sand 815 kg;
- 20mm aggregate 1000 kg;
- Water 210 kg.

Due to the size of the mixer the concrete was mixed in various batches. Each batch was used for casting about 20 samples and three 100x100x100mm cubes that were tested to determine the compressive strength of the concrete. This also controlled that the concrete used in the various samples was of similar strength, as shown in Appendix A. The samples and cubes were casted in one day, and left to dry for a day under sheets of polyethylene to minimise loss of moisture. The formwork was stripped after two days and left to cure for another 26 days before testing. All samples were tested in a day.

After testing, small clear samples were cut from the timber in order to determine the samples density and moisture content.

Details of the fasteners used in the experimental programme are shown in Table 7.5. Along with Helically shaped fasteners typical timber fasteners were tested for comparison purposes, the fasteners used are detailed in Chapter 3.

Table 7.5: *Details of fasteners tested as shear fixings for timber concrete flooring systems*

Fastener	Code	Shank diameter	Root diameter	Length
		mm	mm	mm
Ulti-mate woodscrew 6*100	UM6	6	4.2	100
BZP steel woodscrew No:12	BZPNo12	5.5	3.9	88
Annularly threaded nail 5*100	ATN5	5.6	4.6	100
Round wire nail 4.5*100	RWN45	4.5	N/A	100
Helifix StarTie 10	S10	10	4.25	100
Helifix StarTie 8	S8	8	3.75	100
Helifix InSkew	IN	6	3.35	100
Helifix TimTie	TIM	4.5	3	100

7.4.2 Results and analysis

As for tests of helically shaped fasteners as tie for timber to concrete blocks connections, design calculations were carried out and the results are shown along with experimental results in Figure 7.11. The two methods used for calculating the Eurocode 5 design values were as follow:

a) The average experimental data was used for input in the equations (fastener yield strength, timber density),
b) The characteristic values from Eurocode 5 design equation are used for fasteners yield moment, and C24 characteristic density from BS EN 338:2003.

Table 7.6: Timber to concrete connection results

Fastener	Tests Results	EC5 (a)	EC5 (b)
	N	N	N
StarTie 10	3668.28	2013.01	2503.70
StarTie 8	2458.16	1945.04	2153.68
InSkew	2392.46	1554.37	1887.09
TimTie	1634.66	1471.48	1560.90
UMW6	2994.13	3265.04	2807.41
BZPN°12	2979.09	2892.30	2577.52
ATN5	2872.75	3464.20	2862.54
RWN45	1704.60	3136.86	2521.09

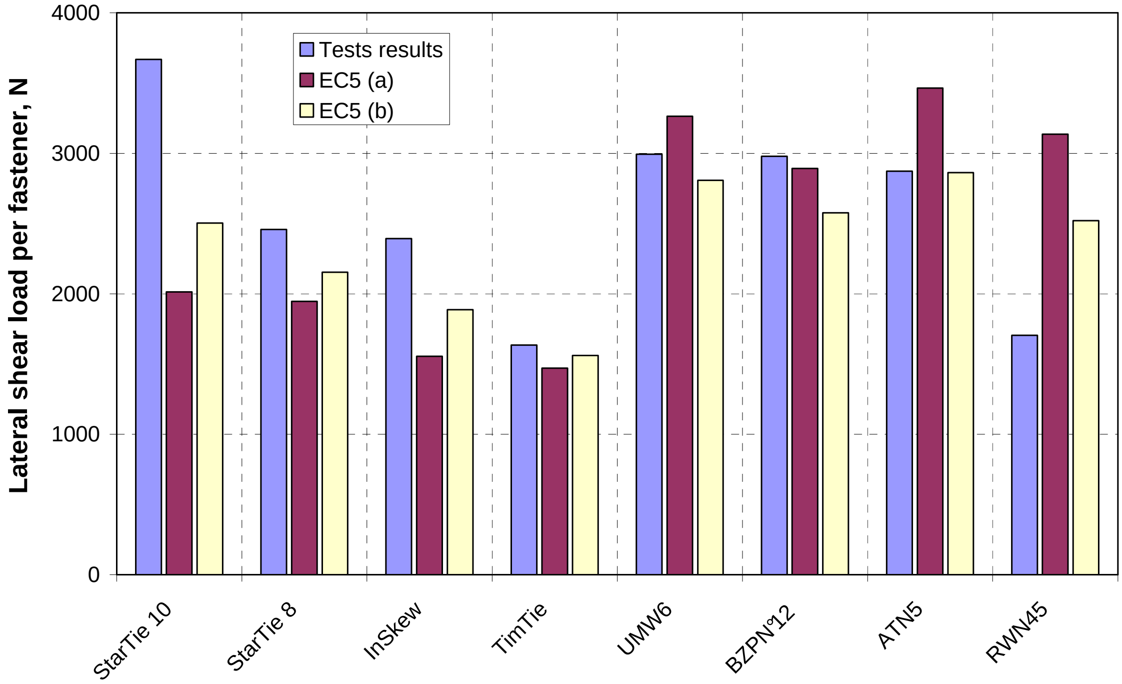


Figure 7.11: Fixings characteristic shear capacity in timber to concrete connections

The results of the experimental tests and design calculations as described above show that:

- Helically shaped StarTie 10 fasteners achieved the highest loads in testing for all fasteners tested;
- Wood screws and threaded nails achieved higher loads than StarTie 8, InSkew and TimTie fasteners;
- The design calculation methods a) and b) resulted in conservative values for Helically shaped fasteners, with method a) resulting in lower values than method b);

- For common timber fasteners the design method a) resulted in design values that over estimated the connection capacities (except in the case of wood screw BZP N°12);
- In the case of round wire nails (RWN 45) both design methods resulted in overestimation of the connection capacity;

Typical load displacement curves of timber to concrete shear connections are shown in Figure 7.12.

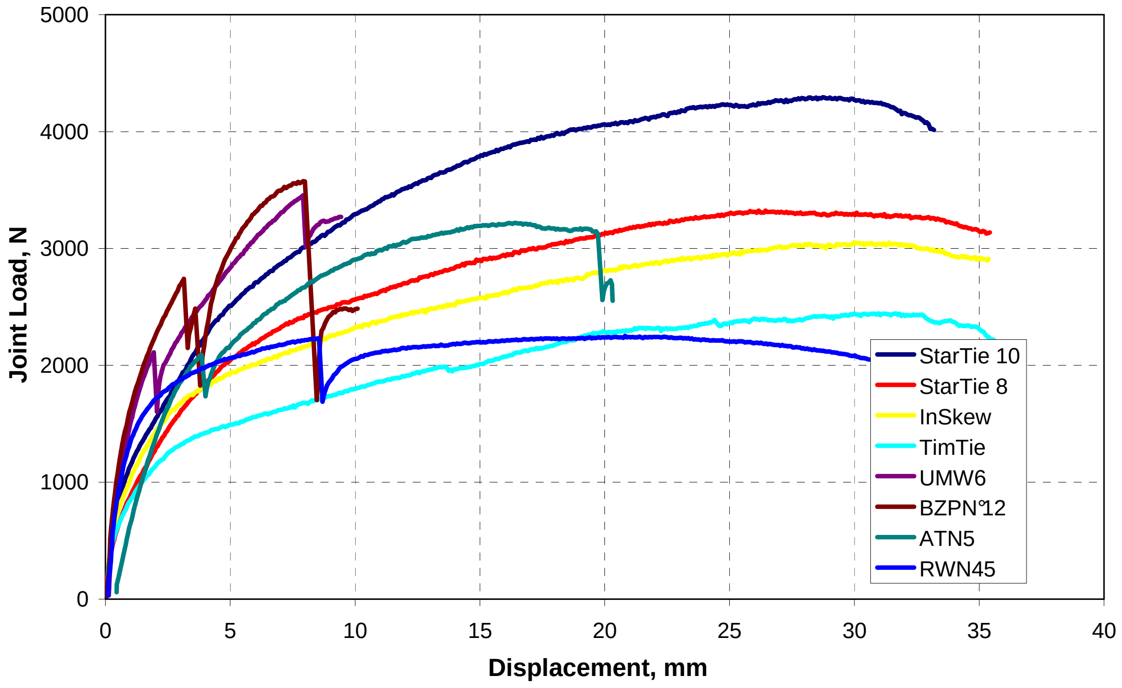


Figure 7.12: Typical load displacement curves of timber to concrete shear connections

As for all tests performed in this study, the typical load displacement relationship show that helically shaped fasteners exhibit a much more ductile behaviour than common timber fasteners. It can be noted that all common fasteners failed in a brittle manner, which was not the case for timber to timber connections tested in the previous chapters where only wood screws exhibited such failure modes. This may be due to the fact that the concrete used in this experimental programme, and the method of fabrication of the samples provided an almost perfect clamp on the fasteners and therefore coupled with the gap between substrate from the plywood induced greater strain on the fasteners which led to brittle failures.

7.4.3 Design of timber to concrete with interlayer

As mentioned above, in many practical applications in timber to concrete composite systems, floor boards are placed between the timber members and concrete to act as formwork and to create clean finishes to the structure. Therefore the joints in such composite structures are considered timber to concrete with interlayer. The numerical models developed for such connections typically consider that the interlayer either is moving freely or is fixed to one of the members, usually the timber. The corresponding failure modes and equations are developed using Johansen's method.

In the experimental programme described above, the interlayer is considered to be moving freely between the timber and concrete. The modes of failures are shown in Figure 7.13.

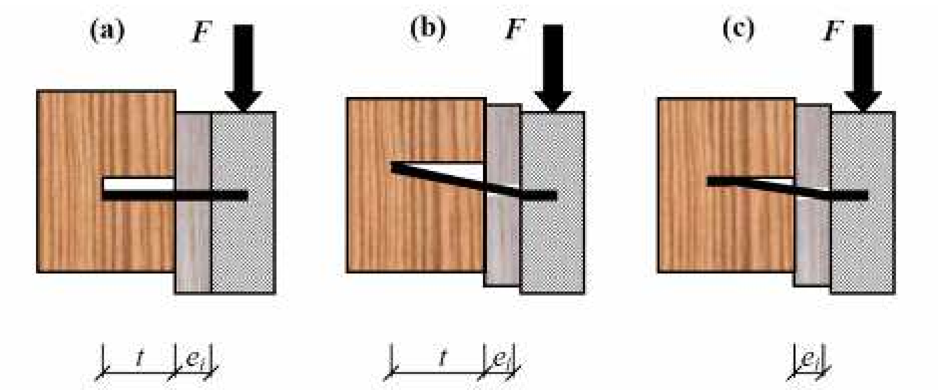


Figure 7.13: Failure modes of timber to thick steel connections with interlayer moving freely

$$F_{tc,\text{int}-free} = \begin{cases} d \cdot t_1 \cdot f_h & \ldots(7.2a) \\ d \cdot f_h \cdot \left(-(t + 2 \cdot e_i) + \sqrt{2e_i^2 + \dfrac{2 \cdot M_y}{d \cdot f_h} + \dfrac{e_i^2 \cdot f_i}{2 \cdot f_h} + t \cdot e_i + \dfrac{t^2}{2}} \right) & \ldots(7.2b) \\ d \cdot f_h \cdot \left(-e + \sqrt{e^2 + \dfrac{4 \cdot M_y}{d \cdot f_h} + \dfrac{e_i^2 \cdot f_i}{2 \cdot f_h}} \right) & \ldots(7.2c) \end{cases}$$

Where: $F_{tc,int-free}$ is the load carrying capacity of the connection with interlayer, in N;

d is the fastener diameter, in mm;

t_1 is the timber member thickness, in mm;

f_h is the embedment strength of the timber member, in N/mm²;

f_i is the embedment strength of the interlayer, in N/mm²;

M_y is the fastener yield moment, in Nmm;

e_i is the gap between the timber member and substrate, in mm.

It can be noted that Equation 7.2 does not include the rope effect due to axial load in the fastener – which was included in Eurocode 5 timber to thick steel connections in the later stages of the code draft – and also does not include any factor to account for parameters that influence the connections such as friction between the substrates and interlayer.

The yield moment, embedment and axial strengths of helically shaped fasteners were determined using the specific equations developed in the previous chapters – Equations 3.7, 3.10 and 6.11. In Chapter 3, the embedment strength of helically shaped fastener was studied for solid timber, as plywood is used as the interlayer in the samples the design equations from Eurocode 5 have to be used for the interlayer – Equation 7.3.

$$f_{h,k,plywood} = 0.11 \cdot \rho_k \cdot d^{0.3} \qquad \textbf{…(7.3)}$$

Table 7.7: *Characteristic load carrying capacity form tests and Equation 7.2*

Fastener	Tests Results	Equation 7.2	Error
	N	N	%
StarTie 10	3668.28	2312.54	-37%
StarTie 8	2458.16	1703.10	-31%
InSkew	2392.46	1315.16	-45%
TimTie	1634.66	1023.58	-37%

The results detailed in Table 7.7 show that the design model with interlayer yields conservative results with the average error between experimental and calculated values being -38%. The results obtained using the more realistic model – i.e. model with interlayer Equation 7.2 – present greater average error than the results obtained for helically shaped fasteners using Equation 7.1 in combination with the specific yield moment, embedment and axial strength. This may be due to the fact that equation 7.1 includes the effect of axial strength in the model. As such the axial strength is an important factor for helically shaped fasteners due to its shape, the rope factor introduced in the timber to timber or steel design equations in Eurocode 5 was included in the design model with interlayer, Equation 7.4.

$$F_{tc,\text{int}-free} = \begin{cases} d \cdot t_1 \cdot f_h & \text{...(7.4a)} \\ d \cdot f_h \cdot \left(-(t + 2 \cdot e_i) + \sqrt{2e_i^2 + \dfrac{2 \cdot M_y}{d \cdot f_h} + \dfrac{e_i^2 \cdot f_i}{2 \cdot f_h} + t \cdot e_i + \dfrac{t^2}{2}} \right) + \dfrac{F_{ax,Rk}}{4} & \text{...(7.4b)} \\ d \cdot f_h \cdot \left(-e + \sqrt{e^2 + \dfrac{4 \cdot M_y}{d \cdot f_h} + \dfrac{e_i^2 \cdot f_i}{2 \cdot f_h}} \right) + \dfrac{F_{ax,Rk}}{4} & \text{...(7.4c)} \end{cases}$$

Using Equation 7.4 for determining the characteristic load carrying capacity of Helically shaped timber to concrete connections with interlayer results in conservative values with an average error of -23%. The percentage error can be attributed to the fact that the rope effect is limited to 25% of its capacity and that the characteristic values of parameters are used in the model. However compared to the design model of timber to thick steel connections equation 7.4 is more realistic and the results are therefore more representative of the true behaviour of the connections, as it can be argued that the results obtained from Equation 7.1 and the relatively low average error can be attributed to parameters not included in the model, and such error can be fortuitous.

7.5 Summary and conclusions

In this chapter the behaviour and resistance of helically shaped fasteners were investigated in timber to concrete connections. Such connections have been realised for many years with helically shaped fasteners, however never in structural systems with timber. A review of the timber to concrete composite systems, and the design methods was first undertaken. It showed that such composite systems were on the increase as developments in the building industry tend to use the materials to their most effective properties, as a results timber to concrete floor systems use the bending capacity of timber in combination with the compressive strength of concrete. The review also revealed that while a large percentage of houses are now timber framed, the structural timber system is always connected to a concrete based platform. Following such findings an experimental programme was performed to investigate the behaviour and resistance of helically shaped fasteners as sole plate connectors and as shear fixings in timber to concrete floors.

Sole plate connections, made from 7 N/mm² concrete blocks connections were investigated with helically shaped and common fasteners used in the building industry.

The results showed that while achieving similar load carrying capacities to most common fixings, helically shaped fasteners exhibited more ductile behaviour with lower stiffness. In addition to comparative tests, the experimental programme investigated the timber to concrete block connections with helically shaped fastener with varying depth of penetration in the concrete and varying sole plate thicknesses. These tests showed that greater resistance is achieved with the sole plates of greater thickness, and that there seem to be a limit at which increase in penetration depth in the concrete does not result in higher connection resistance. However it is to be noticed that these tests can only be informative, as the range of investigation was limited, further study should be undertaken in order to draw more definite conclusions.

However, the experimental results were compared to the results obtained from the design equations from Eurocode 5. This showed that the timber to thick steel design model could be used in combination with the specific equations for yield moment, embedment and axial strength for helically shaped fasteners. The model resulted in acceptable conservative load carrying capacities for helically shaped fasteners.

The research programme was then focused on the shear connection in timber to concrete flooring systems. Samples simulating such connections were fabricated with concrete of target resistance of 25MPa. Similarly to what is done in practice where timber based boards are used between the timber members and concrete to act as formwork, 19mm plywood was used. The tests showed that helically shaped fastener could reach similar load carrying resistance to common timber fasteners while exhibiting a more ductile behaviour. While ductility is a recurring characteristic of joints with helically shaped fasteners, these tests showed that they were the only fastener which did not fail in a brittle manner. The characteristic load carrying capacities obtained from the experimental programme were compared to that obtained using the design equations from Eurocode 5, which showed that Eurocode 5 design equations results in conservative values with relatively low percentage error. Nonetheless, the design equation given in Eurocode 5 do not include all the parameters of the numerical model, in particular the interlayer which is assumed in most composite systems to be able to move freely between the substrates. Taking this observation into consideration the experimental results were compared to characteristic values obtained using a model developed based on Johansen method with an interlayer moving freely. This showed that when used in combination to the specific property parameter equations developed for Helically shaped fasteners the model could predict the load carrying capacity of a connection with an average error of -23%.

In this chapter the study tended to widen the range of structural timber applications for helically shaped fasteners while staying in the applicability range of such fasteners. While the experiments were limited, the study showed that helically shaped fastener could be used for such applications as they achieve greater ductility with similar resistance.

Chapter 8 Conclusions and recommendations for future work

8.1 Introduction

Helically shaped fasteners have been successfully used since 1984 as wall ties and remedial crack fixings for masonry and stone structures. Nowadays the variety of masonry structural applications has widened, and new products have been developed for specific needs – i.e. special ties, grouts, and insertion tools. Along with the development and innovation process for new products, helically shaped fasteners showed that they could be used for fixing various materials from concrete blocks and bricks to timber and timber based materials. While such capacity was observed and has been used in non structural applications, helically shaped fasteners and products have been continuously used and developed in masonry structural applications.

In order to understand the capacity of helically shaped fasteners as timber structural fixings an extensive experimental and analytical programme was undertaken. The main objectives of this study, as listed in Section 1.3, were to investigate the material properties of Helically shaped fasteners for use in timber joints, to compare the resistance and behaviour of Helically shaped fasteners in timber to timber connections in comparison to commonly used timber fasteners such as nails, threaded nails and wood screws, to analyse the design method of timber joints in accordance with Eurocode 5 for timber joints with Helically shaped fasteners, and to investigate possible new application in timber structural systems. These objectives were achieved following an extensive experimental augmented with analytical work programme in accordance with the recommendations of the relevant British and European Standards. In the following sections the principal findings from the investigations performed are summarised.

8.2 Conclusions

8.2.1 On Timber connections:

The first chapter of this study aimed to review the state of the art of timber connections with dowel type fasteners, and lay the basis for the research work undertaken.

1. Timber connections are the most important elements of timber structural systems and therefore need to be evaluated and designed accurately. To overcome the problem numerous jointing methods have been developed with time.
2. The resistance and behaviour of mechanical connections with dowel type fasteners are influenced by a multitude of parameters that can be regrouped in three main categories: material and dimensional properties, joint configuration and loading conditions.
3. The design of timber joints has been the goal of many researchers over the years, and is still being investigated. The main parameters for design according to the current European Standard, Eurocode 5 (BSI, 2004) have been found to be the fastener yield moment, the embedment strength and joint connection.
4. Research on timber connections with dowel type fasteners have often involved experimental programmes on timber samples in “sandwich” construction with dowel type fasteners laterally loaded.

8.2.2 Mechanical properties of helically shaped fasteners

5. The four sizes of helically shaped fasteners used in this study exhibited a characteristic tensile strength above the minimum limit set by Eurocode 5. Common timber fasteners such as screws and threaded nails also achieved higher tensile strength than the required limit; however smooth round nails in some cases did not reach the minimum recommended limit.
6. Yield moment tests on the various fasteners tested showed that the three points bending tests adopted by the American Society of Testing Materials resulted in lower values to that calculated using the design equations of Eurocode 5. The experimentation highlighted the difficulty to assess, either experimentally or in design, the yield moment of fasteners.
7. While the tests did not follow the principle as recommended in BS EN 409:2009 (BSI, 2009 b), four points bending tests resulted in higher yield moment values to those calculated using the design equations from Eurocode 5. In the case of helically shaped fasteners the error between design and tests results indicated that the design equations developed for and from round nail tests could not be applied for design purposes as it would lead to great underestimation of the joint capacity.
8. It was shown that the yield moment is directly related to the fastener diameter and tensile strength, as a result a specific design equation was developed for determining the yield moment of Helically shaped fasteners using both factors. The

equation represented the yield moment strength of helically shaped fasteners accurately.

9. Common timber fasteners such as screws and nails were found to exhibit more elastic and stiffer embedment behaviour than helically shaped fasteners. However, when comparing similar diameters, Helically shaped and common fasteners resulted in equivalent embedment strength.
10. The embedment strength of helically shaped fasteners was found to decrease with increasing fastener diameter, which was true for common timber fasteners tested and as concluded by various studies on the embedment behaviour of dowel type fasteners.
11. The design equation for embedment strength in Eurocode 5 was found to represent erroneously the embedment strength relationship of helically shaped fasteners. Hence a specific design equation was developed for helically shaped fasteners.
12. The analysis of the results and review of previous research work on the subject showed that the timber density and fastener diameter affected the embedment strength of dowel type fasteners. Using these two factors, a specific design equation was developed for calculating the embedment strength of helically shaped fasteners. The design equation represented the relationship between the fastener diameter and timber density to the embedment strength accurately.

8.2.3 Axially loaded helically shaped fasteners in timber

13. The experimental results highlighted the ductile behaviour of helically shaped fasteners compared to common timber connectors when axially loaded. Helically shaped fasteners in withdrawal exhibit a ductile behaviour with an initial elastic phase, then a plastic stage with increase in load relative to the fastener displacement.
14. The analysis of the results for all the fasteners studied concluded a misrepresentation of the withdrawal strength of helically shaped fasteners by the recommended design equations given in Eurocode 5. While the fastener diameter and depth of penetration are accurate parameters for the axial load of common timber fasteners, in the case of helically shaped fasteners, the analysis concluded that the fastener perimeter and projected depth of penetration were to be used.
15. The results of extensive experimental programme on parameters that influence the withdrawal strength and behaviour of helically shaped fastener in timber concluded that the pilot hole diameter had a negative effect on the withdrawal strength.

However the timber density, depth of penetration and angle to the grain (from 0° to 90°) had a positive influence, with the withdrawal strength increasing with increase in the parameter.

16. The analysis of the results showed that the parameters investigated – i.e. pilot hole diameter, timber density, angle to timber grain and depth of penetration – can be combined in semi empirical models that can predict the withdrawal behaviour and capacity of Helically shaped fasteners axially loaded in timber with an average error of 10.4%.

8.2.4 Laterally loaded connections with helically shaped fasteners

17. Preliminary work of laterally loaded connections with helically shaped fasteners emphasised the difficulty to adapt Eurocode 5 fastener spacing recommendations due to the unusual shape of helically shaped fasteners. Indeed, the minimum recommended spacing of *5d* for fasteners parallel to the grain proved to be underestimating the distance required to avoid splitting when *d* is taken as the root diameter. On the other hand, when *d* is taken as the thread diameter the minimum distance between fasteners seem to be overestimated. It was found that, using the fastener root diameter, a spacing between fasteners inserted parallel to the timber grain of *8d* could be used without causing wood splitting.
18. It was found that joints with helically shaped fasteners reached similar loads than woodscrews while exhibiting a much more ductile behaviour. In addition, the yield point for joints with helically shaped fasteners occurred at relatively large displacement compared to the other fasteners used in the study.
19. When analysed in detail, tests on various overlapping configurations showed that fully overlapping helically shaped fasteners did not cause early splitting and brittle failure of the timber members as opposed to other fasteners.
20. The failure modes of the connections investigated could not be accurately investigated due to the large displacement reached during testing. However, the observations on samples after tests showed wood crushing under the fasteners, and fastener yielding, which indicate that the connections modes of failure correspond to the modes II and III as identified by Johansen.
21. It was found, following an extensive experimental programme on parameters that may influence the joint behaviour and resistance, that at a displacement of 3.20mm the fastener diameter, timber density and moisture content were directly proportional to the joint strength. Also, the analysis concluded that the number of

rows of fasteners and row spacing influenced the strength of joints with helically shaped fasteners in a similar way to joints with common timber fasteners.

22. The semi empirical model developed showed that when analysed at a displacement of 3.20mm, contrary to woodscrews and nails, the strength of joints with helically shaped fasteners loaded in single or double shear is not directly proportional to the number of lines in the connection.
23. A semi empirical model based on the connection configuration can be used for predicting the behaviour and strength of a timber joint with helically shaped fasteners loaded in single or double with an average error of 9%.

8.2.5 Design methods for timber joints with helically shaped fasteners

24. The yield theory developed by Johansen and implemented in Eurocode 5 can be used for designing timber connections with helically shaped fasteners. However, due to the discrepancies in their mechanical behaviour between joints with helically shaped fasteners or common timber fasteners, the existing design equations need to be adapted or new equations developed for helically shaped fasteners.
25. For the yield moment and embedment strength of helically shaped fasteners, specific design equations of the same form as those given in Eurocode 5 accurately predict the characteristic values for design purposes.
26. It was found that the design equation for predicting the strength of axially loaded fasteners to Eurocode 5 misrepresented the characteristic resistance of helically shaped fasteners. It was found that the withdrawal resistance of helically shaped fasteners was best represented by a factor including the parameters that have an influence on the withdrawal load. This design method was found to predict accurately the characteristic withdrawal load of axially loaded fasteners.
27. Characteristic experimental lateral shear results of joints with helically shaped fasteners loaded in single and double shear were analysed in comparison to the original design model developed by Johansen, and to the current design model of Eurocode 5. It was found that, when used in combination with the specific equations for yield moment, embedment and axial strength, the design model from EC5 could be used for predicting the characteristic resistance of a timber joint with helically shaped fasteners.
28. The analysis found that the recommended method of calculations for the effective number of fasteners in a line through the factor k_{ef} of Eurocode 5 greatly reduced the predicted characteristic joint capacity. It was demonstrated that with k_{ef} equal to

unity the design model resulted in conservative results reducing the error between experimental characteristic results.

8.2.6 Helically shaped fasteners in timber to concrete composite systems

29. The analysis of the experimental tests on helically shaped fasteners as shear fixings for sole plates in timber-frame construction has demonstrated that they offered a viable alternative to the fasteners currently used. Indeed, the larger sizes achieved similar lateral shear resistance to the other fasteners, while exhibiting greater ductile behaviour.
30. It was found that timber to concrete blocks connections with helically shaped fasteners show low levels of stiffness compared to other connections.
31. It was found that the design equation provided in Eurocode 5 for timber to thick steel predicted reasonably well the performance of common sole plate connectors, on the other hand it underestimated the performances of helically shaped fasteners by about 60%. However the analysis demonstrated that when in use with the specific equations developed in this study, the design method reduced significantly.
32. With shear connectors in timber-concrete floor systems, the investigation demonstrated that helically shaped fasteners compared favourably to other fasteners due to their high ductility and similar resistance.
33. The study showed that the design model for timber to thick steel overestimated the characteristic resistance of timber to concrete connections with interlayer for common timber fasteners; while providing conservative results for helically shaped fasteners.
34. The theoretical design model with interlayer developed on the model from Johansen for timber connections was proved to greatly underestimate the characteristic resistance of connections with helically shaped fasteners. However, as the most realistic model, the introduction of the parameters such as axial loads and friction, the analysis concluded that the model would provide accurate results.

8.3 Recommendations for future work

The investigation undertaken in this research programme allowed the basis for use and the development of helically shaped fasteners in timber structural systems. While the research has fulfilled the objectives as mentioned in section 1.3, and showed that helically shaped fasteners can be used in the structural systems as described in the Chapters 3 to 7, many structural issues are left unknown and should be investigated.

The main areas of research that can be relevant and should be addressed are outlined below.

1) As timber is nowadays often used in numerous situations the behaviour and load carrying resistance of timber connections with helically shaped fasteners should be investigated for parameters that could not be included in depth in this study, such as:

- Conditions of varying timber moisture content. The use of a controllable conditioning chamber and testing facility is recommended in order to accurately measure the effect of moisture fluctuations on connections with helically shaped fasteners.
- Loading perpendicular to the timber grain, as this study focused solely on joints with fasteners loaded parallel to the timber grain.
- Number of fastener in a line. As mentioned in Chapter 5, tests with fasteners inserted in a line parallel to the timber grain failed due to wood splitting.
- Connections with timber based materials such as Laminated Veneer Lumber (LVL), glulam and Orientated Strand Boards (OSB).

2) The structural applications of timber suggest that helically shaped fasteners should be tested as a timber connector in different loading conditions in order to offer a complete structural solution to timber jointing. The investigation should include:

- Combined axial and lateral loading,
- Dynamic and cyclic loading. This is particularly important in the case of timber to concrete floor systems where such loading conditions apply.
- Moment resisting connections.

3) Further study on design applications should first focus on:

- Minimum spacings and distances. This issue was briefly mentioned in this study, however an in depth analysis of the minimum distances should be carried out for design purposes.
- The stiffness of the joints should be assessed and compared to the design equations given in Eurocode 5.

4) Investigation on the uses of helically shaped fastener in mechanically-laminated timber structural systems such as nail laminated beams and nailed floor cassettes systems.

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Appendix A Material Properties

1 - Timber properties

The softwood timber used in this study was supplied by a local building merchant in dressed solid wood planks 4.2m in length with a cross section approximately 185 mm by 47 mm, and was of grade C24 and C16 in accordance with BS EN 338:2003 (BSI, 2003). Hardwood of grade D30 in accordance to BS EN 338:2003 (BSI, 2003) was purchased through a timber supplier and consisted of dressed solid wood planks 2.5m in length with a cross section of 190 mm by 55 mm.

The timber was stored for a period of two months before the tests to achieve constant moisture content. Samples were cut, and clear specimens chosen for the tests, however within a specimen, small knots and variation in the slope of the timber fibres were permitted provided they were unlikely to significantly reduce the specimen strength, or have any influence on the test behaviour or result. The samples were fabricated and tested within one hour.

Following each tests, small clear samples were cut from the timber samples in order to measure the density and moisture content, in accordance to BS 373:1957 (BSI, 1957). The measurements allowed the tests results to be normalised for timber density, this ensured that the effect of timber density was eliminated from the results. A total of 1167 samples were measured for density and moisture content. The results are given in Table A1.

Table A1: *Timber density and moisture content*

Timber grade	Density (kg/m3)			Moisture Content (%)		
	Average	S.D.	C.O.V.	Average	S.D.	C.O.V.
C24	386.13	42.34	0.110	9.76	0.65	0.066
C16	379.67	39.71	0.105	10.80	1.77	0.163
D30	586.57	17.88	0.030	9.88	0.88	0.089
C24 (12% mc)	341.15	15.01	0.044	12.23	0.20	0.016

For information, the timber of grade C24 was tested in accordance to BS 373:1957 (BSI, 1957) to measure its mechanical properties. The results are given in Table A2.

Table A2: Mechanical properties of timber grade C24

Property	Results	S.D.	C.O.V.
	N/mm^2	N/mm^2	%
$E_{m/l}$	10665.91	2975.03	27.89
$E_{m/g}$	10540.50	1919.30	18.21
G	192.35	53.27	27.70
f_m	71.88	8.12	11.29
$E_{c,0}$	4558.95	270.88	5.94
$f_{c,0}$	41.24	4.49	10.89
$E_{c,90}$	101.02	3.77	3.74
$f_{c,90}$	2.35	0.04	1.82

Where $E_{m/l}$ is the local modulus of elasticity,
$E_{m/g}$ is the global modulus of elasticity,
G is the shear modulus (calculated using the single span method),
f_m is the bending strength,
$E_{c,0}$ is the modulus of elasticity in compression parallel to the grain,
$f_{c,0}$ id the compression strength parallel to the grain,
$E_{c,90}$ is the modulus of elasticity in compression perpendicular to the grain,
$f_{c,90}$ is the compression strength perpendicular to the grain.

2 - Concrete properties

As detailed in chapter 7, concrete blocks of nominal strength of 7 N/mm² and in house concrete of target strength of 25MPa were used in the experimental programme in order to investigate timber to concrete connections with Helically shaped fasteners. The materials properties are given in Table A3.

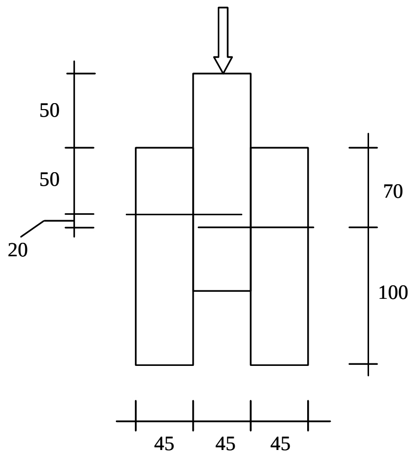
Table A3: Properties of concrete materials

Substrate	Nominal resistance	Density	Compressive strength	S.D.	C.O.V.
	N/mm²	kg/m3	N/mm²	N/mm²	%
Concrete blocks	7	2382	16.11	1.02	6.33
Concrete	25	2237	30.14	1.54	5.10

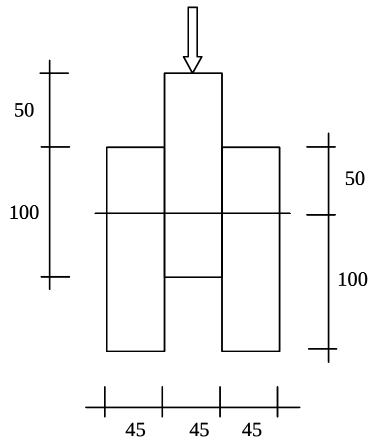
Appendix B Lateral Shear tests nailing configurations

1 – Single fastener connections

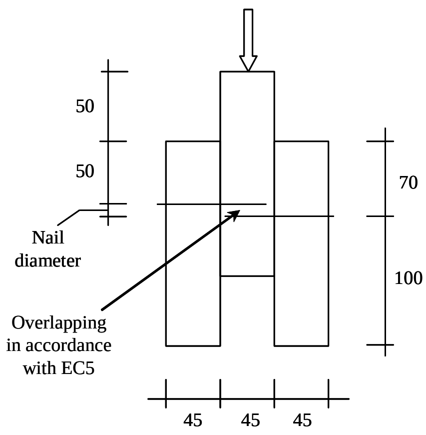
AB



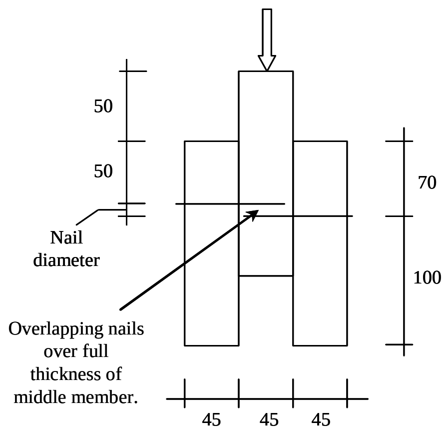
AC



AD



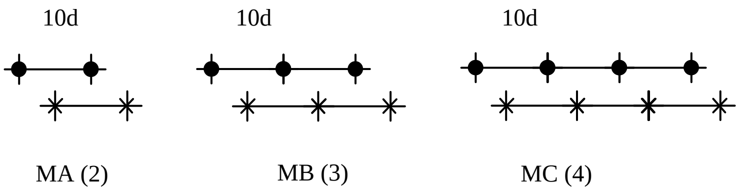
AE

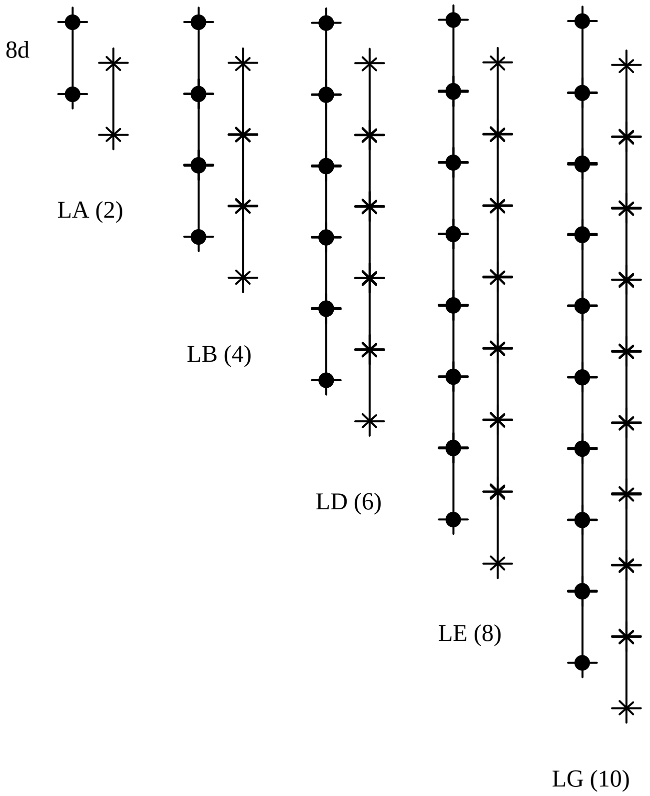


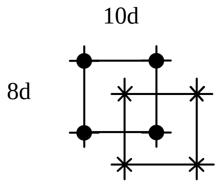
The nailing configurations above were tested with the four sizes of Helically shaped fasteners – i.e. StarTie 10, StarTie 8, InSkew and TimTie. The nailing configuration AB was used for tests with common timber fasteners such as woodscrews, smooth round and threaded nails.

2 – Multiple fasteners joints loaded in single shear

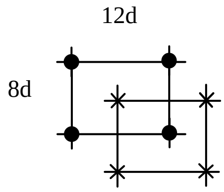
Nailing configurations for lateral shear tests with Helically shaped fasteners in single shear. The figures show the pattern in the connection's middle member for "double single" shear connections as described in Section 5.2.2. The following samples were tested with Helically shaped Inskew fasteners (with d_r = 3.35mm). The Figures show the nailing configurations' code with in parenthesis the number of fastener per shear plane; also, the nail spacing is given as a function of the fastener root diameter.



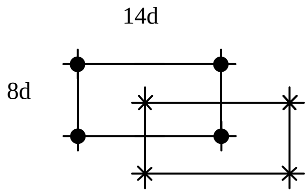




MG (4)



MF (4)



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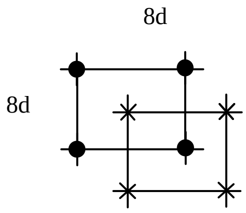

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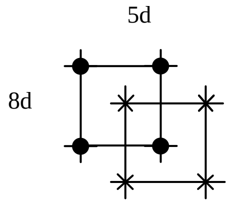

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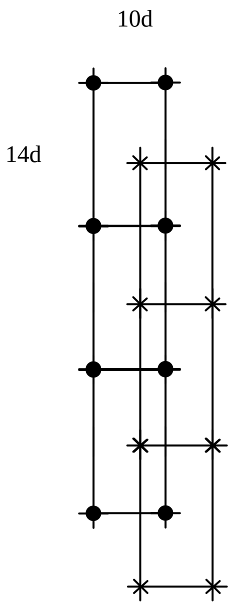
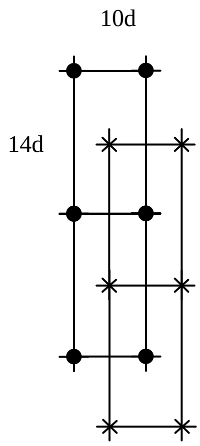
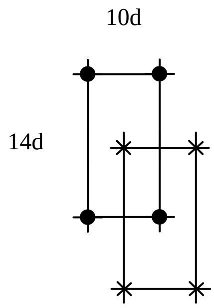


NC (8)



NB (6)



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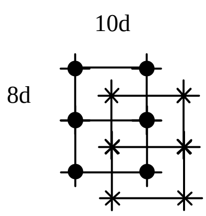

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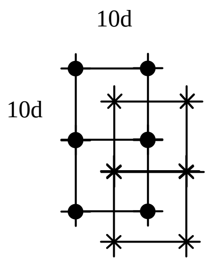

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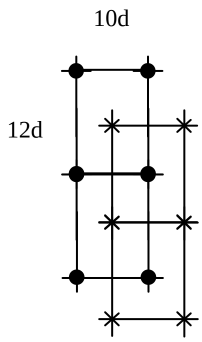
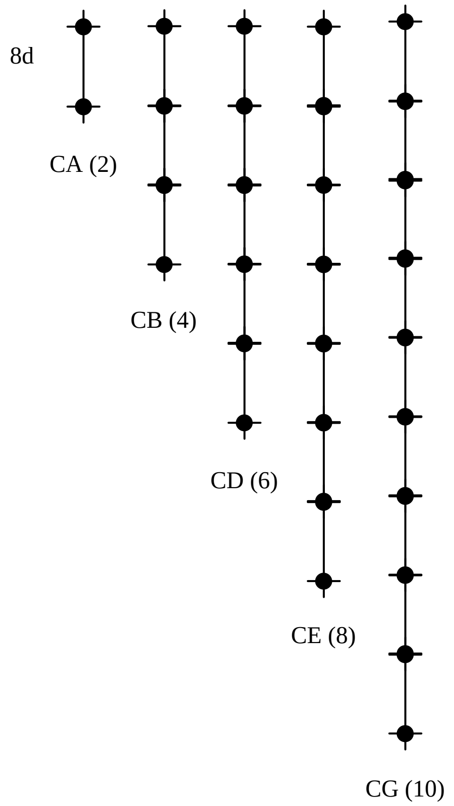
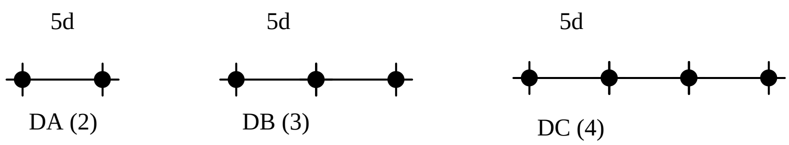


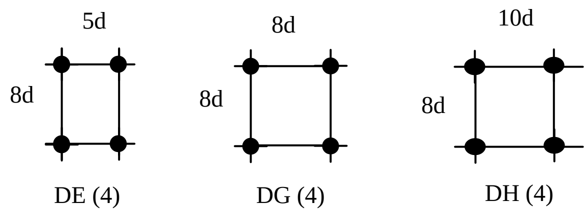
NH (6)

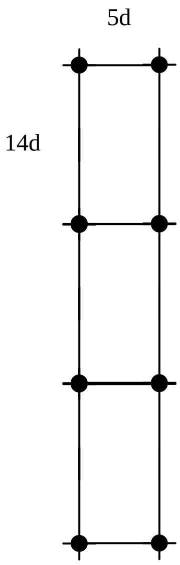
3 – Multiple fasteners joints loaded in double shear

Nailing configurations for lateral shear tests with Helically shaped fasteners in double shear. The following samples were tested with Helically shaped StarTie 8 fasteners (d_r = 3.75mm). The Figures show the nailing configurations' code with in parenthesis the number of fastener per shear plane; also, the nail spacing is given as a function of the fastener root diameter.

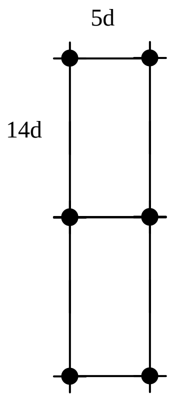




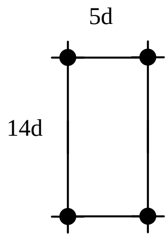




EC (8)



EB (6)



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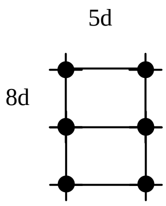

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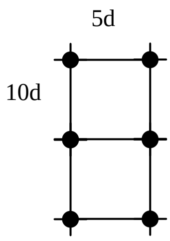

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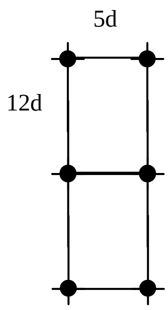


EH (6)

<u>Appendix C</u> The use of MathCAD for determining the displacement function of withdrawal models

This shows how a MathCAD worksheet and its Genfit function was used for determining the parameters of the function $f_1(\delta)$ of the withdrawal semi empirical models developed and detailed in Chapter 4. This particular example shows the equation developed for all four sizes of fasteners.

Numerical model for the withdrawal behaviour of Helifix fasteners.
The joint displacement function is calculated below.

$P = (A.x/2.5 + 1)(1 - e^{c.x/2.5})^D$.

values :=

	0	1
0	0.03	0.05
1	0.07	0.13
2	0.1	0.18
3	0.13	0.24
4	0.17	0.28
5	0.2	0.33
6	0.23	0.37
7	0.27	0.4
8	0.3	0.44
9	0.33	0.47

The column 0 represents the withdrawal displacement in mm.
The column 1 represents the reduced load of the withdrawal tests for the 4 fasteners.

$\text{rload} := \text{values}^{\langle 1 \rangle}$ $\qquad\qquad$ $\text{delta} := \text{values}^{\langle 0 \rangle}$

The function to represents the data will have the following form:

$$f1 = (A \cdot 0.4x + B)\left[\left[1 - e^{(C \cdot 0.4x)}\right]^D\right]$$

with A, B, C and D to be determined.

The partial derivatives of the function are calculated with MathCAD below.

$$\frac{\partial}{\partial A}(A \cdot 0.4x + B)\left[\left[1 - e^{(C \cdot 0.4x)}\right]^D\right] \rightarrow .4 \cdot x \cdot (1 - \exp(.4 \cdot C \cdot x))^D$$

$$\frac{\partial}{\partial B}(A \cdot 0.4x + B)\left[\left[1 - e^{(C \cdot 0.4x)}\right]^D\right] \rightarrow (1 - \exp(.4 \cdot C \cdot x))^D$$

$$\frac{\partial}{\partial C}(A \cdot 0.4x + B)\left[\left[1 - e^{(C \cdot 0.4x)}\right]^D\right] \rightarrow -.4 \cdot (.4 \cdot A \cdot x + B) \cdot (1 - \exp(.4 \cdot C \cdot x))^D \cdot D \cdot x \cdot \frac{\exp(.4 \cdot C \cdot x)}{(1 - \exp(.4 \cdot C \cdot x))}$$

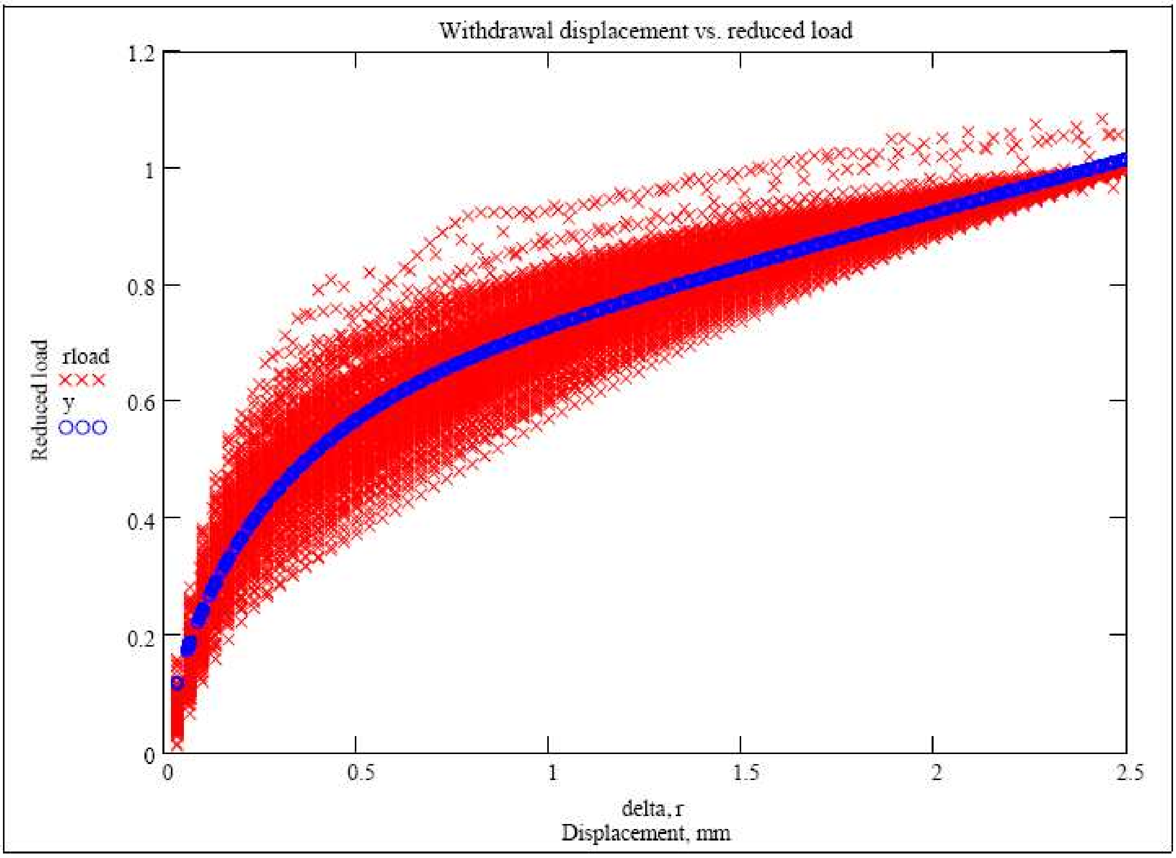
$$\frac{\partial}{\partial D}(A \cdot 0.4x + B)\left[\left[1 - e^{(C \cdot 0.4x)}\right]^D\right] \rightarrow (.4 \cdot A \cdot x + B) \cdot (1 - \exp(.4 \cdot C \cdot x))^D \cdot \ln(1 - \exp(.4 \cdot C \cdot x))$$

P represent guess values for the analysis

$$P := \begin{pmatrix} 0.45 \\ 0.56 \\ -8.4 \\ 0.7 \end{pmatrix} \qquad F(x,P) := \begin{bmatrix} \left(P_0 \cdot 0.4x + P_1\right)\left[\left[1 - e^{\left(P_2 \cdot 0.4x\right)}\right]^{P_3}\right] \\ .4 \cdot x \cdot \left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right)^{P_3} \\ \left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right)^{P_3} \\ -.4 \cdot \left(.4 \cdot P_0 \cdot x + P_1\right) \cdot \left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right)^{P_3} \cdot P_3 \cdot x \cdot \dfrac{\exp\left(.4 \cdot P_2 \cdot x\right)}{\left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right)} \\ \left(.4 \cdot P_0 \cdot x + P_1\right) \cdot \left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right)^{P_3} \cdot \ln\left(1 - \exp\left(.4 \cdot P_2 \cdot x\right)\right) \end{bmatrix}$$

The factors A,B,C and D of the function are calculated in fct1.

$$fct1 := genfit(delta, rload, P, F) \qquad fct1 = \begin{pmatrix} 0.451 \\ 0.565 \\ -8.587 \\ 0.704 \end{pmatrix} \qquad y_i := \left(fct1_0 \cdot 0.4 \cdot r_i + fct1_1\right)\left[\left[1 - e^{\left(fct1_2 \cdot 0.4 r_i\right)}\right]^{fct1_3}\right]$$



$$R := corr(rload, y)^2 \qquad R = 0.94481$$

<u>Appendix D</u> The use of MathCAD for determining the displacement function of lateral shear models

This shows how a MathCAD worksheet and its Genfit function was used for determining the parameters of the function $f_1(\delta)$ of the lateral shear semi empirical models developed and detailed in Chapter 5.

Numerical model for single shear connection behaviour with Helifix fasteners.
The joint displacement function is calculated below.

$P = (A.x/3.2 +1)(1-e^{c.x/3.2})^D$.

value :=

C:\..\Disp Load to 3.2mm.xls

value =

	0	1
0	0.047	0.014
1	0.049	0.06
2	0.077	0.107
3	0.111	0.149
4	0.142	0.183
5	0.165	0.218
6	0.194	0.254
7	0.228	0.286
8	0.27	0.316
9	0.312	0.351

The column 0 represents the connection displacement in mm.
The column 1 represents the reduced load of the tests.

$\text{rload} := \text{value}^{\langle 1 \rangle}$ $\qquad$ $\text{delta} := \text{value}^{\langle 0 \rangle}$

The function to represents the data will have the following form:

$$f1 = (A \cdot 0.3125x + B)\left[\left[1 - e^{(C \cdot 0.3125x)}\right]^D\right]$$

with A, B, C and D to be determined.

The partial derivatives of the function are calculated with MathCAD below.

$$\frac{\partial}{\partial A}(A \cdot 0.3125x + B)\left[\left[1 - e^{(C \cdot 0.3125x)}\right]^D\right] \rightarrow .3125 \cdot x \cdot (1 - \exp(.3125 \cdot C \cdot x))^D$$

$$\frac{\partial}{\partial B}(A \cdot 0.3125x + B)\left[\left[1 - e^{(C \cdot 0.3125x)}\right]^D\right] \rightarrow (1 - \exp(.3125 \cdot C \cdot x))^D$$

$$\frac{\partial}{\partial C}(A \cdot 0.3125x + B)\left[\left[1 - e^{(C \cdot 0.3125x)}\right]^D\right] \rightarrow -.3125 \cdot (.3125 \cdot A \cdot x + B) \cdot (1 - \exp(.3125 \cdot C \cdot x))^D \cdot D \cdot x \cdot \frac{\exp(.3125 \cdot C \cdot x)}{(1 - \exp(.3125 \cdot C \cdot x))}$$

$$\frac{\partial}{\partial D}(A \cdot 0.3125x + B)\left[\left[1 - e^{(C \cdot 0.3125x)}\right]^D\right] \rightarrow (.3125 \cdot A \cdot x + B) \cdot (1 - \exp(.3125 \cdot C \cdot x))^D \cdot \ln(1 - \exp(.3125 \cdot C \cdot x))$$

$$P := \begin{pmatrix} 0.5 \\ 0.5 \\ -4 \\ 1 \end{pmatrix}$$

$$F(x,P) := \begin{bmatrix} \left(P_0 \cdot 0.3125x + P_1\right)\left[\left[1 - e^{\left(P_2 \cdot 0.3125x\right)}\right]^{P_3}\right] \\ .3125 \cdot x \cdot \left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right)^{P_3} \\ \left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right)^{P_3} \\ -.3125 \cdot \left(.3125 \cdot P_0 \cdot x + P_1\right) \cdot \left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right)^{P_3} \cdot P_3 \cdot x \cdot \dfrac{\exp\left(.3125 \cdot P_2 \cdot x\right)}{\left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right)} \\ \left(.3125 \cdot P_0 \cdot x + P_1\right) \cdot \left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right)^{P_3} \cdot \ln\left(1 - \exp\left(.3125 \cdot P_2 \cdot x\right)\right) \end{bmatrix}$$

$fct1 := genfit(delta, rload, P, F)$ $\qquad fct1 = \begin{pmatrix} 0.438 \\ 0.579 \\ -4.013 \\ 0.922 \end{pmatrix}$ $\qquad r :=$ C:\..\Disp 3.2mm.xls $\qquad i := 0..5874$

$$y_i := \left(fct1_0 \cdot 0.3125 \cdot r_i + fct1_1\right)\left[\left[1 - e^{\left(fct1_2 \cdot 0.3125 \, r_i\right)}\right]^{fct1_3}\right]$$

y =

	0
0	0.042
1	0.043
2	0.065
3	0.09
4	0.112

r =

	0
0	0.047
1	0.049
2	0.077
3	0.111
4	0.142